University of Southern Queensland
Faculty of Engineering and Surveying

EFFECTIVENESS OF LIME STABILISATION ON REACTIVE SOILS FOR MAIN ROADS

A dissertation submitted by

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ABSTRACT

The Department of Transport and Main Roads (DTMR) manages the state-controlled road system in Queensland comprising the major traffic carrying and strategic roads in the state. A vast majority of these state-controlled roads are located in areas where the insitu material is dominated by reactive soil. Mixing lime with the reactive subgrade is a stabilisation technology to not only improve construction workability in these soils but to also reduce maintenance frequency on the road system.

Lime stabilisation is used to modify sub-grade soils to improve constructability by improving plastic properties, flocculating particles and drying the material. Studies indicate that lime stabilisation will improve load bearing capacity and will also achieve long term strength retention.

DTMR Specifications and AUSTROADS Pavement design guidelines for lime modified sub-grade recommends the Unconfined Compressive Strength achieve a minimum of 1.5MPa at 28 days (unsoaked). This research will assess current test results against the original test results at completion of construction.

DTMR standards specify that lime stabilisation of subgrade material shall be carried out as 2 passes by applying half the dose in the first pass, waiting 24 hours then applying the remaining dosage in the last pass. This time period between passes is called the Amelioration Period.

The major issue with having a 24 hour period for amelioration is it takes a longer time to finish construction. This delay has potential impacts on the contractor and exposes the open subgrade to potential weather such as rain for a longer period.

Amelioration is necessary in heavy clays as adequate mixing is difficult to achieve in one pass as heavy clays tend to clump together. Allowing 24 hours between passes permits the lime to flocculate clay particles and improve workability. Amelioration results in better mixing in the second run and construction also becomes easier. This research will test a range of reduced time period to determine what length of time will achieve the Unconfined Compressive Strength of 1.5MPa.

This research project will confirm that stabilising the road pavement sub-grade with lime is an effective and economic solution when applied in the appropriate environment and quantity. It is also expected that improvements will continue to be made to lime stabilisation specifications.

University of Southern Queensland

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ENG4111 Research Project Part 1 & ENG4112 Research Project Part 2

LIMITATIONS OF USE

Professor Frank Bullen
Dean
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CERTIFICATION

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

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

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Signature

25 October 2011

Date

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Paul Wilson

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GLOSSARY

Austroads	Australian Road Research Board
AS	Australia Standard
$CaCO_3$	Calcium Carbonate
CaO	Quicklime
$Ca(OH)_2$	Hydrated lime
CO_2	Carbon Dioxide
CBR	California Bearing Ratio
DD	Dry Density
DCP	Dynamic Cone Penetrometer
DTMR	Queensland Department of Transport and Main Roads
EMC	Equilibrium Moisture Content
LL	Liquid Limit
LS	Linear Shrinkage
MC	Moisture Content
MDD	Maximum Dry Density
MRS	Designation of Queensland Main Roads Standard Specifications
NAASRA	National Australian Association of State Road Authorities, now known as Austroads
OMC	Optimum Moisture Content
PI	Plasticity Index, PI = LL – PL
PL	Plastic Limit
UCS	Unconfined Compressive Strength

1 INTRODUCTION

1.1 Introduction

Various forms of lime stabilisation have been used for thousands of years. Early Roman roads utilised lime as a stabilisation agent and until the invention of Portland cement in the 19th Century, lime was widely used for building construction. The Great Wall of China was constructed using a lime stabilised mortar, as were the original buildings at Port Arthur.

Lime stabilised sub-grades have been used for in excess of 30 years in Ipswich City Council with varying results. In fact, lime stabilised sub-grades were used as a substitute for pavement materials, as there were no naturally occurring suitable pavement materials within the old Ipswich City Council boundaries.

Queensland Department of Transport and Main Roads (DTMR) also trialled lime extensively until the late 70's when it fell from favour. Trials by DTMR have recommenced at various sites since the mid 1990's.

Queensland has some 180,500km of public road network of which DTMR manages 33,337kms of state-controlled roads comprising the major traffic carrying and strategic roads in the state. Queensland roads are made up of both flexible and rigid pavement types and these pavements are designed to cater for varying traffic volumes, subgrade types and axle loads. The state-controlled road system plays a vital part in the liveability of our cities and rural communities and is valuable for the movement of people and efficient transportation of goods and services throughout the state and country.

New road pavements are usually designed for a life expectancy of at least 20 years; however, a vast majority of the state-controlled roads are located in areas where the insitu material is dominated by reactive soil. Reactive soils are typically clays that demonstrate extensive volume and strength changes at varying moisture content due to their chemical composition. This change in soil volume has shown through history to cause significant structural damage to road pavements, and structural foundations, due at large to the swelling and shrinking that occurs within the soil.

Engineers and Geologists have studied reactive soils in an effort to determine the most appropriate methods of construction where these soils cannot be avoided. Road pavement design and construction difficulties over expansive soils have been experienced in several countries including Australia, New Zealand, China, India, USA, UK, Israel and South Africa. The history of clays and their engineering significance, dating back to papers written in the early 1930's, is discussed in the Journal, Review of Stabilization of Clays and Expansive Soils in Pavements and Lightly Loaded Structures (Petry & Little 2002).

It is believed that stabilising the road pavement with lime is an effective and economic solution when applied in the appropriate environment and quantity.

1.2 The Problem

Early trials of lime stabilised subgrades adopted the Lime Fixation Percentage (Little 1995) which is defined as the percentage of lime which causes the soil's plastic limit to reach a stable value. The percentage of lime typically added was 2-4% and research has found that these trials were unsuccessful largely due to insufficient quantity of lime being applied to the natural reactive soil and the construction method.

DTMR re-commenced trials of lime stabilised subgrades in 1996 at a couple of sites near Warwick. The Killarney Trial (Warwick – Killarney Road) adopted the Eades and Grim Method (Little 1995) while the Cunningham Highway - Freestone Creek to Eight Mile Intersection Trial adopted the Thompson Method (Little 1995).

These trials are approaching the later part of their 20 year design life expectancy. The Cunningham Highway trial, which was constructed in 1998, has not shown any sign of deterioration to last inspections and is due for another inspection and testing to check the progress of the Unconfined Compressive Strength (UCS) strength of the road subgrade. This will give an indication whether the subgrade is continuing to increase its strength, remaining stable or starting to decrease in strength after 13 years.

Spreading and mixing lime to stabilise highly reactive clay subgrades is generally completed in two passes. This process is called Amelioration where half the lime dosage is applied in the first pass, left to flocculate for specified timeframe, and then apply the remaining half in the last pass.

Amelioration is necessary in heavy clays as adequate mixing is difficult to achieve in one pass as heavy clays tend to clump together. DTMR presently allow 24 hours between passes to permit the lime to flocculate clay particles and improve workability. Amelioration results in better mixing in the second run and construction become easier.

The major issue with having a 24 hour period for amelioration is it takes a longer time to finish construction. This delays the contractor by having to bring a machine back a second day and the contractor can't continue construction until after the second pass. This timeframe also delays construction, delays trafficking and exposes the open subgrade to potential weather such as rain for a longer period.

Another issue is if the mixing machine breaks down, this requires a second machine to be sourced within 24 hours which can be difficult in remote areas. This would extend delays in construction and trafficking, and possibly require the lime mix to be started again.

1.3 Research Aims

The aim of this project is to confirm if the UCS of road subgrade has improved or diminished since construction was completed on the Cunningham Highway (Eight Mile Creek to Freestone Creek) trial site and to verify whether 24 hours Amelioration Period for lime flocculation can be reduced and determine what the optimum time required for amelioration.

1.4 Research Approach

The research approach for this study is divided into two parts:

1. Existing pavement UCS strength

   - Obtain core samples of existing subgrade material;
   - Test UCS of upper, middle and lower layers of the subgrade individually;
   - Analyse UCS test results;
   - Compare results to previous UCS data.

2. Amelioration Period

   - Obtain two samples of black soil using Emerald and Barcaldine soil groups;
   - Conduct soil classification testing to obtain moisture content, particle size distribution, atterberg limits, organic content, sulphate content, lime demand, maximum dry density and optimum moisture of both natural soil and soil with lime at lime demand;
   - Conduct UCS testing with cure for 28 days @ 23deg / 95% Humidity;
   - Test for different amelioration times (0, 6, 12*, 18 and 24 hours).

     * Testing will be conducted as close to 12 hours as possible. Access to laboratory may alter this time to 14 hours.

   The Amelioration process to be undertaken for the UCS samples is as follows:

   Stage 1: Test portion will be mixed with half the total lime needed and with 80% of target water. Test portion will be subjected to half compaction and allowed to cure as per the target amelioration periods.

   Stage 2: Cured test portion will be broken up by hand and rubber mallet and the remaining half of the lime mixed with the remaining 20% water to bring it up to the total needed to achieve target percentage. Test portions will then be compacted to perform UCS testing.

1.5 Dissertation Outline

This dissertation is organised as follows:

Chapter 2 Literature Review

This chapter reviews and summarises literature relating to the use and construction techniques of lime stabilisation in reactive soils undertaken on roads controlled by the Department of Transport and Main Roads.

Chapter 3 Lime Stabilised Subgrade Strength

This chapter gives an overview of the methodology used to analyse the long term strength gains of lime stabilised subgrade and presents the results of unconfined compressive strength testing

Chapter 4 Amelioration Period

This chapter presents an overview of the methodology used and associated results for the analyse of a range of different amelioration time periods between two-stage lime stabilisation mixing in reactive soils when constructing a pavement subgrade.

Chapter 5 Conclusions and Future Works

This chapter documents achievement of project objectives, conclusions from this study and discusses suggested future work.

2 LITERATURE REVIEW

2.1 Introduction

This literature review covers the background on the properties of lime and reactive soils, use of lime as a stabilising product on road pavements and research carried out to date on the issues faced during design and construction of lime stabilised pavements.

