University of Southern Queensland

Faculty of Health, Engineering & Sciences

Comparison of Austroads Guidelines for Lime Stabilisation of Highly Organic Black Soil

A dissertation submitted by

Runis Kashanov

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Abstract

Expansive clay soils, often referred to as 'Cracking Clays' and 'Black Soils', are prevalent in the inland regions of Queensland, Australia, covering around 28% of the state's land. These soils are well known for their high shrink-swell capacity and plasticity, expanding and transitioning from dry and cracked to slippery and yielding when soaked. Their reactivity makes them an undesirable subgrade material for road construction and rehabilitation, which requires specialised approaches like advanced drainage design, material replacement or lime stabilisation.

This follow-up study, based on pavement investigations carried out by the Queensland Department of Transport and Main Roads in 2017, examined black soil samples from a site 40 km south of Cooyar, QLD, mixed with lime following the Austroads (2019) Guide to Pavement Technology Part 4D. After 28 days of curing, this study assessed the Unconfined Compressive Strength (UCS) and soaked California Bearing Ratio (CBR) to determine the causes of the problems observed during the original DTMR investigation and compare the Austroads (2019) mix design methods.

The results showed that it was highly organic and responded well to what Little (1995) has termed a lime modification, indicated by a decrease in Maximum Dry Density and CBR Swell, and an increase in Optimum Moisture Content and CBR with increasing lime content. However, the soil was resistant to lime stabilisation, with UCS results of 0.3 or 0.4 MPa at all lime contents suggesting a negative impact of organic carbon content on the pozzolan fraction and reduction in the production of C-S-H and C-A-H gels. This indirect comparison of the two Austroads (2019) methods has shown that the comparison of CBR results before and after lime incorporation is not a reliable measure of the success of a pozzolanic reaction and only identifies that there has been a cation exchange between the surface of clay particles and calcium ions.

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Limitations of Use

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

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

Runis Kashanov

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Chapter 1

Introduction

1.1 Dissertation Overview

This dissertation is made up of 6 chapters and 6 appendices. A brief description of the content in each of the chapters is as follows.

Chapter 1 is this chapter and will provide background information about the problem, define the problem of the study and provide aims and objectives used to investigate it.

Chapter 2 explores, with the use of publicly available literature, the topics of soil classification and physics, history, mechanics and chemistry of lime stabilisation, documented effects of deleterious materials on lime reactivity, and comparisons between CBR and UCS tests in the context of lime stabilisation.

Chapter 3 provides an overview of the testing regime implemented to obtain the results required for the analysis to be performed and conclusions to be drawn. This chapter also presents risk management strategies that were implemented in preparation for the laboratory testing.

Chapter 4 explores the results and observations made during testing, providing reasonable explanations for unexpected observations and contradictory results between UCS and CBR results from the basic material mechanics and chemistry perspective.

Chapter 5 builds on the conclusions made in Chapter 4 by examining each of the possible causes of the contradictory results between UCS and CBR and investigating the possible implications of the results on pavement performance through the use of empirical and mechanistic-empirical pavement design and analysis methods.

Chapter 6 concludes the dissertation and suggests further work in the areas of the effects of organic contents on the pH and pozzolanic reactivity of the soil, the confining effect of a cylinder mould on laboratory CBR testing, and correlations between CBR and UCS results and bearing capacity.

1.2 Introduction

In recent decades, the practice of improving highly expansive subgrades for road construction by adding hydrated lime to the soil has seen a resurgence in the Darling Downs and South West districts, and other parts of Queensland (Beecroft and Coomer, 2018, Evans et al., 1998). This upturn in the road construction and rehabilitation industry is the result of ongoing review and improvement (Evans et al., 1998) of the processes involved in the practice of lime stabilisation by key bodies such as the Australian state road departments, their South African and American counterparts, and national and global industry associations.

One of the results of this process is the development and ongoing review of the Austroads Guide to Pavement Technology. Specifically, the recent revision of AGTP Part 4D (Austroads, 2019), which summarises the specifications and practises developed by Australian and New Zealand road authorities and industry bodies in order to provide an overview of soil and pavement stabilisation practise using various binders, and guide the user on how to select the appropriate binder and how to determine the appropriate portions of the binder to be used.

Due to the nature of the Austroads (2019) guide, it has become a primary source of information used by engineers and designers outside the state road authorities to implement soil and pavement stabilisation as part of their projects. However, as evidenced by the amount of work required to get the industry to where it is right now (Evans et al., 1998, Heimans, 2004), lime stabilisation is not a panacea solution to manage any expansive subgrade. A road rehabilitation project of a section of the New England Highway (22A) located approximately 40 km south of Cooyar, Queensland, carried out by the Queensland Department of Transport and Main Roads (DTMR) is a recent example of soil that does not respond to lime addition as expected. The precise location of the site has been reproduced in Appendices D.2 and D.3.

During the initial investigation process in 2017 by the Toowoomba DTMR laboratory, soil samples from 18 test pits along the alignment of the highway were tested to determine the suitability of the in situ material for use as a subgrade for a new road replacing the existing pavement that has reached its design life. The summary of the test results during this investigation has been reproduced in Figure 1.1.

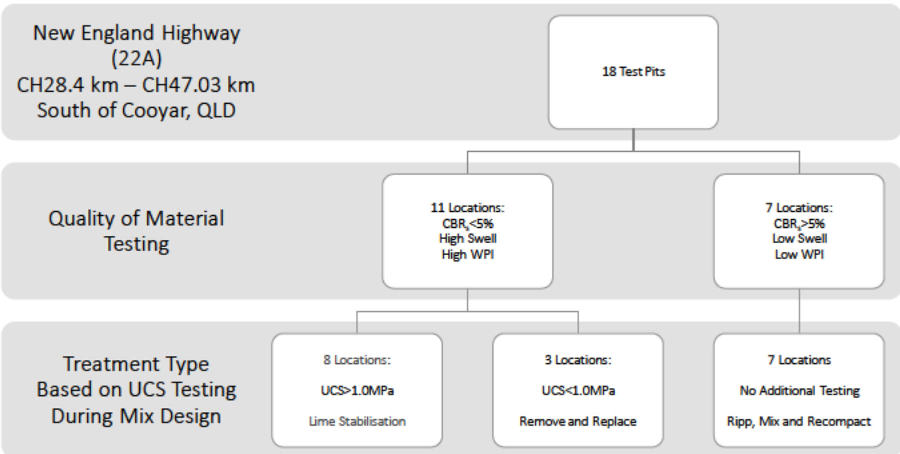


Figure 1.1: Detail of the test results during the initial investigation performed by the Toowoomba DTMR laboratory.

For 7 of 18 sections along the alignment, the existing soil material was considered suitable for use as a subgrade without the need for special treatment methods. For the remaining 11 sections along the existing alignment, the use of lime as a binder for soil stabilisation was determined to be appropriate on the basis of the swell and weighted plasticity index (WPI) criteria.

However, during the unconfined compressive strength (UCS) testing, the samples of existing soil sourced from test pits 9, 11 and 14 coinciding with isolated sections of black soil found along the alignment did not show an increase in the UCS results after the addition of lime and 28-day curing. Because the 28-day cured UCS results did not meet the DTMR (2021*c*) target range between 1.0 and 2.0 MPa, the existing black soil in the corresponding sections of the road was considered unsuitable for stabilisation with lime as a binding material, which resulted in the need to implement a more laborious subgrade treatment method for these sections of the alignment.

Figure 1.2: Surroundings of the original test pit 11. (Department of Transport and Main Roads 2017, project documentation, 5 October)

Figure 1.3: Wall face of the original test pit 11. (Department of Transport and Main Roads 2017, project documentation, 5 October)

Reviewing the geotechnical log and photos of the test pit 11 reproduced in Figures 1.2 and 1.3, it was stipulated that some impurities may have been present in the existing material causing a negative impact on the UCS results. The primary material that is believed to be of high content in the existing soil is organic matter, as it is believed to be the main cause of the black colour in the soil (Government, 2013). However, there is the possibility of iron oxide leaching into the black soil from the overlaying embankment made up of what appears to be orange lateritic soil, as seen on the upper right side of the test pit profile photo in Figure 1.3. As the Toowoomba DTMR laboratory is not equipped to perform deleterious material content testing, at the time of the original study no definitive conclusions were made about the origin of the unexpected UCS results.

1.3 Research Objectives

The following research was carried out under the sponsorship of DTMR in order to confirm and investigate the origins of the low UCS results observed during the initial investigation in 2017. The main aims of this study were to improve the understanding of the factors that affect the effectiveness of lime stabilisation of soils containing deleterious materials and to provide guidance on the use of Austroads (2019) guidelines for lime stabilisation under similar soil conditions to prevent unexpected and early failures of the overlying pavements in future road construction and rehabilitation projects. To satisfy the aims of the investigation, the objectives described in the following list were completed:

Quantify Deleterious Materials

Identify (See Section 2.6) and quantify materials (See Section 4.2) that may affect the performance of lime-stabilised Cooyar black soil.

Compare Two Austroads Methods

Compare the effectiveness of the two Austroads methods for determining the lime content required for stabilisation of Cooyar black soil (See Section 2.4.1).

Assess Effects on Overlaying Pavements

Assess the impact of any variations of the results obtained from the two Austroads methods on overlaying pavement performance (See Section 5.4.2).

Provide Recommendations

Provide recommendations for the best use of the Austroads guidelines for the stabilisation of lime in soils containing deleterious materials (See Section 6.4).

1.4 Chaper Summary

The purpose of this dissertation is to review and build on the findings of the investigation undertaken by DTMR in 2017 by taking samples of Cooyar black soil from the same location. First, by undertaking classification and deleterious contents testing on that material, followed by CBR and UCS teting at various the lime contents. The outcomes of this research will be used by the DTMR to carry out further research to better understand soil behaviour, CBR and UCS testing methods, and lime stabilisation mechanics.

Chapter 2

Literature Review

2.1 Introduction

In the following chapter, the literature relating to the lime stabilisation of expansive clays will be explored to provide background information on the topics of soil classification and physics of vertosols, history, mechanics and chemistry of lime stabilisation, documented effects of deleterious materials on lime reactivity, and comparisons between laboratory CBR and UCS tests in the context of lime stabilisation.

2.2 Highly Expansive Clays

Isbell and NCST (2021*b*) define Vertosols as clay soils with high shrink-swell properties, which can be discerned in the field due to these soils exhibiting strong cracking when dry, and at depth show lenticular peds and slickensides, which are grooves, striations and glossy surfaces on ped faces (Isbell and NCST, 2021*a*). More than half of Australia's vertosols are spread throughout Queensland, occupying 28% of the state's total area (SSA, 2022), making cracking clays the most dominant soil type in Queensland (Vanderstaay, 2020).

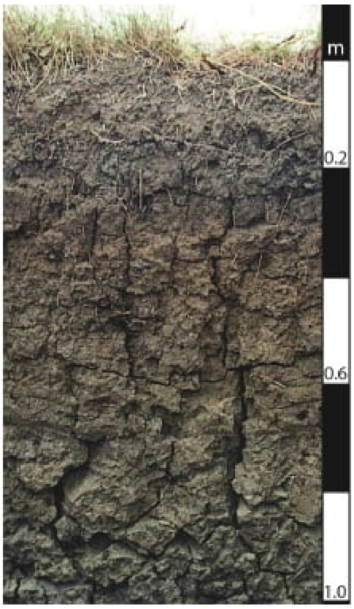


Figure 2.1: Vertosol soil in Beaudesert, Queensland, reproduced from Queensland Ggovernment (2013)

Figure 2.2: Sample of Black Soil displaying peds of varying sizes with slickensides. (R Kashanov 2023, personal photograph, 29 May)

Vanderstaay (2020) describes Vertosols by their more common names ”Cracking Clays” and ”Black Soils”. In his guide Vanderstaay (2020) summarises that Queensland black soils can be classified by soil origin and engineering properties into three groups. These classification groups have been reproduced in Table 2.1.

In his classification, Vanderstaay (2020) describes the residual soils of the Rolling Downs Group found in the Central West region of Queensland, and the alluvial soils, significantly transported from the sedimentary soils of the Rolling Downs Group and Tertiary Sediments to the regions south of the Warrego Highway and north of the Flinders Highway, as having a similar wide range of plastic properties. These soils fall anywhere between the upper limits of low plasticity (CL) to the lower limits of the highly plastic (CH) categories within the USC classification system.

In contrast, residual soils formed from Tertiary Basalt formations found in Darling Downs, Central Highlands, and Northern Tablelands regions are described by Vanderstaay (2020) to show a predominately highly plastic behaviour. This makes Queensland regions where these soils are encountered at greater risk of pavement failures associated with poor management of highly expansive subgrades.

Type Property	Alluvial	Residual on Cretaceous	Residual on Bassalt
Original Material	Rolling Downs Group, Tertiary Sediments	Rolling Downs Group	Tertiary Basalt
Transportation	Significantly Transported	Un-Transported	Un-Transported
Northcote Classification	Ug5.2	Ug5.3	Ug5.1
LL (%)	40-60	40-70	60-90
PI (%)	25-35	25-35	40-60
LS (%)	12-18	15-20	15-25
USC Classification	CL-CH	CL-CH	CH

Table 2.1: Typical origin and engineering properties of black soils encountered across Queensland, reproduced from Vanderstaay (2020)

Pavement Interactive (2023*a*) describes the typical soaked CBR values for medium and high plastic clays as less than 15% and 5%, respectively. This means that for black soils encountered in Queensland that exhibit high plastic properties, especially for residual soils on Tertiary Bassalt, special considerations must be taken when reviewing the test results for the suitability of the soil for use as a subgrade for road construction (DTMR, 2022*c*, cl. 9.2). DTMR (2022*c*, cl. 9.2) sets out that soils with a Plasticity Index (PI) greater than 50% and/or a CBR less than 3% are not suitable for use as a subgrade due to the high expansive nature and low bearing capacity of those soils in saturated condition. This means that before pavement construction over these unsuitable materials, the subgrade layer below and adjacent to the pavement and embankment must first be treated.

DTMR (2022*c*, cl. 18.3.3) describes the common treatment methods for in situ materials. These methods can be generalised as recompacting the material, adding a bridging layer between the subgrade and embankment fill, removing and replacing the subgrade with better materials, and stabilising the subgrade using in situ stabilisation methods. As recompacting and adding bridging layers over expansive subgrades may not be applicable in most situations, it is most common that either the remove and replace method or in situ stabilisation is used to treat expansive subgrades.

With the recent changes to the Queensland Clean Earth Exemption (Queensland Ggovernment, 2023), and the push within the industry for more sustainable and less wasteful road construction and rehabilitation strategies (DTMR, 2023); it is becoming more attractive to use in situ stabilisation to treat unsuitable materials instead of spending money and time excavating, exporting and dumping the unsuitable material, only to then spend more money and time importing a replacement material. Especially in the case of road rehabilitation works where any delay in the works extends the disruptions for the public, who will continue to use the road for personal and commercial purposes throughout the construction process.

2.3 History of Lime Stabilisation

Soil stabilisation and soil modification are two terms that are used interchangeably to describe a process in which soil is modified using a physical, mechanical, chemical, biological or combined process to improve select engineering properties of a soil for use as a construction material (Substrata, 2023). As part of its "Oral History Program" the NSW Road and Traffic Authority has examined 28 hours of interviews with 23 pavement industry professionals and academics to recount the history of pavement recycling and stabilisation practise, and the results of research and cooperation between road authorities and industry (Heimans, 2004).

From the summary report prepared by Heimans (2004) it can be inferred that pavement recycling and stabilisation practise in Australia can be traced back to the Second World War when the American military would import and use P&H stabilisation machines similar to the one depicted in Figure 2.3 to quickly stabilise haul routes, infrastructure hard stands, and airstrips using cement as the binding material. Following the departure of the American military, an Australian company, which was then called Stabilising Australia, acquired the machines that were left behind by the American military and began to promote and undertake pavement stabilisation of Australian pavements throughout the 1950s. In the 1960s, after a significant failure rate was observed in cement-stabilised pavements, cement trials were undertaken to study the causes of these failures.

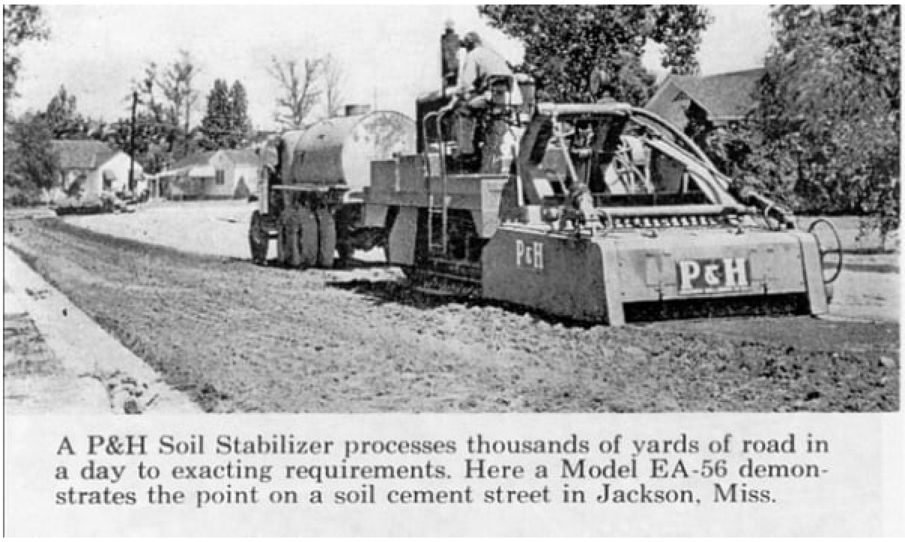


Figure 2.3: Excerpt from an unknown news publication showing a P&H stabilisation machine, reproduced from the collection of historic photos and documents produced by or related to the Harnischfeger, P&H or JoyGlobal prepared by Ray (2017).

In the 1960s, trials also emerged that used lime as a binder to treat the plasticity of subgrades of "black soil country" in New South Wales (Heimans, 2004) and Queensland (Beecroft and Coomer, 2018), with Ipswich City Council using lime stabilisation to treat subgrades as early as 1961 (Beecroft and Coomer, 2018). However, the practice of lime stabilisation quickly fell out of favour within the Queensland industry due to an undesirable failure rate observed throughout the 1970s (Beecroft and Coomer, 2018, Evans et al., 1998).

It was not until 1997 (Evans et al., 1998) that lime stabilisation was again considered for the treatment of highly expansive subgrades when a "Steering Committee" spearheaded by the Queensland Department of Transport and Main Roads had reviewed the recent research by Dallas Little (1995). Little (1995) described lime modification as a technique in which small amounts of quick lime or hydrated lime are mixed through pulverised soil to remove excess moisture and improve the workability and constructability of the soil. Little (1995) then defined that lime stabilisation follows a process similar to modification; however, the amount of lime added is increased to provide long-term strength gain to the soil as measured by UCS testing, in addition to the improvements experienced during lime modification. Based on his recommendations, traditional methods of mix design that relied on lime content that was just sufficient to meet lime demand were revised to include peak UCS testing (Evans et al., 1998).

2.4 Lime Stabilisation Process

The lime stabilisation process follows three basic phases: mix design, lime incorporation, and curing, which are modified and expanded according to the specific requirements and conditions of the project.

2.4.1 Mix Design

The process begins at the design stage of the project, where upon initial investigations the project designers, in consultation with experienced engineers and relevant specifications and guides, determine if lime stabilisation will be an effective and financially appropriate form of treatment for the specific project, soil and environmental conditions. Generally, clay soils with more than 25% particles passing the 75 µm sieve and a Plasticity Index (PI) greater than 10 (Austroads, 2019, Little, 1995, NLA, 2004) or a Weighted Plasticity Index (WPI) between 2200 and 3200 (DTMR, 2022*e*) are believed to respond more successfully to lime stabilisation. For soils that are deemed suitable for lime stabilisation, the amount of lime that must be added to the soil to achieve long-term strength must be determined through a process called mix design.

The lime stabilisation industry authorities outline similar processes for the mix design, which can be grouped with those that recommend the use of the California Bearing Ratio (CBR) or those who prefer UCS testing to quantify the effectiveness of lime stabilisation (Roads and Infrastructure Australia, 2016). The Guide for Pavement Technology produced by Austroads (2019) sets guidelines for the use of both methods, as shown in Figure 2.4. Both methods begin with soil classification and a lime demand test (Austroads, 2019) to determine the optimum amount of lime required to establish an environment that is believed to be favourable for the pozzolanic reaction to occur (Ouhadi et al., 2014, Little, 1995).

CBR methods used in New Zealand and most of Australia typically nominate the lime content for construction as 1% more than the quantity determined during a lime demand test, and use the results of the CBR test for pavement design calculations and to inform the design team about expected improvements to the soil (Austroads, 2019). Meanwhile, the UCS method developed by Little (1995) and preferred by DTMR (2021*c*) uses the

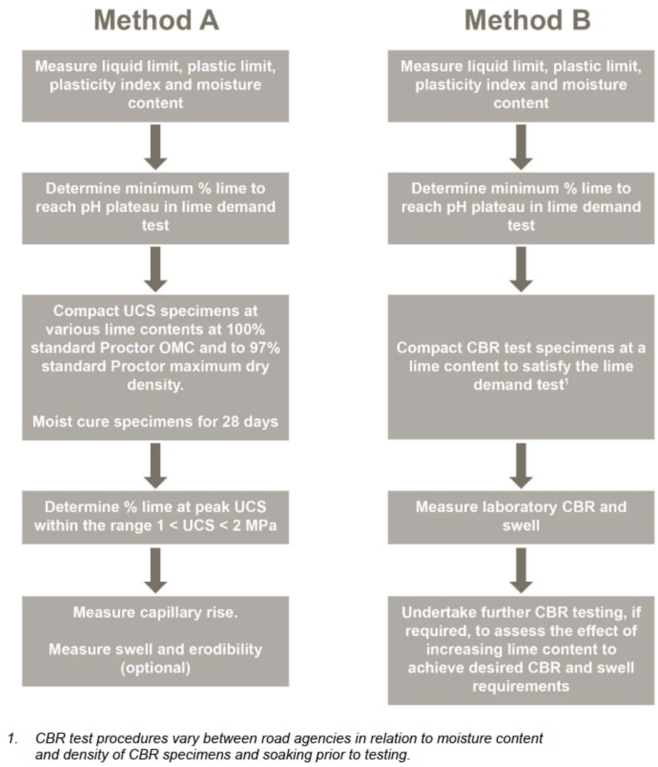


Figure 2.4: Determination of lime content of earthworks materials, reproduced from Austroads (2019, p.29).

results of 28-day cured UCS tests at various lime contents to determine the lime content at which the peak UCS value is observed for construction, provided that the value of the peak UCS is above a threshold of 1.0 MPa to classify the stabilised soil as lightly bound and the lime content at the peak is at least 1% higher than the lime demand of the soil.

2.4.2 Lime Incorporation

The next phase is the soil stabilisation itself. This ground improvement process begins with trimming the subgrade to the desired shape and level, as set out in the project documentation. The subgrade is then covered with lime using a spreader trailer attached to a tractor or a dedicated spreader truck as shown in Figure 2.5. The quantity of lime to spread over the subgrade area is determined based on the lime dosing rate established in the specifications, the thickness of the desired stabilised subgrade layer, and the ratio between the available lime index of the hydrated lime used during the mix design and the hydrated or quicklime used in the field (DTMR, 2022*e*).

Figure 2.5: Quicklime being spread with a spreader truck. (R Kashanov 2022, personal photograph, 9 July)

Figure 2.6: Quicklime being slaked with a modified water tanker. (R Kashanov 2022, personal photograph, 9 July)

Figure 2.7: Slaked lime being incorporated into the subgrade using a Wirtgen soil stabiliser. (R Kashanov 2022, personal photograph, 1 October)

If quicklime is used during the stabilisation process, it must be hydrated with water for the pozzolanic reaction to occur most efficiently. The process by which the quicklime (CaO) is hydrated is called slaking, and depending on the amount of water being added, the hydrated lime ($Ca(OH)_2$) or the lime slurry (saturated $Ca(OH)_2$) is the end product. This process is commonly carried out by spraying the quicklime that has been spread over the subgrade with water from a water cart or a water tanker as shown in Figure 2.6, however, currently there are trials for this process to be carried out in situ (Volker, 2019) as a way to improve the consistency and efficiency of the stabilisation process.

The hydrated lime is then incorporated into the subgrade using a soil stabiliser as shown in Figure 2.7. This machine (Wirtgen Group, 2022) uses a powerful milling rotor that is lowered into the ground, pulverising the soil below the machine and mixing it with overlying lime. The result of the process is a loose homogeneous layer of lime and soil. Sometimes the project specifications may require that the lime be spread and incorporated over the same area multiple times. In those situations between each lime incorporation cycle, the subgrade must be compacted and shaped, and an amelioration period between the drops may even be required (DTMR, 2022*e*, Berger et al., 2001). Once all of the lime required for stabilisation has been incorporated, the subgrade must be compacted and shaped again, and the stabiliser machine connected to a water truck or tanker is used to incorporate water homogeneously into the soil. This final pass with the stabiliser allows the soil to be brought to an optimum moisture content for compaction and will provide an aqueous environment for reactions between the lime and the soil to occur.

2.4.3 Curing

The final stage of the lime stabilisation process is curing. This is an intermittent stage between the actual stabilisation of the subgrade and the construction of the overlaying pavement layers. As the success of lime stabilisation depends on a hydration reaction that only occurs in the presence of water, the stabilised subgrade layer must be protected from drying by maintaining its surface in a damp condition (DTMR, 2022*e*).

Maintenance of the surface layer of the stabilised subgrade in moist conditions prevents excessive contact between the surface and the air, reducing the amount of carbonation that occurs. Carbonation occurs when free calcium (Ca^{2+}) ions react with carbon dioxide

(CO_2) forming calcium carbonates ($CaCO_3$) (Barman and Dash, 2022, Little, 1995). Barman and Dash (2022) describe the carbonation products as cementing agents, which can provide a small increase in soil strength; however, Barman and Dash (2022) and Little (1995) state that the loss of calcium ions available for the pozzolanic reaction will result in a net loss in the long-term strength of the stabilised layer.

2.5 Reactions Between Lime and Soil

AustStab (2008), Little (1995) and NLA (2001) describe some of the expected effects of lime modification and stabilisation on the engineering properties of soil as follows:

- Immediate and Short-Term Benefits from Lime Modification and Stabilisation:
  - Improved plastic properties by reducing the PI of the soil.
  - Reduction in the moisture holding capacity of the soil.
  - Improvement of the shrink-swell behaviour of the soil, reducing heave and cracking.
  - Increase in the stability of the soil, allowing for the construction of temporary working platforms.
  - Increase in immediate CBR values as a measure of shear strength improvement.
- Long-Term Benefits from Lime Stabilisation Only:
  - Increase in the resilient modulus values, which are a measure of the soil stiffness.
  - Increase in the compressive, tensile, and flexural strengths of the soil.
  - Continued strength gain, even after curing is complete.

The immediate improvements in the workability and constructability of the soil perceived after lime modification and stabilisation of the soil are mainly attributed to calcium (Ca^{2+}) ions that replace the cations of the surface of the clay (NLA, 2001). Little (1995) reasons that the effectiveness of lime in altering the surface chemistry of clay particles is because calcium ions released from lime have higher valence and are higher in lyotropic series than most of the cations present in clay, resulting in a high rate of cation exchange.

Arguing that the rate of cation exchange increases further with an increase in the concentrations of calcium ions, allowing for the exchange between calcium ions and surface cations of the same valence and lower concentrations.

Little (1995) then describes that replacement of the cations on the surface of the clay particles with calcium ions results in a decrease in the size of the diffused water layer around the clay particles, allowing for edge-to-edge attraction and flocculation, and a subsecquential textural change of the clay from a cohesive mass to a friable material similar to sand.

Meanwhile, the long-term strength gain experienced after lime stabilisation is a result of a hydration reaction between the pozzolans and lime. The process starts as soon as the lime is mixed with the clay soil and a sufficient amount of water is added during the final incorporation pass of the soil stabiliser machine. In the aqueous solution, hydroxide ions (OH^-) are released from lime (Barman and Dash, 2022), neutralising any acids present in the soil and increasing the alkalinity of the soil.

Once the pH of the soil is above 10.5, the silicate (SiO_2) and aluminate (Al_2O_3) pozzolans present in clay particles become soluble (Sargent, 2015). When in an alkaline solution with a pH of ≥ 12.4 (Ouhadi et al., 2014, Little, 1995) the calcium ions and hydroxide ions remaining after initial reactions will partake in the following pozzolanic hydration reactions (Barman and Dash, 2022, Little, 1995):

$$\mathrm{Ca^{2+} + 2OH^- + SiO_2 \rightarrow 3CaO \cdot 2SiO_2 \cdot 3H_2O} \tag{2.1}$$

$$\mathrm{Ca^{2+} + 2OH^- + Al_2O_3 \rightarrow 3CaO{\cdot}Al_2O_3 \cdot 3H_2O} \tag{2.2}$$

The products of these hydration reactions are cementitious compounds called calcium silicate hydrate (C-S-H) and calcium aluminate hydrate (C-A-H) gels (Barman and Dash, 2022, Little, 1995). These gels coat and bind clay particles together (Barman and Dash, 2022). Provided a sufficient amount of calcium ions is available for the pozzolanic reaction and the pH of the solution remains above 12.4 a strength gain (Little, 1995) and self-healing are observed through the ongoing dissolution of pozzolans (Groot et al., 2022) throughout the life of the subgrade.

2.6 Factors Influencing Pozzolanic Reactivity

Two crucial criteria must be met for a successful pozzolanic reaction and the subsequent development of long-term strength. There must be a sufficient amount of SiO_2 and Al_2O_3 available to enter the solution and form C-S-H and C-A-H gels. A sufficient amount of lime must be added to establish a pH of ≥ 12.4 (Ouhadi et al., 2014, Little, 1995), which ensures that there is enough calcium available to maintain the solubility of pozzolans and that there are enough calcium ions to satisfy the initial and pozzolanic reactions (Little, 1995).

Therefore, the processes that can adversely affect the lime reactivity of the soil are as follows:

1. The initial pH of the soil before lime incorporation being below 7 (Thompson, 1966), indicating that a substantial amount of lime will need to be added to bring SiO_2 and Al_2O_3 into solution and maintain the pozzolanic reaction (Little, 1995).
2. Elevated presence of calcium-reactive impurities will reduce the amount of Ca^{2+} ions available for the pozzolanic reaction, resulting in the need for additional amounts of lime to produce the same quantities of C-S-H and C-A-H gels. (Little, 1995, Sargent, 2015, Mitchell and Dermatas, 1992)
3. Natural blending of the soil with impurities will decrease the concentration of reactive clay particles in the soil (Hampton and Edil, 1998). This coupled with the fact that some impurities have been described by Eisazadeh et al. (2011), Little (1995) and Mohd yunus et al. (2013) to coat clay particles means that an increase in impurities will directly lead to less SiO_2 and Al_2O_3 available for the pozzolanic reaction and that less C-S-H and C-A-H gels will be produced in the same volume of the soil.

Reviewing the publications and technical notes produced by the Australian and American lime stabilisation industry bodies, there are three main characteristics of the in situ soil that are believed to be the primary sources of poor lime reactivity of soil. Specifically, AustStab (2008) and NLA (2004) state that organic impurities and soil sulphate contamination can prevent reactions between lime and clay. Austroads (2019) and DTMR (2021*c*) also include the degree of weathering in the form of ferrous oxide concentrations as the last factor on the list. The maximum contents of these materials for soil to be considered suitable for lime stabilisation, as established by these industry bodies, are summarised in Table 2.2. The mechanisms by which each of these factors influences the reactivity of the soil are described below.