It is important to mention that similar topics have been undertaken by the University of Southern Queensland students over the past few years. The year and topics are outlined below: -

- 2010 – Effective Road Pavement Design for Expansive Soils in Ipswich by Catherine Caunce;
- 2009 - Alternate Pavement Types on Reactive Soils in the Ipswich Council Area by Jeffrey Crone;
- 2008 - Investigation of Construction Practices and Test Procedures for Road Pavements on Expansive Subgrades by Kieren Walters;
- 2005 - Road Stabilisation Issues in Southern District of the Department of Main Roads, Queensland by Elissa Harrison.

Every effort will be made to ensure that relevant data is utilised and this research does not duplicate previous research. The stated outcomes from these previous papers by fellow students indicated that further research could be undertaken in the following areas (with respect to lime stabilisation and clay soils) -:

- Investigation into the cause of pavement failures;
- Advantages of performance based testing over traditional empirical testing;
- Determination of the strength gain relationship between lime stabilization and black soils;
- Longer term testing of stabilisation efforts;
- Trial test methods such as soil suction during seasonal peaks to determine the active depth of expansive soils;
- Classification of clays using various methods.

2.2 Properties of Lime

Lime is produced from Limestone (Calcium Carbonate $CaCO_3$) which occurs naturally in sedimentary rock. The term “lime” can refer to calcium carbonate (agricultural lime), calcium hydroxide (slaked or hydrated lime) and calcium oxide (quicklime and dolomite). Agricultural lime is generally a finely ground limestone which is suitable for soil amendment but it is not chemically active enough to be effective for soil stabilisation.

Lime stabilisation in road pavements requires a more reactive form of lime. This is achieved with quicklime or hydrated lime.

2.2.1 Quicklime

Quicklime (CaO) is formed by heating calcium carbonate ($CaCO_3$) at high temperatures until carbon dioxide is driven off.

$CaCO_3 + \text{Heat } (\sim 1315°C) \rightarrow CaO + CO_2$

Quicklime can be manufactured in varying sizes from quite fine to very coarse. Some of the different types available are Lump Lime, Pebble Lime, Granular Lime and Pulverised lime (Little 1995).

Quicklime has a high heat of hydration ($\Delta H = -15.6$ kcal/mol) which makes it difficult to handle and store. In humid climates it may be necessary to slake the lime immediately to form hydrated lime. It is of a caustic nature and must be handled with caution because it will corrosively attack equipment and can cause severe skin burns (Metcalf & Ingles 1972).

2.2.2 Hydrated Lime

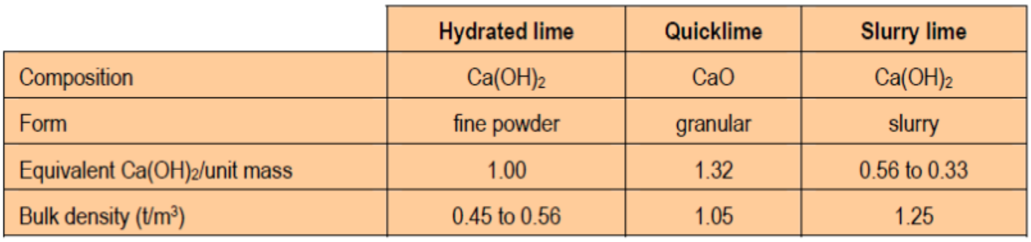
Hydrated lime ($Ca\,(OH)_2$) is a fine dry powder and is formed by “slaking” quicklime (CaO) by the addition of water.

$CaO + H_2O \rightarrow Ca(OH)_2 + \text{Heat}$

Hydrated lime is less sensitive to humid climates and therefore makes handling and storage easier than with quicklime. However, prolonged exposure can still cause skin irritations.

For lime stabilisation, the quantity of calcium hydroxide is the active component that reacts with the subgrade or pavement material. In the laboratory, hydrated lime is used to determine the amount of lime to achieve the desired material attributes. However, in construction, quicklime (calcium oxide) is often used which if added at the laboratory determined application rate will result in an increased amount of calcium hydroxide being available. Table 2-1 (Austroads 2006) indicates that quicklime has approximately 30% more effective lime for stabilisation than hydrated lime (i.e. equivalent $Ca(OH)_2$/unit mass when slaked). This needs to be taken into account in the mix design of stabilised materials.

Table 2-1: Properties of quicklime and hydrated lime (assuming pure lime)



	Hydrated lime	Quicklime	Slurry lime
Composition	$Ca(OH)_2$	CaO	$Ca(OH)_2$
Form	fine powder	granular	slurry
Equivalent $Ca(OH)_2$/unit mass	1.00	1.32	0.56 to 0.33
Bulk density (t/m^3)	0.45 to 0.56	1.05	1.25

Pure lime is calcium carbonate containing 40% calcium and no magnesium, whilst good commercial agricultural lime contains 35%–38% calcium and very little magnesium.

2.3 Properties of Reactive Soils

Reactive or expansive soil is described as any soil that consist of clays which will expand or contract with a variation of moisture content (West 1995). Three of the main types of clay minerals that are found in Australian soil are:

- Kaolinite;
- Illite;
- Montmorillonite.

Kaolinite is a low plastic clay and tends to be a non-swelling. Illite is an expansive clay of medium plasticity and low permeability. The most active of the three clays is Montmorillonite which is highly expansive, highly plastic and is extremely impermeable.

The in-situ moisture content of a soil, and its consistency, can be useful in determining the shear strength, compressibility and bearing capacity of the soil. When the soil's natural moisture level is near the liquid limit (LL), the soils will exhibit low strength. However, when the moisture content is near the plastic limit (PL), firmness is more apparent and greater strengths are exhibited.

Lime reacts more quickly with Montmorillonitic clays than with Kaolinitic clays. The difference may amount to a few weeks (Metcalf & Ingles 1972). The addition of lime will improve soil properties such as plasticity index (PI) reduction in Montmorillonitic clays however, this may not occur in Kaolintic clays.

X-ray diffraction testing can be used as a method to determine whether a soil is suitable for lime stabilisation as it identifies the percentages of clay minerals present in a soil.

2.4 Chemistry of Lime Stabilisation

2.4.1 Soil Modification

There has been a lot of work done to understand the processes of lime stabilisation. It is now accepted that small amounts of lime modify the properties of soils through a rapid rate process of ion exchange and flocculation or edge to face reorientation of the clay plate-like particles. Refer Figure 2.1.

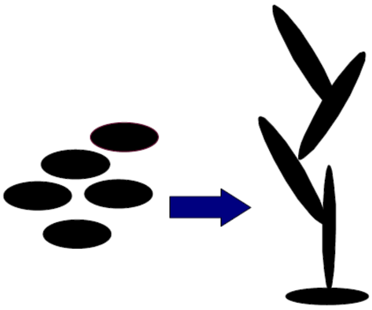


Figure 2.1: Flocculation - Reorientation of Clay Particles

Rapid pozzolanic reactions may also occur during the process. Pozzolanic reaction is discussed further in Section 2.4.3. These reactions cause a reduction in the PI and volumetric change which improves the workability and shear strength of the soil.

After initial mixing, the calcium ions (Ca++) from hydrated lime migrate to the surface of the clay particles and displace water and other ions. The soil becomes friable and granular, making it easier to work and compact (Figure 2.2). At this stage the Plasticity Index of the soil decreases dramatically, as does its tendency to swell and shrink. The process, which is called flocculation or agglomeration, generally occurs in a matter of hours.

Figure 2.2: Lime Flocculated Clay

Soil modification using low levels of lime can be a most economic construction expedient. It permits the use of heavy construction machinery to be used in wet conditions when it is necessary to dry out saturated materials, to bridge across poor subgrade soil or to provide a stable working platform for placement and compaction of pavement layers over the subgrade.

However subgrades modified with small amounts of lime may be more susceptible to moisture entry due to increased permeability, to such an extent that these initial strength gains may be short term and can be reversed through leaching of calcium from the subgrade upon water entry.

Extensive research conducted by McCallister and Petry (1990) demonstrated that the permeability of soils increased significantly after initial lime additions, then decreased as further concentrations were added. These effects seem to be due to flocculation of the clay plates and the development of some pozzolanic reaction products. The resultant soil structure is an open matrix.

Such effects have been confirmed by recent local testing and are suspected to be the main cause of some unsatisfactory historical experiences due to the addition of inadequate amounts of lime.

It is recommended that lime modification should be used with caution, particularly if subgrades are likely to be exposed to water entry (e.g. low lying areas). Successful usage of low concentrations of lime has often been associated with well-drained embankments, or subgrades protected from moisture ingress by extensive subsoil drainage systems.

2.4.2 Soil Stabilisation

If there are sufficient amounts of calcium and pozzolans (silica and alumina particles) available in the soil then the soil will continue to gain strength though the stabilisation process. The silica and alumina particles only become available for pozzolanic reactions to occur in high pH environments. Therefore there must be a sufficient amount of lime available after soil modification to elevate the pH to at least 12.4.

The pozzolanic reactions are slow and permanent but will continue to take place while there are sufficient quantities of lime and pozzolans available. These reactions improve the durability of the material and increase the flexural, tensile and compressive strengths of the bound layer. A densely bonded matrix is less susceptible to moisture, and has improved resistance to carbonation, leaching of lime or erosion (DTMR 2007).

2.4.3 Pozzolanic Reactions

"A pozzolan is defined as a finely divided siliceous or aluminous material which in the presence of water and calcium hydroxide will form a cemented product. The cemented products are calcium-silicate hydrates and calcium-aluminate hydrates" (Little 1995).

The following equations represent the pozzolanic reactions:

$Ca^{++} + OH^{-}$ + Soluble Clay Silica → Calcium Silicate Hydrate (CSH)
$Ca^{++} + OH^{-}$ + Soluble Clay Alumina → Calcium Aluminate Hydrate (CAH)

In the case of montmorillonite clays (generally present in black soil), the pozzolanic reaction does not provide a permanent effect until enough cation exchange has occurred to saturate the layers between the clay minerals.

2.5 Design

DTMR Pavement Design Manual requires all new flexible pavements to be designed for a design life of at least 20-40 year, depending on volume of traffic. All rehabilitation treatments shall be designed for a design life of at least 20 years.

The percentage of lime required to reduce the plasticity index and improve workability is known as the 'Lime Fixation' percentage. The 'Lime Fixation' percentage is the maximum percentage of lime which causes a change in the plasticity index. Beyond this percentage no further change in plasticity index will occur.

The Mixture Design and Testing Protocol (Little 1999, 2000) is designed to produce a mixture that possess the desired structural properties and durability in a pavement layer. The procedure measures engineering properties that are critical to the performance of the lime-stabilised mixture as a structural layer in a pavement system.

DTMR currently use the Lime Demand Test to establish whether a soil is reactive to lime and to determine the amount of lime required to "satisfy cation exchange and short term reactions" (DTMR 2007).

The long term reactivity and durability of the soil is further assessed by conducting 28-day Unconfined Compressive Strength (UCS) Testing. Long Term Reactivity is determined by calculating the UCS at Lime Demand minus the UCS at zero percent lime. A soil is considered non reactive when this value is less than 0.35MPa and reactive when greater or equal to 0.35MPa (DTMR 2007).

The optimum lime content is established from the UCS verses lime content plot. The peak value is determined as the optimum lime content and generally an additional 1% of lime will be added in the field to account for variations in mixing and losses.

Sound specifications such as Main Roads Standard Specification for Insitu Stabilised Subgrades using Quicklime or Hydrated lime (DTMR 2006) use best construction practice for lime stabilisation. It also gives designers the confidence that construction would be carried out to ensure that lime reactions develop and sufficient compaction of the stabilised materials is achieved (AustStab 2008).

Research indicates that accelerated curing (at higher than field temperatures) can alter the mechanism of normal pozzolanic reaction. This is the reason why curing for less than 28 days before testing UCS cylinders is not recommended.

Laboratory testing is always carried out using hydrated lime. If quicklime is used in the field, which is the common practise, the laboratory design dosage will need to be decreased (to compensate for the increased concentration of CaO in quicklime). The level of impurities will depend on the source of the quicklime and the application rate should be adjusted to compensate for impurities.

2.6 Properties and Characteristics of Lime-Treated Soils

In general, most fine-grained soils exhibit improved plasticity (i.e. lower plasticity), workability and volume change characteristics when mixed with lime. An attempt has been made to summarise these effects on the soil due to addition of lime as follows:

2.6.1 Compaction Characteristics

It is desirable to achieve a high state of compaction if the full benefits of adding lime are to be realised. For the same compaction effort, lime treated soil has a lower maximum density than the original untreated soil. In addition, the optimum moisture content increases with increasing lime content. This is illustrated in Figure 2.3.

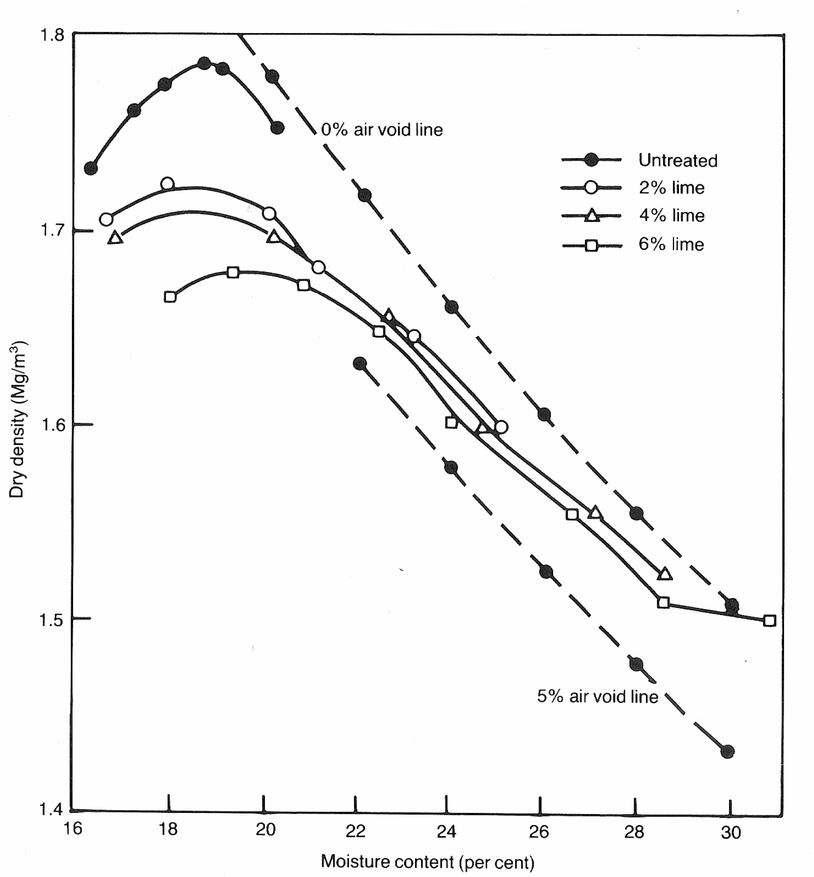


Figure 2.3: Effect of lime content on the moisture content / dry density relation

Source: (Littleton 1988)

The shift in density and optimum moisture content is evidence of the physical changes that occur during lime treatment. If a mixture is allowed to cure and gain strength prior to compaction, a further reduction in the maximum dry density and an additional increase in optimum moisture content may be noted. It is important that the appropriate moisture-density curve, in terms of percent lime used and time of curing, be used for field control purposes.

2.6.2 Plasticity and Workability

The most striking and immediate effect from lime treatment is the action on the plasticity properties of clay soils. The beneficial effect that lime can have on wet heavy clay soils is shown graphically in Figure 2.4 and pictorially in Figure 2.5.

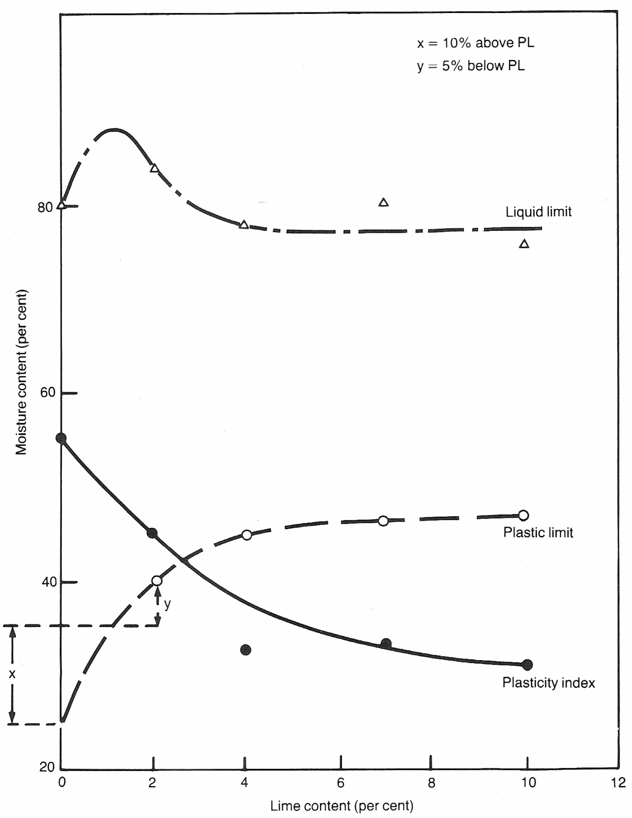
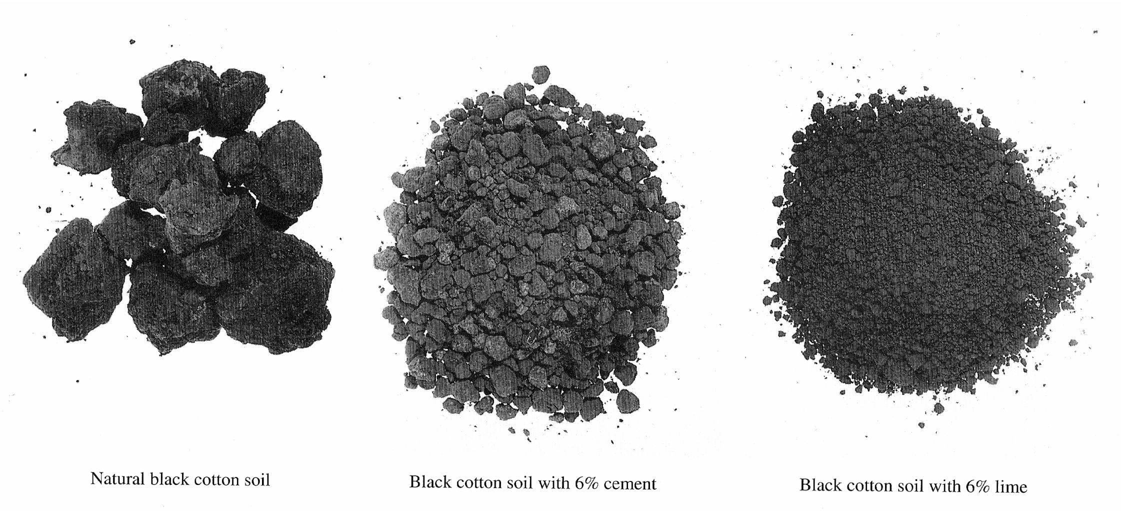


Figure 2.4: Effect of the addition of lime on the plasticity properties of London Clay

Source: (Sherwood 1993)


Natural black cotton soil

Black cotton soil with 6% cement

Black cotton soil with 6% lime

Figure 2.5: Effect of lime and cement on a clay soil after 10 minutes mixing in a laboratory mixer (initial moisture content 29%)

Source: (Littleton 1988)

2.6.3 Swell Potential

Soil swell potential and swelling pressure are normally significantly reduced by lime treatment. These reduced swell characteristics are generally attributed to a decreased affinity for water of the calcium-saturated clay, a reduction in the number of inter-corrected voids, and the formation of a cementitious matrix that resists volumetric expansion.