Industry Body	Factor: Organic Matter	Factor: Sulphate	Factor: Degree of Weathering
Austroads (2019)	$\leq$ 10.0%	$\leq$ 0.9 g/L SO_4	$\leq$ 2.0% FeO
DTMR (2017, 2021*c*, 2022*e*)	$\leq$ 1.0/10.0%[1]	$\leq$ 0.3%	$\leq$ 2.0% FeO
AustStab (2008)	No Limit Stated[2]	$\leq$ 0.3%	No Mention
NLA (2004) and Little (1995)	$\leq$ 10.0%	$\leq$ 0.3%	No Limit Stated[3]

[1] DTMR (2017) recomended a limit of $\leq$ 1.0% organic carbon, until the more recent specifications, where DTMR (2021*c*, 2022*e*) sets-out a more broader limit of $\leq$ 10.0% organic carbon.

[2] No limits of sulphate contents were stated by AustStab (2008), however the deleterious effect from the presence of the organic matter on the reactivity of the soil to lime was mentioned.

[3] No content limits of metals indicating the degree of weathering were stated, however, the negative effects of weathering and leaching are well described by Little (1995).

Table 2.2: Limits of deleterious material contents in soil considered suitable for lime stabilisation.

2.6.1 Organic Matter

The deleterious effects of organic compounds in soils stabilised with calcium-based binders have been extensively studied, with several theories and explanations being reported over the last half a century. Some of the reported sources of problems correlated with increased organic content are the acid-base reaction of calcium-based binders with humic acids, reduced fraction of clay minerals available for the hydration reaction to occur, and the potential of organic compounds promoting the growth of ettringite.

Humic acids are one of the two types of organic acid polymers found in organic soil and are the result of the breakdown of organic matter by microorganisms (The Editors of Encyclopaedia Britannica, 2020). Humic acids act as a retardant for pozzolanic and cementitious reactions in three ways. When in solution, the acids release free hydrogen ions, which react with free hydroxil groups lowering the pH of the solution, which means that a more calcium-based binder must be added to the soil to maintain the pH of ≥ 12.4 required for a successful pozzolanic reaction (Ouhadi et al., 2014).

Harris et al. (2009) have observed that variation in humic acid content in the soil does not appear to affect the lime demand test. That is, humic acid does not appear to immediately release free hydrogen ions, resulting in similar pH values at the same levels of lime content at low and high concentrations of humic acid during the test. However, this means that if hydrogen ions were to be released from humic acids after stabilisation had occurred lowering the pH, halting the pozzolanic reaction, and potentially causing the reverse of cation exchange between calcium ions and other free cations of higher concentration and valence back to the surface clay minerals.

Furthermore, humic acids have been reported to have an affinity for calcium ions (Shiroya and Kumada, 1976), which removes them from the solution (Sargent, 2015) and reduces the amount of free calcium ions available for the pozzolanic reaction. Finally, humic acids under SEM and XRD analysis have been shown to coat clay minerals (Mohd yunus et al., 2013). This means that in a strong base environment these acids coagulate (Bleam, 2017) around the minerals, preventing cation exchange between calcium ions and surface cations of clay minerals, and preventing SiO_2 and Al_2O_3 from entering the solution and reacting with the remaining calcium ions.

The organic carbon content in the soil typically ranges between 0.5% and 3.0% in the top soils, but it can be greater than 18% in the organic soils (Wikipedia Contributors, 2023). The increase in the concentration of impurities such as organic carbon is directly related to a decrease in the fraction of clay minerals in the soil, resulting in a decrease in SiO_2 and Al_2O_3 available for the pozzolanic reaction. As found by Hampton and Edil (1998) in soils with high concentrations of organic carbon, the addition of lime as a calcium-based binder will not induce a pozzolanic reaction due to the lack of pozzolans present in the soil and being added as part of the binder.

Rawls et al. (2003) have also shown that high concentrations of organic carbon have been correlated with an increase in water retention in fine and coarse-grained soils. As the abundance of water can draw additional sulphates from the adjacent soil (Ouhadi and Yong, 2008), which are essential for the formation of ettringite, Hampton and Edil (1998) have found that high organic content can promote the growth of ettringite. The effects of ettringite growth are described in the following subsection.

2.6.2 Sulphates

In their industry-changing research after the 1975 Stewart Avenue failure Mitchell and Dermatas (1992) summarise that trisulphate hydrate (ettringite) and monosulphate hydrate are the only two stable products of the calcium-aluminium-sulphate hydration reaction. They then describe ettringite as a mineral of greater concern for the performance of lime-stabilised soils, because, unlike monosulphate hydrate, it is stable in both dry and wet conditions, can cause heave of the soil during formation, as well as the possibility of ettringite being transformed into the similarly deleterious mineral thaumasite under cold weather conditions below 15°C and in the presence of soluble carbonates.

The formation of ettringite is predominant when the amount of soluble sulphate is significantly greater than the amount of Al_2O_3 available for the reaction (Mitchell and Dermatas, 1992). This means that if there is a sufficient supply of sulphate from gypsum or by influx through water percolation (Berger et al., 2001, Mitchell and Dermatas, 1992) even at high pH, the addition of calcium-based binders will result in the growth of ettringite minerals instead of the formation of C-S-H and C-A-H gels. Therefore, as confirmed by Cheshomi et al. (2017), in the presence of water, the addition of a calcium-based binder to soil with a high sulphate content will increase the unwanted expansive behaviour of the soil.

Little (1995) summarises, that the sulphate heave can be less detrimental to pavement performance if the following "good" construction techniques are followed:

- Ettringite formation is allowed to happen during the mixing and mellowing periods in abundance of water, to allow for the free sulphate to be used up before compaction. Thus, ensuring that minimal hydration of calcium-aluminium-sulphate occurs after compaction.

- The method of lime incorporation produces a homogeneous mix of in situ materials, lime, and water. This is done to prevent the formation of regions with high sulphate content and the possibility of long-term formation of calcium-aluminium-sulphate hydrate.
- The pavement design allows sufficient drainage and diversion of water from the stabilised soil layer, to minimise the amount of sulphate entering the stabilised material from the nearby contaminated soils and resulting in a calcium-aluminium-sulphate hydration reaction.

Following on this Berger et al. (2001) devised to rank the sulphate content in the soil according to the amount of risk expected as a result of the stabilisation process, to be used as a guide for the best practices on how to mitigate these risks during the lime stabilisation practise. The sulphate content ranges in ppm of the soil mass and the related risk mitigation strategies suggested by Berger et al. (2001) and later confirmed by Harris et al. (2004) are as follows:

No Concern ($SO_4^{2-} < 3,000$)

Proceed with the usual good mix design and construction practises. Adequate water should be used during mixing (OMC plus 3%). If any phosphates are detected, substitution of dry lime with lime slurry is recommended.

Moderate Risk ($3,000 \leq SO_4^{2-} \leq 5,000$)

Can be successfully treated if situation-specific methods and controls are established early and followed throughout the design and construction process. Extra water should be used during mixing (OMC plus 3% to 5%), mellowing and curing. Substitution of dry quicklime and hydrated lime with lime slurry is recommended. A mellowing period of at least 72 hours is recommended.

Moderate to High Risk ($5,000 \leq SO_4^{2-} \leq 8,000$)

Generally can be treated following the same principles as the sulphate content posing a moderate risk, however, swell potential testing is recommended to establish the expected amount of swelling and the mellowing period required before compaction.

Unacceptable Risk ($8,000 < SO_4^{2-} \leq 10,000$)

Occasionally can be treated, only if experienced contractors are engaged, educated about the stabilisation of lime in high-sulphate soils and double application techniques, and provided that thorough initial laboratory tests are carried out. It will require lime slurry to be used as a source of lime and must have a high water content during mixing, mellowing, and curing. Density monitoring is recommended, as the mellowing period may last as long as 7 days.

Generally Not Suitable ($10,000 < SO_4^{2-}$)

Usually, concentrations are localised to the seams. Field electrical conductivity testing may help characterise the seams and allow problem-specific strategies, such as removal or blending, to be devised.

2.6.3 Degree of Weathering

Little (1995) describes the degree of weathering and drainage as another major set of attributes that can have negative effects on the lime reactivity of the soil. Weathering of well-drained soils causes leaching of minerals and other constituents from the upper levels of the soil through water percolation (The Editors of Encyclopaedia Britannica, 2010). Along with the type of soil parent material and the breakdown of organic matter, leaching is one of the main sources of soil acidity (Zhang, 2017).

Little (1995) detailed that leaching of the metallic cations is followed by absorption of the hydrogen cations and a consequent decrease in pH. As previously described, acidification of the soil results in an increase in the amount of calcium-based binder that must be added to the soil to reach the levels required for SiO_2 and Al_2O_3 to become soluble and for the pozzolanic reaction to be sustained successfully. Thompson (1966) summarised this by categorising soils with a pH less than 7 as soils that indicate poorer lime reactivity.

In addition, soil weathering can cause laterisation, where continued percolation can alter the mineral profile of a well-drained highly weathered soil to contain less SiO_2 due to leaching and more Al_2O_3 and Fe_2O_3 caused by deposition (Eisazadeh et al., 2011), compared to a similar soil that is only slightly or moderately weathered (Thompson, 1966). If a laterite soil is contaminated with phosphate, the extra concentration of Al_2O_3 will lead to a higher growth potential of ettringite, compared to a less weathered counterpart.

Eisazadeh et al. (2011) and Little (1995) write that free Fe_2O_3, similarly to humic acids, can coat clay particles, preventing the dissolution of Al_2O_3 and Fe_2O_3 or the exchange of cations between calcium hydroxide and clay minerals.

2.7 Laboratory CBR and UCS Tesing

Roads and Infrastructure Australia (2016) writes that the choice between the use of design procedures based on UCS and CBR testing is a contentious topic within the Australian civil construction industry. The DTMR (2021*c*) design procedure emphasises a strength improvement as a result of a successful pozzolanic reaction by adding additional lime to achieve a UCS result target of 1.0 MPa over methods that use CBR testing and rely on thicker pavement layers to protect lime-treated subgrades at lower lime contents (Roads and Infrastructure Australia, 2016). As the choice to use either of the methods is a critical financial decision, it is important to distinguish between how the UCS and CBR tests are performed in the laboratory, what these methods measure, and if there are any correlations between the results.

2.7.1 How are UCS and CBR tests performed

The Q113C test method is the standard test method used by DTMR (2022*b*) to determine the California Bearing Ratio of stabilised soils. In this method, soil compacted into a standard cylinder at a nominated level of dry density and moisture content is soaked in water over a set period before being penetrated using a standard CBR machine, depicted in Figure 2.8 consisting of a force measuring device, penetration gage, penetration piston, and a movable platen with a uniform rate of movement. During the test, the CBR machine measures the applied force and penetration into the cylinder and plots the values on a force-penetration curve. After the test is complete and the force-penetration curve has been adjusted by drawing a tangent through the steepest part of the curve, the values of force applied corresponding to the adjusted 2.5 mm and 5.0 mm penetration are reported as a percentage of 13.2 kN and 19.8 kN respectively.

The Unconfined Compressive Strength is determined by DTMR (2022*b*) using the Q115 test method. This method uses the same CBR testing machine, modified with a platen on a spherical seat in the leu of a penetrator, depicted in Figure 2.9, so that during the test two plates compress an unsaturated soil sample that has been compacted into a standard cylinder, taken out of the mould, and cured for a set period. During the test, the force applied to the cylinder is monitored and recorded upon failure of the cylinder. The force applied at the time of failure is then co-related using the specified method to determine and report the Unconfined Compressive Strength of the soil.

Figure 2.8: Example of a CBR test being performed. (R Kashanov 2023, personal photograph, 6 September)

Figure 2.9: Example of a UCS test being performed. (R Kashanov 2023, personal photograph, 2 May)

2.7.2 What do UCS and CBR test results determine

The use of the UCS test of a cohesive soil to measure compressive shear strength and estimate tensile and flexural strength, as well as other structural properties of the soil(Little, 1995) is a common practice in the field of geotechnical engineering (Hossain et al., 2021). In civil engineering practice during the pavement design process using lime-stabilised subgrades, DTMR (2021*c*), and many state agencies in the United States (Little, 1995), use the magnitude of unconfined compressive strength after a 28-day cure as a measure of the effectiveness of a pozzolanic reaction and to confirm whether the subgrade layer can be treated as a lightly bound structural pavement layer.

In comparison, CBR is a purely empirical measure that estimates how similar the mechanical properties of the tested soil are to a standard well-graded crushed stone (Pavement Interactive, 2023*a*) based on the stress exerted at a given depth of penetration. As the CBR results do not correlate directly with any basic material property (Little, 1995), the CBR is only used in civil engineering practice, primarily as part of flexible pavement design methods.

2.7.3 Correlations between UCS and CBR test results

When comparing the results of two strength testing methods, it is important to distinguish any correlations that might exist between the UCS and CBR results from what effects the addition of lime has on UCS and CBR individually.

Although UCS is a measure of the axial compressive strength of a cohesive material, expressed in SI units, and can be used to estimate other basic structural properties, CBR is only an indirect measure of the shear strength of a material (Little, 1995) and cannot be directly related to other structural properties. However, several attempts have been made over the years to make statistical correlations between CBR and UCS values (Baig, 1962, Eme et al., 2016, O'Flaherty et al., 1961, Saputra and Putra, 2020). But the results of these attempts are that statistically significant correlations cannot be made (Baig, 1962, Eme et al., 2016), or that if there is any correlation between UCS and CBR results, it is very specific to the materials used during testing and cannot be generalised (O'Flaherty et al., 1961, Saputra and Putra, 2020).

This sensitivity in the CBR results to changes in material properties observed in the literature extends to studies trying to predict the CBR results based on the index properties of the soil. Rehman et al. (2017) have collated many previous studies on this topic, showing that depending on the materials used and the analysis techniques, each attempt to establish a relationship between the index properties and the CBR has resulted in unrelated results of varying accuracy. Despite this, the models presented by Rehman et al. (2017) and the artificial intelligence analysis and modelling carried out by Kassa and Wubineh (2023) and Taskiran (2010) show that the CBR of soil is most sensitive to the workability of the soil as measured by maximum dry density, optimum moisture content, and plasticity measures in the form of fines contents and atterberg limits.

2.8 Applicability of Laboratory CBR Testing

In his older study Black (1961) has also identified inconsistencies between results from three methods by which the percentage CBR of the same material can be determined, even if the variations in compaction are controlled for. He described that the result of a CBR test on an undisturbed sample cut from in situ soil and trimmed to the size and shape of a standard mould will be different when compared to the result of a CBR test on a remoulded sample of the same soil at the same density and moisture content. Black (1961) has elaborated that methods of compaction in the laboratory and in situ may result in different pore water pressures affecting CBR values.

Black (1961) then further described how the compaction method and the confining effect of the CBR mould can affect the results of the laboratory CBR tests compared to the in situ tests. Black (1961) was able to determine that the confining effect of the mould is marginal in cohesive soils, by establishing a theoretical relationship between the CBR results, the bearing capacity, and the measurements of the cohesion, suction and internal friction angle, and with the help of the empirical data. However, for soils with an internal friction angle greater than 30°, a restrictive effect provided by the mould was observed as a result of the circular shear surface, which develops under the load applied by the penetration piston, crossing the wall of the mould instead of reaching the surface of the sample.

As previously established, provided that a certain amount of lime is added to a soil, a modification of mechanical properties occurs as a result of calcium ions replacing the cations on the surface of clay minerals (NLA, 2001). This cation exchange reduces the defused water layer around clay particles, allowing the particles to come closer, have a more structured alignment, and allow flocculation to occur (Little, 1995). The result of this is a clay mass that has been broken down into individual clots of reduced plasticity that act more like fine-grained sand (Little, 1995). Therefore, the addition of lime to clayey soil will result in CBR values that are more similar to those of a granular material. Additionally, since the internal angle of friction of cohesive soil is related to the plasticity index, the apparent increase in the laboratory CBR results will be further exacerbated due to the "mould effect" described by Black (1961). This means that as the results of the laboratory CBR test do not rely on a strength improvement associated with a successful pozzolanic reaction there will be an increase in CBR in both modified and stabilised soils.

In light of this, when flexible pavement design using lime-stabilised subgrades is performed, care should be taken to select an appropriate CBR value to be assigned to the subgrade. Consistent with the recommendations made by the Steering Committee (Evans et al., 1998), Little (1995) and DTMR (2021*c*) if there is no evidence of long-term strength improvement, the soil cannot be considered stabilised. Since an increase in laboratory CBR values does not automatically mean that a successful pozzolanic reaction has occurred, if no UCS testing is performed, extra caution should be used if any CBR value other than the original percentage of CBR before lime addition is used.

2.9 Chapter Summary

Vertosols are highly expansive clay soils of moderate to high plasticity and the most common type of soil encountered in Queensland. The most effective methods to treat these highly expansive soils for use in road construction are to replace the topmost layers of the subgrade with less reactive materials or to stabilise the subgrade with a binding material such as cement or lime. The first and most important step in the soil stabilisation process is the mix design; during this step, the soil is tested to determine if the soil is suitable for stabilisation and what type and quantity of binder is most appropriate for the application. Austroads (2019) describes two methods to determine the lime content to be used in the stabilisation, with the main difference between the two that Method A follows the guidelines provided by Dallas Little (1995) and emphasises the strength performance of the soil stabilised with lime through laboratory UCS testing instead of the CBR testing used in Method B.

With lime stabilisation, three key elements are required for a successful outcome to be observed. There shall be a sufficient amount of hydroxyl groups provided by lime to raise the pH to a level high enough to cause the dissolution of silicon and aluminium oxides from the clay particles and to sustain a pozzolanic reaction. There shall be enough silicone and aluminium oxides and calcium ions remaining after the initial reactions with impurities are completed to produce a sufficient amount of C-S-H and C-A-H gels through a pozzolanic reaction. Any imbalance in free hydroxyl groups, pozzolans, and calcium ions will result in a reduced fraction of C-S-H and C-A-H and reduced binding of clay particles.

The literature indicates that three impurities are believed to have the greatest effect on lime reactivity, organic matter, iron oxide and sulphate. The organic matter is described in the literature in two forms, organic carbon and humic acids. The common concerns with organic carbon contents are that elevated impurity contents directly affect the fraction of clay minerals and pozzolans available for the pozzolanic reaction, as well as that organic carbon can cause elevated levels of water retention posing a risk for sulphate being drawn out from adjacent soils promoting ettringite growth. Meanwhile, the humic acids have been reported to negatively impact pozzolanic reactivity through acid-base reactions with hydrated lime, removing both calcium and free hydroxyl groups from the solution, as well as coating the clay particles preventing the release of pozzzolans.

Likewise, iron oxide has also been reported to coat the clay particles, preventing the release of pozzolans and catio exchange between clay minerals and calcium ions. Additionally, iron oxide is deposited in the soil as a result of laterisation, where due to continuous speculation the aluminium oxides and metallic cations have been leached out of the clay minerals, reducing its pozzolanic reactivity and the pH respectively.

Ettringite is a product of a hydration reaction between calcium aluminium and sulphate, competing for the same elements required for a pozzolanic reaction to occur and requiring additional lime contents and pozzolan dosing. Additionally, when ettringite crystalises it rapidly grows in size, causing heave to the soil around it, making soils with high sulphate contents of more than 3% an undesirable candidate for lime stabilisation, requiring special construction methods and controls to avoid ettringite growth after compaction.

Chapter 3

Methodology

3.1 Introduction

The material testing regime presented in the following chapter has been prepared with help from DTMR Toowoomba laboratory staff, following recommendations set out in the Austroads (2019) guidelines, DTMR (2022*b*) material testing manual and by Little (1995). The regime was designed to best simulate the steps that would be undertaken by an Engineer that would be following either of the two mix design methodologies presented in the Austroads (2019) guide and to allow for a fair comparison between these two methods. First, the materials used in the testing and their sources will be explored, followed by descriptions and sequences of the testing methodologies employed. This chapter will then conclude with an exploration of the risk factors identified before tests were performed and how these risks were managed during the testing process.

3.2 Materials

In the following section, the sources and important details of the materials used will be outlined. All materials used were sourced, handled and stored by trained DTMR personnel following internal procedures and the recommendations set out in the DTMR (2022*b*) material testing manual.

The soil material on which the tests were performed was sourced during the road rehabilitation works on the New England Highway (22A) south of Cooyar in 2022, following the **AS 1289.1.2.1** method. The soil material was collected from a new test pit located near the original test pit 11 on the left side of the road, at a chainage of 40.04 km. No logging of the ground conditions and stratigraphy was performed in the new test pit, as only material that appeared similar to the material tested during the initial investigation was collected. A total of 15 soil sample buckets, weighing 10 to 20 kg each, were collected from this new test pit. The soil material was then transported to the DTMR laboratory in Toowoomba, air dried in the sun to slow microbial activity (Woods, 2022), and divided into 9 kg subsamples for testing as shown in Figure 3.2.

Figure 3.1: Example of a soil being air-dried in metal trays. (R Kashanov 2023, personal photograph, 1 May)

Figure 3.2: Air-dried samples of the Cooyar black soil in 9 kg moisture-proof bags. (R Kashanov 2023, personal photograph, 1 May)

The lime for this study was obtained from Wagners in the form of a hydrated lime powder, which has been tested according to the **AS 4489.6.1** method to have an available lime index of 81.68% $Ca(OH)^2$. DTMR (2022*b*) recommends the use of hydrated lime for laboratory use, mentioning the safety concerns associated with the handling of quick lime due to its high corrosivity and reactivity (Greymont, n.d.). It is also more advantageous to use hydrated lime powder over quicklime or lime slurry as it does not require mixing with water before incorporation, resulting in a cleaner work area and a more streamlined process.

Furthermore, since hydrated lime is supplied as a dry powder, when lime is incorporated into the soil sample, the addition of moisture to the sample is marginal. That is, when hydrated lime powder is used, the calculations of how much water must be added to the soil do not need to account for the water that would have been added if slaked lime or lime slurry were used. The water used in the preparation and soak of the soil samples was potable water sourced from the Toowoomba Bulk Water Supply. As the impurities and pH of the potable water supply generally comply with public health regulations (TRC, 2022), the water at the point of delivery is not regularly tested by the DTMR laboratory.

3.3 Testing Regime

As part of the DTMR conditions of the research sponsorship, all of the black soil material was conditioned, divided into sub-samples, mixed with lime and tested by trained personnel in three NATA-accredited laboratories. Most of the tests were performed in the DTMR laboratory in Toowoomba, the organic content and sulphate tests were performed by the DTMR Bulwer Island laboratory and the ferrous iron tests were performed by ALS Environmental in Brisbane. As all testing methodologies used, except for **Fe-VOL05**, are documented with step-by-step instructions in the publicly available DTMR (2022*b*) materials testing manual and well-documented standard testing procedures (Standards Australia, 1997, 1998, 2008, 2009*a*,*b*,*c*,*d*, 2020), only the outlines of the testing methodologies used were reproduced as part of the Appendix D.1.

With the testing being conducted in commercial laboratories that have multiple ongoing projects, the sequence in which the testing is performed is very important because it ensures that the day-to-day activity in the laboratory is the least affected. This was done by dividing the tests into sets of related tests that could be performed in half-day blocks. The sequence chosen in which the tests were performed is shown in Figure 3.3.

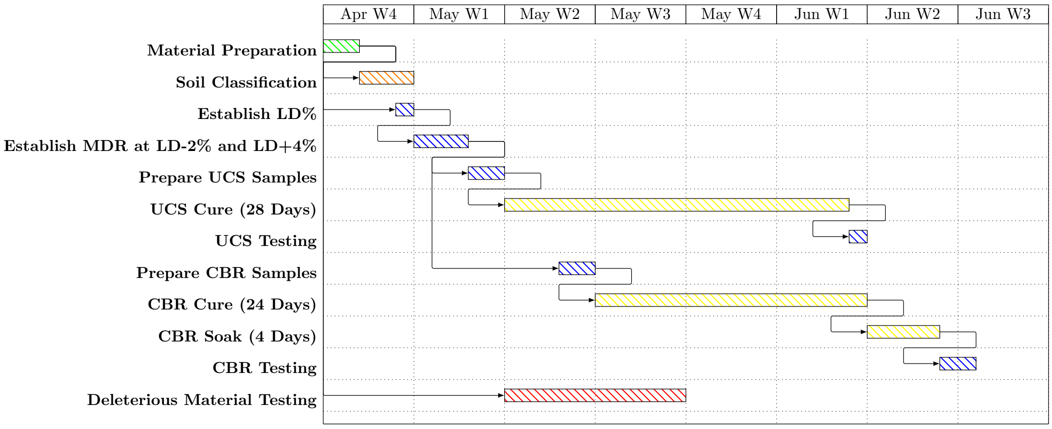


Figure 3.3: Detailed section of the Project Timeline showing the sequence in which the testing was undertaken.

The first lot of testing to be performed were tests used to classify the soil as it appears in situ using basic material characteristics and identify what behaviour is expected of the soil material as a subgrade if no stabilisation or modification with lime were to be performed. The characteristics of the soil material to be determined and testing methodologies used in the first lot of testing are as follows:

A1289.3.6.1 – Particle Size Distribution. Due to the cohesive nature of clay, for the classification of particles passing through sieve sizes below 4.75mm, wet wash sieves were used.

AS1289.3.1.1, 3.2.1, 3.3.1, 3.4.1 – Atterberg Limits and Linear Shrinkage.

Q142A – Moisture Density Relationship (MDR).

Q133 – Lime Demand (LD%).

Based on the results of the initial tests, 12 sets of samples were prepared and tested in the following order, using the soil material passing a 19mm sieve, which has been air-dried and split into manageable 9 kg bags. The optimal moisture content (OMC) values determined as part of the first step of the following list were used during the cylinder compaction for the UCS and CBR tests, where the samples mixed at the lime contents of LD-2% and LD-1% were compacted at an OMC of the LD-2% MDR curve and the samples mixed at the lime contents of LD%, LD+2% and LD+4% were compacted at an OMC of the LD+4% MDR curve.

Figure 3.4: Wet soil being spread inside the Casagrande cup to determine the liquid limit as part of Atterberg limits test. (R Kashanov 2023, personal photograph, 1 May)

Figure 3.5: Example of a clay soil after drying in a 250mm mould to determine the linear shrinkage as part of Atterberg limits test. (R Kashanov 2023, personal photograph, 1 May)

1. Twelve sample bags were individually mixed with lime and water so that there were two sets of samples with lime contents of LD-2% and LD+4% and each set had one sample with a moisture content of either 22%, 28%, 26%, 30% or 34%. These samples were then tested to determine the MDR and OMC of the soil with the two lime contents using the **Q142A** testing method.

2. Five sets of three cylinders were compacted with soil mixed at lime contents of LD-2%, LD-1%, LD%, LD+2% and LD+4% and allowed to cure for one day before being taken out of the mould and further cured for 27 days in preparation for UCS testing following the **Q115** testing method.

3. Five sets of three cylinders were compacted with soil at lime contents of LD-2%, LD-1%, LD%, LD+2% and LD+4% and allowed to cure for 24 days followed by a 4-day soak prior to CBR testing following the **Q113C** testing method.

Each time the soil was mixed with lime and water, the mixing was performed as per the **Q135A** testing method on a metal mixing tray depicted in Figure 3.6 over a two-day period, where half of the prescribed total weight of water and hydrated lime was added to the soil each day with an amelioration period of at least 12 hours between additions. The two-part addition of hydrated lime and water allows moisture to be evenly distributed throughout the soil, preventing areas with high and low moisture content and simulates the construction processes described by Berger et al. (2001) and DTMR (2022*e*) employed to minimise the impact of sulphates and maximise long-term strength development. All of the compaction was undertaken using **Q145A** and **Q251A** testing methods as nominated in other relative testing methods and all curing of the moulded specimens was carried out under controlled conditions following the **Q135B** method.

Figure 3.6: Water being added to a mixture of soil and hydrated lime. (R Kashanov 2023, personal photograph, 1 May)

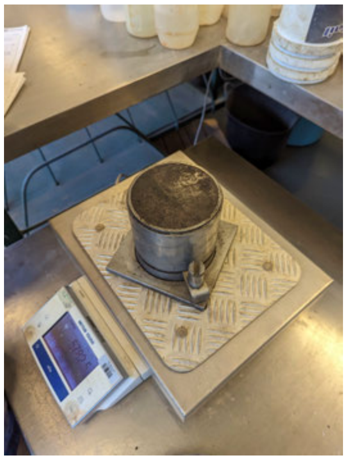


Figure 3.7: Soil compacted into a cylinder mould, being weight as part of MDR testing. (R Kashanov 2023, personal photograph, 1 May)

While UCS and CBR samples were curing, the subsamples for deleterious testing were packed and transported to the DTMR Bulwer Island and ALS Environmental laboratories, where the organic, sulphate and ferrous iron contents were determined using the **Q120B**, **AS1289.4.2.1** and **Fe-VOL05** testing methodologies respectively, as prescribed in the (DTMR, 2022*b*) material testing manual. The material used for these tests was taken from the same set of subsamples prepared for soil classification testing, to identify the deleterious material contents as they would be in situ before stabilisation with lime.

3.4 Omitted Testing

As shown in Figure 2.4, Austroads (2019) recommends that when following Method A, swell and erodibility testing is undertaken as the final step during the mix design process. However, since the test regime used in this study includes CBR testing using the Q113C test method, which includes the determination of swelling after 4 days of soaking, no additional swell testing was performed on soil mixed with lime. Regarding erodibility testing, Austroads (2019) recommends the use of the 2012f Roads and Maritime Test Method, which is not performed in DTMR laboratories. Therefore, being an optional test to perform erodibility testing was omitted from forming part of the testing regime for this study.

3.5 Risk Management

The identification of risks and development of strategies to manage these serve a critical role in any day-to-day activity of commercial, public and household settings. Lack of due diligence can not only result in lost production and injuries, but can cause serious emotional, financial, and ethical damages to everyone involved in the activity, both directly and indirectly. As part of the planning process for this study, two activities that require formal risk management strategies to be implemented were identified. The risk assessments for these activities were completed through the Riskware safety risk management system. Copies of these risk assessments can be found in the Appendix B.

3.5.1 Risk Assessment 2314: Fatigue Management

The first activity requiring a formal review is the daily travel between home and the laboratory where the testing was to be performed. Considering the commute to the laboratory being 1.5 hours and taking into account fatigue breaks, if laboratory visits were 8 hours to match the laboratory working hours, on days when tests were required to be performed, the door-to-door work time could have been as long as 12 hours. Safe Work Australia (2013) identifies that for daily work hours, including a commute of 10 or more hours, the risk of fatigue is greatly increased. Therefore, in order to reduce the risk associated with daily commute, the guidelines established by Safe Work Australia (2013) were followed by ensuring that all commutes were carried out in daylight hours and laboratory visit times were reduced so that door-to-door time, including breaks during commute and throughout the day, did not exceed 10 hours.

3.5.2 Risk Assessment 2316: Material Testing

The second activity to be reviewed was the execution of the testing. The material testing required for this investigation involved manual handling, the use of testing equipment, and the risk of exposure to lime and silica particles in the air. As all tests were to be performed in the laboratory operated by DTMR, the visitor safe work method statement used by the laboratory, which can be found in Appendix B.3 was reviewed and included in the preparation of the risk assessment. The comprehensive list of risks identified in the laboratory and how they were managed is included in Appendix B.1.