2.6.4 Strength

The strength of lime-soil mixtures can be evaluated in many ways. The UCS test is the most popular procedure and CBR tests are used to a lesser extent. The strength is a function of soil type, lime type, lime percentage and the curing conditions (i.e. time and temperature). The difference between the compressive strengths of the natural and lime-treated soil has been used as an indication of the degree to which the soil-lime reactions have proceeded. A substantial strength increase indicates that the soil is reactive with lime and can probably be stabilised to produce a higher quality road building material.

The major effect of lime on the shear strength of a reactive fine-grained soil is to produce a substantial increase in cohesion with some minor increase in the angle of internal friction.

At the low confining pressures normally considered to exist in a flexible pavement structure, the cohesion increase is of the greatest significance. It is apparent that large shear strengths could be developed in cured soil-lime mixtures.

The tensile strength of a lime-soil mixture increases as the UCS increases. Two test procedures are commonly used to measure the tensile strength: - the indirect tensile test and the flexural beam test. The ratio of indirect tensile strength to UCS is approximately 0.13 which can then be used for normal design purposes (Little 1987).

The common method for evaluating the flexural tensile strength is the flexural beam test as this value can be related to the stabilised slab which bends under the action of traffic loading in the field. A realistic estimate of the flexural strength (modulus of rupture) is 0.25 times the cured UCS (Little 1987).

2.6.5 Deleterious Materials

The presence of deleterious materials such as organic matter and sulphates can respectively, interfere with the hydration process and pozzolanic reactions. For stabilisation to be effective the sulphate content (water soluble), and organic content must be equal to or less than 0.3% and 1% respectively (DTMR 2007).

Organic molecules 'can absorb calcium cations or interact with soil exchange sites and hence prevent them from reacting with the soil as they normally would to produce cation exchange and pozzolanic reaction' (Little 1995). Lime stabilisation relies on an increase in the pH of the soil for pozzolanic reactions to occur. Depending on the type and amount of organic material present in the soil, it may retard or completely inhibit the change in pH (Sherwood 1993). Lime stabilisation of soils with greater than one percent of organic materials may still be possible with the addition of higher percentages of lime.

Any sulphates that are present in the soil or water can be harmful to the lime stabilisation process due to the formation of calcium-sulphate-aluminate-hydrate crystals. If calcium-sulphate-aluminate-hydrate crystals form after compaction it can destroy the pavement because it will result in heaving of the stabilised layer (Little 1995).

2.6.6 Permeability

Permeability of some lime reactive soils will tend to initially increase upon lime treatment. The initial increase will then usually be followed by a decrease in permeability, although it will often remain significantly higher than the level prior to lime treatment. However, it has also been shown that curing may have a positive effect on permeability. Together, curing and ongoing pozzolanic reactions can decrease the permeability of some lime stabilised soils. In some instances the permeability will decrease to approximately that of the natural soil (Little 1995).

McCallister & Petry (1990) undertook some research on three lime treated North Central Texas soils and found that the permeability of these soils were 7 to 300 times higher than the natural soils. They also found that upon leaching, the permeability decreased when the percentage of lime was low (3% – 4%). However when the lime content was at optimum for strength gain (6% – 7%), the affect on permeability upon leaching was insignificant (McCallister & Petry 1990).

2.7 Stabilised Subgrade Construction

The construction steps involved in stabilization and modification are similar. Generally, stabilization requires more lime and more thorough processing and job control than modification.

Basic steps include:
- scarifying or partially pulverizing soil;
- spreading lime;
- adding water and mixing;
- compacting to maximum practical density; and
- curing prior to placing the next layer or wearing course.

In the past it was common practice to scarify before spreading. Today, because of the availability of superior mixers, lime is often applied without scarification. Lime trucks can also negotiate the roadway more readily if it is compacted, rather than scarified, particularly on wet soils.

Mixing of the lime is undertaken by in-place mixing, plant mixing or pressure injection. The most common method is in-place mixing which involves mixing the lime with the existing material at the construction site.

Adequate mixing is absolutely essential to achieve satisfactory results in lime stabilisation. Whilst some soils may only require one-stage mixing, heavier and more plastic soils require two-stage mixing.

The two-stage process consists of preliminary mixing, moist curing for a period of time and final mixing or remixing. In the preliminary mixing operation, the objective is to distribute the lime throughout the soil and thereby allow the lime to start breaking down the clay particles. In order to optimise the chemical reactions of cation exchange and pozzolanic reactivity it is essential to leave the mixed soil to ameliorate or “mellow” for a period of time. Generally after a 24 to 48 hour delay (Little 1995) the clay becomes friable enough to easily allow better mixing in the final mixing.

2.7.1 Department of Main Road Construction Procedures

Department of Transport and Main Road Technical Specification MRTS07A (DTMR 2006) specifies the following construction procedures for lime stabilising a pavement subgrade.

2.7.1.1 GENERAL

Lime stabilisation of subgrade material shall be carried out as a two day operation to facilitate the two-stage mixing process.

The stabilising agent shall be spread using a purpose-built spreader and the stabilising agent and water shall be incorporated into the material using a reclaimer/stabiliser or stabiliser.

Alternatively, a reclaimer/stabiliser with a calibrated integrated spreader/applicator may be used to incorporate the stabilising agent and water directly into the material to be stabilised. Where a reclaimer/stabiliser with calibrated integrated spreader/applicator is used, quicklime shall not be used as the stabilising agent.

2.7.1.2 DAY 1

A single lime pass at a rate of up to half the required rate shall be applied. Where quicklime is spread over the subgrade, it shall be slaked. In situations where hydrated lime is used, slaking is not required before mixing the stabilising agent into the soil. The hydrated lime or lime slurry formed from the slaking of quicklime shall then be mixed into the material. The depth of mixing shall not exceed 90% of the specified stabilisation thickness. The material shall be lightly rolled to seal the surface prior to the completion of work on that particular day.

2.7.1.3 DAY 2

After the overnight amelioration period, the balance of the required lime shall be spread. Where quicklime is spread over the subgrade, it shall be slaked. In situations where hydrated lime is used, slaking is not required before mixing the stabilising agent into the soil. The hydrated lime or lime slurry formed from the slaking of quicklime shall then be mixed into the material. Notwithstanding this, a minimum of two mixing passes shall be completed.

2.7.1.4 SPREADING OF STABILISING AGENT

Lime shall be spread at a maximum spread rate of 12 kg/m^2. The number of passes shall be calculated to comply with this requirement. The stabilising agent shall be uniformly spread over the insitu material at a controlled rate (mass per unit area, kg/m^2).

The total rate of spread shall be such that the stabilising agent spread rate for the compacted material is within the specified tolerances.

Once the stabilising agent has been spread, no traffic, other than the construction plant employed for the stabilisation work, shall travel over it.

After each spreading run at least one mixing run, trimming and/or compaction, as required, shall be completed. Further mixing operations between spreading runs shall be completed.

2.7.1.5 SLAKING

Quicklime shall be slaked with sufficient water to allow complete hydration such that the material remains friable after slaking and no further exothermic reaction occurs when further water is added to the lime.

All through traffic shall be stopped during any slaking operation.

2.7.1.6 MIXING

Mixing shall be achieved using a reclaimer/stabiliser or stabiliser. Where quicklime is used as the stabilising agent mixing shall not commence until slaking is complete.

On day two, all lime spread shall be mixed into the soil within 6 hours of each application.

The depth of each mixing pass, except the final mixing pass, shall not exceed 90% of the full depth of stabilisation specified. The final mixing pass shall be to the full depth of stabilisation specified. Mixing shall proceed until all material other than stones can pass a 19 mm AS sieve, at least 60% of such material can pass a 9.5 mm AS sieve and the lime is uniformly mixed through the soil. Notwithstanding this the minimum number of mixing passes shall be two.

The distribution of the stabilising agent and water shall be uniform throughout the full depth, and over the entire area, of the material to be stabilised. The moisture content shall be adjusted as necessary during the mixing process to maintain moisture content of between 97% and 101% of OMC (optimum moisture content) of the stabilised soil as determined by tests using standard compactive effort.

Water shall be added by means of a controlled pressure feed distribution system located inside the mixing chamber of the reclaimer/stabiliser or stabiliser. This system shall be capable of spraying varying rates across its width.

Where test results or visual inspections indicates that any of the requirements stated in this Clause have not been met, additional mixing passes shall be carried out to improve the uniformity of the –

a) Materials to be stabilised;
b) Distribution of the stabilising agent; and
c) Distribution of water.

After each spreading run, mixing, trimming and/or compaction, as required, shall be completed.

2.7.1.7 TRIMMING BEFORE COMPACTION

After completion of stabilisation and before compaction commences, the surface shall be trimmed to approximately the alignment, heights and shapes specified for the completed work, and any depressions shall be filled with additional stabilised material that is mixed and placed within its allowable working time.

2.7.1.8 COMPACTION

Light compaction of the stabilised material shall be completed after each application of lime has been incorporated into the material. This shall be carried out using an appropriate roller that is capable of achieving relatively uniform compaction over the depth of the stabilised layer.

Final compaction shall be completed within the allowable working time and the stabilised layer shall be compacted.

2.7.1.9 TRIMMING AFTER COMPACTION

No marks caused by a pad foot or other roller shall remain on the surface after final trimming.

All trimming shall involve cutting to waste. All material cut to waste shall be disposed of in accordance with Clause 11 of MRTS01 *Introduction to Technical Standards*.

2.7.1.10 PERIOD FOR COMPACTION AND TRIMMING

Compaction and trimming shall be completed within the allowable working time.

2.7.1.11 CURING

A curing operation shall commence immediately after the completion of compaction. The stabilised layer shall be cured using water by maintaining the layer surface and edges in a continuously damp condition, using a uniformly applied fine mist, until the stabilised layer is covered by an overlying pavement layer or a sprayed bituminous surfacing with cover aggregate. Water shall be applied in a manner such that slurrying of the surface, pavement instability and erosion and/or leaching of the stabilising agent are all avoided.

2.7.1.12 PROTECTION

The surface of the compacted layer shall be kept moist, in good order, in good condition and free from contamination until an overlying pavement layer or a sprayed bituminous surfacing with cover aggregate is placed. Construction or other traffic shall not use a compacted layer where damage to the surface may occur. Placement and compaction of any subsequent layer shall be carried out within 48 hours of density testing subject to the stabilised later complying with all specified requirements.

2.7.1.13 CONDITIONS UNDER WHICH STABILISATION SHALL NOT PROCEED

The entire stabilisation process shall not proceed in any of the following situations –

a) during rainfall;
b) when rainfall appears to be imminent;
c) during periods when the wind is strong enough to cause particles of the stabilising agent to become airborne;
d) during conditions that may result in the work causing nuisance or danger to people, property, the environment, or live stock;
e) when the pavement temperature, measured 50 mm below the surface, drops below 10ºC; or
f) when the air temperature, measured in the shade, exceeds 40ºC.

2.8 Amelioration

Two-stage mixing process consists of preliminary mixing, moist curing for a period of time and final mixing or remixing. In the preliminary mixing or first application, the objective is to distribute the lime throughout the soil and allowing the amelioration or mellowing operation to take place.

Highly reactive clay soils require a high percent of lime to be mixed for effective soil modification. The recommended maximum spreading rate for hydrated lime is 20kg/m^2 and for quicklime is 12 to 15kg/m^2 (AustStab 2008). This generally requires the mixing to occur in two passes.

This two-stage mixing process is now standard DTMR practice which involves applying half the lime dosage in the first pass then the remaining half is applied in the second pass after waiting a specified amelioration time period.

DTMRs current standard specifies this amelioration period between passes should be 24 hours, whilst AUSTROAD Guide to Pavement Technology Part 4D: Stabilised Materials manual indicates a “mellowing” period of 4 – 72 hours between passes (Austroads 2006). This broad timeframe is dependent on the plasticity of the clay.

Conversely, a mellowing time period of 24 to 72 hours or more is indicated in Dallas Little's Handbook for Stabilisation of Pavement Subgrades and Base Courses with Lime (Little 1995).

Amelioration is necessary in heavy clays as adequate mixing is difficult to achieve in a single pass as heavy clays tend to clump together. Historically, amelioration was needed to breakdown clay so that low capacity rotary hoes could work in the clay soil environment. This problem has largely been overcome with modern purpose built machines.

The two-stage mixing standard has been set by the maximum quantity of lime, but in some circles is considered conservative. It is important to note that subgrades are the foundation for roads which will be buried for a long time.

Allowing 24 hours between passes permits the lime to flocculate clay particles and improve workability. Amelioration results in better mixing in the second run and construction becomes easier.

The major issue with a 24 hour period for amelioration is it takes a longer time to finish construction. This delays the contractor by having to bring a machine back a second day or having machinery sitting idle which is expensive especially on small projects. Construction is delayed as the contractor can't continue construction until after the second pass.

During the amelioration period the open subgrade is exposed to potential weather such as rain for a longer period. The subgrade soils mixed with lime in the first pass become more permeable and if rain falls on the subgrade, water infiltration will become a problem. This will delay construction as this wet material is difficult to compact and may even require the stabilisation process to recommence.

Another issue is if the mixing machine breaks down, a second machine would have to be sourced within 24 hours which can be difficult in remote areas. This would extend construction delays and possibly require the lime stabilisation to be started again.

Traffic delays can be expected and could be extended if the subgrade stabilisation has to be recommenced.

2.9 Pavement Performance

The material properties of lime-stabilised soils, as related to their overall pavement performance, can be divided into four categories (Little 1999):

2.9.1 Strength

The most obvious improvement in a lime reactive soil is strength gained over time. The various strength parameters impacted by pozzolanic reactions that occur include UCS, tensile strength, flexural strength and CBR.

2.9.2 Resilient Modulus / Stiffness

Concurrent with the strengthening of a soil brought about by pozzolanic reactions, are changes in the stress-strain relationship of the material (Little 1999). Materials in the laboratory (repeated-load triaxial and indirect tensile test) and in the field (impulse deflection test and vibrational test) both confirm significant increases over time in the resilient properties of lime treated materials.

2.9.3 Fracture and Fatigue

Flexural fatigue strength is related to the number of loads that can be carried by a material at a given stress level and is an important consideration in the evaluation of lime-soil mixtures. The strength gain effects produced by pozzolanic reactions are often substantial for reactive soils.

2.9.4 Durability

The ability of lime stabilised materials to resist the detrimental effects of moisture and freeze-thaw cycling over time has been evaluated in several ways in both the laboratory and field. The results of these evaluations have often shown only slight detrimental effects from the environment on the levels of strength / stiffness produced by the addition of lime.

2.10 Lime Stabilisation in Queensland

The stabilisation of subgrades with lime is not a new innovation. Ipswich City Council extensively used lime stabilisation in roadworks for many years from the 1950’s to 1970’s.

The Queensland Department of Main Roads extensively trialled lime stabilisation in roadwork's until the late 1970's when it fell from favour. A couple of projects that brought about the downfall of lime stabilisation were the Gladfield project in the Border District and the Palmerston Highway project in the Peninsula District. The design of the lime content during this era was based on the Lime Fixation Percentage method.

Current research into lime stabilisation has found that the Lime Fixation method grossly underestimates the percentage of lime actually required for stabilisation. The lime percentages applied during the late 70's were only enough to promote soil modification. Leaching of the lime during wet conditions would have rapidly reversed any soil improvements.

Research into lime stabilisation was not again conducted in Queensland until after the 1996 Transport Technology Forum. A steering committee was formed to review past lime stabilisation projects and to oversee the design and performance of two new trial projects near Warwick, Queensland. The steering committee consisted of representatives from Industry, Local Government, DTMR Transport Technology and DTMR Border District.

The paper 'Update on Lime Stabilisation', written by Peter Evans (Evans 1997) reports on the design and performance of the Killarney Project and the design of the Cunningham Highway - Freestone Creek to Eight Mile Intersection Project. Evans reports on the performance of the Freestone Creek to Eight Mile Intersection Job in a later paper, 'Rethink of the Design Philosophy of Lime Stabilisation (1998)'. At the time of Evans' paper (Evans et al. 1998) coming to print, the DTMR Technology and Environment Division were commissioning a review into the research project to determine whether it had been a success.

The Killarney Project

Two laboratory methods were investigated prior to deciding on the lime percentage for the Killarney Project. The first method investigated was the Lime Fixation Method. Data showed that 3-4% of lime was required to stabilise the soil's plastic limit. Data from the second method, the Eades and Grim Method showed that 4 percent of lime was required to reach a pH of 12.4 and 8 percent was required to stabilise the pH level (Eades & Grim 1960).

Due to perceived risks, the DTMR had specified the use of hydrated lime for the project. However, after negotiations between the principal and the contractor, the use of quicklime was eventually approved. During the application of the quicklime no incidents occurred and it was noted that quicklime did not cause the dust problems which were previously encountered when using hydrated lime.

Sections of the project were trialled with concentrations of 3 to 6 percent quicklime in an effort to determine the actual lime demand percentage. After 9 months of monitoring by the steering committee the following conclusions were made:

- Lime stabilisation was an effective method for expediting the construction process during wet conditions;
- There was a noticeable difference in appearance and insitu CBR between the 3% and 6% lime stabilised sections. The 6% quicklime section had an insitu CBR of almost triple that of the 3% stabilised section.

In Evans' paper it was proposed that the steering committee continue performing deflection testing on the pavement and monitoring its long-term performance. Assessments will continue to be made of the modulus to assess whether it improves over time.

Cunningham Highway - Freestone Creek to Eight Mile Intersection

The Freestone Creek to Eight Mile Intersection project consisted of widening a 4km section of the Cunningham Highway, 13 km north from Warwick. Due to the presence of highly expansive clays it was necessary to provide a deep pavement to bridge over the poor subgrade. However, this was not feasible due to being constrained by the 300mm existing embankment. A more viable option was to lime stabilise the subgrade to a depth of 300mm.

Following recommendations in the Handbook for Stabilization of Pavement Subgrades and Base Courses with Lime (Little 1995), 28-day UCS tests were carried out for various lime contents. This laboratory test method is referred to as the Thompson Method.

During laboratory testing for this project, comparisons were made between the Lime Fixation, Eades and Grim and Thompson methods. Both the Lime Fixation and Eades and Grim Methods demonstrated lime percentages that were considerably less than the Thompson Method. After applying the hydrated lime to quicklime conversion factor and allowing for losses, 8% quicklime was adopted in the design. A 20m section of subgrade at the Eight Mile end of the works was not stabilised so that comparisons of UCS could be made.

Six weeks after the project was completed deflectometer testing was conducted on the pavement using a 40kN falling weight. Analysis showed that the modulus of the stabilised subgrade was significantly higher than that of the overlying base course material.

UCS cylinders were taken during the stabilisation process and tested during the initial twenty six months following completion. Results indicate that the initial strength gains are permanent and ongoing.

2.11 Chapter Summary

Subgrades are the foundation upon which road pavements are built. Pavement failures largely tend to occur as a result of problems with the subgrade. They are typically due to loss in strength which can be attributed to the poor strength of reactive clay material and moisture infiltration into the pavement material from the insitu clay material below or ingress from the top or side of pavement.

Initial results from the Cunningham Highway lime stabilisation trial project undertaken by DTMR tended to indicate that strength gains were permanent and ongoing. There is not sufficient data to confirm whether strength gains are long term. This research will undertake testing and analysis to determine whether the UCS of the subgrade has maintained an acceptable strength on the Cunningham Highway lime stabilised trial site.

There has been a significant amount of research conducted in the interaction of lime with clay soils but an area of research that appears deficient is the assessed of the amelioration time period between lime mixing passes when constructing the lime stabilised subgrade. Mixing is required to be completed in two passes to allow the lime to breakdown the clay particles and to meet specified maximum allowable quantities of lime in each pass. The amelioration time period has been set based on tradition and experience. This research will test and analyse a range of time periods less than 24 hours to determine what time period will achieve 1.5 MPa UCS.

3 LIME STABILISED SUBGRADE STRENGTH

3.1 Background

Subgrade stabilisation was completed on the Freestone Creek to Eight Mile Intersection section of the Cunningham Highway in 1998. The project consisted of widening a 4km section, however as a result of the presence of highly expansive clays, the most viable option was to lime stabilise the subgrade to a depth of 300mm.