Chapter 4

Test Results

4.1 Introduction

In the following chapter, the results of soil classification, moisture density relationship and strength testing of Cooyar black soil, and observations noted during testing and material handling process are explored. The results presented in this chapter have been reproduced from the NATA-accredited laboratory test reports, which can be found in Appendices E.1 through E.4. The results have been grouped into four appendices, namely material test reports, maximum dry density reports, CBR reports, and UCS reports, and will be explored in this chapter in the same order.

4.2 Soil Calssification Testing

The summary of the quality of the material test reports, together with the limits of the material properties for lime stabilisation established by the industry authorities (Austroads, 2019, DTMR, 2022*e*, Little, 1995, NLA, 2004) is presented in Table 4.1. Of all properties tested, only one result falls outside the prescribed limits, the organic content of 10.5% is higher than the limit of 10%, meaning that if deleterious testing had been performed as a first step, lime stabilisation would have been ruled out as an appropriate treatment from the beginning. The following steps further summarise the test results by classifying the soil using the Unified Soil Classification System (USCS), as described in

Test Method	Property Tested	Result	Recommended Limit	Comment
AS 1289.3.6.1	Passing 2.36 mm Sieve	100 %	N/A	–
AS 1289.3.6.1	Passing 425 µm Sieve	96 %	N/A	–
AS 1289.3.6.1	Passing 75 µm Sieve	90 %	> 25	OK
AS 1289.3.1.1	Liquid Limit	80 %	N/A	–
AS 1289.3.1.2	Plastic Limit	26 %	N/A	–
AS 1289.3.3.1	Plasticity Index	54	> 10	OK
AS 1289.3.4.1	Linear Shrinkage	22 %	N/A	–
–	Weighted Plasticity Index	5176	> 2200	OK
Q120B	Organic Content	10.5 %	$\leq 1/10$	Over Limit
AS 1289.4.2.1	Sulfate Content	0.01 %	≤ 0.3	OK
Fe-VOL05	FeO Content	0.66 %	≤ 2.0	OK

Table 4.1: Summary of the Quality of Material test results, reproduced from test reports in Appendix E.1.

AS 1726 (Standards Australia, 2017) for use in geotechnical field investigations throughout Australia. The use of USCS minimises ambiguity in the description of soil and rock materials during geotechnical investigations and in the interpretation of the investigation results by third parties who do not have physical access to the material. As the laboratory did not record notes during the sampling of the material used in the testing, information such as in situ consistency or moisture condition will not be included in the following description of the soil.

AS 1726 Clause 6.1.4.5: Primary Component

Based on the Particle Size Distribution data, the highest proportion (90%) of the soil passes through the 75 µm sieve, therefore, the soil is fine-grained. As the soil has 10.5% organic content, it is organic and will have an "O" prefix. When plotting the results of the Liquid Limit and Plasticity Index tests on Figure 4.1 used for the classification of silts and clays, the intersection of these values is above the A line in the high plastic clay zone. Therefore, the group symbol for the tested Cooyar soil material is OH.

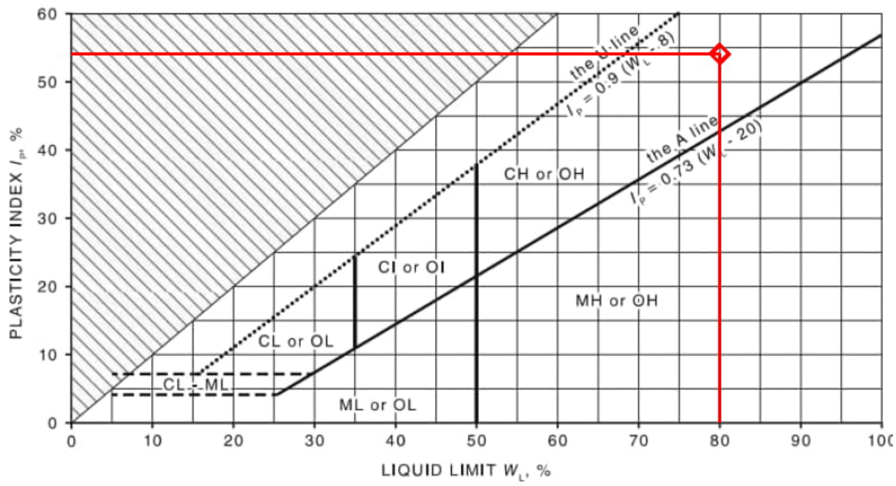


Figure 4.1: Modified Cassagrande chart for classifying silts and clays according to their behaviour, reproduced from Standards Australia (2017, pg. 29).

AS 1726 Clause 6.1.4.6: Acessory Component

The remaining 10% of material that is retained on the 75 µm sieve passes through the 2.36 mm sieve in full, which means that the accessory component is sand. As the sand content is less than 15%, the sand is a minor component with the designation of trace. And since the 600 µm and 210 µm sieve sizes used to describe the coarseness of the sand were not used during the PSD test, since 6% of the sand has passed through the 425 µm sieve compared to 4% that were retained, the sand can only be classified as medium to fine-grained.

AS 1726 Clause 6.1.5: Colour

As seen in Figures 1.3, 3.4 and 3.6, the colour of the soil is dark brown in dry conditions and black when wetted with water. As the standard states that the colour must be described in moist conditions, select black as the primary colour of the soil.

AS 1726 Clause 6.1.9: Soil Origin

The soil origin or depositional environment can be determined by interpreting the geological survey maps. Appendix D.4 is the detailed surface geology map around the site where the soil was sampled, printed from the Queensland Globe (DR, 2023) GIS web services. From this map, the location of the site at Ch 40.04 is in close proximity to three major geological formations: Main Range Volcanics (Tm), Marburg Subgroup (Jbm) and Quaternary Alluvium (Qa). As the black clay encountered at the site does not appear to be residual soil from the sandstone and siltstone of the Jbm formation, and since the plastic properties of this clay fall within the typical ranges of the residual black soils in the basalt layers described to occur in the Darling Downs region (Vanderstaay, 2020), the clay sampled is assumed to be residual in nature and part of the Tm geological formation.

Following the steps from AS 1726 (Standards Australia, 2017) as described above, the final classification of the soil sampled at the subgrade level on the New England Highway (22A) CH 40.04 km is OH Organic CLAY trace sand, black, high plasticity; sand 10%, fine to medium grained; residual Bassalt [Tm].

The results of the individual pH test results produced during the lime demand test are reproduced in Figure 4.2. As previously discussed, the purpose of the lime demand test is to determine a target lime content at which the pH of the solution is brought above the target pH of 12.4. As can be seen in the results, the soil prior to lime addition is slightly acidic with a pH of 6.5, which means that there is a minimal presence of acids, indicating that the soil has undergone minimal weathering (Little, 1995) as also indicated by low FeO contents, but the presence of humic acid in the soil cannot be ruled out. After the addition of only 1% lime, the pH of the soil increases rapidly to 9.92. Due to the nature of the pH scale and the fact that the hydrated lime used was tested to have an average pH of 12.69, an additional increase in pH as a result of the increase in the lime content follows a logarithmic pattern, plateauing as the pH of 12.4 is reached at the lime content of 4%.

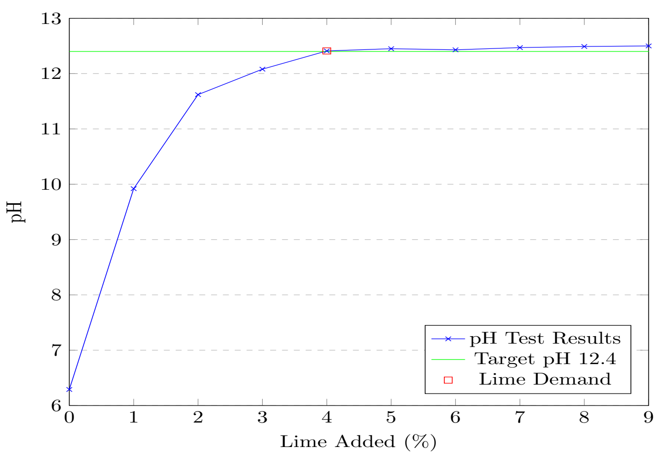


Figure 4.2: Lime Demand Test working results of Cooyar Black Soil.

4.3 Moisture Density Relationships

The moisture density relationship reports for the black clay with weighted lime contents of 0%, 3% and 6% can be found in the Appdendix E.2. The Maximum Dry Density at the 100% compaction and the Optimum Moisture Contents required to achieve 100% compaction were reproduced in Table 4.2. Additionally, the table includes target density at the 97% compaction based on these MDD results, these values are used as a quality control measure to assess whether the cylinders used in strength testing were compacted within the limits specified in the test methods.

Lime Content (%)	Standard MDD (t/m^3)	Target Density (t/m^3)	OMC (%)
0	1.43	1.39	27.5
3	1.40	1.36	28.0
6	1.38	1.34	30.0

Table 4.2: Summary of Moisture Density Relationship test results.

These results show that despite the high organic content, the addition of lime to the clay soil still alters the physical properties of the clay, and confirm the observations recorded in the literature (Austroads, 2019, Little, 1995) that the addition of lime decreases the MDD and increases the OMC of the soil. One of the notable observations made during the testing process was that when handling the soil after the MDR testing, the clay material that had been mixed with lime felt less cohesive and more friable, similar to clayey sand. The changes observed are consistent with the mechanical and textural improvements associated with the lime modification process.

4.4 Strength Testing

The sample preparation for the UCS and CBR testing was staggered with a week in between. Since the UCS cylinders were cured for 28 days and the CBR cylinders were cured for 24 days and soaked for 4 days, the testing of these samples was also performed a week apart. In Figure 4.3 the UCS cylinders after the testing was performed are depicted. The cylinders are lined from left to right by the lime content, starting at 2% on the left through to 8% on the right.

An observation was made when the UCS cylinders were removed from the environmental chamber, as can be seen in Figure 4.3 cylinders with lower lime content had experienced mold growth. As expected, with an increase in lime content from 2% to 3% and 4% the extent of molding decreased from full coverage at 2% to minor spotting at 4% due to increased alkalinity. The mold growth was observed only on the surface of the cylinders and did not appear to extend deep into the soil. It is not uncommon to see mold growth in an environmental chamber, as the same warm and humid environment that promotes curing is also an environment where bioactivity occurs, provided there are appropriate levels of secondary environmental factors such as oxygen and pH (Vereecken and Roels, 2012).

Figure 4.3: UCS cylinders following UCS testing, lined up with cylinders containing 2% lime on the left through to cylinders containing 8% lime on the right. (B Waters 2023, personal photograph, 2 June)

However, since molds are not expected to thrive in environments with a pH greater than 9 (Tournas et al., 2023), and the pH results during the lime demand test show that with the lime content of 2% and 3% the soil pH is expected to be well above 11.5, it is concerning that mold growth were observed. These results indicate that during the 28-day curing period, a cation exchange reaction or dissolution of acids such as humic acid has occurred, using free hydroxyl groups and decreasing the pH of the soil enough to cause the growth of mold. Considering that this decrease in pH was not observed during the lime demand test, the reaction in question occurs gradually over time, which is similar to observations made by Harris et al. (2009) where the presence of humic acid did not have an effect on the pH of the soil during the lime demand test. As the pozzolanic reaction is not believed to be sustained at pH levels below 12.4 (Ouhadi et al., 2014, Little, 1995), and mold growth occurred in samples with an initial pH below 12.4, the pozzolanic reaction can be ruled out as the cause of the decrease in pH. Similarly, since the sulphate content is minor, the growth of ettringite is not expected to have occurred.

Meanwhile, since the CBR cylinders are confined in cylinder moulds and covered with an absorbent paper on the top for the entire curing and soaking periods, there was restricted access to oxygen for mold growth to occur. Therefore, no mold was observed on the surface of the material when the cylinders were draining to test or when the soil was extracted from the moulds after the testing was complete.

The CBR and UCS test results have been reproduced in Table 4.3 from the NATA accredited reports that can be found in the Appendices E.3 and E.4. The table presents relationships between the lime content and the target density, swell, CBR and UCS of the 28-day cured Cooyar black soil. As previously indicated during the MDR tests, the increase in lime content causes a reduction in the achievable density at maximum and target compaction levels; therefore, different target density values were used during the compaction of the CBR and UCS cylinders.

The CBR Swell results indicate a successful reduction in soil moisture reactivity with the addition of lime, with a complete negation of the swell observed at the 3% lime content. Similarly, the significant increase in CBR with the increase in lime content successfully indicates the immediate textural changes reported as a result of the cation exchange between calcium (Ca^{2+}) ions provided by the addition of lime and cations to the surface of clay particles (NLA, 2001, TRC, 2022). Therefore, based on the results in Table 4.3, the modification of the Cooyar black soil with lime can be achieved to improve the plasticity, workability, and reactivity of the soil at a lime content of 3% or greater.

Meanwhile, the UCS results indicate that with an increase in the lime fraction in Cooyar black soil there is no significant strength improvement that is typically observed in stabilised soil. When plotted on a graph in Figure 4.4 it is clearly apparent that the results form a relatively flat line unlike the inverse parabolic or positively inclined linear shape usually seen during UCS testing of successfully lime-stabilised soil, nor do the results at any lime content be close to the target range of 1.0 to 2.0 MPa prescribed by DTMR (2021*c*). As the target UCS range was not reached, at the tested lime contents the Cooyar black soil cannot be classified as lightly bound at 28 days of curing. Since no growth pattern can be seen in the UCS results, there is no apparent reason to perform additional testing with higher lime content to determine whether the target is ever reached.

Lime Content (%)	Target Density (t/m^3 at 97% Compaction)	CBR Swell (%)	Soaked CBR (%)	Average UCS (MPa)
0	1.39	3.0	4.5	N/A
2	1.36	0.5	20	0.3
3	1.36	0.0	35	0.4
4	1.34	0.0	40	0.3
6	1.34	0.0	50	0.4
8	1.34	0.0	50	0.4

Table 4.3: Summary of CBR and UCS test results.

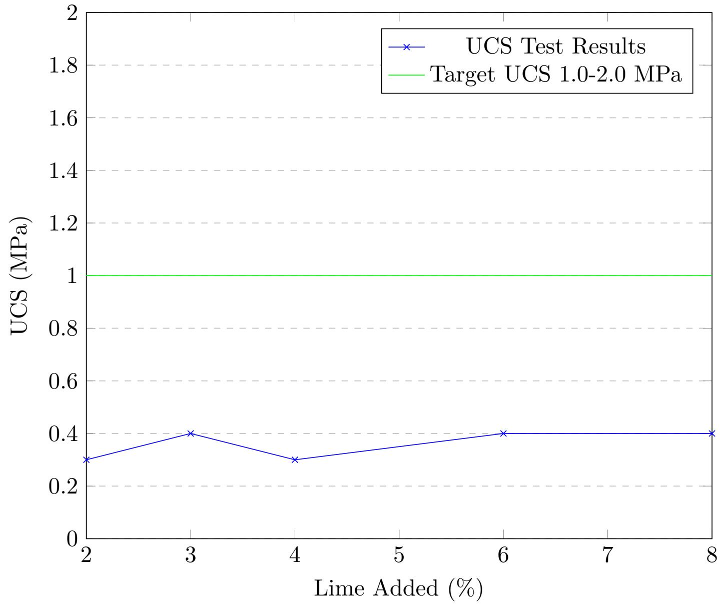


Figure 4.4: 28 day cured UCS test results of Cooyar Black Soil mixed with lime.

4.5 Chapter Summary

The material test results prior to mixing with lime have been analysed and used to classify the soil sampled at the subgrade level on the New England Highway (22A) CH 40.04 km is OH Organic CLAY trace sand, black, high plasticity; sand 10%, fine to medium; alluvial [Qa]. With the organic content results being outside the limits prescribed by DTMR (2021*c*) and as evidenced by the subpar UCS results after the addition of lime to the soil in the original work and this study, it was determined that if deleterious testing was included in the test suit carried out during the preliminary investigations in 2017, the use of lime stabilisation as the treatment method would have been ruled out in the first place. However, it is apparent from the MDR and CBR results that the improvements in workability and reactivity associated with soil modification through the addition of lime are not affected by the process responsible for halting the pozzolanic reaction.

Chapter 5

Discussion

5.1 Chapter Overview

In this chapter, the variance between the CBR and UCS results and the impact of test results on pavement design and performance are discussed. At the start of this chapter, two pavement designs are prepared using the CBR and UCS results, respectively. The theoretical performance of these two configurations is then analysed under variable conditions using the CIRCLY software. Followed by a discussion about variance in the CBR and UCS results, and an assessment of the processes by which deleterious materials have been described in Section 2.6 to affect the lime stabilisation process, to determine the most plausible causes of the low UCS results observed in the Cooyar black soil.

5.2 Pavement Design based on CBR Results

In the following section, a pavement design utilising a lime-stabilised subgrade along a design lane described in Appendix D.5 will be performed. The pavement design method used in this section is the Austroads (2017, chap. 8.3) empirical design of granular pavements with thin bituminous surfacing. This is a commonly used iterative pavement design method, which can be characterised by its simplicity, as the only data inputs required for this method are the design traffic expressed in ESA, the thicknesses of the pavement layers and the CBR values of the subgrade and pavement aggregates.

As this design will be carried out for comparison purposes between Austroads (2019) Methods A and B, only the steps set out in Method B will initially be followed. Furthermore, since Method B recommends using the difference between the CBR results before and after lime treatment as an indication of the effectiveness of lime stabilisation, initially no consideration will be given to the UCS results, and it will be assumed that a successful pozzolanic reaction has occurred.

5.2.1 Definition of Pavement Layers

Consider a simple pavement consisting of a base and sub-base layers of thickness t_B and t_{SB} respectively, overlaying a lime-stabilised subgrade with a thickness of t_{SS}, and topped with a bituminous surfacing with a thickness of less than 40mm (Austroads, 2017, chap. 8.3) as shown in Figure 5.1. As the structural performance of the pavement layers is the main concern of this analysis, no design of the bituminous surfacing will be carried out. The pavement layers and the CBR values assigned to those layers to be used in the pavement design are described in the following list:

Bitumenous Surfacing It is assumed that a sprayed bituminous seal surface of no more than 40mm thick will be used as a bituminous surfacing layer based on restrictions established in the Austroads (2017, chap. 8.3) empirical design method.

Base The base pavement layer will consist of a Type 2.1 aggregate with an assumed CBR of 80% (DTMR, 2022*d*, tab. 7.2.5).

Sub-Base The sub-base pavement layer will consist of a Type 2.3 aggregate with an assumed CBR of 45% (DTMR, 2022*d*, tab. 7.2.5).

Lime-Stabilised Sub-Grade The Austroads (2019) Method B recommends that the lime content used in for construction is LD+0.5% or LD+1%. However, since the UCS and CBR testing was undertaken at LD-2%, LD-1%, LD%, LD+2%, LD+4% with the results available in Appendix E.3, it will be assumed that the stabilised lime subgrade has been stabilised at 6% lime content, representing LD + 2%, and will be considered to have a CBR of 50% based on the laboratory results.

Sub-Grade The remainder of the subgrade will be taken as having a CBR of 4.5% based on the laboratory test results in Appendix E.3.

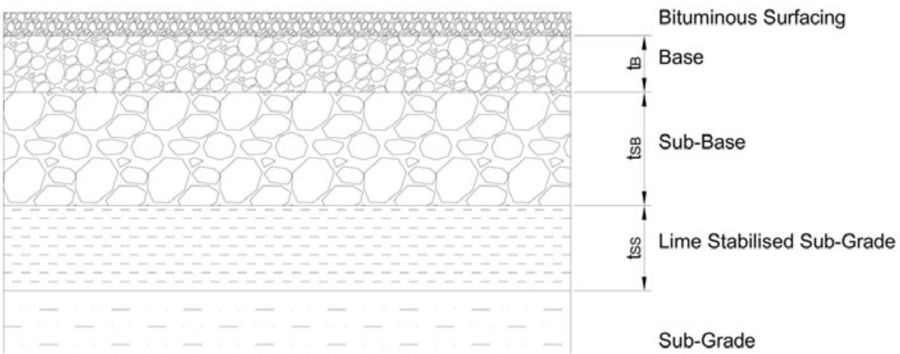


Figure 5.1: Definition of Pavement Layers.

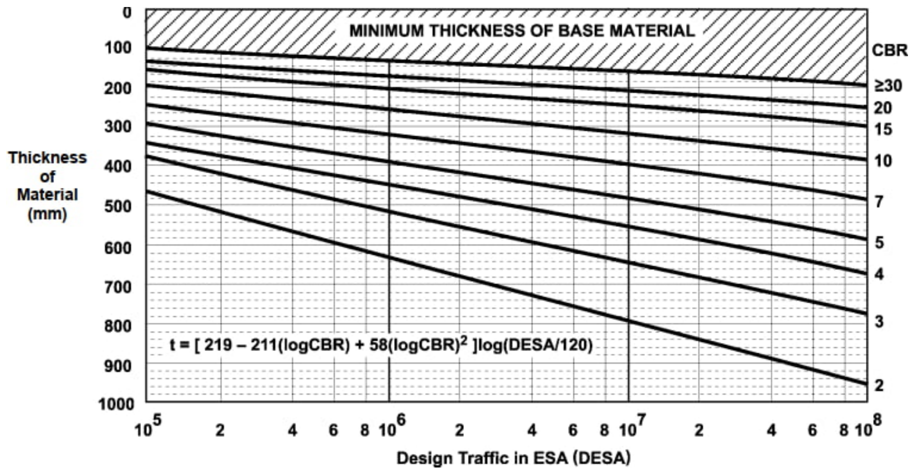


Figure 5.2: Design chart for granular pavements with thin bituminous surfacing, reproduced from Austroads (2017, p.122).

5.2.2 Trial Pavement Configuration

Following the development of the design traffic undertaken in Appendix D.5, the first step in the Austroads (2017, chap. 8.3.2) empirical pavement design process is to establish a trial pavement configuration and check against the design chart in Figure 5.2 whether there is sufficient cover for the individual pavement and subgrade layers of select CBR.

Consider a pavement over a subgrade that has been stabilised to a depth of $t_{SS} = 200$ mm. The design CBR of the lime stabilised subgrade is then the lesser of 50% determined from a CBR test, the support provided by the underlying subgrade as determined by the Equation 5.1, or 15% (Austroads, 2017, chap.8.3.2).

$$\begin{aligned} CBR_{Sup} &= CBR_{SB} * 2^{\frac{t_{SS}}{150}} \\ CBR_{Sup} &= 4.5 * 2^{200/150} = 11\% \end{aligned} \tag{5.1}$$

Therefore, the design CBR is: $CBR_{Design} = MIN(50, 11, 15) = \mathbf{11}\%$. Now that the design CBR of the lime-stabilised subgrade is known, the minimum cover required and the thickness of the overlaying pavement layers can be calculated using the design chart in Figure 5.1. Under a design traffic of $5.4 * 10^6$ ESA the minimum cover over the lime-stabilised subgrade is $(219 - 211 * (log(11)) + 58 * (log(11))^2) * log(\frac{5.4*10^6}{120}) \approx \mathbf{290}$ mm and the minimum base thickness over a sub-base with a CBR $> 30\%$ is $t_B = (219 - 211 * (log(30)) + 58 * (log(30))^2) * log(\frac{5.4*10^6}{120}) \approx \mathbf{160}$ mm. Therefore, the thickness of the sub-base is $t_{SB} = 290 - 160 = \mathbf{130}$ mm.

To check whether the selected pavement configuration provides satisfactory protection of the subgrade, first, the minimum cover over the subgrade with a CBR of 4.5% must be determined. From the design chart in Figure 5.1, the minimum cover is $(219 - 211 * (log(4.5)) + 58 * (log(4.5))^2) * log(\frac{5.4*10^6}{120}) \approx \mathbf{500}$ mm. The total thickness of the pavement, including lime-stabilised subgrade is $200 + 160 + 130 = \mathbf{490}$ mm, which is 10 mm less than the minimum cover required, meaning that the current pavement configuration is not satisfactory.

5.2.3 Refined pavement Configuration

Increasing the thickness of the lime stabilised subgrade and/or addition of thickness to the subbase to provide minimal cover to a subgrade leads to the three satisfactory configurations as shown in Table 5.1. Without undertaking estimates of construction costs related to each individual configuration, neither of the pavement configurations presented can be considered the most optimal.

Configuration 1 requires an additional thickness of the sub-base in order to satisfy minimal cover over the subgrade, leading to a pavement thickness being 20mm greater than minimal cover over the lime-stabilised subgrade layer. For configuration 3, in order to satisfy minimal cover over the lime-stabilised subgrade layer, the total thickness of pavement is 40mm greater than the minimal cover required over the subgrade. Taking into account the highly expansive nature of the subgrade based on the material test results and the DTMR (2021*c*, chap. 6.1) preference for the depth of lime-stabilisation design to be 300mm, pavement configuration 1.3 as shown in Figure 5.3 will be chosen as a conservative option for the purposes of analysis that will be further discussed in this chapter.

Configuration 1.1	t (mm)	CBR (%)	Mininal Cover (%)
Base	160	80	-
Sub-Base	140	45	160
Lime-Stabilised Sub-Grade	200	11	290
Total Pavement	500		
Sub-Grade	-	4.5	500

Configuration 1.2	t (mm)	CBR (%)	Mininal Cover (%)
Base	160	80	-
Sub-Base	90	45	160
Lime-Stabilised Sub-Grade	250	14	250
Total Pavement	500		
Sub-Grade	-	4.5	500

Configuration 1.3	t (mm)	CBR (%)	Mininal Cover (%)
Base	160	80	-
Sub-Base	90	45	160
Lime-Stabilised Sub-Grade	300	15	240
Total Pavement	540		
Sub-Grade	-	4.5	500

Table 5.1: Summary of the pavement design calculations for three satisfactory pavement configurations using lime stabilised subgrade.

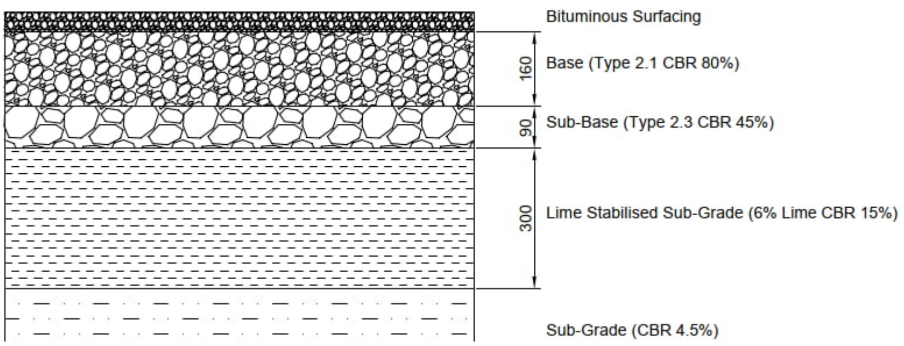


Figure 5.3: Design pavement configuration incorporating a lime-stabilised sub-grade.

5.3 Pavement Design based on UCS Results

The pavement design undertaken in the following section will follow similar steps to those in Section 5.2, however now the recommendations set out in the Austroads (2019) Method A and the guidelines set out by DTMR (2021*c*) will be followed and the UCS results will be considered.

5.3.1 Definition of Pavement Layers

For this design, a similar pavement configuration will be used, consisting of a base and sub-base layers of thickness t_B and t_{SB} respectively, topped with a bituminous surfacing with a thickness of less than 40mm (Austroads, 2017, chap. 8.3). However, since UCS results as shown in Appendix E.4 did not meet the minimum strength criteria of 1.0 MPa as established by DTMR (2021*c*), a DTMR (2022*c*) subgrade treatment type B consisting of a select fill layer between the sub-base and subgrade, of a thickness of t_{SS} will be used in lieu of the lime-stabilised subgrade layer to achieve minimal cover requirements over the subgrade. The revised list of pavement layers and assigned CBR values for this design are as follows.

Bitumenous Surfacing It is assumed that a sprayed bituminous seal surface of no more than 40mm thick will be used as a bitumenous surfacing layer based on restrictions established in the Austroads (2017, chap. 8.3) empirical design method.

Base The base pavement layer will consist of a Type 2.1 aggregate with an assumed CBR of 80% (DTMR, 2022*d*, tab. 7.2.5).

Sub-Base The sub-base pavement layer will consist of a Type 2.3 aggregate with an assumed CBR of 45% (DTMR, 2022*d*, tab. 7.2.5).

Select Fill Sub-Grade The select fill material used will be assumed to be a cohesive Class A1 or B earthfill material (DTMR, 2022*c*, chap.14.2.2) with a minimum CBR of 10% (DTMR, 2022*c*, chap.18.2.1).

Sub-Grade The remained of the subgrade will be taken as having a CBR of 4.5% based on the test results in Appendix E.3.

Configuration 2.1	t (mm)	CBR (%)	Mininal Cover (%)
Base	160	80	-
Sub-Base	190	45	160
Select Fill Sub-Grade	150	9	330
Total Pavement	500		
Sub-Grade	-	4.5	500

Configuration 2.2	t (mm)	CBR (%)	Mininal Cover (%)
Base	160	80	-
Sub-Base	150	45	160
Select Fill Sub-Grade	200	10	310
Total Pavement	510		
Sub-Grade	-	4.5	500

Table 5.2: Summary of the pavement design calculations for two satisfactory pavement configurations using select fill subgrade.

5.3.2 Pavement Configuration

Again following the Austroads (2017, chap. 8.3) empirical design method, two pavement configurations as shown in Table 5.2 were calculated to provide sufficient cover for the expansive subgrade against a design traffic of $5.4 * 10^6$ ESA. Taking a closer look at configuration one, to provide a minimal cover for the select fill subgrade layer of 360 mm, the thickness of the sub-base only had to be 170 mm, however, that resulted in a total pavement thickness of 480 mm, 20 mm less than that required to provide a protection for the expansive subgrade. Therefore, in configuration 2.1, the sub-base thickness had to be increased to 190mm. In comparison, pavement configuration 2.2, as shown in Figure 5.4, allows for a more economical design by increasing the select fill subgrade by 50mm, improving the CBR of that layer by 1% and allowing the sub-base thickness to be reduced to a minimal required value, and producing a more conservative result with the total pavement thickness being 10mm over the minimal required cover over the subgrade.

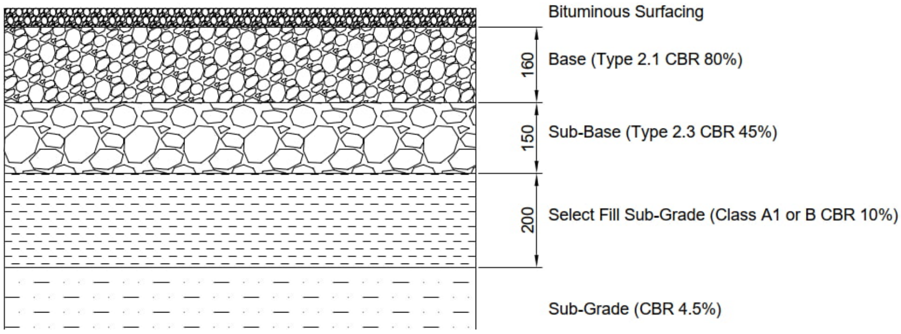


Figure 5.4: Design Pavement Configuration Incorporating a Select Fill Sub-Grade.