The pavement design overlaying the stabilised subgrade consisted of 300mm of gravel and a wearing surface of 100mm asphalt. A further 100mm of asphalt was laid to the full width of road in 2005. Visual evidence indicates the pavement is performing well.

Figure 3.1 shows the lime being spread to the subgrade of the widening on the Cunningham Highway trial.

Figure 3.1 Lime Spreading on Widening to Cunningham Highway Trial Project

3.2 Design of Lime Content

For lime stabilisation, the recommended method conducted by DTMR is the Thompson Method. This involves the use of pH testing (Lime Demand) to determine whether soil is “reactive” to lime and to estimate approximate lime content and pH testing augmented by 28 day UCS testing to establish the optimum lime content (Lime Stabilisation Optimum). The optimum lime design occurs when the plot of the UCS versus the lime percentage attains a maximum. An additional 1% lime should be added in the field to allow for variations due to losses and uneven mixing.

The design method results are summarised as follows:

a) Lime Fixation Percentage indicates the percentage of lime to cause the plastic limit to stabilise (refer to Figure 3.2).
b) Lime Demand indicates the percentage of lime required to achieve pH of 12.4 (refer to Figure 3.3).
c) Strength indicates the percentage of lime required to achieve a minimum 28 day UCS of 1.5MPa in accordance with Thompson's Method (refer to Figure 3.4).

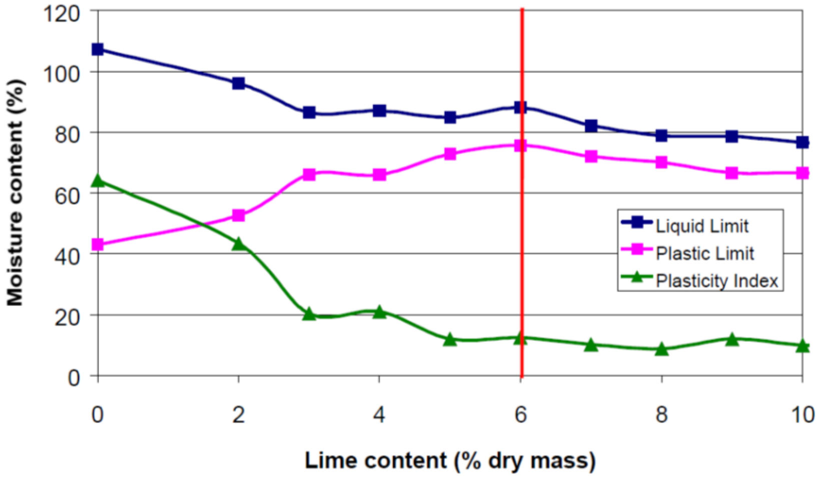


Figure 3.2: Lime Fixation Percentage

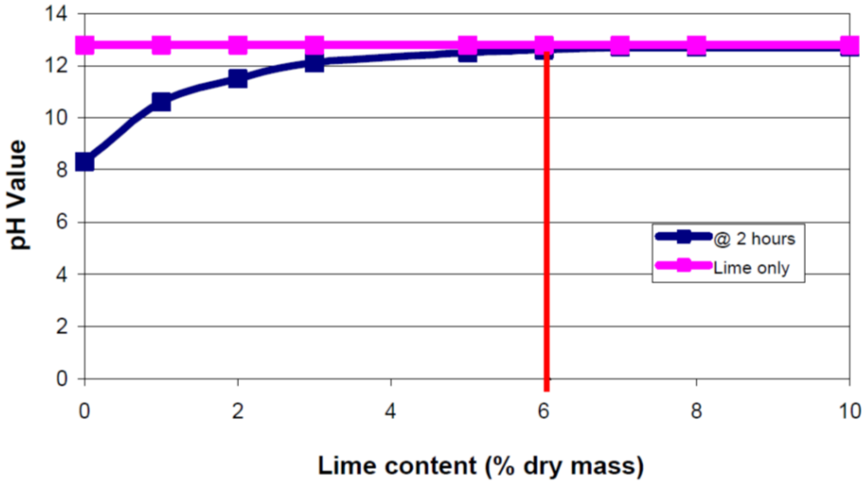


Figure 3.3: Lime Demand Method

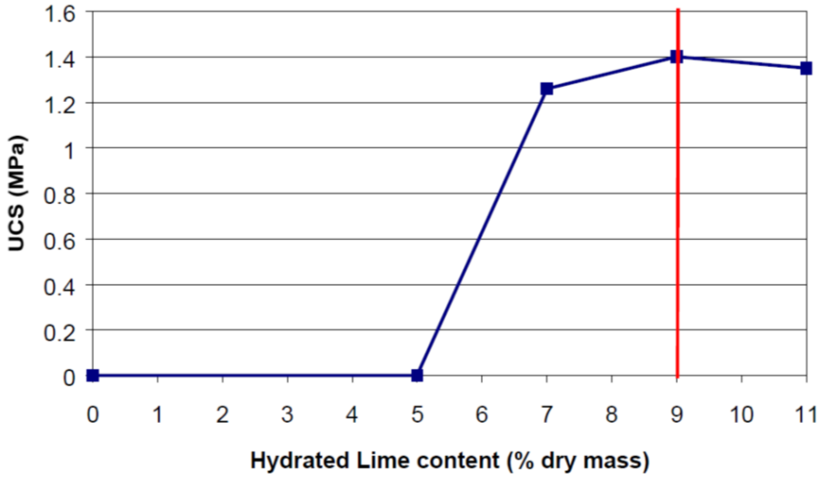


Figure 3.4: Thompson's Strength Method

For the Thompson method, the peak 28 day UCS results occur with 9% Hydrated Lime (Figure 3.4). Applying a conversion factor of 0.757 (adjustment of hydrated lime to quicklime used in the field), this equates to 6.8% quicklime. Allowing an additional 1% for variations, the design adopted was 8% quicklime.

3.3 Subgrade Strength at Construction

Deflectometer tests were conducted using the 40kN Heavy Weight Deflectometer, approximately 6 weeks following job completion, and these were back analysed from deflection data to determine the Modulus. The results of this analysis are shown in Figure 3.5 and indicate that the stabilised subgrade was in fact stronger than base course gravel (CBR 80) which is used as the top layer in all DTMR road pavements.

The UCS values are typically derived by dividing the Modulus values by 1000, hence the UCS of the stabilised subgrade ranged from approximately 0.6 to 1.0MPa.

The CBR value of subgrade can be determined by using the empirical relationship:
Modulus (MPa) = 10 x CBR. This equation is, at best, an approximation and modulus has been found to vary in the range 5 x CBR to 20 x CBR. A maximum value of 150MPa is normally adopted for subgrade materials. (Austroads 2010).

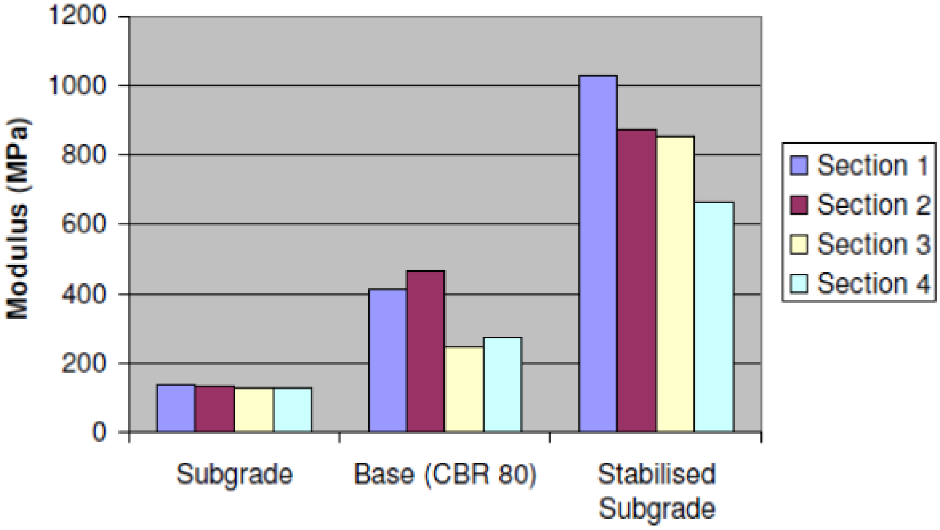


Figure 3.5: Comparison of Modulus

UCS cylinders were taken at various sections during the stabilisation process and tested during the initial twenty six months following completion. Results indicate that the initial strength gains appeared to be permanent and ongoing. This is clearly shown in Figure 3.6.

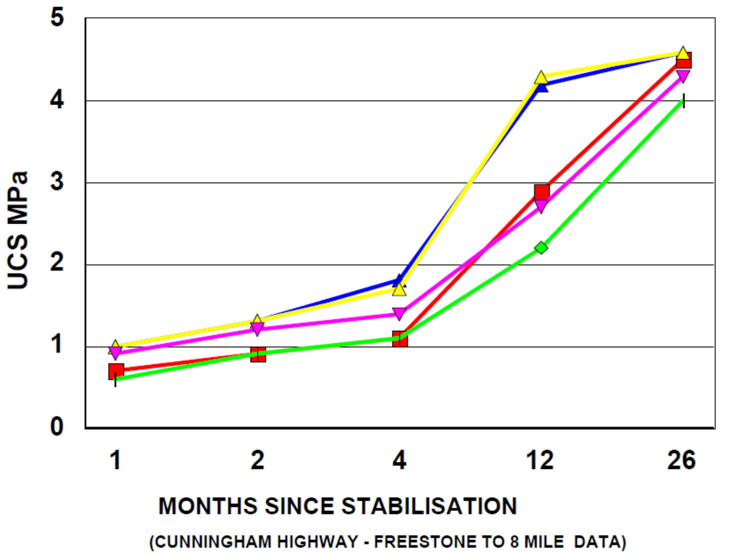


Figure 3.6: UCS Results after Construction

3.4 Present Subgrade Strength

The aim of this part of the research was to determine the strength of the stabilised subgrade in comparison to previous testing. This would confirm whether the strength still achieves the minimum requirement of 1.5Mpa. This investigation includes the following:

- Obtain core samples of existing subgrade material;
- Test UCS of upper, middle and lower layers of the pavement individually;
- Analyse UCS test results;
- Compare results to previous data.