5.4 Pavement Performance Analysis

In this section, the performance of the pavement configurations prepared as part of the Sections 5.2 and 5.3 will be analysed using the CIRCLY 7.0 mechanistic-empirical pavement design and analysis software. The analysis will be performed considering presumptive values published by Austroads (2017) and DTMR (2021*d*) and all test results obtained as part of the testing regime and using the same design traffic developed in Appendix D.5, expressed as a number of cumulative heavy axle groups rounded up to $7.3 * 10^6$ HVAG and a traffic load distribution presented in Appendix D.6.

5.4.1 Theoretical Performance of the Two Configurations Excluding UCS Results

The pavement configurations 1.3 and 2.2 presented in Figures 5.3 and 5.4 respectively were modelled in the CIRCLY 7.0 software. DTMR (2021*d*, tab.6.2.3(a)) recommends a presumptive vertical modulus for the base layer consisting of a Type 2.1 aggregate be 350 MPa. Similarly, Austroads (2017, tab.6.3) recommends that the presumptive vertical modulus for sub-base material should be 250 MPa. The vertical modulus of the select fill, lime stabilised, and in situ subgrades were chosen to be 100, 150 and 45 MPa, respectively, based on the $E_v = 10CBR$ correlation used by Austroads (2017, chap. 8.2.2). All of the pavement layers apart from the in situ subgrade were sub-layered using the sub-layering built into CIRCLY 7.0 software by using the Austroads material library. The inputs and outputs of the performance analysis with a project reliability level of 90% (DTMR, 2021*d*) can be found in the Appendices F.1 and F.2 and have been summarised in Table 5.3.

Pavement with Select Fill Sub-Grade, Figure 5.4						
NDT=$7.3 * 10^6$, ESA/HVAG=0.743 (Project Specific CTLD), Project Reliability=90%						
t (mm)	Material ID	Ev (MPa)	P.Ratio	Perf. Const.	Perf. Exp.	CDF
160	Gran_350	350	0.35	N/A	N/A	N/A
150	Gran_250	250	0.35	N/A	N/A	N/A
200	subsltCB10	100	0.45	0.009150	7	$5.947 * 10^{-1}$
Semi Inf.	Sub_CBR4.5	45	0.45	0.009150	7	$8.044 * 10^{-1}$
Pavement with Lime-Stabilised Sub-Grade, Figure 5.3						
NDT=$7.3 * 10^6$, ESA/HVAG=0.743 (Project Specific CTLD), Project Reliability=90%						
t (mm)	Material ID	Ev (MPa)	P.Ratio	Perf. Const.	Perf. Exp.	CDF
160	Gran_350	350	0.35	N/A	N/A	N/A
90	Gran_250	250	0.35	N/A	N/A	N/A
300	sublimeCB15	150	0.45	0.009150	7	$4.152 * 10^{-1}$
Semi Inf.	Sub_CBR4.5	45	0.45	0.009150	7	$3.392 * 10^{-1}$

Table 5.3: Summary of the inputs and outputs from CIRCLY 7.0 software for analysis of the pavement configurations using select fill and lime stabilised subgrades.

From the results of the pavement analysis in Table 5.3 it can be seen that after application of the design traffic of $7.3 * 10^6$ HVAG, both pavement configurations prepared using the Austroads (2017, chap. 8.3) empirical design methods had CDF values less than 1. This means that these configurations are conservative and it can be expected that they will not fail within the design life of 25 years (Pavement Science, 2015). It can be estimated with 90% confidence that at 25 years, both pavement configurations have $\left(1 - 8.044 * 10^{-1}\right) * 100 = \mathbf{19.56}\%$ and $\left(1 - 4.152 * 10^{-1}\right) * 100 = \mathbf{58.48}\%$ of cumulative damage capacity remaining, respectively.

5.4.2 Theoretical Performance of the Lime-Stabilised Sub-Grade Acknowledging UCS Testing

Based on the recommendations brought up by the Steering Committee (Evans et al., 1998), Little (1995) and DTMR (2021*c*), when analysing the performance of the lime stabilised subgrade it is important to look at the 28-day cured UCS results as a measure of the success of the pozzolanic reaction. As already established, the UCS results are below 1 MPA, which means that the effect of mould on the laboratory CBR results (Black, 1961) discussed in Section 2.8 must be considered.

Both of the Austroads (2017, chap.8.3.2) pavement design methods already partially account for elevated CBR values by limiting the design CBR of the select fill and the lime stabilised subgrades to be no more than 15%. However, as shown in Figure 5.5, this appears to be due to the visible disagreement in the high range of CBR values between the AASHTO/AAI correlation method added by Austroads (2017, chap.8.3.2) and the models used by other agencies for the correlation between CBR and Resilient Modulus. Considering the CBR of the subgrade prior to stabilisation is 4.5%, if there is no strength improvement from a lack of a successful pozzolanic reaction, a design CBR of 15% corresponding to a three times increase in CBR is implausible.

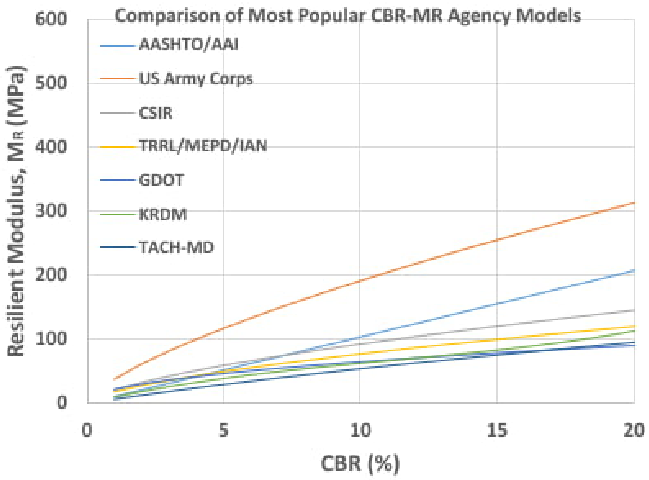


Figure 5.5: Comparison of popular Agency adopted models correlating CBR and resilient modulus, M_R @ low stiffness Sub-Base and low $\sim$ high stiffness Sub-Grade levels, reproduced from Mukabi (2015, p.3).

In the absence of triaxial testing to directly measure the resilient modulus as an estimate of the modulus of elasticity (Pavement Interactive, 2023*b*), and to avoid using the CBR values of unmodified soil, an attempt can be made to estimate the modulus of elasticity using the 28-day cured UCS as the only other measure of the material strength included in the testing regime. This is done under the assumption that the reported (NLA, 2001, AustStab, 2008, Little, 1995) mechanical and shear strength improvement associated with lime modification of the soil will have a positive effect on the elastic modulus of the soil even when no strength gain associated with a pozzolanic reaction is observed.

Hossain and Kim (2014) have proposed two models for the correlation between the UCS results ($\mathrm{Q_u}$ (psi)) of samples prepared using a Proctor hammer and a resilient modulus ($\mathrm{M_r}$ (psi)). One of these models builds on the previous work undertaken by Thompson and Robnett (1979) by including the plasticity index and the percentage fines as variables. However, as plasticity testing and classification of the material after lime incorporation were not carried out, the less accurate model as shown in Equation 5.2 will be used to estimate the resilient modulus of the clay material that has been mixed with 6% lime and has been tested to have a UCS of 0.4 MPa corresponding to the LD + 2% data used in the pavement design process.

$$\begin{aligned} M_r \text{ psi} &= 4283 + 143 * Q_u \text{ psi}; r^2 = 0.73 \\ M_r \text{ MPa} &= \frac{4283 + 143 * 145.038 * Q_u \text{ MPa}}{145.038} \\ M_r &= \frac{4283 + 143 * 145.038 * 0.4}{145.038} = 86.73 \text{ MPa} \end{aligned} \tag{5.2}$$

Pavement with Lime-Stabilised Sub-Grade, Figure 5.3

NDT=$7.3 * 10^6$, ESA/HVAG=0.743 (Project Specific CTLD), Project Reliability=90%

t (mm)	Material ID	Ev (MPa)	P.Ratio	Perf. Const.	Perf. Exp.	CDF
160	Gran_350	350	0.35	N/A	N/A	N/A
90	Gran_250	250	0.35	N/A	N/A	N/A
300	sublimeE85	85	0.45	0.009150	7	$1.4 * 10^1$
Semi Inf.	Sub_CBR4.5	45	0.45	0.009150	7	$6.896 * 10^{-1}$

Table 5.4: Summary of the inputs and outputs from CIRCLY 7.0 software for analysis of the pavement configuration using lime stabilised subgrade, accounting for low UCS results.

The pavement configuration 1.3 in Figure 5.3 with a lime stabilised subgrade has been again analysed in CIRCLY 7.0, now with a revised vertical modulus for the lime stabilised layer taken as $E_v = 85$ MPa, rounding down from the Equation 5.2. The results of the analysis are presented in Table 5.4. Looking at the results, the CDF of the lime stabilised layer is more than 1, which means that with 90% at 25 years from opening (Pavement Science, 2015), the cumulative damage to the pavement is above the capacity and that the pavement is predicted to fail before all design traffic loads are applied (Pavement Science, 2015). The time from the opening of the pavement section to when a failure is expected to occur can be estimated by calculating the number of heavy vehicles required that apply the same amount of cumulative damage as the capacity of the stabilised pavement layer and then readjusting Equation D.3 to calculate the design life.

$$N_{dt} = \frac{n_{dt}}{CDF} \tag{5.3}$$

$$N_{dt} = \frac{7.3 * 10^6}{14} \approx 521,429 \text{ HVAG}$$

$$NHV = \frac{N_{dt}}{N_{HVAG}} \tag{5.4}$$

$$NHV = \frac{521,429}{2.49} \approx 209,409 \text{ HV}$$

$$P = \frac{\log(\dfrac{0.01i * NHV}{365 * N_i} + 1)}{\log(1 + 0.01i)} \tag{5.5}$$

$$P = \frac{\log(\dfrac{0.01 * 2 * 209,409}{365 * 249} + 1)}{\log(1 + 0.01 * 2)} = 2.275$$

$$P \approx 2 \text{ years } 3 \text{ month}$$

Therefore, the analysis of the pavement configuration 1.3 in Figure 5.3 with a lime stabilsied subgrade presented in the CIRCLY 7.0 analysis data in Table 5.4 and Equation 5.5 has shown that; when considering the UCS results below 1.0 and the mould effect on the laboratory CBR results (Black, 1961), if the vertical modulus (E_v) of the lime stabilised layer is estimated by correlating the UCS results with a resilient modulus (M_r) there is 90% confidence that the stabilised pavement layer will fail within the first 2 years and 3 months of the design life of the pavement. This is a significant decrease in design life from the 25 years for which the original pavement design was prepared using the laboratory CBR results.

5.5 Variance in CBR and UCS Results

As summarized in Section 2.5 both CBR and UCS of clay soils are expected to increase following an increase in the fraction of lime content above the value determined as part of the lime demand test. However, as discussed in Section 2.7 the laboratory CBR and UCS tests do not measure the same strength properties of the material tested, as the CBR test measures the force required to penetrate a confined sample with a plunger to a specific depth inserted at a uniform rate, while the UCS test measures the amount of compressive force required to apply to an unconfined cylinder before a plastic collapse is observed.

When applied to clay soil mixed with lime, the laboratory CBR test measures how well the lime improves the workability of the soil compared to a well-graded quality pavement aggregate, whereas the UCS tests mainly determine how effective the pozzolanic reaction was at forming the C-S-H and C-A-H gels and binding the clay particles together. And as there is no consensus in the literature on whether a reliable correlation method between CBR and UCS (Baig, 1962, Eme et al., 2016, Saputra and Putra, 2020, O'Flaherty et al., 1961) due to these tests that measure unrelated properties of a material, it should be no surprise that if one of the processes that is expected to occur during lime stabilisation is not induced, one of the improvements in the test results of one measure will be affected without impacting the other measure.

Therefore, as discussed in Section 4.4 the incorporation of lime into Cooyar black soil modifies the workability of the soil as indicated by improvements in the plasticity, swelling and CBR of the soil. However, due to the presence of the deleterious materials in the soil, the pozzolanic reaction is not induced, failing to produce sufficient amounts of C-S-H and C-A-H gels to coat the clay particles and provide effective binding to resist early plastic collapse during the UCS test.

5.6 Effect of Deleterious Materials

Each of the mechanisms by which the deleterious materials are described in the literature to affect pozzolanic reactivity is evaluated against the results and observations made during the testing in this section. By evaluating each mechanism individually, the most plausible explanations for the low UCS results can be separated from the rest.

5.6.1 Ferrous Oxide

The deleterious material testing has determined that the Cooyar black soil sampled contains 0.66% ferrous oxide by weight, which means that the soils in the area have undergone partial weathering and leaching. But since the ferrous oxide content is below the limit of 2.0% prescribed by Austroads (2019) and DTMR (2021*c*, 2022*e*), the literature does not concern that soil weathering has directly caused the abnormal UCS results observed. However, as leaching can cause a decrease in SiO_2, one of the pozzolans, and a decrease in pH it is possible that weathering is a contributing factor to the decrease in lime reactivity.

5.6.2 Sulphate

Similarly, the sulphate content in the Cooyar black soil is only 0.01%, which is well below the limit of 3% established by DTMR (2021*c*, 2022*e*), AustStab (2008), NLA (2004) and Little (1995). As the sulphate content is well within the limit of no concern (Berger et al., 2001, Harris et al., 2004), there is no reason for ettringite to grow, remove free Calcium (CA^{2+}) ions, cause heaving problems described by (Cheshomi et al., 2017) and ultimately affect the UCS results. Especially considering that the good construction practises described by Berger et al. (2001) and Little (1995) for the treatment of soils containing sulphate form part of DTMR (2022*e*) specifications and have been used during this study where applicable, even if a minor sulphate content was observed, the mitigation strategies employed would have prevented the heaving problems.

5.6.3 Organic Carbon

The only method by which the industry authorities (Austroads, 2019, DTMR, 2021*c*, 2022*e*, NLA, 2001) determine the amount of organic matter in the soil is the organic carbon content, which they specified to generally not exceed 10% by weight to allow a soil to be suitable for lime stabilisation. Therefore, the only measure by which organic matter was tested as part of the regime developed for this study is organic carbon, which was determined to make up 10.5% of the Cooyar black soil by weight. As the amount of organic carbon is considered excessive, there is no doubt that the Cooyar black soil will have a decreased fraction of pozzolans compared to other residual nonorganic clays in the area. The lack of pozzolans can cause a pozzolanic reaction to not be induced (Hampton and Edil, 1998) and the C-S-H and C-A-H gels to not be produced in enough quantity to bind the clay particles.

The high organic carbon content has been shown by Rawls et al. (2003) to also increase water retention in the soil, increasing the potential for sulphate attack and ettringite growth over the life of the pavement. However, since the sulphate content of Cooyar black soil is low, there would not have been any concern that this would pose a risk of serviceability to the pavement and since for the sulphate attack to occur, the soil must draw sulphate from the neighbouring materials, the increased water retention would not have any impact on the laboratory results.

5.6.4 Humic Acid

As humic acid is the result of the breakdown of organic matter (The Editors of Encyclopaedia Britannica, 2020), and Cooyar black soil has a high organic carbon content, it is therefore expected that humic acid will be present in the soil. In their testing, Harris et al. (2009) have established that the content of humic acid as little as 1% can negatively impact the pozzolanic reaction, similar to the research of Sargent (2015) and Shiroya and Kumada (1976) which describe the tendency of humic acid to bind and remove Calcium (CA^{2+}) ions from the solution. Harris et al. (2009) have also reported that the presence of humic acid does not affect the results of the lime demand test, which means that without conducting specific tests to determine the amount of humic acid in the soil, the possibility of humic acid affecting the UCS results cannot be ignored.

Assuming that humic acid exists in Cooyar black soil, if it coated the clay particles and prevented the release of the compounds required for a pozzolanic reaction as described by Bleam (2017), it would also prevent the exchange of cations associated with immediate textural improvements as a result of the modification of a soil with lime. However, on the basis of the MDR, swell, and CBR results, it has been concluded that these improvements are still observed despite the lack of strength improvement associated with a successful pozzolanic reaction. Therefore, the theory of humic acid coating the clay particles can be ruled out as a reason for the low UCS results.

5.7 Chapter Summary

The key findings of the analysis of variance in the response of the laboratory CBR and UCS test results to the addition of lime to Cooyar black soil are as follows:

1. The addition of lime to the Cooyar black soil provides textural changes through cation exchange between Calcium ions and surface cations of the clay particles. As reported by Rehman et al. (2017), Kassa and Wubineh (2023) and Taskiran (2010) changes in maximum dry density, optimal moisture content, and plasticity associated with lime modification also have a strong influence on laboratory CBR results, resulting in an increase in laboratory CBR results with an increase in lime content.

2. However, the addition of lime fails to induce a successful pozzolanic reaction, produce sufficient amounts of C-S-H and C-A-H gels to bind the clay particles together and provide an increase in the UCS test results used to confirm that stabilisation of the soil has occurred. It is most likely to be caused by a high organic carbon content, which reduces the amount of pozzolans available for a reaction. However, it is also probable that there is humic acid in the soil that has been reported to negatively impact the pozzolanic reaction (Harris et al., 2009, Shiroya and Kumada, 1976, Sargent, 2015), while not noticeably affecting the results of the lime demand test.

3. Laboratory CBR results should not be used to confirm the effectiveness of lime stabilisation, and there should not be any assertion of strength increase without UCS test results above 1.0 MPa. Because a pavement design with a design life of 25 years, produced using the CBR test results of Cooyar black soil mixed with lime, when considering the inability of the pozzolanic reaction to be induced in this soil, the factors that influence the CBR results, and the confining effect of a CBR mould, it was determined using the CIRCLY 7.0 pavement design and analysis software that the pavement will fail within the first 2 years and 3 months of opening.

Chapter 6

Conclusions and Further Work

6.1 Introduction

The following chapter will conclude the dissertation by summarising the key findings of the study in relation to the objectives and aims established in Chapter 1. This will be followed by a presentation of the limitations and implications of the research undertaken. In conclusion of this chapter, opportunities and recommendations for future research on the topic of lime stabilisation of highly organic clay soils will be proposed.

6.2 Conclusions

This dissertation was completed under the sponsorship of DTMR in order to confirm and investigate the origins of the low UCS results observed during the initial investigation in 2017 and use the findings to improve the understanding of the factors that affect the effectiveness of lime stabilisation of soils containing deleterious materials and to provide guidance on the use of Austroads (2019) guidelines for lime stabilisation under similar soil conditions to prevent unexpected and early failures of the overlying pavements in future road construction and rehabilitation projects. In order to achieve these aims, the following work was completed.

6.2.1 Identify and Quantify Deleterious Materials

In Section 2.6 as part of the review of publicly available literature and guides by soil and pavement stabilisation industry authorities, sulphate, organic content in the form of organic carbon and humic acid, and degree of weathering measured as ferrous oxide content have been identified as the main factors attributed to having a negative impact on the performance of lime-stabilised soil by affecting soil alkalinity and the availability of pozzolans and calcium ions required for a successful pozzolanic reaction to occur.

When conducting material classification testing on the Cooyar black soil, as summarised in Section 4.2, organic carbon just above the limit established in the literature (Austroads, 2019, DTMR, 2022*e*, 2021*c*, NLA, 2004, Little, 1995) and minor ferrous oxide contents at approximately a quarter of the maximum described by Austroads (2019) and DTMR (2022*e*, 2021*c*) were identified to be present in the soil, with the sulphate content being negligible.

Upon further review of the results and the literature, as described in Section 5.6, it was determined that the high organic carbon content in the Cooyar black soil affects the amount of pozzolans available for the successful pozzolanic reaction to occur, preventing the binding of the clay particles together and the development of a unconfined compressive strength. It was also stipulated that, because humic acid is a by-product of the breakdown of organic carbon, there is a potential presence of humic acid, which has been reported to negatively impact the pozzolanic reaction without affecting the pH and lime content results during the lime demand testing.

However, the results of the CBR testing at varying lime contents were not affected by the high organic carbon content in the soil, showing an increase in CBR with an increase in lime content. This means that the CBR results are primarily a measure of the textural changes associated with the lime modification that are the result of the reduction of the defused water layer and the increased flocculation caused by the cation exchange between the surface of clay minerals and calcium ions. It is expected that, because the cation exchange does not require an abundance of pozzolans or hydroxyl groups to occur, the presence of organic matter will not have a significant effect on the CBR results, to where the comparison between CBR results prior to and after stabilisation would reveal that a pozzolanic reaction has not occurred.

6.2.2 Compare Two Austroads Methods

As described in Section 2.4.1, when following Austroads (2019) Method A for mix design methods, UCS results at various lime contents below and above the lime demand point are plotted to find the peak value above the threshold of 1.0 MPa (DTMR, 2021*c*) to be used as the target lime content. However, during the testing of the Cooyar black soil, the UCS values did not produce a peak and were all below the threshold of 1.0 MPa. Therefore, no direct comparison could be made between the two mix design methods, although Roads and Infrastructure Australia (2016) have reported those mix design methods that use the UCS results tend to require a higher lime content compared to CBR methods due to the requirement to use the lime content at a peak UCS result, while for CBR methods, the required lime content is usually taken as the lime demand test plus 1%.

6.2.3 Assess Effects on Overlaying Pavements

As the UCS results did not reach the threshold of 1.0 MPa, the Austroads (2017) mechanistic-empirical pavement design methods could not be used to prepare a pavement configuration that would be comparable to the pavement that was developed using the Austroads (2017) empirical pavement design methods and CBR results. Instead of performing direct computations, two pavement configurations prepared using Austroads (2017) empirical pavement design methods were analysed using the mechanistic-empirical pavement design and analysis software CYRCLY 7.0. As discussed in Section 5.4.2, the results of this analysis when estimating the vertical modulus of Cooyar black soil from the UCS results, after lime incorporation and curing, show that a pavement configuration that was prepared based on the CBR results for a design life of 25 years under growing design traffic is actually expected to fail within the first 2 years and 3 months of operation under design traffic.

6.3 Limitations

As the research prepared as part of this dissertation was carried out as a capstone project over a period of 35 weeks, there are a number of limitations that have affected the amount of work that could be carried out on time and within the generous, however, still limited funding that could be provided by the Department of Transport and Main Roads, which is a Queensland Government body that funded this research in its entirety.

It is an internal policy within DTMR that for any research undertaken internally and externally to be considered in the preparation or revision of specifications, the testing involved in the said research must be performed in NATA-accredited laboratories. As there is only a limited budget available for each individual project sponsored by the department, in order to stay within the budget, only the necessary tests could be performed, and the testing methods used had to be within the scope that each laboratory involved had NATA accreditation for. This meant that as part of the testing regime employed, no XRD or SRM imaging could be performed to help identify the actual effects of the organic contents on the pozzolanic reactivity of the soil. Additionally, because the current DTMR specifications do not include a method for testing for humic acids, the humic acid contents could not be quantified as part of this study.

Similarly, since the main aim of this study was to develop a basic understanding of what was wrong with the samples used during pavement investigations carried out in 2017 to develop the aims and methods of future research on this topic, this study only dealt with representative samples of the soil, as it was in situ, only modified by adding water and hydrated lime, and did not involve the addition, removal or any other variation of the contents of deleterious materials. Meaning that no testing was performed to explore the relationships between CBR and UCS results in response to variations in the deleterious material contents.

6.4 Implications

The findings of this study and the suggestions presented in the following section of the dissertation are expected to form part of the ongoing research undertaken by the Department of Transport and Main Roads into sustainable and responsible methods of road construction and rehabilitation. It is also believed that this study and future research by the DTMR that arises from these findings will allow the DTMR representatives who are members of the Austroads board to provide evidence-based suggestions for improvements to the Austroads (2019) guide, including the following recommendations on how best to use the Austroads (2019) guidelines in the context of highly organic black soil.

Care must be taken when undertaking the mix design procedures for use in the stabilisation of clay soils with lime as the calcium-based binder, ensuring that all the recommendations established in the Austroads (2019) guide and the relevant specifications provided by the local road authorities are adhered to. This recommendation includes that all efforts are made to ensure the identification of the presence of deleterious materials in the soil in situ, as the consequences as a result of a lack of pozzolanic reaction are not yet well understood.

If in certain circumstances it is not possible to carry out deleterious testing, the mix design methods employing UCS testing must be used to ensure that a successful pozzolanic reaction has been induced since the CBR results are not a reliable indicator of this. This recommendation should be applied even if the pavement design methods used do not assume that the lime-stabilised subgrade layer will be used as a structural layer. This is because the main reason for this recommendation is that even if it is assumed that there has been no improvement in soil strength after lime stabilisation, there is no certainty that the calcium ions on the surface of clay particles, responsible for the flocculation and textural changes seen in lime modification, will not be leached out through percolation or cation exchange from deposited impurities with cations of concentration and valence higher than that of calcium ions and that the expansive behaviour of the soil will not be reversed over time.

6.5 Further Research and Recomendations

Despite the amount of research that has been undertaken across the world on the topic of lime stabilisation, there are still gaps in the knowledge on the interactions between the deleterious materials and lime, and how to mitigate the negative effects to allow for the use of lime stabilisation across a broader spectrum of soils. The following are two research topics and recommendations developed based on the findings of this dissertation, which, if undertaken and led to positive outcomes, are believed to help benefit industrial knowledge for a more sustainable future in road construction and rehabilitation practice.

Based on the conclusions drawn in this dissertation, the lack of pozzolans appears to be the best possible cause of the low UCS results in the organic clay soil examined in the study. An effective method to test this hypothesis would be to perform an experimental study that assumes that this is the primary cause and looks at the best way to solve the issue without the need to remove and replace the organic soil. Some of the ways this may be achieved would be to study the effect of incorporating pozzolan-rich additives along with the hydrated lime, the primary candidates for such additives are the byproducts of iron production and coal combustion, the blast furnace slag for its ability to turn into C-S-H upon contact with water, and the fly ash as it tends to contain aluminium, silicon, and calcium oxides which can fuel a pozzolanic reaction.

If the addition of pozzolans does not provide sufficient improvements in the UCS results, further research will be required, including XRD and SRM imaging at various stages of the mix design process to deduce the actual causes and management strategies. Furthermore, if a testing method for quantifying the content of humic acids is approved for use in future research projects, it may be beneficial to monitor the pH and the humic acid content in lime-stabilised organic soil samples at set intervals for a period of 28 days, which is longer than the pH monitoring period used during the lime demand testing to identify whether humic acids have a delayed impact on the pH, which may explain the lack of pozzolanic activity and develop a better understanding of the findings made by Harris et al. (2009).

References

ALS Global 2023, *Iron Ore Analysis*, viewed viewed 10 June 2023, <https://www.alsglobal.com/en/geochemistry/bulk-commodity-analysis/iron-ore-analysis>.

Austroads 2017, *Guide to Pavement Technology Part 2: Pavement Structural Design*, 4.3 edn, Austroads, Sydney, New South Wales.

Austroads 2019, *Guide to Pavement Technology Part 4D: Stabilised Materials*, 2.1 edn, Austroads, Sydney, New South Wales.

AustStab 2008, *AustStab Technical Note No.1F: Lime stabilisation practice*, Chatswood, New South Wales, viewed 26 February 2023, <https://auststab.com.au/wp-content/uploads/2017/02/Lime-Stabilisation-Practice.pdf>.

Baig, M. N. 1962, CBR and unconfined compressive strength tests on a lime stabilized clay soil, PhD thesis, Virginia Polytechnic Institute.

Barman, D. and Dash, S. K. 2022, 'Stabilization of expansive soils using chemical additives: A review', *Journal of Rock Mechanics and Geotechnical Engineering* **14**(4), 1319–1342.

Beecroft, A. and Coomer, J. 2018, Report p54: Effective expansive subgrade treatments across queensland, Technical Report TC-710-4-4-8, National Asset Centre of Excellence, Brisbane, Queensland, viewed 11 March 2023, <http://nacoe.com.au/wp-content/uploads/2018/07/PRP16030-P54_Effective-treatments-for-expansive-subgrades_FINAL.pdf>.

Berger, E., Little, D. and Graves., R. 2001, Guidelines for stabilization of soils containing sulfates, Technical Memorandum, The Lime Association of Texas, viewed

2 April 2023, <https://www.lime.org/documents/publications/free_downloads/technical-memorandum.pdf>.

Black, W. P. M. 1961, 'The calculation of laboratory and in-situ values of california bearing ratio from bearing capacity data', *Géotechnique* **11**(1), 14–21.

Bleam, W. 2017, Chapter 7 - natural organic matter, *in* 'Soil and Environmental Chemistry', 2 edn, Academic Press, pp. 333–384.

Cheshomi, A., Eshaghi, A. and Hassanpour, J. 2017, 'Effect of lime and fly ash on swelling percentage and atterberg limits of sulfate-bearing clay', *Applied Clay Science* **135**, 190–198.

DR 2023, Queensland Globe, GIS Database, Queensland Department of Resources, Brisbane, Queensland, viewed 16 August 2023, <https://qldglobe.information.qld.gov.au/>.

DTMR 2017, Testing of Materials for Lime Stabilisation, Technical Note 151, Queensland Department of Transport and Main Roads, Brisbane, Queensland.

DTMR 2021*a*, CIRCLY 7.0 Materials Database, Technical Database, Queensland Department of Transport and Main Roads, Brisbane, Queensland, viewed 15 July 2023, <https://www.tmr.qld.gov.au/business-industry/technical-standards-publications/pavement-design-supplement>.

DTMR 2021*b*, Class-Specific Traffic Load Distributions Spreadsheet, Calculation Worksheet, Queensland Department of Transport and Main Roads, Brisbane, Queensland, viewed 15 July 2023, <https://www.tmr.qld.gov.au/business-industry/technical-standards-publications/pavement-design-supplement>.

DTMR 2021*c*, Structural design procedure for lime stabilised subgrade, guidline, Queensland Department of Transport and Main Roads, Brisbane, Queensland, viewed 26 February 2023, <https://www.tmr.qld.gov.au/business-industry/technical-standards-publications/pavements-guidelines>.

DTMR 2021*d*, Supplement to 'Part 2: Pavement Structural Design' of the Austroads Guide to Pavement Technology, Paveemnt Design Supplement, Queensland Department of Transport and Main Roads, Brisbane, Queensland, viewed 15 July 2023, <https://www.tmr.qld.gov.au/business-industry/technical-standards-publications/pavement-design-supplement>.

DTMR 2022*a*, Average Annual Daily Traffic for Queensland State Controlled Roads 2012 to 2021, Traffic Census Data, Queensland Department of Transport and Main Roads, Brisbane, Queensland, viewed 15 July 2023, <https://www.data.qld.gov.au/dataset/traffic-census-for-the-queensland-state-declared-road-network/resource/1f52e522-7cb8-451c-b4c2-8467a087e883>.

DTMR 2022*b*, Materials Testing Manual, 5.8 edn, Technical Manual, Queensland Department of Transport and Main Roads, Brisbane, Queensland, viewed 26 February 2023, <https://www.tmr.qld.gov.au/business-industry/Technical-standards-publications/Materials-testing-manual>.

DTMR 2022*c*, Transport and Main Roads Specifications: MRTS04 General Earthworks, Technical Specification, Queensland Department of Transport and Main Roads, Brisbane, Queensland, viewed 26 February 2023, <https://www.tmr.qld.gov.au/business-industry/technical-standards-publications/specifications/3-roadworks-drainage-culverts-and-geotechnical>.

DTMR 2022*d*, Transport and Main Roads Specifications: MRTS05 Unbound Pavements, Technical Specification, Queensland Department of Transport and Main Roads, Brisbane, Queensland, viewed 15 July 2023, <https://www.tmr.qld.gov.au/business-industry/technical-standards-publications/specifications/5-pavements-subgrade-and-surfacing>.

DTMR 2022*e*, Transport and Main Roads Specifications: MRTS07A Insitu Stabilised Subgrades using Quicklime or Hydrated Lime, Technical Specification, Queensland Department of Transport and Main Roads, Brisbane, Queensland, viewed 26 February 2023, <https://www.tmr.qld.gov.au/business-industry/technical-standards-publications/specifications/5-pavements-subgrade-and-surfacing>.

DTMR 2023, *Building sustainable roads*, Brisbane, Queensland, viewed 26 September 2023, <https://www.tmr.qld.gov.au/Buildingsustainableroads>.

Eisazadeh, A., Kassim, K. A. and Nur, H. 2011, 'Characterization of phosphoric acid- and lime-stabilized tropical lateritic clay', *Environmental Earth Sciences* **63**(5), 1057–1066.

Eme, D., Nwofor, T. and Sule, S. 2016, 'Correlation between the california bearing ratio (cbr) and unconfined compressive strength (ucs) of stabilized sand-cement of the niger delta', *International Journal of Civil Engineering* **3**, 7–13.

Evans, P., Smith, W. and Vorobieff, G. 1998, Rethink of the design philosophy of lime stabilisation, *in* 'ARRB TRANSPORT RESEARCH LTD CONFERENCE, 19TH', ARRB, Sydney, New South Wales, viewed 12 March 2023, <https://auststab.com.au/wp-content/uploads/2017/02/Rethink-of-the-design-philosophy-of-lime-stabilisation-1998.pdf>.

Government, Q. 2013, *Soil colour*, Brisbane, Queensland, viewed 23 August 2023, <https://www.qld.gov.au/environment/land/management/soil/soil-properties/colour>.

Greymont n.d., *High calcium quicklime*, Safety Data Sheet, Greymont, viewed 10 June 2023, <https://www.graymont.com/sites/default/files/pdf/msds/high_calcium_quicklime_graymont_-_sds_us-can_4.9.3.4english_us.pdf>.

Groot, C., Veiga, R., Papayianni, I., Hees, R., Secco, M., Alvarez, J., Faria, P. and Stefanidou, M. 2022, 'Rilem tc 277-lhs report: lime-based mortars for restoration -– a review on long-term durability aspects and experience from practice', *Materials and Structures* **55**.

Hampton, M. and Edil, T. 1998, 'Strength gain of organic ground with cement-type binders', *Geotechnical Special Publication* pp. 135–148.

Harris, P., Harvey, O., Puppala, A., Sebesta, S., Chikyala, S. R. and Saride, S. 2009, Mitigating the effects of organics in stabilized soils, Technical Report 0-5540-1, Texas Transportation Institute, College Station, Texas, viewed 31 June 2023, <https://rosap.ntl.bts.gov/view/dot/38672>.

Harris, P., Scullion, T. and Sebesta, S. 2004, Hydrated lime stabilization of sulfate-bearing soils in texas, Technical Report 0-4240-2, Texas Transportation Institute, College Station, Texas, viewed 8 March 2023, <https://static.tti.tamu.edu/tti.tamu.edu/documents/0-4240-2.pdf>.

Heimans, F. 2004, Oral History Program: Pavement recycling & stabilisation, Summary report, NSW Roads and Traffic Authority, viewed 3 October 2023, <https://www.transport.nsw.gov.au/operations/roads-and-waterways/environment-and-heritage/heritage/oral-history/pavement-recyclingl>.

Hossain, M. S. and Kim, W. S. 2014, Estimation of Subgrade Resilient Modulus Using the Unconfined Compression Test, Report on Federally Funded Project FHWA/VCTIR 15-R12, Virginia Center for Transportation Innovation and Research, Charlottesville, Virginia, viewed 31 July, <http://www.virginiadot.org/vtrc/main/online_reports/pdf/15-r12.pdf>.

Hossain, S., Islam, A., Badhon, F. F. and Imtiaz, T. 2021, *Unconfined Compressive Strength Test*, Mavs Open Press. viewed 1 July 2023, <https://uta.pressbooks.pub/soilmechanics/chapter/unconfined-compressive-strength-test/>.

International Energy Agency 2020, *Global Energy Review 2020*, International Energy Agency, Paris, Île-de-France, viewed 15 July 2023, <https://www.iea.org/reports/global-energy-review-2020>. License: CC BY 4.0.

Isbell, R. F. and NCST 2021*a*, Glossary, *in* 'Australian Soil Classification', CSIRO Publishing, Clayton South, Victoria, viewed 24 September 2023, <https://www.soilscienceaustralia.org.au/asc/soilglos.htm>.

Isbell, R. F. and NCST 2021*b*, Vertosols, *in* 'Australian Soil Classification', CSIRO Publishing, Clayton South, Victoria, viewed 24 September 2023, <https://www.soilscienceaustralia.org.au/asc/ve/vertsols.htm>.

Kassa, S. M. and Wubineh, B. Z. 2023, 'Use of machine learning to predict california bearing ratio of soils', *Advances in Civil Engineering* **2023**, 1–11.

Little, D. N. 1995, *Handbook for Stabilization of Pavement Subgrades and Base Courses with Lime*, Kendall/Hunt Publishing Company.

Maheswaramma, K. S. 2016, Estimation of ferrous iron by dichrometry, *in* 'Engineering Chemistry', Pearson Education India, viewed 27 September 2023, <https://www.oreilly.com/library/view/engineering-chemistry/9789332579163/xhtml/35_Lab6.xhtml>.

Mitchell, J. K. and Dermatas, D. 1992, Clay soil heave caused by lime-sulfate reactions, *in* 'Innovations and uses for lime', Vol. 1135, ASTM International West Conshohocken, PA, USA, pp. 41–64.

Mohd yunus, N. Z., Wanatowski, D., Abdullah, N. and Abdullah, R. 2013, 'Effect of humic acid on microstructure of lime-treated organic clay', *International Journal of Engineering Research and Technology* **2**, 1827–1833.

Mukabi, J. 2015, Evaluation of limitations of some popular cbr - ucs based resilient modulus models for applications in the structural design of pavements, viewed 31 July 2023, <https://www.researchgate.net/publication/309160450_Evaluation_of_Limitations_of_Some_Popular_CBR_-_UCS_Based_Resilient_Modulus_Models_for_Applications_in_the_Structural_Design_of_Pavements>.

NLA 2001, *Using lime for soil stabilization and modification. A proven solution!*, viewed 26 February, <https://www.lime.org/publications/free-downloads/>.

NLA 2004, Lime-treated soil construction manual: Lime stabilization & lime modification, Technical manual, The National Lime Association, Arlington, Virginia, viewed 26 February, <https://www.lime.org/publications/free-downloads/>.

Ouhadi, V. and Yong, R. 2008, 'Ettringite formation and behaviour in clayey soils', *Applied Clay Science* **42**(1), 258–265.

Ouhadi, V., Yong, R., Amiri, M. and Ouhadi, M. 2014, 'Pozzolanic consolidation of stabilized soft clays', *Applied Clay Science* **95**, 111–118.

O'Flaherty, C. A., David, H. T. and Davidson, D. T. 1961, 'Relationship between the california bearing ratio and the unconfined compressive strength of sand-cement mixtures', *Proceedings of the Iowa Academy of Science* **68**(1), 341–356.

Pavement Interactive 2023*a*, *California Bearing Ratio*, viewed 1 July 2023, <https://pavementinteractive.org/reference-desk/design/design-parameters/california-bearing-ratio/>.

Pavement Interactive 2023*b*, *Resilient Modulus*, viewed 31 July 2023, <https://pavementinteractive.org/reference-desk/design/design-parameters/resilient-modulus/>.

Pavement Science 2015, 2970 — cumulative damage factor (cdf), CIRCLY 7.0 Tutorial, Pavement Science, viewed 1 August 2023, <https://ops.pavement-science.com.au/courses/circly-7-0/lessons/how-to-use-austroads-design-mode-7-0a/topic/2970-cumulative-damage-factor-cdf/>.

Queensland Ggovernment 2013, *Common soil types*, Brisbane, Queensland, viewed 24 September 2023, <https://www.qld.gov.au/environment/land/management/soil/soil-testing/types>.

Queensland Ggovernment 2023, *Exempt waste*, Brisbane, Queensland, viewed 26 September 2023, <https://www.qld.gov.au/environment/management/waste/recovery/disposal-levy/about/exempt>.

Rawls, W., Pachepsky, Y., Ritchie, J., Sobecki, T. and Bloodworth, H. 2003, 'Effect of soil organic carbon on soil water retention', *Geoderma* **116**(1), 61–76.

Ray, C. 2017, *P&H Legacy*, viewed 3 October 2023, <https://home4c.com/ray/phlegacy.html>.

Rehman, Z. u., Khalid, U., Farooq, K. and Mujtaba, H. 2017, 'Prediction of cbr value from index properties of different soils', *Technical Journal* **22**, 17–26.

Roads and Infrastructure Australia 2016, *Design of lime stabilised subgrades – CBR or UCS?*, viewed 22 April 2023, <https://roadsonline.com.au/design-of-lime-stabilised-subgrades-cbr-or-ucs/>.

Safe Work Australia 2013, *Guide for managing the risk of fatigue at work*, viewed 22 April 2023, <https://www.safeworkaustralia.gov.au/doc/guide-managing-risk-fatigue-work>.

Saputra, N. A. and Putra, R. 2020, 'The correlation between cbr (california bearing ratio) and ucs (unconfined compression strength) laterite soils in palangka raya as heap material', *IOP Conference Series: Earth and Environmental Science* **469**(1), 012093.

Sargent, P. 2015, 21 - the development of alkali-activated mixtures for soil stabilisation, *in* F. Pacheco-Torgal, J. Labrincha, C. Leonelli, A. Palomo and P. Chindaprasirt, eds, 'Handbook of Alkali-Activated Cements, Mortars and Concretes', Woodhead Publishing, Oxford, pp. 555–604.

Shiroya, R. and Kumada, K. 1976, 'Combination reaction between humic acid and calcium ions', *Soil Science and Plant Nutrition* **22**(3), 345–349.

SSA 2022, *State Soils*, Soil Science Australia, Braeside, Victoria, viewed 24 September 2023, <https://www.soilscienceaustralia.org.au/about/about-soil/state-soils/>.

Standards Australia 1997, *Test methods for limes and limestones Lime index – Available lime*, Standard AS 4489.6.1-1997, Standards Australia, Sydney, New South Wales.

Standards Australia 1998, *Methods of testing soils for engineering purposes Sampling and preparation of soils – Disturbed samples – Standard method*, Standard AS 1289.1.2.1-1998, Standards Australia, Sydney, New South Wales.

Standards Australia 2008, *Methods of testing soils for engineering purposes Soil classification tests – Determination of the linear shrinkage of a soil – Standard method*, Standard AS 1289.3.4.1-2008, Standards Australia, Sydney, New South Wales.

Standards Australia 2009*a*, *Methods of testing soils for engineering purposes Soil classification tests – Calculation of the plasticity index of a soil*, Standard AS 1289.3.3.1-2009, Standards Australia, Sydney, New South Wales.

Standards Australia 2009*b*, *Methods of testing soils for engineering purposes Soil classification tests – Determination of the liquid limit of a soil – Four point Casagrande method*, Standard AS 1289.3.1.1-2009, Standards Australia, Sydney, New South Wales.

Standards Australia 2009*c*, *Methods of testing soils for engineering purposes Soil classification tests – Determination of the particle size distribution of a soil – Standard method of analysis by sieving*, Standard AS 1289.3.6.1-2009, Standards Australia, Sydney, New South Wales.

Standards Australia 2009*d*, *Methods of testing soils for engineering purposes Soil classification tests – Determination of the plastic limit of a soil – Standard method*, Standard AS 1289.3.2.1-2009, Standards Australia, Sydney, New South Wales.

Standards Australia 2017, *Geotechnical site investigations*, Standard AS 1726:2017, Standards Australia, Sydney, New South Wales.

Standards Australia 2020, *Methods of testing soils for engineering purposes Soil chemical tests – Determination of the sulfate content of a natural soil and the sulfate content of the groundwater – Normal method*, Standard AS 1289.4.2.1:2020, Standards Australia, Sydney, New South Wales.

Substrata 2023, *Soil Stabilization: The Ultimate Guide to Soils and Soil Stabilization*, viewed 15 April 2023, <https://www.substrata.us/soil-stabilization>.

Taskiran, T. 2010, 'Prediction of california bearing ratio (cbr) of fine grained soils by ai methods', *Advances in Engineering Software* **41**(6), 886–892.

The Editors of Encyclopaedia Britannica 2010, *Leaching*, Encyclopædia Britannica, inc., viewed 9 April 2023, <https://www.britannica.com/science/leaching-geochemistry-of-soil>.

The Editors of Encyclopaedia Britannica 2020, *Humic acid*, Encyclopædia Britannica, inc., viewed 25 March 2023, <https://www.britannica.com/science/humic-acid>.

Thompson, M. R. 1966, 'Lime reactivity of illinois soils', *Journal of the Soil Mechanics and Foundations Division* **92**(5), 67–92.

Thompson, M. R. and Robnett, Q. L. 1979, 'Resilient properties of subgrade soils', *Transportation Engineering Journal of ASCE* **105**(1), 71–89.

Tournas, V., Stack, M., Mislivec, P. and H.A. Koch, R. B. 2023, Chapter 18: Yeasts, Molds and Mycotoxins, *in* 'Bacteriological Analytical Manual', U.S. Food and Drug Administration, Silver Spring, Maryland, viewed 22 August 2023, <https://www.fda.gov/food/laboratory-methods-food/bam-chapter-18-yeasts-molds-and-mycotoxins>.

TRC 2022, *DRINKING WATER QUALITY MANAGEMENT PLAN – ANNUAL REPORT – 2021-2022*, 1 ver, Report DM #10635424, Toowoomba Regional Council, Toowoomba, Queensland, viewed 10 June 2023, <https://www.tr.qld.gov.au/environment-water-waste/water-supply-dams/dams-bores/13299-water-quality>.

Vanderstaay, A. G. B. 2020, Soils of western queensland, Western Queensland Best Practice Guidelines WQ32, Queensland Department of Transport and Main Roads, Brisbane, Queensland, viewed 24 September 2023, <https://www.tmr.qld.gov.au/business-industry/Technical-standards-publications/Western-Queensland-best-practice-guidelines>.

Vereecken, E. and Roels, S. 2012, 'Review of mould prediction models and their influence on mould risk evaluation', *Building and Environment* **51**, 296–310.

Volker, D. J. 2019, *Intention of field trials to explore innovative methods of slaking quicklime*, AusStab AGM, viewed 22 April 2023, <https://web.archive.org/web/20200309183331/https://auststab.com.au/wordy/wp-content/uploads/2019/08/Volker_conference_presentation_2019.pdf>.

Wikipedia Contributors 2023, *Soil carbon — Wikipedia, The Free Encyclopedia*, viewed 25 March 2023, https://en.wikipedia.org/wiki/Soil_carbon.

Wirtgen Group 2022, *How state-of-the-art cold recyclers and soil stabilizers work*, viewed 22 April 2023, <https://www.wirtgen-group.com/en-us/products/wirtgen/technologies/recycling-and-soil-stabilization/soil-stabilization-wr-series/>.

Woods, M. 2022, *Reminder to air dry soil samples*, Asian Turfgrass Center, Knoxville, Tennessee, viewed 11 June 2023, <https://www.asianturfgrass.com/post/reminder-to-air-dry-soil-samples/>.

Zhang, H. 2017, *Oklahoma Cooperative Extension Fact Sheet PSS-2239: Cause and effects of soil acidity*, Oklahoma State University, viewed 9 April 2023, <https://extension.okstate.edu/fact-sheets/cause-and-effects-of-soil-acidity.html>.

Appendix A

Project Specification

ENG 4111/2 Research Project

Project Specification

For:	**Runis Kashanov**
Topic:	Comparison of Austroads Guidelines for Lime Stabilisation of Highly Organic Black Soil
Major:	Civil Engineering
Supervisors:	H. Seligmann, UniSq - S2, 2023 A. Nataatmadja, UniSQ - S1, 2023 D. Volker, DTMR - S1&2, 2023
Sponsorship:	Queensland Department of Transport and Main Roads
Enrolment:	ENG4111 - EXT S1, 2023 ENG4112 - EXT S2, 2023
Project Aim:	To increase understanding of the factors influencing the effectiveness of lime stabilisation in soils containing deleterious materials, and to provide guidance on the use of Austroads guidelines for lime stabilisation in similar soil conditions in order to avoid failures in future road construction and rehabilitation projects.

Programme: **Version 3, 3$^{\text{nd}}$ June 2023**

1. Undertake background research into lime stabilisation of expansive soils.
2. Undertake research into types of deleterious materials and their effects onto lime stabilisation.
3. Undertake research into pavement design principles utilising lime stabilised subgrades.
4. Perform testing of Cooyar black soil utilising the two mix design methods set-out in Austroads guidelines.
5. Perform analysis of the test results.
6. Compare the expected results form the two Austroads methods for stabilisation of soils with deleterious materials.

As time and resources permit:

1. Perform research and develop a theoretical traffic model for a new highway construction.
2. Undertake pavement designs utilising the traffic model and two sub-grade models produced using the test results.
3. Compare the two Austroads methods based on the pavement designs produced.

Agreed:

Student Name:	Runis Kashanov
Supervisor Name:	Hannah Seligmann
Supervisor Name:	Damian Volker

Appendix B

Risk Assessment

B.1 Fatigue Management

2314	RISK DESCRIPTION	TREND	CURRENT	RESIDUAL
	Fatigue Management of regular travel between BNE and TWB to undertake testing		Medium	Low

RISK OWNER	RISK IDENTIFIED ON	LAST REVIEWED ON	NEXT SCHEDULED REVIEW
Runis Kashanov	01/05/2023	03/05/2023	03/11/2023

RISK FACTOR(S)	EXISTING CONTROL(S)	CURRENT	PROPOSED CONTROL(S)	TREATMENT OWNER	DUE DATE	RESIDUAL
Daily work hours and work-related travel, including commute.	Control: If visists to the laboratory are 8hrs long the matching their working hours. Door to Door working time is expected to be 12 working hours.	Medium	Arrange with the laboratory that visits can only last for 6hours to reduced the Door to Door time down to maximum of 10 hours.			Low
			Start commute no earlier than 6am, monitor the time during the day to ensure that there is enough time to be back home before 4pm.		08/05/2023	
Work-related travel.	Control: The commute between Home and Laboratory is going to be 1.5 hours.	Low	Schedule a 15minute fatigue break during the drive. Ensure 8hours of sleep prior to departure. Drive in day-light hours only.		08/05/2023	Low
Frequency of breaks during work and recovery time between work periods.	Control: Adequate and regular breaks and sleep.	Very Low	No Control:			Very Low

ATTACHMENTS
managing-the-risk-of-fatigue.pdf

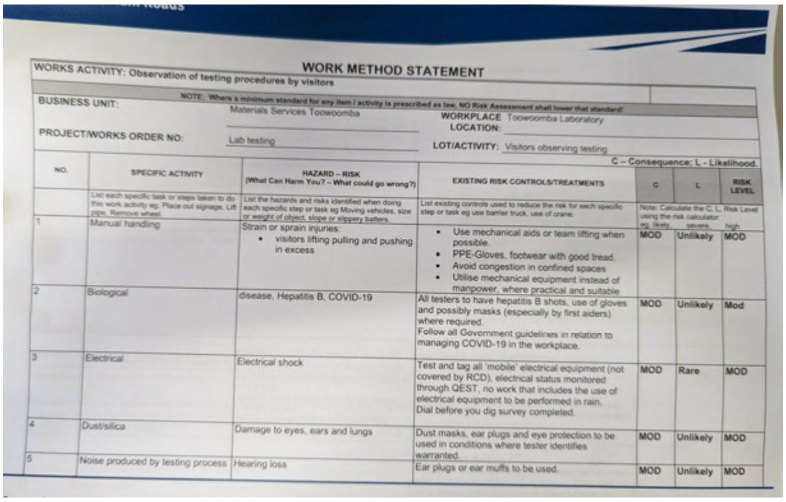
B.2 Undertaking of Material Testing

2316	RISK DESCRIPTION	TREND	CURRENT	RESIDUAL
	Undertaking of material testing at RoadTek Toowoomba Laboratory		Medium	Low

RISK OWNER	RISK IDENTIFIED ON	LAST REVIEWED ON	NEXT SCHEDULED REVIEW
Runis Kashanov	01/05/2023	02/05/2023	02/11/2023

RISK FACTOR(S)	EXISTING CONTROL(S)	CURRENT	PROPOSED CONTROL(S)	TREATMENT OWNER	DUE DATE	RESIDUAL
Strains, sprains and abrasions from manual handling and tool use.	Control: Use mechanical aids and team lifting for moving heavy loads, where possible. Use mechanical equipment for repetitive or strenuous activities, where possible. ______________ Control: PPE - Gloves, footwear with good tread	Medium	Only work in open areas to avoid congestion.			Low
Potential of Hepatitis B from working with soil.	Control: Use hand-gloves and masks when handling soil.	Medium	All testers and visitors to have up-to-date hepatitis B shots. Treat and clean any cut or abrasion and cover prior and after working with soil.			Low
Electrical shock from computers and other electrical lab equipment.	Control: Test and tag all electrical equipment.	Very Low				Very Low
Airborne Lime or Silica Dust when mixing dry soil samples and working with lime powder.	Control: Dust masks, goggles and ear plugs to be worn when working with dry soil and hydrated lime powder.	Medium	Work with dry soil and hydrated lime only in dedicated ventilated areas.			Low
Hearing damage from noise produced by testing process.	Control: Earplugs or muffs to be used.	Medium	Schedule testing in way where prolonged exposure to the noise is minimised.			Low

B.3 DTMR SWMS - Visitors Observing Testing



WORK METHOD STATEMENT

WORKS ACTIVITY: Observation of testing procedures by visitors

NOTE: Where a minimum standard for any item / activity is prescribed as law, NO Risk Assessment shall lower that standard!

BUSINESS UNIT: Materials Services Toowoomba

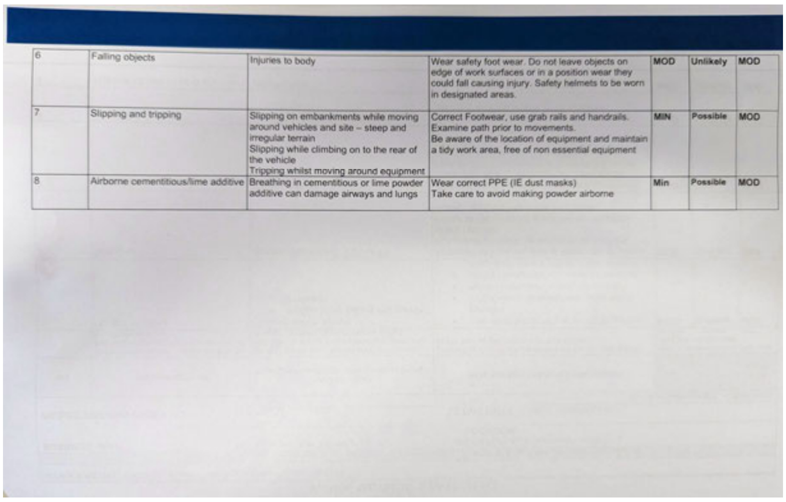
WORKPLACE LOCATION: Toowoomba Laboratory

PROJECT/WORKS ORDER NO: Lab testing

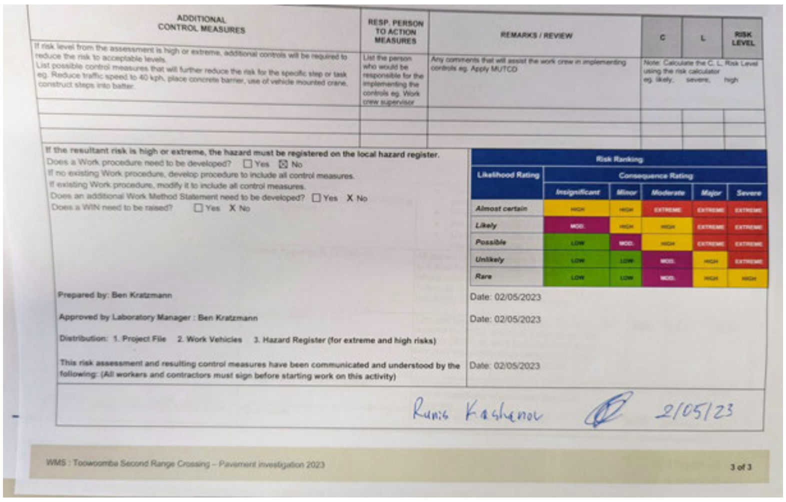
LOT/ACTIVITY: Visitors observing testing

C – Consequence; L - Likelihood.

NO.	SPECIFIC ACTIVITY	HAZARD – RISK (What Can Harm You? – What could go wrong?)	EXISTING RISK CONTROLS/TREATMENTS	C	L	RISK LEVEL
	List each specific task or steps taken to do this work activity eg. Place out signage. Lift pipe. Remove wheel.	List the hazards and risks identified when doing each specific step or task eg Moving vehicles, size or weight of object, slope or slippery batters.	List existing controls used to reduce the risk for each specific step or task eg use barrier truck, use of crane.	Note: Calculate the C, L, Risk Level using the risk calculator eg. likely,	severe,	high
1	Manual handling	Strain or sprain injuries: • visitors lifting pulling and pushing in excess	• Use mechanical aids or team lifting when possible. • PPE-Gloves, footwear with good tread. • Avoid congestion in confined spaces • Utilise mechanical equipment instead of manpower, where practical and suitable	MOD	Unlikely	MOD
2	Biological	disease, Hepatitis B, COVID-19	All testers to have hepatitis B shots, use of gloves and possibly masks (especially by first aiders) where required. Follow all Government guidelines in relation to managing COVID-19 in the workplace.	MOD	Unlikely	Mod
3	Electrical	Electrical shock	Test and tag all 'mobile' electrical equipment (not covered by RCD), electrical status monitored through QEST, no work that includes the use of electrical equipment to be performed in rain. Dial before you dig survey completed.	MOD	Rare	MOD
4	Dust/silica	Damage to eyes, ears and lungs	Dust masks, ear plugs and eye protection to be used in conditions where tester identifies warranted.	MOD	Unlikely	MOD
5	Noise produced by testing process	Hearing loss	Ear plugs or ear muffs to be used.	MOD	Unlikely	MOD



6	Falling objects	Injuries to body	Wear safety foot wear. Do not leave objects on edge of work surfaces or in a position wear they could fall causing injury. Safety helmets to be worn in designated areas.	MOD	Unlikely	MOD
7	Slipping and tripping	Slipping on embankments while moving around vehicles and site – steep and irregular terrain Slipping while climbing on to the rear of the vehicle Tripping whilst moving around equipment	Correct Footwear, use grab rails and handrails. Examine path prior to movements. Be aware of the location of equipment and maintain a tidy work area, free of non essential equipment	MIN	Possible	MOD
8	Airborne cementitious/lime additive	Breathing in cementitious or lime powder additive can damage airways and lungs	Wear correct PPE (IE dust masks) Take care to avoid making powder airborne	Min	Possible	MOD



ADDITIONAL CONTROL MEASURES	RESP. PERSON TO ACTION MEASURES	REMARKS / REVIEW	C	L	RISK LEVEL
If risk level from the assessment is high or extreme, additional controls will be required to reduce the risk to acceptable levels. List possible control measures that will further reduce the risk for the specific step or task eg. Reduce traffic speed to 40 kph, place concrete barrier, use of vehicle mounted crane, construct steps into batter.	List the person who would be responsible for the implementing the controls eg. Work crew supervisor	Any comments that will assist the work crew in implementing controls eg. Apply MUTCD	Note: Calculate the C, L, Risk Level using the risk calculator eg. likely,	severe,	high

If the resultant risk is high or extreme, the hazard must be registered on the local hazard register.

Does a Work procedure need to be developed? ☐ Yes ☒ No

If no existing Work procedure, develop procedure to include all control measures.

If existing Work procedure, modify it to include all control measures.

Does an additional Work Method Statement need to be developed? ☐ Yes X No

Does a WIN need to be raised? ☐ Yes X No

Risk Ranking

Likelihood Rating	Consequence Rating: Insignificant	Minor	Moderate	Major	Severe
Almost certain	HIGH	HIGH	EXTREME	EXTREME	EXTREME
Likely	MOD.	HIGH	HIGH	EXTREME	EXTREME
Possible	LOW	MOD.	HIGH	EXTREME	EXTREME
Unlikely	LOW	LOW	MOD.	HIGH	EXTREME
Rare	LOW	LOW	MOD.	HIGH	HIGH

Prepared by: Ben Kratzmann Date: 02/05/2023

Approved by Laboratory Manager : Ben Kratzmann Date: 02/05/2023

Distribution: 1. Project File 2. Work Vehicles 3. Hazard Register (for extreme and high risks)

This risk assessment and resulting control measures have been communicated and understood by the following: (All workers and contractors must sign before starting work on this activity) Date: 02/05/2023

Runis Kashenov 2/05/23

WMS : Toowoomba Second Range Crossing – Pavement investigation 2023 3 of 3

Appendix C

Ethical Clearance

There are no Ethical Clearances applicable to this project.