3.4.1 Methodology

In consultation with DTMR staff a site was chosen to obtain undisturbed cylindrical samples suitable for UCS testing. This site was located at the eastern end of the section and the coring was undertaken on 16 June 2011 by DTMR Soils Laboratory staff from the Southern Districts office.

It was decided to take the core samples from the shoulder to minimise impacts on the existing pavement in the travel lanes of the highway. The process involved scarifying the existing asphalt (Figure 3.7) then excavating the pavement gravels to the subgrade. Six core samples were taken from the subgrade (Figure 3.8) for laboratory testing to test three layers within the subgrade material. These were top, middle and bottom sections of the core samples.

Figure 3.7: Asphalt Excavation

Figure 3.8: Core Holes in Subgrade

The visual appearance of the core samples indicated that the subgrade was "extremely hard" and akin to a concrete look as seen in Figure 3.9 and Figure 3.10. This was confirmed when attempting to conduct a dynamic cone penetrometer (DCP) test which resulted in refusal.

Figure 3.9: Core Sample (End View)

Figure 3.10: Core Sample (Side View)

DTMR laboratory specimens are typically compacted in standard compaction moulds with length-to-diameter (L/D) ratio approximately equal to 1. This ratio will influence the UCS values that are obtained from core samples, so it was important that the post construction core samples should have a similar L/D ratio for the values to be relative.

3.4.2 Laboratory Testing

Laboratory testing was performed by the DTMR Materials Section in Herston, Brisbane on 21 June 2011 in accordance with DTMR Material Testing Manual (DTMR 2010).

The core samples were cut to provide representative samples for the top, middle and bottom portions of selected core samples. The core sample details are indicated in Table 3-1.

Table 3-1: Core Sample Details

Core No. (Location)	Core Diameter Mm	Core Height mm	L/D	Insitu Density T/m^3	Mass Kg
2 (Top)	143	127	0.89	1.844	3.762
2 (Bot)	143	134	0.94	1.794	3.861
6 (Mid)	143	199.4	1.39	1.760	5.635

Unconfined Compressive Strength of Compacted Material was undertaken generally in accordance with Q115 – 2010 from the Department of Main Roads Materials Testing Manual (DTMR 2010).

This method describes the procedure to determine the UCS of compacted specimens of soils and crushed rock which have been either modified or stabilised with a stabilising agent or are in their natural state. The method has particular application as a laboratory design procedure but can also be used to test field-moulded specimens in order to check field processes.

This method was developed in-house by DTMR using techniques evolved through internal departmental research investigations.

The steps to determine the UCS of core samples were as follows:

- Determine the average diameter (D) of each test specimen to the nearest 0.1 mm from two diameters measured at right angles to each other;
- Place the test specimen centrally on the lower platen of the CBR machine such that the top of the specimen is uppermost;
- Apply a compressive force to the specimen and record the maximum applied force (F) in kN;
- Using the achieved compaction moisture content, calculate the achieved compacted dry density and the achieved relative compaction for each specimen as detailed in Test Method Q145A;
- Calculate the unconfined compressive strength for each specimen to the nearest 0.05 MPa as follows:

$$UCS = \frac{1273 * F}{D^2}$$

Where,
UCS = unconfined compressive strength (MPa)
F = applied force at failure (kN)
D = average diameter of specimen (mm)

The UCS formula above is identical to the stress formula which was used to obtain results in this testing.

$$Stress = \frac{Load}{Area}$$

In addition to the UCS test, it was decided to conduct a Capillary Rise test on the core samples to determine the water absorption, swell and capillary rise of water in the compacted stabilised subgrade materials. This test was undertaken in accordance with AS 5101.5 – 2008: Absorption, swell and capillary rise of compacted materials.

3.4.3 Results

UCS samples were taken from the representative samples for the top, middle and bottom portions of selected core samples. The test specimen were placed in the CBR machine, loaded with a compressive force until the specimen failed as shown in Figure 3.11.

Figure 3.11 Test Specimen Compressed in CBR Machine until Failure

Compressive force results obtained are summarised in Table 3-2 and pictorially represented in Figure 3.12 below.

From these results it can be seen that the target 28 day strength of 1.5MPa was easily achieved.

Table 3-2: Core Sample Strength Results

Core No. (Location)	Core Area mm^2	Maximum Force kN	UCS MPa
2 (Top)	16061	87.97	5.50
2 (Bot)	16061	129.9	8.10
6 (Mid)	16061	47.088	2.90

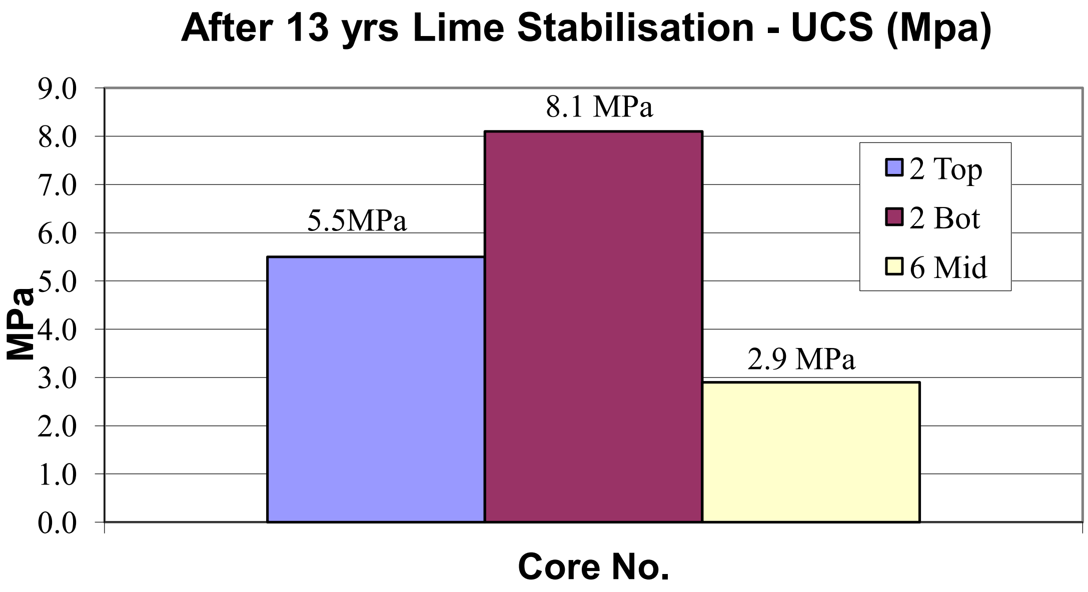


Figure 3.12: Unconfined Compressive Strength Results

3.4.4 Long Term Unconfined Compressive Strength

The UCS test results from June 2011 were averaged and collated against previous UCS test result data which were also averaged between sections. The analysis of these results is shown in Figure 3.13.

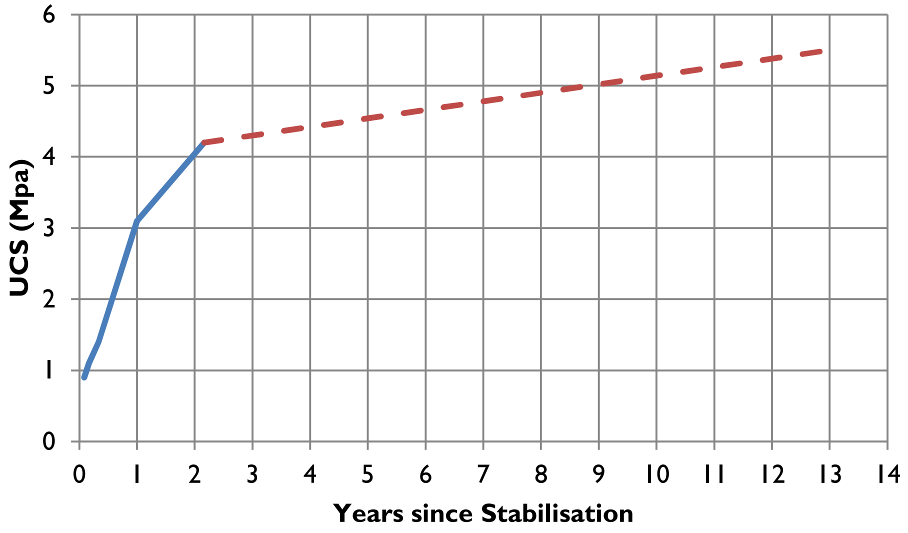


Figure 3.13 Strength after 13 Years

Analysis of test results from June 2011 shows that the average UCS has increased to approximately 5.4MPa compared to strength of approximately 4.1MPa two years after construction.