Appendix D

Suporting Information

D.1 Scopes of Testing Methods Used

D.1.1 Soil Classification Testing

AS 1289.1.2.1: *Methods of testing soils for engineering purposes Sampling and preparation of soils – Disturbed samples – Standard method*:

> This Standard outlines considerations and specifies procedures for taking disturbed samples of soils for engineering purposes such as earthworks and pavements, subdividing the samples and details for packing and forwarding them for examination and testing. In order to obtain appropriate representative samples, sampling is to be carried out by operators properly trained in the procedures and considerations given in this Standard and in general sampling techniques. The Standard does not cover undisturbed sampling of soils or sampling soils for tests for environmental purposes. Disturbed soil samples may be suitable for visual identification and for classification, chemical, density tests and strength tests on remoulded specimens. The selection of sites from where samples are to be taken is not covered by this Standard. Random selection of sampling sites, when required, is covered by AS 1289.1.4.1 or 1.4.2.
> Standards Australia (1998)

AS 1289.3.6.1: *Methods of testing soils for engineering purposes Soil classification tests – Determination of the particle size distribution of a soil – Standard method of analysis by sieving*:

> This Standard sets out the method for the quantitative determination by sieve analysis of the particle size distribution in a soil, down to the 75 (m sieve. By using this Method the combined silt and clay fraction can be obtained by difference. For particle sizes smaller than 75 (m the sedimentation method described in AS 1289.3.6.3, using a hydrometer to secure the necessary data, applies.
> Standards Australia (2009*c*)

AS 1289.3.1.1: *Methods of testing soils for engineering purposes Soil classification tests – Determination of the liquid limit of a soil – Four point Casagrande method*:

> This Standard sets out a method for determining the liquid limit of soil (the moisture content at which a soil passes) from the plastic to the liquid state).
> Standards Australia (2009*b*)

AS 1289.3.2.1: *Methods of testing soils for engineering purposes Soil classification tests – Determination of the plastic limit of a soil – Standard method*:

> This Standard sets out a method for determining the plastic limit of a soil (the moisture content at which a soil passes from the semi-solid to the plastic state).
> Standards Australia (2009*d*)

AS 1289.3.3.1: *Methods of testing soils for engineering purposes Soil classification tests – Calculation of the plasticity index of a soil*:

> This Standard sets out a method to calculate the plasticity index of a soil as derived from the liquid limit and the plastic limit of a soil.
> Standards Australia (2009*a*)

AS 1289.3.4.1: *Methods of testing soils for engineering purposes Soil classification tests – Determination of the linear shrinkage of a soil – Standard method*:

> This Standard sets out the method to determine the linear shrinkage of a soil.
> Standards Australia (2008)

Q142A: *Dry density-moisture relationship of soils and crushed rock — standard*:

This method sets out a procedure for the determination of the relationship between the moisture content and the dry density of a soil or a crushed rock material, including mixtures containing stabilising agents, when compacted using standard compactive effort (596 kJ/m3). Perform compaction over a range of moisture contents to establish the maximum mass of dry material per unit volume achievable for this compactive effort and its corresponding moisture content.
The procedure is applicable to that portion of a material that passes the 37.5 mm sieve. Material that all passes the 19.0 mm sieve is compacted in a 105 mm diameter mould. Material that has more than 20% rock retained on the 19.0 mm sieve is compacted in a 152 mm diameter mould. Corrections for oversize are not directly included in this method but are detailed in Test Method Q140A when required for compaction control.
DTMR (2022*b*)

D.1.2 Deleterious Material Testing

Q120B: *Organic content of soil - loss on ignition*:

This method describes the procedure for the determination of the organic content of soil by loss on ignition. It determines the total organic content of a sample (including any undecomposed organic matter such as particles of grass, sticks, and so on) by igniting the sample at 500°C in a furnace and calculating the resultant percentage mass loss.
DTMR (2022*b*)

AS 1289.4.2.1: *Methods of testing soils for engineering purposes Soil chemical tests – Determination of the sulfate content of a natural soil and the sulfate content of the groundwater – Normal method*:

> This Standard covers the determination of the water-soluble sulfate content of natural soil and the sulfate content of the groundwater. The results obtained give the sulfate contents at the time of sampling only and both these values are subject to seasonal fluctuations. The two sulfate contents and the moisture content of the soil are mutually interdependent.
> Standards Australia (2020)

Fe-VOL05: *Determination of ferrous iron by titration*: As this is an in-house method performed by ALS Global (2023), there is no publicly available standard that would provide the scope of the test method. However, Maheswaramma (2016) describes the aim and principle of the test as follows:

> To estimate the amount of ferrous iron present in 100 ml of the given solution by using approximately 0.05N potassium dichromate solution. The estimation is based on redox titration.
> Ferrous iron is estimated by dichrometry using diphenylamine indicator. Potassium dichromate oxidises ferrous iron to ferric iron in an acidic medium.

$$K_2Cr_2O_7 + 4\ H_2SO_4 \rightarrow K_2SO_4 + Cr_2(SO_4)_3 + 4H_2O + 3(O)$$

$$[2FeSO_4 + H_2SO_4 + (O) \rightarrow Fe_2\ (SO_4)_3 + (H_2O)] \times 3$$

$$K_2Cr_2O_7 + 6FeSO_4 + 7H_2SO_4 \rightarrow K_2SO_4 + Cr_2(SO_4)_3 + 3\ Fe_2\ (SO_4)_3 + 7H_2O$$

> The addition of orthophosphoric acid is to reduce the redox potential of the ferrous to ferric iron and to obtain the sharp change of the end point of titration from a colourless to a permanent violet blue colour solution.
> Maheswaramma (2016)

D.1.3 Strength Testing

AS 4489.6.1: *Test methods for limes and limestones Lime index – Available lime*:

The available lime index of quicklime and hydrated lime designates those constituents that enter into the reaction under the conditions of this specified test method, otherwise known as the 'rapid sugar test method'. The interpretation of results obtained by this test method is restricted by this definition. This test method is based on ASTM C 25, Test Methods for Chemical Analysis of Limestone, Quicklime and Hydrated Lime.
Standards Australia (1997)

Q133: *Lime demand of soil*:

This method describes the procedure to determine the degree to which a soil will react with calcium hydroxide through cationic exchange and pozzolanic responses from reactive clay minerals. The method provides for the determination of the lime demand (percent lime), as measured using an extended pH test. The lime demand is a minimum lime content in determining the design lime content. The lime demand test provides lime contents that correspond well with minimum lime contents required for effective long-term stabilisation.
DTMR (2022*b*)

Q135A: *Addition of stabilising agents*:

This method describes the procedure for calculating the quantity of stabilising agent(s) and any specified admixture(s) to add to a host soil or crushed rock as well as the procedures for mixing, by either hand or machine, the constituent materials and the conditioning of the mixture prior to compaction. The mixing process allows for the incorporation of one or more stabilising agents and provides the techniques for the addition of hydraulic, bituminous or ionic agents.
DTMR (2022*b*)

Q142A: *Dry density–moisture relationship of soils and crushed rock — standard*:

This method sets out a procedure for the determination of the relationship between the moisture content and the dry density of a soil or a crushed rock material, including mixtures containing stabilising agents, when compacted using standard compactive effort (596 kJ/m3). Perform compaction over a range of moisture contents to establish the maximum mass of dry material per unit volume achievable for this compactive effort and its corresponding moisture content.
The procedure is applicable to that portion of a material that passes the 37.5 mm sieve. Material that all passes the 19.0 mm sieve is compacted in a 105 mm diameter mould. Material that has more than 20% rock retained on the 19.0 mm sieve is compacted in a 152 mm diameter mould. Corrections for oversize are not directly included in this method but are detailed in Test Method Q140A when required for compaction control.
DTMR (2022*b*)

Q145A: *Laboratory compaction to nominated levels of dry density and moisture content*:

This method describes the procedure for compacting specimens to a nominated dry density and nominated moisture content when specimens are required by a reference test method for further testing. The nominated levels of dry density and/or moisture content often relate to some percentage of the MDD and/or OMC or DoS respectively.
The procedure relies on the reference method to provide essential procedural information such as apparatus and compaction details.
DTMR (2022*b*)

Q251A: *Preparation and compaction of laboratory mixed stabilised materials*:

This method describes the procedure to prepare and compact UCS specimens of soils and crushed rock which have been either modified or stabilised with a stabilising agent. The method has particular application as a laboratory design procedure. In the laboratory, prepare test specimens by compacting passing 19.0 mm material by standard or modified compactive effort or to a nominated dry density and moisture content, as detailed in Test Method Q145A. Where density/moisture parameters are not directly specified, use standard compactive effort.
DTMR (2022*b*)

Q135B: *Curing moulded specimens of stabilised material*:

This method describes the procedures for curing laboratory and field moulded specimens containing stabilising agents under standard conditions. The procedure provides for the curing of specimens which are demoulded before testing.
DTMR (2022*b*)

Q115: *UCS of stabilised materials*:

This method describes the procedure to determine the UCS of compacted specimens of soils, crushed rock and recycled material blends which have been either modified or stabilised with a stabilising agent or are in their natural state. The method has application as a laboratory design procedure, testing field-moulded specimens in order to check field processes, testing laboratory-moulded samples of soils or recycled material blends or core specimens removed from a stabilised material by dry coring.
DTMR (2022*b*)

Q113C: *California Bearing Ratio of soil at nominated levels of dry density and moisture content*:

This method sets out the procedure for the single point determination of the CBR of soils used for estimating design subgrade strength. California Bearing Ratio is the ratio of the force required to cause a circular plunger of 1932 mm^2 area to penetrate the material for a specified distance, expressed as a percentage of a standard force. The standard forces used in this method are 13,200 and 19,800 newtons for penetrations of 2.5 and 5.0 mm respectively.
Prepare test specimens by compacting passing 19.0 mm material to a nominated dry density and moisture content using standard compactive effort in accordance with Test Method Q145A. They are then tested either in a soaked or unsoaked condition. The duration of soaking is 4 days.
DTMR (2022*b*)

D.2 Approximate Site Location

9/27/23, 8:54 PM 27°03'58.9"S 151°51'11.4"E - Google Maps

Google Maps 27°03'58.9"S 151°51'11.4"E

Approximate Location of the Site

Imagery ©2023 TerraMetrics, Map data ©2023 Google 2 km

27°03'58.9"S 151°51'11.4"E

Directions Save Nearby Send to phone Share

Thornville QLD 4352

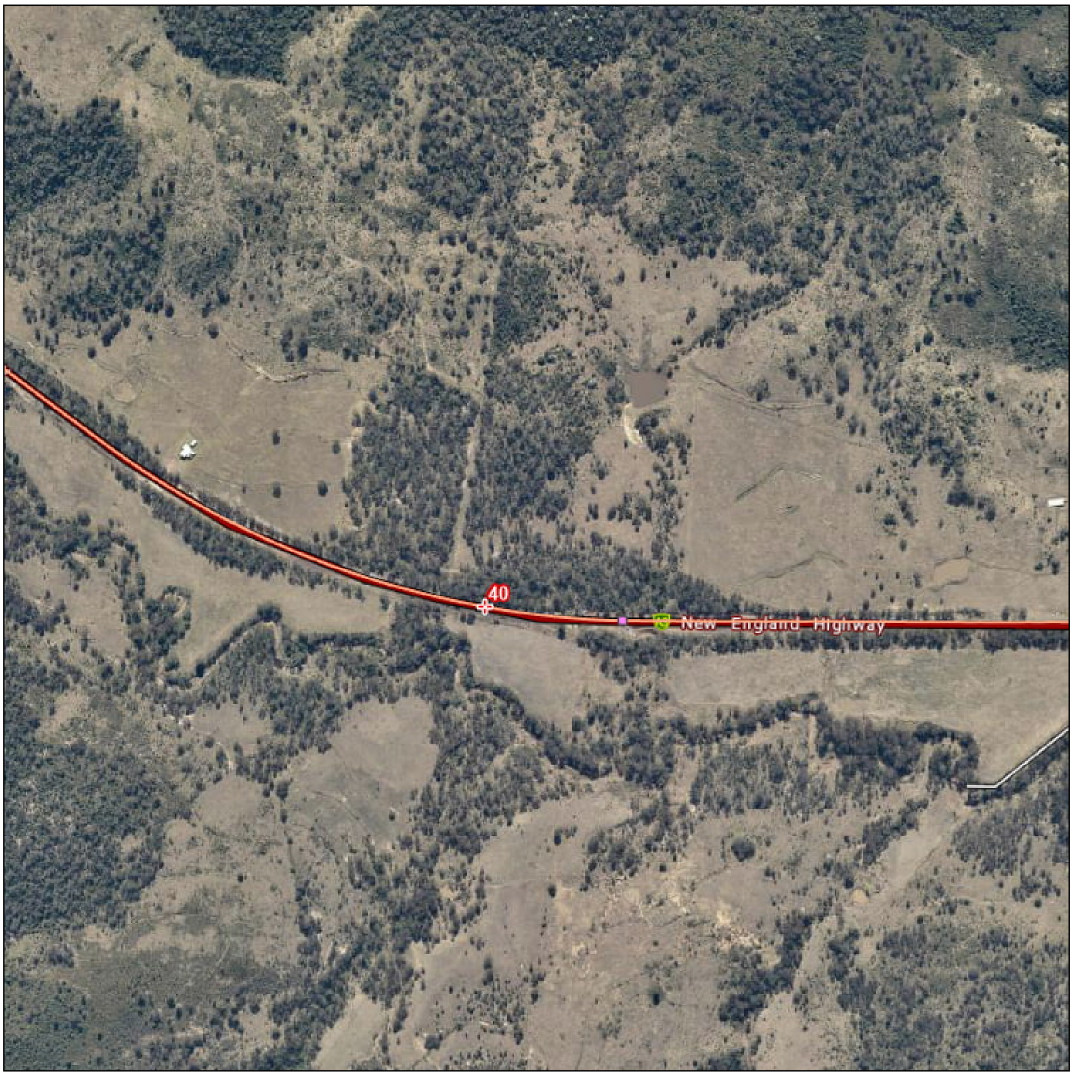
WVM3+F73 Thornville, Queensland

https://www.google.com/maps/place/27°03'58.9"S+151°51'11.4"E/@-27.0491626,151.8288771,21046m/data=!3m1!1e3!4m4!3m3!8m2!3d-27.066... 1/1

D.3 Satelite View of Test Pit Location

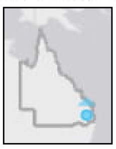
Approximate Site Location

27°3'26"S 151°50'39"E 27°3'26"S 151°51'43"E



27°4'23"S 151°50'39"E 27°4'23"S 151°51'43"E

A product of
Queensland Globe



Legend located on next page

N

0 250 metres

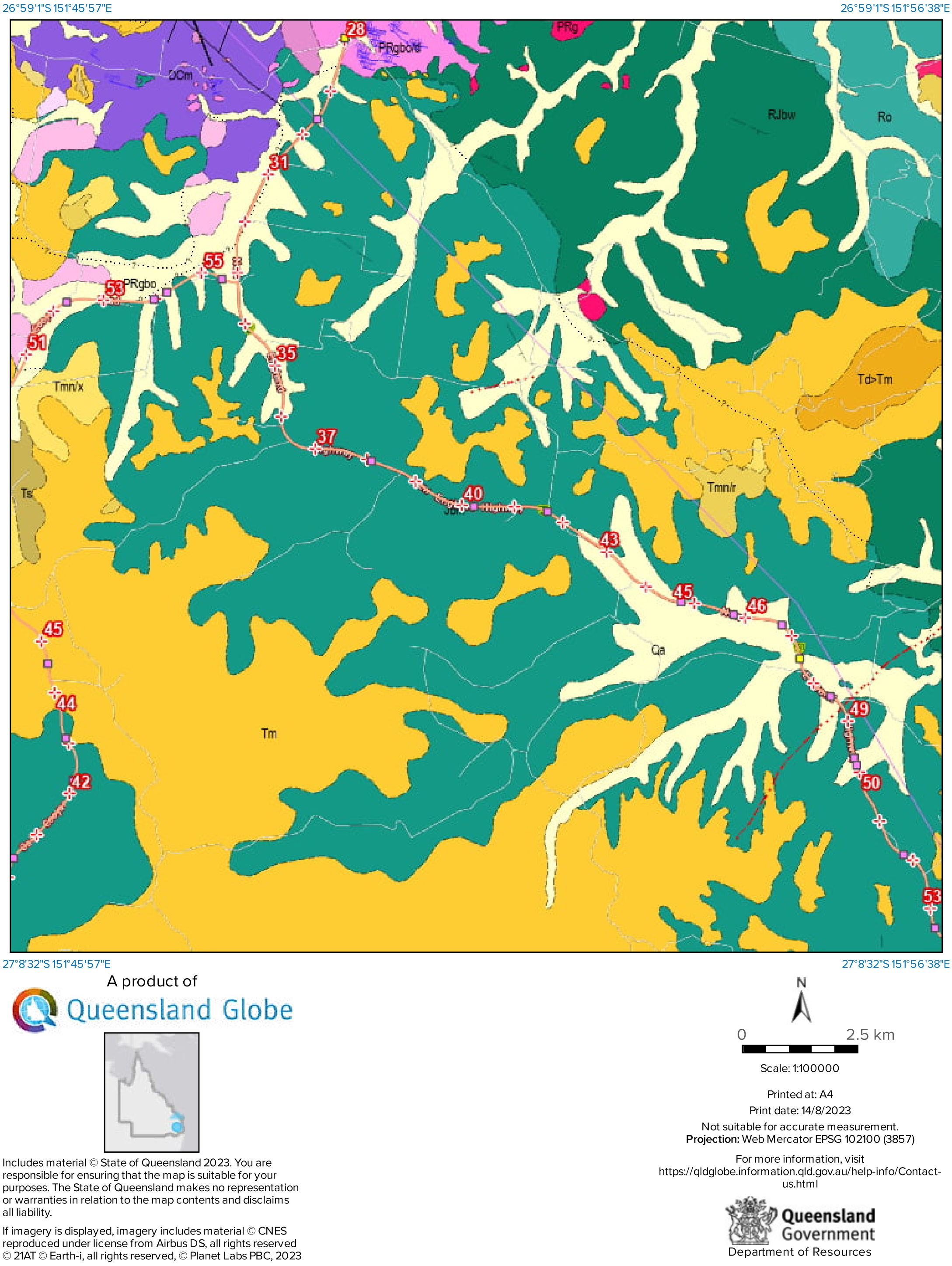
Scale: 1:10000

Printed at: A4
Print date: 14/8/2023
Not suitable for accurate measurement.
Projection: Web Mercator EPSG 102100 (3857)

For more information, visit
https://qldglobe.information.qld.gov.au/help-info/Contact-us.html

Queensland Government
Department of Resources

D.4 Surface Geology Map



D.5 Design Traffic for Data Analysis and Modelling

Design traffic is an estimation of the number of standard single axles with dual tyres that will apply a design load of 80 kN to the pavement over the design life of the pavement (Austroads, 2017). When expressed in Equivalent Standard Axles (ESA), it is a primary input by which the contemporary empirical and mechanistic-empirical pavement design methods determine the expected performance of a select pavement and sub-grade configuration. The summary of the steps taken to prepare the design traffic for the flexible and rigid pavement design is reproduced in Figure D.1.

The following section will outline the steps taken to calculate the design traffic for the section of New England Highway (22A) at an approximate chainage of 40.04 km located south of Cooyar, following the procedure presented by Austroads (2017, chap. 7) with the additions by DTMR (2021*d*, chap. 7), and using publicly available traffic data published by Austroads (2017) and DTMR (2022*a*).

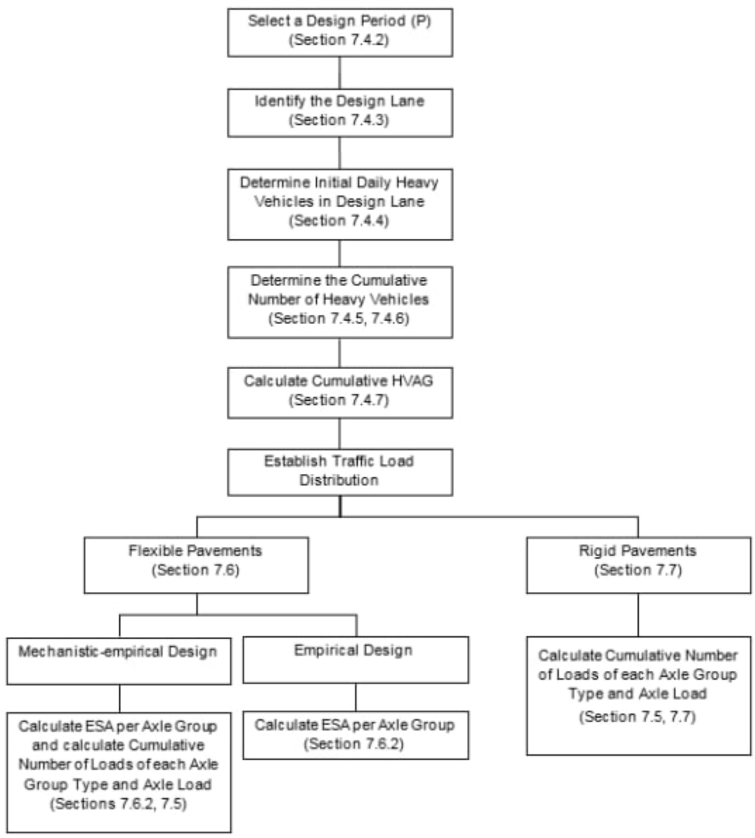


Figure D.1: Procedure for determining design traffic, reproduced from Austroads (2017, p.95).

D.5.1 Design Lane

Before the design traffic can be calculated, the design lane along which traffic will travel through the design period will need to be described. As can be seen from Figure 1.2, the section of the New England Highway (22A) for which pavement design will be carried out is a two-lane two-way road and has a posted speed limit of 100 km/hr. The results of the traffic census carried out by DTMR (2022*a*) in 2021 on the closest section of the New England Highway (22A) between chainages of 19 and 33.19 km are presented in Table D.1. From the data shown the following characteristics can be attributed to the design lane:

$\mathbf{AADT_{21}}$ − The estimated Annual Average Daily Traffic during census through the section in both directions is **1943** vehicles per day.

i − The vehicle growth rate per annum is **2.16** %, based on the Annual Growth from five years previous to the 2021 count. Five-year growth was chosen over the ten-year growth as a more conservative value because it had a larger value. The one-year growth value was ignored, since vehicle counts in 2019 and 2020 have been greatly affected by COVID-19 restrictions (International Energy Agency, 2020), resulting in inflated growth values between 2020 and 2021 census data.

DF − The traffic flow in both directions is approximately equivalent, making the Direction Gactor equal to **0.5**.

%HV − The average percentage of heavy vehicles based on the data for both directions of travel is approximately $\frac{AADT_{0B}}{AADT} * 100 = \frac{467.0972}{1943} * 100 = \mathbf{24.04}\%$.

LDF − As the highway configuration only has one lane in each direction, the Lane Distribution Factor for the design lane is **1**.

Design Period − As the AADT total in both directions is below 30,000 DTMR (2021*d*, chap. 7.4.2), design period of **25** years will be adopted.

Travel Direction	AADT	$AADT_{0A}$	$AADT_{0B}$	i_{1yr}	i_{5yr}	i_{10yr}
Against Gazettal	971	701.9359	269.0641	19.58	2.16	1.66
Both Directions	1943	1475.9028	467.0972	19.2	2.17	1.71
With Gazettal	972	773.4204	198.5796	18.83	2.18	1.77

Table D.1: New England Highway (22A - Yarraman to Toowoomba) 2021 Traffic Census Data, reproduced from DTMR (2022*a*).

The classified traffic count of the 2021 DTMR (2022*a*) census was analysed using the spreadsheet (DTMR, 2021*b*) provided as part of the DTMR (2021*d*, app. E) supplement. The Class-Specific Traffic Load Distributions (CTLD) output of the spreadsheet based on a presumptive CTLD is available in the Appendix D.6. The CTLD data also provide two factors that will be needed in the calculations of the design traffic. These factors are the average number of heavy vehicle axle groups ($\mathbf{N_{HVAG}}$) per heavy vehicle which is **2.49** HVAG and the average number of equivalent standard axles (**ESA/HVAG**) per heavy vehicle axle group of **0.74** ESA.

As there is no publicly available WIM data for the Queensland section of the New England Highway (Austroads, 2017, app. E), the presumptive CTLD was used for the above calculations as a conservative alternative to the data available from the nearby WIM sites at Gatton, Southbrook, and Oakey.

D.5.2 Initial Daily Heavy vehicles in the Design Lane

The pavement design undertaken will be based on an opening year of 2024. Since the latest publicly available traffic census data is from 2021, before the design traffic calculations can be made, the AADT must be projected 3 years forward using the reported growth rate.

$$\begin{aligned} AADT_{24} &= (1 + 0.01i)^x * AADT_{21} \\ AADDT_{24} &= (1 + 0.01 * 2.16)^3 * 1943 \approx 2072 \text{ V} \end{aligned} \tag{D.1}$$

Therefore, the expected initial number of heavy vehicles daily in the design lane is as follows.

$$\begin{aligned} N_i &= AADT_{24} * DF * \%HV/100 * LDF \\ N_i &= 2072 * 0.5 * 24.04/100 * 1 \approx 249 \text{ HV} \end{aligned} \tag{D.2}$$

D.5.3 Design Number of Equivalent Standard Axless

The first part of the design traffic calculations is the estimation of the total number of heavy vehicles expected to travel in the design lane throughout the design life of the pavement, accounting for a yearly growth in the daily traffic of heavy vehicles. As New England Highway section 22A is not a key freight route, the annual growth rate of heavy vehicles will be presumed to be 2% (DTMR, 2021*d*, chap. 7.4.5). Therefore, the cumulative number of heavy vehicles when under capacity is as follows.

$$\begin{aligned} N_{HV} &= 365 * CGF * N_i = 365 * \frac{(1+0.01i)^P - 1}{0.01i} * N_i \\ N_{HV} &= 365 * \frac{(1+0.01*2)^{25} - 1}{0.01*2} * 249 \approx 2,911,074 \text{ HV} \end{aligned} \tag{D.3}$$

As the value of N_{HV} does not exceed 10^7 heavy vehicles, the value of the cumulative number of heavy vehicles considering capacity will not need to be checked (Austroads, 2017, chap. 7.4.5).

The next step in the design traffic calculations is to determine a cumulative number of heavy vehicle axle groups, based on the total number of heavy vehicles and the average of axle groups calculated from classified traffic counts.

$$\begin{aligned} N_{DT} &= N_{HV} * N_{HVAG} \\ N_{DT} &= 2,911,074 * 2.49 \approx 7,248,575 \text{ HVAG} \end{aligned} \tag{D.4}$$

Finally, the design traffic expressed in the equivalent standard axles of traffic loading can be calculated using the cumulative number of axle groups expected to traverse the design lane and the average number of equivalent standard axles per heavy vehicle axle group calculated using the presumptive CTLD and classified traffic counts.

$$
\begin{aligned}
DESA &= ESA/HVAG * N_{DT} \\
DESA &= 0.74 * 7,248,575 \approx 5,363,946 \text{ ESA}
\end{aligned}
\tag{D.5}
$$

Rounding up the value calculated in the Equation D.5 leads to a design traffic of $5.4 * 10^6$ ESA with a traffic load distribution as presented in Appendix D.6.

D.6 Class Specific Traffic Load Distributions

The following two pages are a print copy of the DTMR (2021*b*) Class-Specific Traffic Load Distributions spreadsheet, including recombined Weight-in-Motion data and Traffic Load distributions used in calculations of the design traffic and pavement analysis with CIRCLY respectively.

Class-Specific Traffic Load Distributions Spreadsheet (Transport and Main Roads, July 2021)

This is the calculation worksheet. Follow Steps 1 to 4 below to fill in the yellow cells. The resulting traffic load distribution can then be exported for use in pavement design.

STEP 1: Insert project details

Project site details	Project Name	USQ Student Project
	Road Section(s)	New England Highway 22A - CH40.04km
	Project Info	

STEP 2: Insert classified traffic count details

Classified Traffic Count site details	Road ID/Name	NEW ENGLAND HIGHWAY (YARRAMAN - TOOWOOMBA) (22A)	Chainage	19-33.19km
	Site ID	30083	Traffic Lane	All Lanes
	Site Location	22A-20m South Boyde Crk Culvert	Data Year	2021

Classification Type: Select either 12 bin or 4 bin classification inputs from the drop-down list (12 bin preferred) (click cell I14) — 12 bin

Classified Traffic Counts or Proportions: Based on your selection above, enter either 12 bin or 4 bin inputs below. Enter as either counts or proportions (% or decimal).

	Class 3	Class 4	Class 5	Class 6	Class 7	Class 8	Class 9	Class 10	Class 11	Class 12
Counts/proportions (12 bin)	274.7	19.6	4.5	21.4	35 2	6.0	48.4	55.8	1.6	0.0
Counts/proportions (4 bin)										
Heavy Vehicle Proportions	58.82%	4.20%	0.96%	4.58%	7.53%	1.29%	10.36%	11.94%	0.33%	0.00%

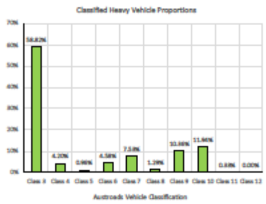
STEP 3: Select a WIM site from the drop-down menu (click cell A24)

District - Road ID - Lane - WIM Site Description	Road Name	Chainage	WIM Site ID & Lane
Presumptive CTLD (2015-2019)	Presumptive Class-specific Traffic Load Distributions (Year 2015 - 2019)	N/A	Presumptive CTLD

		Weigh-in-Motion data based on selected WIM site details									
	Recombined	Class 3	Class 4	Class 5	Class 6	Class 7	Class 8	Class 9	Class 10	Class 11	Class 12
NHVAG	2.49	2.00	2.00	2.00	3.00	3.06	3.03	3.03	4.00	5.05	7.10
ESA / HVAG	0.74	0.53	1.05	1.24	0.30	0.60	0.84	1.02	1.20	1.02	0.78
ESA / HV	1.85	1.05	2.10	2.47	0.90	1.84	2.54	3.07	4.80	5.15	5.54
WIM recorded heavy vehicles		5,436,572	3,009,984	890,639	477,600	788,191	1,112,627	8,089,124	4,387,007	601,687	131,242

STEP 4: Click the "Export Recombined TLD" button to export the recombined TLD to a Comma Separated Values (CSV) file

Alternatively, the recombined TLD can be copied from the cells below in AGPT02 format (cells F89:L141).



	Recombined - Traffic Load Distribution - AustPADS / CIRCLY format					
Load (kN)	SAST	SADT	TAST	TADT	TRDT	QADT
10	0.000282	0.004693		0.000765	0.000071	0.000003
20	0.018272	0.008994	0 000002	0.009410	0.000055	0.000001
30	0.100756	0.051258	0 000020	0.006664	0.000273	0.000001
40	0.081725	0.067490	0 000060	0.010512	0.001418	0.000001
50	0.077835	0.058323	0 000137	0.012399	0.004697	0.000002
60	0.079203	0.046321	0 000364	0.016961	0.008276	0.000003
70	0.031542	0.027993	0 000637	0.018827	0.010982	0.000005
80	0.005158	0.018249	0 000764	0.016252	0.010193	0.000010
90	0.001625	0.011716	0 000779	0.011852	0.007155	0.000013
100		0.006392	0 000665	0.008661	0.005157	0.000020
110		0.003297	0 000562	0.007410	0.003930	0.000022
120		0.001651	0 000335	0.007192	0.003229	0.000008
130		0.001080	0 000141	0.008070	0.002893	0.000007
140		0.000627	0 000069	0.008758	0.002744	0.000006
150		0.000390	0 000046	0.009864	0.002950	0.000007
160		0.000209	0 000038	0.012406	0.003481	0.000010
170		0.000163	0 000026	0.011731	0.004009	0.000012
180		0.000120		0.006347	0.004968	0.000014
190				0.002650	0.006068	0.000014
200				0.001225	0.007053	0.000013
210				0.000647	0.006559	0.000016
220				0.000322	0.004331	0.000019
230				0.000180	0.002569	0.000024
240				0.000106	0.001340	0.000023
250				0.000069	0.000763	0.000020
260				0.000046	0.000458	0.000015
270				0.000026	0.000261	0.000010
280				0.000013	0.000161	0.000006
290				0.000006	0.000103	0.000004
300				0.000003	0.000066	0.000004
310				0.000002	0.000047	0.000003
320				0.000001	0.000029	0.000002
330				0.000001	0.000019	0.000001
340					0.000011	0.000001
350					0.000007	0.000001
360					0.000005	0.000001
370					0.000003	0.000001
380					0.000002	0.000001
390					0.000001	0.000001
400					0.000001	0.000001
410						0.000001
420						0.000001
430						0.000001
440						
450						
460						
470						
480						
490						
500						

Export Recombined TLD

Appendix E

NATA Accredited Test Reports

E.1 Material Test Reports

Materials Services - Toowoomba
Department of Transport and Main Roads
427 Greenwattle Street, TOOWOOMBA QLD 4350

Phone: (07) 4699 9329

www.tmr.qld.gov.au

Queensland Government

Materials Test Report

Report No: MAT:TWB22W-0217-S01-1
Issue No: 1

Client: TMR Pavements, Materials and Geotechnical
Level 2 Bulwer Island Office Building
[redacted]

Project: 2023 USQ Student Project
Location:

Accredited for compliance with ISO/IEC 17025 - Testing

NATA

[redacted]

NATA Accredited Laboratory Number 2302

Approved Signatory: Ben Kratzmann (Materials Technologist)
Date of Issue: 20/06/2023

THIS DOCUMENT SHALL NOT BE REPRODUCED EXCEPT IN FULL

Sample Details

Sample ID:	TWB22W-0217-S01
Sampling Method:	AS 1289.1.2.1 Cl 6.5.4
Date Sampled:	07/12/2022
Sampled By:	This lab
Source:	Insitu (Existing Material)
Material:	Sub Grade
Sampled From:	Test Pit
Specification:	Nil
Location:	CH 40.04km LHS
Material Description:	Black Clay

Other Test Results

Description	Method	Result	Limits
Atterberg Limits [AS 1289.3.1.1, AS 1289.3.2.1, AS 1289.3.3.1, AS 1289.3.4.1]			
Sample History	AS 1289.1.1		
Preparation	AS 1289.1.1	Dry Sieved	
Linear Shrinkage (%)	AS 1289.3.4.1	22.0	
Mould Length (mm)		250.6	
Crumbling		No	
Curling		Yes	
Cracking		No	
Liquid Limit (%)	AS 1289.3.1.1	80	
Method		Four Point	
Plastic Limit (%)	AS 1289.3.2.1	26	
Plasticity Index (%)	AS 1289.3.3.1	54	
Date Tested		26/04/2023	
Lime Demand of Soil [Q133]			
Lime Demand (%):		4.0	
Date Tested		28/04/2023	
Atterberg Limits Casagrande [Q252]			
Sample History	AS 1289.1.1		
Preparation	AS 1289.1.1	Dry Sieved	
Linear Shrinkage (%)	AS 1289.3.4.1	22.0	
Mould Length (mm)		250.6	
Crumbling		No	
Curling		Yes	
Cracking		No	
Liquid Limit (%)	AS 1289.3.1.2	80	
Plastic Limit (%)	AS 1289.3.2.1	26	
Plasticity Index (%)	AS 1289.3.3.1	54	
Passing 0.425 mm sieve	AS 1289.3.6.1	96	
Weighted Plasticity Index		5176	
Date Tested		26/04/2023	

Particle Size Distribution

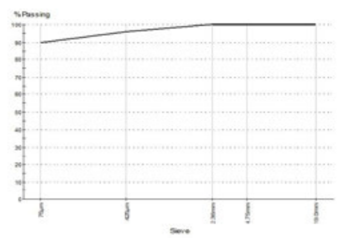
Method: Grading [AS 1289.3.6.1]

Date Tested: 19/04/2023

Note: Sample Washed

Sieve Size	% Passing	Limits
19.0mm	100	
4.75mm	100	
2.36mm	100	
425µm	96	
75µm	90	

Chart



Comments

Variation to AS1289.3.6.1 - material washed passing 4.75mm sieve
Results by AS4489.6.1 Lime Index - Available Lime obtained from TMR Bulwer Island Accred no. 2302. Report number MAT:BIL23W-0094-S01-1.
Result: 81.68%

Queensland Government

Materials Test Report

Materials Services - Brisbane
Department of Transport and Main Roads
Bulwer Island Laboratory
398 Tingira Street
Pinkenba, Qld, 4008

Telephone: (07) 3066 3345

Report No: MAT:BIL23W-0150-S01-1
Issue No: 1

Client: TMR Toowoomba
[redacted]
Toowoomba QLD 4350

Project: Materials Testing
Location:

Accredited for compliance with ISO/IEC 17025 - Testing

NATA
[redacted]

NATA Accredited Laboratory Number 2302
Approved Signatory: Robyn Devitt (Chemist)
Date of Issue: 19/05/2023

Sample Details

Sample ID:	BIL23W-0150-S01
Client Sample ID:	TWB22W-0217-1
Sampling Method:	Sample tested as received
Date Sampled:	07/12/2022
Sampled By:	TMR Staff
Source:	Insitu subgrade
Material:	
Location:	22A New England Highway
TRN:	52-02035312.O.OW.2.2
Chainage:	Ch 40.04 km
Material Description:	Black soil subgrade

Test Results

Description	Method	Result	Limits
Sulfate Content of Natural Soil and Groundwater (normal method) [AS 1289.4.2.1]			
Sulfate Content of Soil (%):		0.01	
Sulfate Content of Groundwater (g/l):			
Date Tested		11/05/2023	
Organic Content of Soil - Loss on Ignition [Q120B]			
Organic Content (%):		10.50	
Date Tested		11/05/2023	

Comments

WBS: 52-02035312.O.OW.2.2

Australian Laboratory Services Pty. Ltd.
82 Shand Street
Stafford
Brisbane QLD 4058
Phone: +61 7 8248 7222 Fax: +61 7 8248 7218
www.alsglobal.com/geochemistry

To: DEPARTMENT OF TRANSPORT AND MAIN ROADS
PMG DIVISION
35 BUTTERFIELD STREET
HERSTON QLD 4006

Finalized Date: 19-MAY-2023
Account: MAIROAPMG

CERTIFICATE BR23125202

Project: TWB22W-0217

This report is for 1 sample of Soil submitted to our lab in Brisbane, QLD, Australia on 10-MAY-2023.

The following have access to data associated with this certificate:

BELINDA J WATERS		

SAMPLE PREPARATION

ALS CODE	DESCRIPTION
LEV-01	Waste Disposal Levy
DRY-22	Drying - Maximum Temp 60C
LOG-22	Sample login - Rcd w/o BarCode
WEI-21	Received Sample Weight
PUL-21	Pulverize entire sample
TRA-21	Transfer sample
BAG-01	Bulk Master for Storage
PUL-QC	Pulverizing QC Test

ANALYTICAL PROCEDURES

ALS CODE	DESCRIPTION	INSTRUMENT
Fe-VOL05	FeO (Ferrous Iron)	

This is the Final Report and supersedes any preliminary report with this certificate number.Results apply to samples as submitted.All pages of this report have been checked and approved for release.
***** See Appendix Page for comments regarding this certificate *****

Signature: [redacted]

Shaun Kenny, Brisbane Laboratory Manager

Australian Laboratory Services Pty. Ltd.
32 Shand Street
Stafford
Brisbane QLD 4053
Phone: +61 7 3243 7222 Fax: +61 7 3243 7218
www.alsglobal.com/geochemistry

To: DEPARTMENT OF TRANSPORT AND MAIN ROADS
PMG DIVISION
35 BUTTERFIELD STREET
HERSTON QLD 4006

Finalized Date: 19-MAY-2023
Account: MAIROAPMG

Project: TWB22W-0217

CERTIFICATE OF ANALYSIS BR23125202

Sample Description	Method / Analyte / Units / LOD	WEI-21 Recvd Wt. kg 0.02	PUL-QC Pass75um % 0.01	Fe-VOL05 FeO % 0 01
TWB22W-0217.1		2.39	97.3	0.66

***** See Appendix Page for comments regarding this certificate *****

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Stafford
Brisbane QLD 4053
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www.alsglobal.com/geochemistry

To: DEPARTMENT OF TRANSPORT AND MAIN ROADS
PMG DIVISION
35 BUTTERFIELD STREET
HERSTON QLD 4006

Finalized Date: 19-MAY-2023
Account: MAIROAPMG

Project: TWB22W-0217

CERTIFICATE OF ANALYSIS BR23125202

CERTIFICATE COMMENTS

LABORATORY ADDRESSES

Processed at ALS Brisbane located at 32 Shand Street, Stafford, Brisbane, QLD, Australia. Processed at ALS Brisbane Sample Preparation at 23 Pineapple Street, Zillmere, QLD, 4034, Australia

Applies to Method:				
	BAG-01	DRY-22	Fe-VOL05	LEV-01
	LOG-22	PUL-21	PUL-QC	TRA-21
	WEI-21			

E.2 Maximum Dry Density Test Reports

Queensland Government

Materials Services - Toowoomba
Department of Transport and Main Roads
427 Greenwattle Street, TOOWOOMBA QLD 4350

Phone: (07) 4699 9329

www.tmr.qld.gov.au

Maximum Dry Density Report

Report No: MDD:TWB22W-0217-S01-1
Issue No: 1

Client: TMR Pavements, Materials and Geotechnical
Level 2 Bulwer Island Office Building
[redacted]

Project: 2023 USQ Student Project
Location:

Accredited for compliance with ISO/IEC 17025 - Testing

NATA

[redacted]

NATA Accredited Laboratory Number 2302

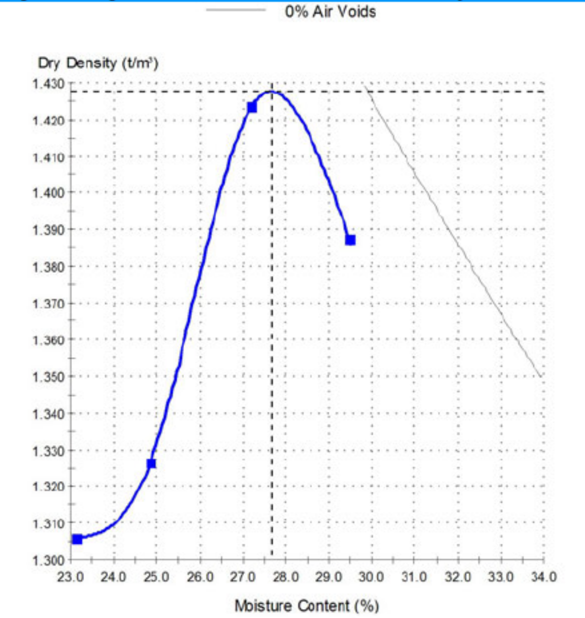
Approved Signatory: Ben Kratzmann (Materials Technologist)
Date of Issue: 20/06/2023

THIS DOCUMENT SHALL NOT BE REPRODUCED EXCEPT IN FULL

Sample Details

Sample ID:	TWB22W-0217-S01		
Sampling Method:	AS 1289.1.2.1 Cl 6.5.4		
Date Sampled:	7/12/2022	**Sampled By:**	This lab
Source:	Insitu (Existing Material)	**Material:**	Sub Grade
Sampled From:	Test Pit		
Specification:	Nil		
Location:	CH 40.04km LHS		
Material Description:	Black Clay	**Tested By:**	Philip Franke
Date Tested:	28/04/2023		

Dry Density - Moisture Content Relationship



Test Results

Maximum Dry Density - Standard [AS 1289.5.1.1]

Standard MDD (t/m³):	**1.43**
Standard OMC (%):	**27.5**
Retained Sieve (mm):	
Oversize Material (%):	
Curing Time (h):	241
LL Method:	AS 1289.3.1.1

Comments

Queensland Government

Materials Services - Toowoomba
Department of Transport and Main Roads
427 Greenwattle Street, TOOWOOMBA QLD 4350

Phone: (07) 4699 9329

www.tmr.qld.gov.au

Maximum Dry Density Report

Report No: MDD:TWB22W-0217-S03-3
Issue No: 1

Client: TMR Pavements, Materials and Geotechnical
Level 2 Bulwer Island Office Building
[redacted]

Project: 2023 USQ Student Project
Location:

Accredited for compliance with ISO/IEC 17025 - Testing

NATA

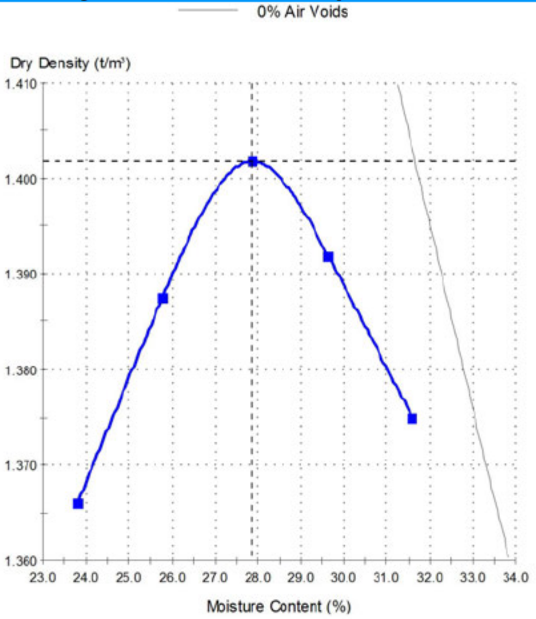
[redacted]

NATA Accredited Laboratory Number 2302
Approved Signatory: Ben Kratzmann (Materials Technologist)
Date of Issue: 20/06/2023

Sample Details

Sample ID:	TWB22W-0217-S03		
Sampling Method:	AS 1289.1.2.1 Cl 6.5.4		
Date Sampled:	7/12/2022	**Sampled By:**	This lab
Source:	Insitu (Existing Material)	**Material:**	Sub Grade
Sampled From:	Test Pit		
Specification:	Nil		
Location:	CH 40.04km LHS		
Material Description:	Black Clay with 3% Hydrated Lime	**Tested By:**	Philip Franke
Date Tested:	3/05/2023		

Dry Density - Moisture Relationship



Test Results

Maximum Dry Density - Standard [Q142A]

Standard MDD (t/m³):	**1.40**
Standard OMC (%):	**28.0**
MC Test Method:	AS 1289.2.1.1
Retained Sieve (mm):	
Oversize Material (%):	
Additive Type:	Hydrated Lime
Additive Source:	Wagners
Additive Proportion (%):	3.0
Additive Sampled From:	Test Pit
Sample Preparation:	Q101

Comments

Materials Services - Toowoomba
Department of Transport and Main Roads
427 Greenwattle Street, TOOWOOMBA QLD 4350

Phone: (07) 4699 9329

www.tmr.qld.gov.au

Queensland Government

Maximum Dry Density Report

Report No: MDD:TWB22W-0217-S05-2
Issue No: 1

Client: TMR Pavements, Materials and Geotechnical
Level 2 Bulwer Island Office Building
[redacted]

Project: 2023 USQ Student Project
Location:

Accredited for compliance with ISO/IEC 17025 - Testing

NATA

[redacted]

NATA Accredited Laboratory Number 2302

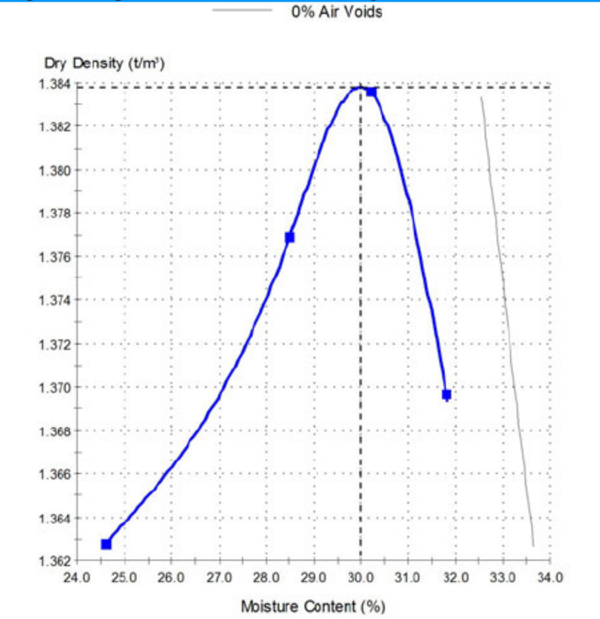
Approved Signatory: Ben Kratzmann (Materials Technologist)
Date of Issue: 20/06/2023

THIS DOCUMENT SHALL NOT BE REPRODUCED EXCEPT IN FULL

Sample Details

Sample ID:	TWB22W-0217-S05		
Sampling Method:	AS 1289.1.2.1 Cl 6.5.4		
Date Sampled:	7/12/2022	**Sampled By:**	This lab
Source:	Insitu (Existing Material)	**Material:**	Sub Grade
Sampled From:	Test Pit		
Specification:	Nil		
Location:	CH 40.04km LHS		
Material Description:	Black Clay with 6% Hydrated Lime	**Tested By:**	Philip Franke
Date Tested:	3/05/2023		

Dry Density - Moisture Relationship



Test Results

Maximum Dry Density - Standard [Q142A]

Standard MDD (t/m³):	**1.38**
Standard OMC (%):	**30.0**
MC Test Method:	AS 1289.2.1.1
Retained Sieve (mm):	
Oversize Material (%):	
Additive Type:	Hydrated Lime
Additive Source:	Wagners
Additive Proportion (%):	6.0
Additive Sampled From:	Test Pit
Sample Preparation:	Q101

Comments

E.3 CBR Test Reports

Materials Services - Toowoomba
Department of Transport and Main Roads
427 Greenwattle Street, TOOWOOMBA QLD 4350

Phone: (07) 4699 9329

www.tmr.qld.gov.au

Queensland Government

California Bearing Ratio Test Report

Report No: CBR:TWB22W-0217-S01-2
Issue No: 1

Client: TMR Pavements, Materials and Geotechnical
Level 2 Bulwer Island Office Building
[redacted]

Project: 2023 USQ Student Project
Location:

Accredited for compliance with ISO/IEC 17025 - Testing

NATA

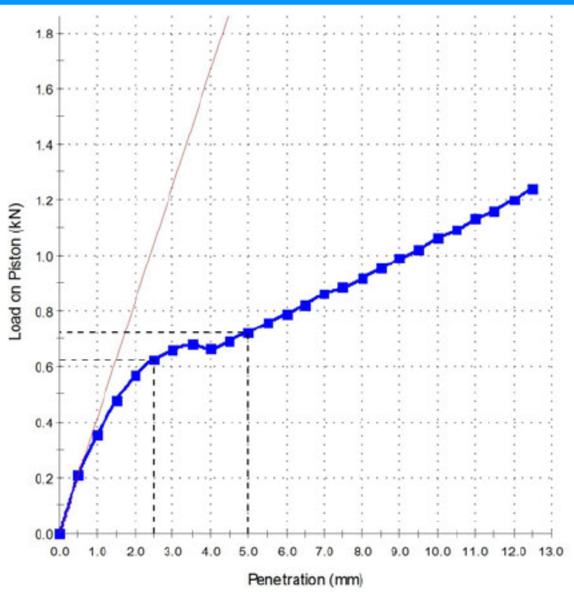
[redacted]

NATA Accredited Laboratory Number 2302
Approved Signatory: Ben Kratzmann (Materials Technologist)
Date of Issue: 20/06/2023

Sample Details

Sample ID:	TWB22W-0217-S01		
Sampling Method:	AS 1289.1.2.1 Cl 6.5.4		
Date Sampled:	7/12/2022	**Sampled By:**	This lab
Source:	Insitu (Existing Material)	**Material:**	Sub Grade
Sampled From:	Test Pit	**Specification:**	Nil
Location:	CH 40.04km LHS	**Material Description:**	Black Clay
Tested By:	Philip Franke	**Date Tested:**	29/05/2023

Load vs Penetration



Test Results

California Bearing Ratio [AS 1289.6.1.1]

CBR at 2.5mm (%):	**4.5**
Dry Density before Soaking (t/m³):	1.39
Density Ratio before Soaking (%):	97.5
Moisture Content before Soaking (%):	27.1
Moisture Ratio before Soaking (%):	98.0
Dry Density after Soaking (t/m³):	1.35
Density Ratio after Soaking (%):	94.5
Swell (%):	3.0
Moisture Content of Top 30mm (%):	35.3
Moisture Content of Remaining Depth (%):	28.7
Compaction Hammer Used:	Standard AS 1289.5.1.1
Surcharge Mass (kg):	4.51
Period of Soaking (Days):	4
Retained on 19 mm Sieve (%):	
CBR Moisture Content Method:	AS 1289.2.1.1
Sample Curing Time (h):	144
Plasticity Determination Method:	AS 1289.3.1.2

Comments

Materials Services - Toowoomba
Department of Transport and Main Roads
427 Greenwattle Street, TOOWOOMBA QLD 4350

Phone: (07) 4699 9329

www.tmr.qld.gov.au

Queensland Government

California Bearing Ratio Test Report

Report No: CBR:TWB22W-0217-S02-1
Issue No: 1

Client: TMR Pavements, Materials and Geotechnical
Level 2 Bulwer Island Office Building
[redacted]

Project: 2023 USQ Student Project
Location:

Accredited for compliance with ISO/IEC 17025 - Testing

NATA

[redacted]

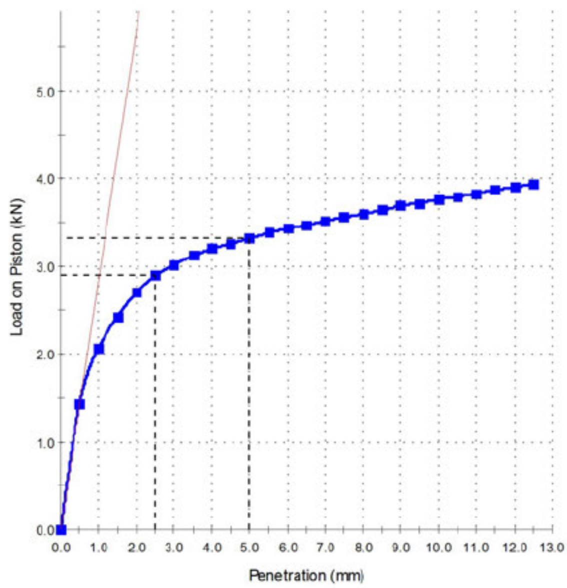
NATA Accredited Laboratory Number 2302

Approved Signatory: Ben Kratzmann (Materials Technologist)
Date of Issue: 20/06/2023

Sample Details

Sample ID:	TWB22W-0217-S02		
Sampling Method:	AS 1289.1.2.1 Cl 6.5.4		
Date Sampled:	7/12/2022	**Sampled By:**	This lab
Source:	Insitu (Existing Material)	**Material:**	Sub Grade
Sampled From:	Test Pit	**Specification:**	Nil
Location:	CH 40.04km LHS	**Material Description:**	Black Clay with 2% Hydrated Lime
Tested By:	Kevin Bennett	**Date Tested:**	12/05/2023

Load vs Penetration



Test Results

California Bearing Ratio [AS 1289.6.1.1]

CBR at 2.5mm (%):	**20**
Dry Density before Soaking (t/m³):	1.36
Density Ratio before Soaking (%):	97.0
Moisture Content before Soaking (%):	27.6
Moisture Ratio before Soaking (%):	99.0
Dry Density after Soaking (t/m³):	1.36
Density Ratio after Soaking (%):	96.5
Swell (%):	0.5
Moisture Content of Top 30mm (%):	31.4
Moisture Content of Remaining Depth (%):	32.9
Compaction Hammer Used:	Standard AS 1289.5.1.1
Surcharge Mass (kg):	4.51
Period of Soaking (Days):	4
Retained on 19 mm Sieve (%):	
CBR Moisture Content Method:	AS 1289.2.1.1
Sample Curing Time (h):	
Plasticity Determination Method:	AS 1289.3.1.2

Comments

Material cured per Q135A (2% Hydrated Lime used)
Moulded specimen cured per Q135B at 23 degrees Celcius for 24 days prior to soaking

Queensland Government

Materials Services - Toowoomba
Department of Transport and Main Roads
427 Greenwattle Street, TOOWOOMBA QLD 4350

Phone: (07) 4699 9329

www.tmr.qld.gov.au

California Bearing Ratio Test Report

Report No: CBR:TWB22W-0217-S03-1
Issue No: 1

Client: TMR Pavements, Materials and Geotechnical
Level 2 Bulwer Island Office Building
[redacted]

Project: 2023 USQ Student Project
Location:

Accredited for compliance with ISO/IEC 17025 - Testing

NATA

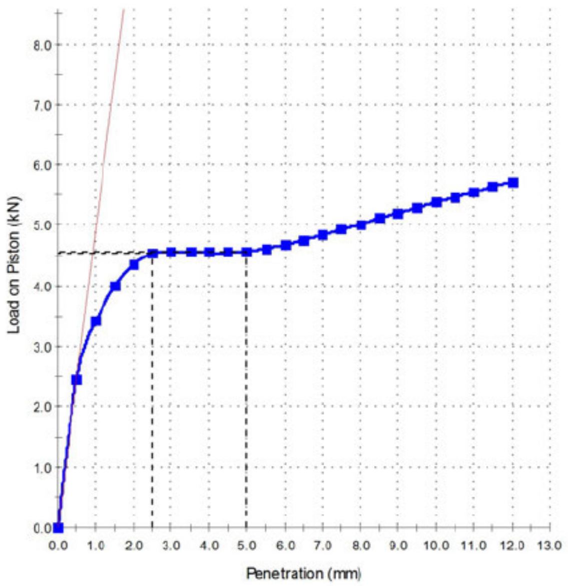
[redacted]

NATA Accredited Laboratory Number 2302
Approved Signatory: Ben Kratzmann (Materials Technologist)
Date of Issue: 20/06/2023

Sample Details

Sample ID:	TWB22W-0217-S03		
Sampling Method:	AS 1289.1.2.1 Cl 6.5.4		
Date Sampled:	7/12/2022	**Sampled By:**	This lab
Source:	Insitu (Existing Material)	**Material:**	Sub Grade
Sampled From:	Test Pit	**Specification:**	Nil
Location:	CH 40.04km LHS	**Material Description:**	Black Clay with 3% Hydrated Lime
Tested By:	Kevin Bennett	**Date Tested:**	12/05/2023

Load vs Penetration



Test Results

California Bearing Ratio [AS 1289.6.1.1]

CBR at 2.5mm (%):	**35**
Dry Density before Soaking (t/m³):	1.36
Density Ratio before Soaking (%):	97.0
Moisture Content before Soaking (%):	27.9
Moisture Ratio before Soaking (%):	100.0
Dry Density after Soaking (t/m³):	1.36
Density Ratio after Soaking (%):	96.5
Swell (%):	0.0
Moisture Content of Top 30mm (%):	29.4
Moisture Content of Remaining Depth (%):	30.6
Compaction Hammer Used:	Standard AS 1289.5.1.1
Surcharge Mass (kg):	4.51
Period of Soaking (Days):	4
Retained on 19 mm Sieve (%):	
CBR Moisture Content Method:	AS 1289.2.1.1
Sample Curing Time (h):	
Plasticity Determination Method:	AS 1289.3.1.2

Comments

Swell reading taken prior to air curing (Q135B) - 2.86mm
Variation to AS 1289.6.1.1 - Penetration terminated at 12mm due to software error
Curing of material prior to compaction in accordance with Q135A (3% Hydrated Lime used)
Moulded specimen cured at 23 degrees Celcius for 24 days prior to soaking

Queensland Government

Materials Services - Toowoomba
Department of Transport and Main Roads
427 Greenwattle Street, TOOWOOMBA QLD 4350

Phone: (07) 4699 9329

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Report No: CBR:TWB22W-0217-S04-1
Issue No: 1

California Bearing Ratio Test Report

Client: TMR Pavements, Materials and Geotechnical
Level 2 Bulwer Island Office Building
[redacted]

Project: 2023 USQ Student Project
Location:

Accredited for compliance with ISO/IEC 17025 - Testing

NATA

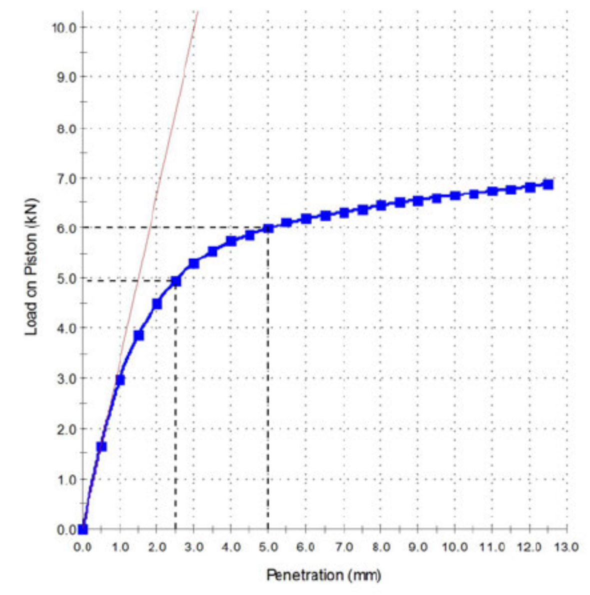
[redacted]

NATA Accredited Laboratory Number 2302
Approved Signatory: Ben Kratzmann (Materials Technologist)
Date of Issue: 20/06/2023

Sample Details

Sample ID:	TWB22W-0217-S04		
Sampling Method:	AS 1289.1.2.1 Cl 6.5.4		
Date Sampled:	7/12/2022	**Sampled By:**	This lab
Source:	Insitu (Existing Material)	**Material:**	Sub Grade
Sampled From:	Test Pit	**Specification:**	Nil
Location:	CH 40.04km LHS	**Material Description:**	Black Clay with 4% Hydrated Lime
Tested By:	Kevin Bennett	**Date Tested:**	12/05/2023

Load vs Penetration



Test Results

California Bearing Ratio [AS 1289.6.1.1]

CBR at 2.5mm (%):	**40**
Dry Density before Soaking (t/m³):	1.34
Density Ratio before Soaking (%):	97.0
Moisture Content before Soaking (%):	29.9
Moisture Ratio before Soaking (%):	99.5
Dry Density after Soaking (t/m³):	1.34
Density Ratio after Soaking (%):	97.0
Swell (%):	0.0
Moisture Content of Top 30mm (%):	30.3
Moisture Content of Remaining Depth (%):	33.1
Compaction Hammer Used:	Standard
	AS 1289.5.1.1
Surcharge Mass (kg):	4.51
Period of Soaking (Days):	4
Retained on 19 mm Sieve (%):	
CBR Moisture Content Method:	AS 1289.2.1.1
Sample Curing Time (h):	
Plasticity Determination Method:	AS 1289.3.1.2

Comments

Swell reading taken prior to air curing (Q135B) - 2.23mm
Curing of material prior to compaction in accordance with Q135A (4% Hydrated Lime used)
Moulded specimen cured at 23 degrees Celcius for 24 days prior to soaking

Queensland Government

Materials Services - Toowoomba
Department of Transport and Main Roads
427 Greenwattle Street, TOOWOOMBA QLD 4350

Phone: (07) 4699 9329

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California Bearing Ratio Test Report

Report No: CBR:TWB22W-0217-S05-1
Issue No: 1

Client: TMR Pavements, Materials and Geotechnical
Level 2 Bulwer Island Office Building
[redacted]

Project: 2023 USQ Student Project
Location:

Accredited for compliance with ISO/IEC 17025 - Testing

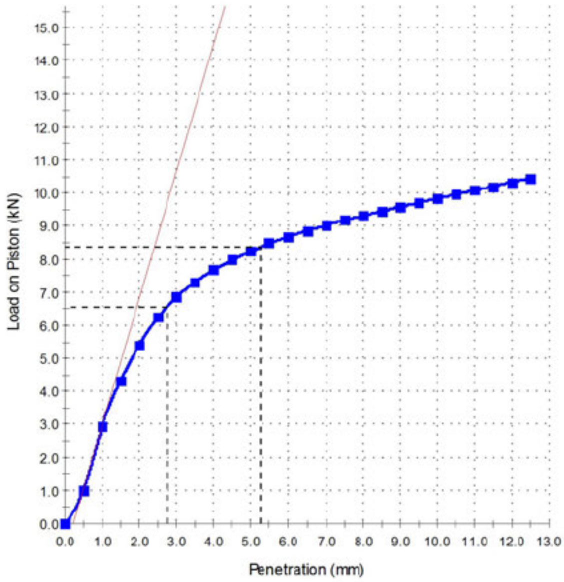
NATA
[redacted]

NATA Accredited Laboratory Number 2302
Approved Signatory: Ben Kratzmann (Materials Technologist)
Date of Issue: 20/06/2023

Sample Details

Sample ID:	TWB22W-0217-S05		
Sampling Method:	AS 1289.1.2.1 Cl 6.5.4		
Date Sampled:	7/12/2022	**Sampled By:**	This lab
Source:	Insitu (Existing Material)	**Material:**	Sub Grade
Sampled From:	Test Pit	**Specification:**	Nil
Location:	CH 40.04km LHS	**Material Description:**	Black Clay with 6% Hydrated Lime
Tested By:	Kevin Bennett	**Date Tested:**	12/05/2023

Load vs Penetration



Test Results

California Bearing Ratio [AS 1289.6.1.1]

CBR at 2.5mm (%):	**50**
Dry Density before Soaking (t/m³):	1.34
Density Ratio before Soaking (%):	97.0
Moisture Content before Soaking (%):	29.7
Moisture Ratio before Soaking (%):	99.0
Dry Density after Soaking (t/m³):	1.34
Density Ratio after Soaking (%):	97.0
Swell (%):	0.0
Moisture Content of Top 30mm (%):	30.0
Moisture Content of Remaining Depth (%):	32.4
Compaction Hammer Used:	Standard AS 1289.5.1.1
Surcharge Mass (kg):	4.51
Period of Soaking (Days):	4
Retained on 19 mm Sieve (%):	
CBR Moisture Content Method:	AS 1289.2.1.1
Sample Curing Time (h):	
Plasticity Determination Method:	AS 1289.3.1.2

Comments

Swell reading taken prior to air curing (Q135B) - 4.21mm
Curing of material prior to compaction in accordance with Q135A (6% Hydrated Lime used)
Moulded specimen cured at 23 degrees Celcius for 24 days prior to soaking

Queensland Government

Materials Services - Toowoomba
Department of Transport and Main Roads
427 Greenwattle Street, TOOWOOMBA QLD 4350

Phone: (07) 4699 9329

www.tmr.qld.gov.au

California Bearing Ratio Test Report

Report No: CBR:TWB22W-0217-S06-1
Issue No: 1

Client: TMR Pavements, Materials and Geotechnical
Level 2 Bulwer Island Office Building
[redacted]

Project: 2023 USQ Student Project
Location:

Accredited for compliance with ISO/IEC 17025 - Testing

NATA

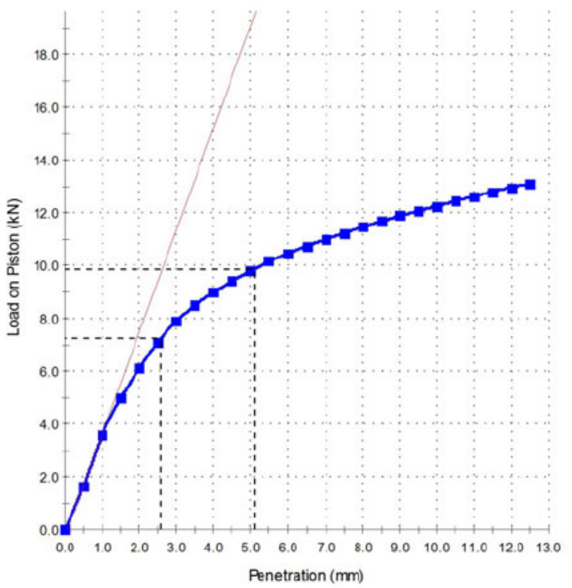
[redacted]

NATA Accredited Laboratory Number 2302
Approved Signatory: Ben Kratzmann (Materials Technologist)
Date of Issue: 20/06/2023

Sample Details

Sample ID:	TWB22W-0217-S06		
Sampling Method:	AS 1289.1.2.1 Cl 6.5.4		
Date Sampled:	7/12/2022	**Sampled By:**	This lab
Source:	Insitu (Existing Material)	**Material:**	Sub Grade
Sampled From:	Test Pit	**Specification:**	Nil
Location:	CH 40.04km LHS	**Material Description:**	Black Clay with 8% Hydrated Lime
Tested By:	Kevin Bennett	**Date Tested:**	12/05/2023

Load vs Penetration



Test Results

California Bearing Ratio [AS 1289.6.1.1]

CBR at 2.5mm (%):	**50**
Dry Density before Soaking (t/m³):	1.35
Density Ratio before Soaking (%):	97.0
Moisture Content before Soaking (%):	29.5
Moisture Ratio before Soaking (%):	98.0
Dry Density after Soaking (t/m³):	1.34
Density Ratio after Soaking (%):	97.0
Swell (%):	0.0
Moisture Content of Top 30mm (%):	29.6
Moisture Content of Remaining Depth (%):	31.5
Compaction Hammer Used:	Standard AS 1289.5.1.1
Surcharge Mass (kg):	4.51
Period of Soaking (Days):	4
Retained on 19 mm Sieve (%):	
CBR Moisture Content Method:	AS 1289.2.1.1
Sample Curing Time (h):	
Plasticity Determination Method:	AS 1289.3.1.2

Comments

Swell reading taken prior to air curing (Q135B) - 2.16mm
Curing of material prior to compaction in accordance with Q135A (8% Hydrated Lime used)
Moulded specimen cured at 23 degrees Celcius for 24 days prior to soaking

E.4 UCS Test Reports

Queensland Government

Materials Services - Toowoomba
Department of Transport and Main Roads
427 Greenwattle Street, TOOWOOMBA QLD 4350

Phone: (07) 4699 9329

www.tmr.qld.gov.au

Report No: UCS:TWB22W-0217-S02-1
Issue No: 1

Unconfined Compressive Strength

Client: TMR Pavements, Materials and Geotechnical
Level 2 Bulwer Island Office Building
[redacted]

Project: 2023 USQ Student Project
Location:

Accredited for compliance with ISO/IEC 17025 - Testing

NATA

[redacted]

NATA Accredited Laboratory Number 2302

Approved Signatory: Ben Kratzmann (Materials Technologist)
Date of Issue: 20/06/2023

THIS DOCUMENT SHALL NOT BE REPRODUCED EXCEPT IN FULL

Sample Details

Sample ID:	TWB22W-0217-S02	**Sampling Method:**	AS 1289.1.2.1 Cl 6.5.4
Date Sampled:	7/12/2022	**Sampled By:**	This lab
Source:	Insitu (Existing Material)	**Material:**	Sub Grade
Sampled From:	Test Pit	**Specification:**	Nil
Location:	CH 40.04km LHS	**Material Description:**	Black Clay with 2% Hydrated Lime

General Details

Unconfined Compressive Strength [Q115]

Sample Type:	Laboratory Mixed	**Specimen Preparation To:**	Q251A
Compaction Standard:	Standard	**MDD/OMC determined by:**	Q142A
Nominated Relative Compaction (%):	97.0	**Target and Achieved Values as Per:**	Q145A
Nominated Relative Moisture (%):	100	**Target Dry Density (t/m³):**	1.36
Target Moisture Content (%):	27.9	**Moisture Content Test Method:**	AS 1289.2.1.1

Stabilising Agent and Curing Details

Unconfined Compressive Strength [Q115]

Curing Method:	Q135B		
Stabilising Agent Type:	Hydrated Lime	**ATIC Registration:**	
Source:	Wagners	**Proportion (%):**	2.00
Air Curing in Mould Period (days):	1	**Temperature (°C):**	23
Air Curing Period (days):	27	**Temperature (°C):**	23

Results

Unconfined Compressive Strength [Q115]

Achieved Moisture Content (%):	28.6	**Achieved Percent of OMC (%):**	102
Average UCS (MPa):	0.3		

Specimens

Achieved Dry Density (t/m³)	Achieved Relative Compaction (%)	Unconfined Compressive Strength (MPa)	Tested Capped (Yes/No)	Exclude this result (Yes/No)
1.35	96.5	0.3	No	No
1.35	96.5	0.3	No	No
1.35	96.5	0.4	No	No

Comments

Q115 - Specimens cured in moist condition.

Materials Services - Toowoomba
Department of Transport and Main Roads
427 Greenwattle Street, TOOWOOMBA QLD 4350

Phone: (07) 4699 9329

www.tmr.qld.gov.au

Queensland Government

Unconfined Compressive Strength

Report No: UCS:TWB22W-0217-S03-1
Issue No: 1

Client: TMR Pavements, Materials and Geotechnical
Level 2 Bulwer Island Office Building
[redacted]

Project: 2023 USQ Student Project
Location:

Accredited for compliance with ISO/IEC 17025 - Testing

NATA

[redacted]

NATA Accredited Laboratory Number 2302

Approved Signatory: Ben Kratzmann (Materials Technologist)
Date of Issue: 20/06/2023

THIS DOCUMENT SHALL NOT BE REPRODUCED EXCEPT IN FULL

Sample Details

Sample ID:	TWB22W-0217-S03	**Sampling Method:**	AS 1289.1.2.1 Cl 6.5.4
Date Sampled:	7/12/2022	**Sampled By:**	This lab
Source:	Insitu (Existing Material)	**Material:**	Sub Grade
Sampled From:	Test Pit	**Specification:**	Nil
Location:	CH 40.04km LHS	**Material Description:**	Black Clay with 3% Hydrated Lime

General Details

Unconfined Compressive Strength [Q115]

Sample Type:	Laboratory Mixed	**Specimen Preparation To:**	Q251A
Compaction Standard:	Standard	**MDD/OMC determined by:**	Q142A
Nominated Relative Compaction (%):	97.0	**Target and Achieved Values as Per:**	Q145A
Nominated Relative Moisture (%):	100	**Target Dry Density (t/m³):**	1.36
Target Moisture Content (%):	27.9	**Moisture Content Test Method:**	AS 1289.2.1.1

Stabilising Agent and Curing Details

Unconfined Compressive Strength [Q115]

Curing Method:	Q135B		
Stabilising Agent Type:	Hydrated Lime	**ATIC Registration:**	
Source:	Wagners	**Proportion (%):**	3.00
Air Curing in Mould			
Period (days):	1	**Temperature (°C):**	23
Air Curing			
Period (days):	27	**Temperature (°C):**	23

Results

Unconfined Compressive Strength [Q115]

Achieved Moisture Content (%):	28.5	**Achieved Percent of OMC (%):**	102
Average UCS (MPa):	0.4		

Specimens

Achieved Dry Density (t/m³)	Achieved Relative Compaction (%)	Unconfined Compressive Strength (MPa)	Tested Capped (Yes/No)	Exclude this result (Yes/No)
1.35	96.5	0.3	No	No
1.35	96.5	0.4	No	No
1.35	96.5	0.4	No	No

Comments

Q115 - Specimens cured in moist condition.

Materials Services - Toowoomba
Department of Transport and Main Roads
427 Greenwattle Street, TOOWOOMBA QLD 4350

Phone: (07) 4699 9329

www.tmr.qld.gov.au

Queensland Government

Unconfined Compressive Strength

Report No: UCS:TWB22W-0217-S04-1
Issue No: 1

Client: TMR Pavements, Materials and Geotechnical
Level 2 Bulwer Island Office Building
[redacted]

Project: 2023 USQ Student Project
Location:

Accredited for compliance with ISO/IEC 17025 - Testing

NATA

[redacted]

NATA Accredited Laboratory Number 2302

Approved Signatory: Ben Kratzmann (Materials Technologist)
Date of Issue: 20/06/2023

THIS DOCUMENT SHALL NOT BE REPRODUCED EXCEPT IN FULL

Sample Details

Sample ID:	TWB22W-0217-S04	**Sampling Method:**	AS 1289.1.2.1 Cl 6.5.4
Date Sampled:	7/12/2022	**Sampled By:**	This lab
Source:	Insitu (Existing Material)	**Material:**	Sub Grade
Sampled From:	Test Pit	**Specification:**	Nil
Location:	CH 40.04km LHS	**Material Description:**	Black Clay with 4% Hydrated Lime

General Details

Unconfined Compressive Strength [Q115]

Sample Type:	Laboratory Mixed	**Specimen Preparation To:**	Q251A
Compaction Standard:	Standard	**MDD/OMC determined by:**	Q142A
Nominated Relative Compaction (%):	97.0	**Target and Achieved Values as Per:**	Q145A
Nominated Relative Moisture (%):	100	**Target Dry Density (t/m³):**	1.34
Target Moisture Content (%):	30.0	**Moisture Content Test Method:**	AS 1289.2.1.1

Stabilising Agent and Curing Details

Unconfined Compressive Strength [Q115]

Curing Method:	Q135B		
Stabilising Agent Type:	Hydrated Lime	**ATIC Registration:**	
Source:	Wagners	**Proportion (%):**	4.00
Air Curing in Mould			
Period (days):	1	**Temperature (°C):**	23
Air Curing			
Period (days):	27	**Temperature (°C):**	23

Results

Unconfined Compressive Strength [Q115]

Achieved Moisture Content (%):	30.3	**Achieved Percent of OMC (%):**	101
Average UCS (MPa):	0.3		

Specimens

Achieved Dry Density (t/m³)	Achieved Relative Compaction (%)	Unconfined Compressive Strength (MPa)	Tested Capped (Yes/No)	Exclude this result (Yes/No)
1.34	97.0	0.3	No	No
1.33	96.5	0.3	No	No
1.34	97.0	0.3	No	No

Comments

Q115 - Specimens cured in moist condition.

Materials Services - Toowoomba
Department of Transport and Main Roads
427 Greenwattle Street, TOOWOOMBA QLD 4350

Phone: (07) 4699 9329

www.tmr.qld.gov.au

Queensland Government

Unconfined Compressive Strength

Report No: UCS:TWB22W-0217-S05-1
Issue No: 1

Client: TMR Pavements, Materials and Geotechnical
Level 2 Bulwer Island Office Building
[redacted]

Project: 2023 USQ Student Project
Location:

NATA

Accredited for compliance with ISO/IEC 17025 - Testing

[redacted]

NATA Accredited Laboratory Number 2302

Approved Signatory: Ben Kratzmann (Materials Technologist)
Date of Issue: 20/06/2023

Sample Details

Sample ID:	TWB22W-0217-S05	**Sampling Method:**	AS 1289.1.2.1 Cl 6.5.4
Date Sampled:	7/12/2022	**Sampled By:**	This lab
Source:	Insitu (Existing Material)	**Material:**	Sub Grade
Sampled From:	Test Pit	**Specification:**	Nil
Location:	CH 40.04km LHS	**Material Description:**	Black Clay with 6% Hydrated Lime

General Details

Unconfined Compressive Strength [Q115]

Sample Type:	Laboratory Mixed	**Specimen Preparation To:**	Q251A
Compaction Standard:	Standard	**MDD/OMC determined by:**	Q142A
Nominated Relative Compaction (%):	97.0	**Target and Achieved Values as Per:**	Q145A
Nominated Relative Moisture (%):	100	**Target Dry Density (t/m³):**	1.34
Target Moisture Content (%):	30.0	**Moisture Content Test Method:**	AS 1289.2.1.1

Stabilising Agent and Curing Details

Unconfined Compressive Strength [Q115]

Curing Method:	Q135B		
Stabilising Agent Type:	Hydrated Lime	**ATIC Registration:**	
Source:	Wagners	**Proportion (%):**	6.00
Air Curing in Mould			
Period (days):	1	**Temperature (°C):**	23
Air Curing			
Period (days):	27	**Temperature (°C):**	23

Results

Unconfined Compressive Strength [Q115]

Achieved Moisture Content (%):	30.3	**Achieved Percent of OMC (%):**	101
Average UCS (MPa):	0.4		

Specimens

Achieved Dry Density (t/m³)	Achieved Relative Compaction (%)	Unconfined Compressive Strength (MPa)	Tested Capped (Yes/No)	Exclude this result (Yes/No)
1.33	96.5	0.4	No	No
1.34	96.5	0.4	No	No
1.34	96.5	0.4	No	No

Comments

Q115 - Specimens cured in moist condition.

Materials Services - Toowoomba
Department of Transport and Main Roads
427 Greenwattle Street, TOOWOOMBA QLD 4350

Phone: (07) 4699 9329

www.tmr.qld.gov.au

Queensland Government

Report No: UCS:TWB22W-0217-S06-1
Issue No: 1

Unconfined Compressive Strength

Client: TMR Pavements, Materials and Geotechnical
Level 2 Bulwer Island Office Building
[redacted]

Project: 2023 USQ Student Project
Location:

Accredited for compliance with ISO/IEC 17025 - Testing

NATA

[redacted]

NATA Accredited Laboratory Number 2302
Approved Signatory: Ben Kratzmann (Materials Technologist)
Date of Issue: 20/06/2023
THIS DOCUMENT SHALL NOT BE REPRODUCED EXCEPT IN FULL

Sample Details

Sample ID:	TWB22W-0217-S06	**Sampling Method:**	AS 1289.1.2.1 Cl 6.5.4
Date Sampled:	7/12/2022	**Sampled By:**	This lab
Source:	Insitu (Existing Material)	**Material:**	Sub Grade
Sampled From:	Test Pit	**Specification:**	Nil
Location:	CH 40.04km LHS	**Material Description:**	Black Clay with 8% Hydrated Lime

General Details

Unconfined Compressive Strength [Q115]

Sample Type:	Laboratory Mixed	**Specimen Preparation To:**	Q251A
Compaction Standard:	Standard	**MDD/OMC determined by:**	Q142A
Nominated Relative Compaction (%):	97.0	**Target and Achieved Values as Per:**	Q145A
Nominated Relative Moisture (%):	100	**Target Dry Density (t/m³):**	1.34
Target Moisture Content (%):	30.0	**Moisture Content Test Method:**	AS 1289.2.1.1

Stabilising Agent and Curing Details

Unconfined Compressive Strength [Q115]

Curing Method:	Q135B		
Stabilising Agent Type:	Hydrated Lime	**ATIC Registration:**	
Source:	Wagners	**Proportion (%):**	8.00
Air Curing in Mould			
Period (days):	1	**Temperature (°C):**	23
Air Curing			
Period (days):	27	**Temperature (°C):**	23

Results

Unconfined Compressive Strength [Q115]

Achieved Moisture Content (%): 30.4 **Achieved Percent of OMC (%):** 101
Average UCS (MPa): 0.4

Specimens

Achieved Dry Density (t/m³)	Achieved Relative Compaction (%)	Unconfined Compressive Strength (MPa)	Tested Capped (Yes/No)	Exclude this result (Yes/No)
1.33	96.5	0.4	No	No
1.33	96.5	0.3	No	No
1.33	96.5	0.4	No	No

Comments

Q115 - Specimens cured in moist condition.

Appendix F

CIRCLY 7.0 Analysis Outputs

F.1 Selected Fill Sub-Grade, CBR=10%

```
CIRCLY  - Version 7.0 (1 February 2022)

Job Title: Runis Kashanov Thesis

Design Method: Austroads 2017

NDT (cumulative heavy vehicle axle groups over design period): 7.30E+06

Traffic Load Distribution:

   ID: Runis Kashanov TLD
   Name: Runis Kashanov Thesis TLD
   ESA/HVAG: 0.743

Details of Load Groups:

   Load   Load                  Load                 Load            Radius    Pressure/    Exponent
   No.    ID                    Category             Type                      Ref. stress
    1     ESA750-Full           ESA750-Full          Vertical Force     92.1    0.75         0.00
    2     SAST53                SAST53               Vertical Force    102.4    0.80         0.00

   Load Locations:
   Location   Load                  Gear          X          Y      Scaling     Theta
   No.        ID                    No.                             Factor
    1         ESA750-Full            1         -165.0        0.0   1.00E+00      0.00
    2         ESA750-Full            1          165.0        0.0   1.00E+00      0.00
    3         ESA750-Full            1         1635.0        0.0   1.00E+00      0.00
    4         ESA750-Full            1         1965.0        0.0   1.00E+00      0.00
    1         SAST53                 1            0.0        0.0   1.00E+00      0.00
    2         SAST53                 1         2130.0        0.0   1.00E+00      0.00

Details of Layered System:

   ID: Runis Kashanov 1 Title: Pavement Layers 1

   Layer  Lower    Material               Isotropy   Modulus    P.Ratio
   No.    i/face   ID                                (or Ev)    (or vvh)  F          Eh         vh
    1     rough    Gran_350               Aniso.     3.50E+02   0.35      2.59E+02   1.75E+02   0.35
    2     rough    Gran_250               Aniso.     2.50E+02   0.35      1.85E+02   1.25E+02   0.35
    3     rough    subsltCB10             Aniso.     1.00E+02   0.45      6.90E+01   5.00E+01   0.45
    4     rough    Sub_CBR4.5             Aniso.     4.50E+01   0.45      3.10E+01   2.25E+01   0.45

   Performance Relationships:
   Layer  Location Material               Component  Perform.   Perform.   Shift
   No.             ID                                Constant   Exponent   Factor
    3     top      subsltCB10             EZZ         0.009150    7.000
    4     top      Sub_CBR4.5             EZZ         0.009150    7.000

   Reliability Factors:
   Project Reliability: Austroads 90%
   Layer  Reliability  Material
    No.   Factor       Type
    3      1.00       Subgrade (Selected Material) (Austroads 2017)
    4      1.00       Subgrade (Austroads 2017)

   Details of Layers to be sublayered:
   Layer no.  1:  Austroads (2004) sublayering
   Layer no.  2:  Austroads (2004) sublayering
   Layer no.  3:  Austroads (2004) sublayering

Strains:

   Layer  Thickness  Material               Axle       Unitless
   No.               ID                                Strain
    3      200.00    subsltCB10
                                            SADT(80): 9.270E-04
    4        0.00    Sub_CBR4.5
                                            SADT(80): 9.679E-04

Results:

   Layer  Thickness  Material               Axle       CDF
   No.               ID                     Group
    1      160.00    Gran_350                               n/a

    2      150.00    Gran_250                               n/a

    3      200.00    subsltCB10             Total:     5.947E-01

    4        0.00    Sub_CBR4.5             Total:     8.044E-01
```

F.2 Lime Stabilised Sub-Grade, CBR=15%

```
CIRCLY  - Version 7.0 (1 February 2022)

Job Title: Runis Kashanov Thesis

Design Method: Austroads 2017

NDT (cumulative heavy vehicle axle groups over design period): 7.30E+06

Traffic Load Distribution:

   ID: Runis Kashanov TLD
   Name: Runis Kashanov Thesis TLD
   ESA/HVAG: 0.743

Details of Load Groups:

   Load   Load                 Load                 Load           Radius    Pressure/    Exponent
   No.    ID                   Category             Type                     Ref. stress
    1     ESA750-Full          ESA750-Full          Vertical Force     92.1    0.75         0.00
    2     SAST53               SAST53               Vertical Force    102.4    0.80         0.00

   Load Locations:
   Location   Load                 Gear          X          Y      Scaling     Theta
   No.        ID                   No.                             Factor
    1         ESA750-Full           1         -165.0        0.0   1.00E+00      0.00
    2         ESA750-Full           1          165.0        0.0   1.00E+00      0.00
    3         ESA750-Full           1         1635.0        0.0   1.00E+00      0.00
    4         ESA750-Full           1         1965.0        0.0   1.00E+00      0.00
    1         SAST53                1            0.0        0.0   1.00E+00      0.00
    2         SAST53                1         2130.0        0.0   1.00E+00      0.00

Details of Layered System:

   ID: Runis Kashanov 1 Title: Pavement Layers 1

   Layer  Lower    Material               Isotropy  Modulus    P.Ratio
   No.    i/face   ID                               (or Ev)    (or vvh)  F          Eh         vh
    1     rough    Gran_350               Aniso.    3.50E+02   0.35      2.59E+02   1.75E+02   0.35
    2     rough    Gran_250               Aniso.    2.50E+02   0.35      1.85E+02   1.25E+02   0.35
    3     rough    sublimestabCB15        Aniso.    1.50E+02   0.45      1.03E+02   7.50E+01   0.45
    4     rough    Sub_CBR4.5             Aniso.    4.50E+01   0.45      3.10E+01   2.25E+01   0.45

   Performance Relationships:
   Layer  Location Material               Component  Perform.   Perform.   Shift
   No.             ID                                Constant   Exponent   Factor
    3     top      sublimestabCB15        EZZ         0.009150    7.000
    4     top      Sub_CBR4.5             EZZ         0.009150    7.000

   Reliability Factors:
   Project Reliability: Austroads 90%
   Layer  Reliability  Material
    No.   Factor        Type
    3      1.00       Subgrade (Selected Material) (Austroads 2017)
    4      1.00       Subgrade (Austroads 2017)

   Details of Layers to be sublayered:
   Layer no.  1:  Austroads (2004) sublayering
   Layer no.  2:  Austroads (2004) sublayering
   Layer no.  3:  Austroads (2004) sublayering

Strains:

   Layer  Thickness  Material               Axle      Unitless
   No.               ID                               Strain
    3      300.00    sublimestabCB15
                                            SADT(80): 8.807E-04
    4        0.00    Sub_CBR4.5
                                            SADT(80): 8.556E-04

Results:

   Layer  Thickness  Material               Axle      CDF
   No.               ID                     Group
    1      160.00    Gran_350                              n/a

    2       90.00    Gran_250                              n/a

    3      300.00    sublimestabCB15        Total:    4.153E-01

    4        0.00    Sub_CBR4.5             Total:    3.392E-01
```

F.3 Lime Stabilised Sub-Grade, CBR=4.5%

```
CIRCLY  - Version 7.0 (1 February 2022)

Job Title: Runis Kashanov Thesis

Design Method: Austroads 2017

NDT (cumulative heavy vehicle axle groups over design period): 7.30E+06

Traffic Load Distribution:

   ID: Runis Kashanov TLD
   Name: Runis Kashanov Thesis TLD
   ESA/HVAG: 0.743

Details of Load Groups:

   Load   Load                 Load                 Load            Radius    Pressure/    Exponent
   No.    ID                   Category             Type                      Ref. stress
    1     ESA750-Full          ESA750-Full          Vertical Force     92.1    0.75          0.00
    2     SAST53               SAST53               Vertical Force    102.4    0.80          0.00

   Load Locations:
   Location   Load                 Gear          X          Y      Scaling     Theta
   No.        ID                   No.                             Factor
    1         ESA750-Full           1         -165.0        0.0   1.00E+00      0.00
    2         ESA750-Full           1          165.0        0.0   1.00E+00      0.00
    3         ESA750-Full           1         1635.0        0.0   1.00E+00      0.00
    4         ESA750-Full           1         1965.0        0.0   1.00E+00      0.00
    1         SAST53                1            0.0        0.0   1.00E+00      0.00
    2         SAST53                1         2130.0        0.0   1.00E+00      0.00

Details of Layered System:

   ID: Runis Kashanov 1 Title: Pavement Layers 1

   Layer  Lower    Material               Isotropy  Modulus    P.Ratio
   No.    i/face   ID                               (or Ev)    (or vvh)  F          Eh         vh
    1     rough    Gran_350               Aniso.    3.50E+02   0.35      2.59E+02   1.75E+02   0.35
    2     rough    Gran_250               Aniso.    2.50E+02   0.35      1.85E+02   1.25E+02   0.35
    3     rough    sublimestabCB4.5       Aniso.    4.50E+01   0.45      3.10E+01   2.25E+01   0.45
    4     rough    Sub_CBR4.5             Aniso.    4.50E+01   0.45      3.10E+01   2.25E+01   0.45

   Performance Relationships:
   Layer  Location Material               Component  Perform.   Perform.   Shift
   No.             ID                                Constant   Exponent   Factor
    3     top      sublimestabCB4.5       EZZ         0.009150    7.000
    4     top      Sub_CBR4.5             EZZ         0.009150    7.000

   Reliability Factors:
   Project Reliability: Austroads 90%
   Layer  Reliability  Material
    No.   Factor        Type
    3      1.00       Subgrade (Selected Material) (Austroads 2017)
    4      1.00       Subgrade (Austroads 2017)

   Details of Layers to be sublayered:
   Layer no.  1:  Austroads (2004) sublayering
   Layer no.  2:  Austroads (2004) sublayering
   Layer no.  3:  Austroads (2004) sublayering

Strains:

   Layer  Thickness  Material               Axle      Unitless
   No.               ID                               Strain
    3       300.00   sublimestabCB4.5
                                            SADT(80): 2.787E-03
    4         0.00   Sub_CBR4.5
                                            SADT(80): 1.110E-03

Results:

   Layer  Thickness  Material               Axle      CDF
   No.               ID                     Group
    1       160.00   Gran_350                              n/a

    2        90.00   Gran_250                              n/a

    3       300.00   sublimestabCB4.5       Total:    1.321E+03

    4         0.00   Sub_CBR4.5             Total:    2.092E+00
```

F.4 Lime Modified Sub-Grade, UCS=0.4 MPa

```
CIRCLY  - Version 7.0 (1 February 2022)

Job Title: Runis Kashanov Thesis

Design Method: Austroads 2017

NDT (cumulative heavy vehicle axle groups over design period): 7.30E+06

Traffic Load Distribution:

   ID: Runis Kashanov TLD
   Name: Runis Kashanov Thesis TLD
   ESA/HVAG: 0.743

Details of Load Groups:

   Load   Load                 Load                 Load            Radius    Pressure/    Exponent
   No.    ID                   Category             Type                      Ref. stress
    1     ESA750-Full          ESA750-Full          Vertical Force     92.1    0.75          0.00
    2     SAST53               SAST53               Vertical Force    102.4    0.80          0.00

   Load Locations:
   Location   Load                 Gear          X          Y      Scaling     Theta
   No.        ID                   No.                              Factor
    1         ESA750-Full           1         -165.0        0.0   1.00E+00      0.00
    2         ESA750-Full           1          165.0        0.0   1.00E+00      0.00
    3         ESA750-Full           1         1635.0        0.0   1.00E+00      0.00
    4         ESA750-Full           1         1965.0        0.0   1.00E+00      0.00
    1         SAST53                1            0.0        0.0   1.00E+00      0.00
    2         SAST53                1         2130.0        0.0   1.00E+00      0.00

Details of Layered System:

   ID: Runis Kashanov 1 Title: Pavement Layers 1

   Layer  Lower    Material              Isotropy   Modulus    P.Ratio
   No.    i/face   ID                               (or Ev)    (or vvh)  F          Eh         vh
    1     rough    Gran_350              Aniso.     3.50E+02   0.35      2.59E+02   1.75E+02   0.35
    2     rough    Gran_250              Aniso.     2.50E+02   0.35      1.85E+02   1.25E+02   0.35
    3     rough    sublimeE85            Aniso.     8.50E+01   0.45      5.86E+01   4.25E+01   0.45
    4     rough    Sub_CBR4.5            Aniso.     4.50E+01   0.45      3.10E+01   2.25E+01   0.45

   Performance Relationships:
   Layer  Location Material              Component  Perform.   Perform.   Shift
   No.             ID                               Constant   Exponent   Factor
    3     top      sublimeE85            EZZ         0.009150    7.000
    4     top      Sub_CBR4.5            EZZ         0.009150    7.000

   Reliability Factors:
   Project Reliability: Austroads 90%
   Layer  Reliability  Material
    No.   Factor        Type
    3      1.00       Subgrade (Selected Material) (Austroads 2017)
    4      1.00       Subgrade (Austroads 2017)

   Details of Layers to be sublayered:
   Layer no.  1:  Austroads (2004) sublayering
   Layer no.  2:  Austroads (2004) sublayering
   Layer no.  3:  Austroads (2004) sublayering

Strains:

   Layer  Thickness  Material              Axle      Unitless
   No.               ID                              Strain
    3      300.00    sublimeE85
                                           SADT(80): 1.456E-03
    4        0.00    Sub_CBR4.5
                                           SADT(80): 9.469E-04

Results:

   Layer  Thickness  Material              Axle      CDF
   No.               ID                    Group
    1      160.00    Gran_350                             n/a

    2       90.00    Gran_250                             n/a

    3      300.00    sublimeE85            Total:    1.400E+01

    4        0.00    Sub_CBR4.5            Total:    6.896E-01
```