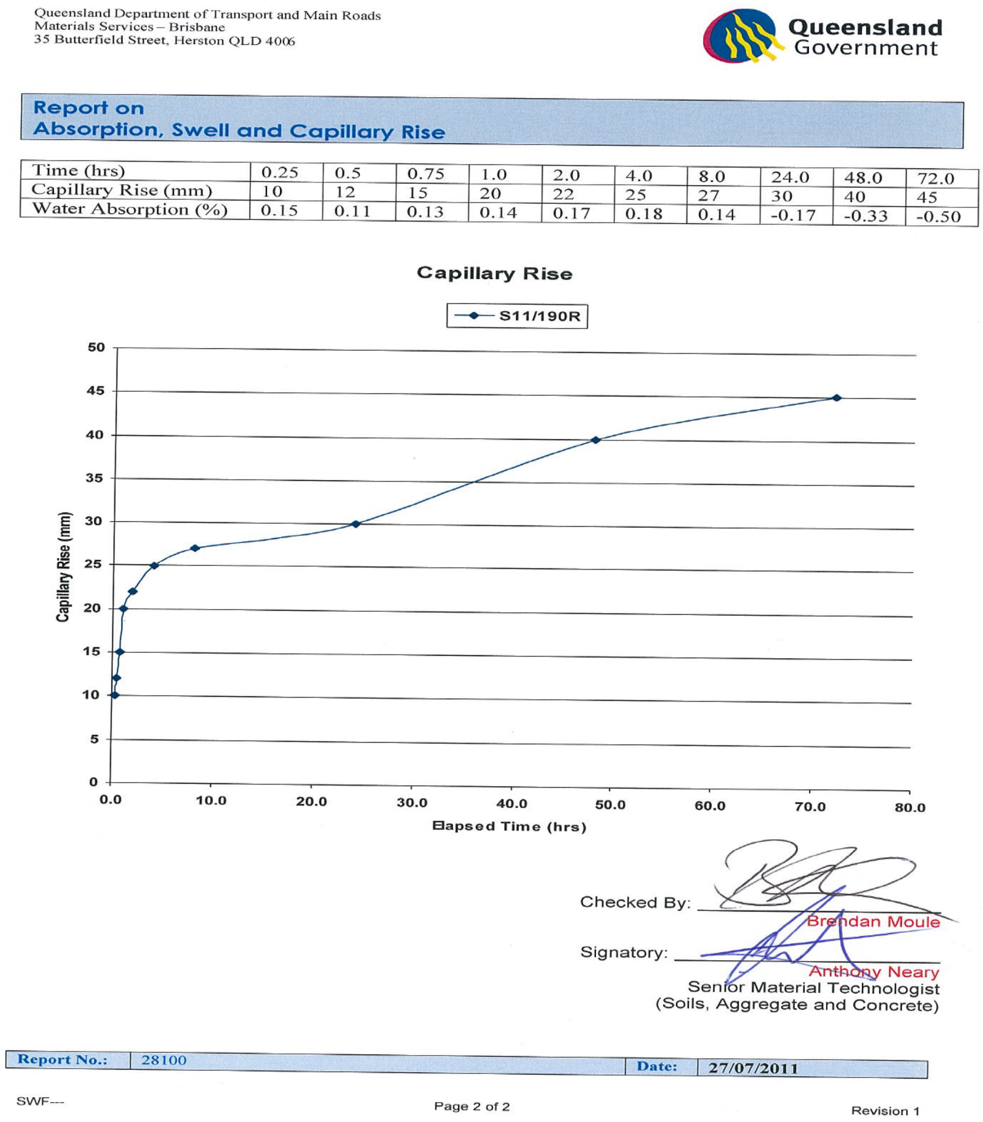
The current strength is still well above DTMRs required 1.5MPa and appears to be on target to reach the expected design life of 20 years and possibly well beyond.

3.4.5 Capillary Rise

An assessment was also undertaken of the long term effect on Capillary Rise. There was some sign of visual moisture rising up the side of samples, as seen toward the bottom of sample in Figure 3.14. Moisture appeared to rise all the way to the top of one sample.

Figure 3.14: Capillary Rise Test

The samples were air dried and the capillary rise test was repeated again. The samples were weighed with time and the resulting percentage of water absorption was less than 1%. The moisture rise to the top of one sample in the initial testing was only on the outer skin of sample and did not enter the material at all. In fact some of the results after 24 hours had negative absorption. This indicated that they were drying faster than absorbing water. An example of these results is shown in Figure 3.15.



Queensland Department of Transport and Main Roads
Materials Services – Brisbane
35 Butterfield Street, Herston QLD 4006

Queensland Government

Report on
Absorption, Swell and Capillary Rise

Time (hrs)	0.25	0.5	0.75	1.0	2.0	4.0	8.0	24.0	48.0	72.0
Capillary Rise (mm)	10	12	15	20	22	25	27	30	40	45
Water Absorption (%)	0.15	0.11	0.13	0.14	0.17	0.18	0.14	-0.17	-0.33	-0.50

Checked By: Brendan Moule

Signatory: Anthony Neary
Senior Material Technologist
(Soils, Aggregate and Concrete)

Report No.:	28100	**Date:**	27/07/2011

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Revision 1

Figure 3.15: Capillary Rise Test Result

Deterioration of road conditions in reactive soils mostly results from water entering the pavement gravels which weakens the strength of material. This test clearly demonstrates that the stabilised subgrade material on the Cunningham Highway trial is impermeable to moisture ingress. The lime stabilised subgrade has protected the pavement gravels from moisture rise that could emanate from the insitu material below the subgrade.

The lack of permeability of the lime stabilised subgrade material on this trial section appears to have the significant benefit of extending the strength and durability of the road pavement.

3.5 Discussion

Test results for the initial 28 month period after construction of the widening in 1998 indicated strength gains were ongoing. After 13 years the strength of the subgrade has also continued to show increases. However, due to a lack of test data in the last 11 years it is not certain whether the strength is in fact increasing or if a peak has been reached and the strength is on the decline. Further UCS testing should be conducted in the next 12 to 24 months to confirm that the subgrade strength is in fact increasing.

The length-to-diameter (L/D) ratio will influence the UCS values that are obtained from the core samples. DTMR laboratory specimens are compacted in standard compaction moulds with L/D ratios approximately equal to 1. The UCS test results for the top and bottom sections of the stabilised subgrade could be slightly elevated due to the L/D ratio being less than 1. The bottom section was closest to ratio of 1 but also recorded the highest strength value. Conversely, the UCS for the middle section could be slightly low as the L/D ratio of this sample was greater than 1.

The reason for the significant variation in strength between layers is unknown. The design thickness for the stabilised subgrade layer was 300mm and the mixing and compaction process was carried out in one layer for the full 300mm.

The high UCS value for the bottom layer is not unexpected as this layer is in direct contact with insitu material and continued moisture from the insitu material could have contributed to continued pozzolanic reactions further increasing the strength. The lower strength value for the middle layer could be as a result of lack of compactive effort during construction.

During laboratory testing for this project, comparisons were made between the Lime Fixation, Eades and Grim and Thompson methods. Both the Lime Fixation and Eades and Grim Methods demonstrated lime percentages that were considerably less than the Thompson Method. All UCS test results obtained indicate justification for the use of Thompson Method as the higher quantity of lime added appears to have contributed to higher strengths in the subgrade.

It was noted during literature review that a 20m section of subgrade at the Eight Mile end of the works was not stabilised so that comparisons could be made against the stabilised section. During recent reviews there was no recent data found and it is recommended that this section should be strength tested to give a comparison to recent lime stabilised subgrade strength testing.

3.6 Chapter Summary

The aim of this part of the research was to determine the strength of the stabilised subgrade in comparison to previous testing. This would confirm whether the subgrade strength still achieves the minimum DTMR requirement of 1.5Mpa.

For stabilisation, the recommended method conducted by DTMR is the Thompson Method. This involves the use of pH testing (Lime Demand) to determine whether soil is "reactive" to lime and to estimate approximate lime content and pH testing augmented by 28 day UCS testing to establish the optimum lime content.

This investigation involved obtaining core samples of existing subgrade material from the Cunningham Highway trial site, testing the UCS of upper, middle and lower layers of the subgrade samples, analysing the UCS test results and comparing results to previous UCS data.

Laboratory testing was performed by the DTMR Materials Section in Herston, Brisbane on 21 June 2011 in accordance with DTMR Material Testing Manual (DTMR 2010).

Test results of the upper, middle and lower sections of core samples ranged from 2.9MPa to 8.1MPa. The lowest strength value is still well above DTMRs required 1.5MPa and the average of these latest test results is above all previous UCS test results.

Capillary rise testing was conducted on the subgrade samples and the results confirmed that the material was impermeable to moisture. This has the benefit of protecting the pavement gravels above and will significantly contribute to the pavements ability to retain strength and durability.

The test results from this investigation indicate that lime stabilised subgrade and road pavement are on target to reach the expected design life of 20 years and possibly well beyond. This research has a major benefit for DTMR particularly in life cycle costing. If pavements that are laid on lime stabilise subgrades that are achieving expected design life then DTMR can benefit from significant saving if they do not have to rehabilitate or reconstruct road pavements prematurely.

4 AMELIORATION PERIOD

4.1 Background

Two-stage mixing process is currently DTMRs standard practice. This process consists of mixing half the lime dosage to the subgrade in the first pass, moist curing for a period of 24 hours then mixing the remaining half of lime to the subgrade in the second pass. In the first pass, the objective is to distribute the lime throughout the soil and allowing the amelioration or mellowing operation to take place.

To optimise the chemical reactions of cation exchange and pozzolanic reactivity essential to facilitate the amelioration process, it is necessary to break down the clay clods down to sizes of less than 50mm (Little 1995).

Allowing 24 hours between passes permits the lime to flocculate clay particles and the clay becomes friable enough to improve workability in the second pass.

4.2 Amelioration Issues

The major issue with having a 24 hour period for amelioration is it takes a longer time to finish construction. This delays the contractor by having to bring a machine back a second day or having machinery sitting idle which is expensive especially on small projects. Construction is delayed as the contractor can’t continue construction until after the second pass.

During the amelioration period the open subgrade is exposed to potential weather such as rain for a longer period. The subgrade soils mixed with lime in the first pass become more permeable and if rain falls on the subgrade, water infiltration will become a problem. This will delay construction as this wet material is difficult to compact and may even require lime stabilisation to recommence.

Another issue is if the mixing machine breaks down, this requires a second machine to be sourced within 24 hours which can be difficult in remote areas. This would extend construction delays and possibly require the lime stabilisation to be started again.

Traffic delays can be expected and could be extended if the subgrade stabilisation has to be recommenced.

4.3 Methodology

The aim of this part of the research was to assess whether the amelioration period can be reduced to less than 24 hours whilst achieving the minimum strength requirement of 1.5Mpa. This investigation includes the following:

- Obtain two samples of black soil using Emerald and Barcaldine soil groups;
- Conduct soil classification testing to obtain moisture content, particle size distribution, atterberg limits, organic content, sulphate content, lime demand, maximum dry density and optimum moisture of both natural soil and soil with lime at lime demand;
- Conduct UCS testing with cure for 28 days @ 23deg / 95% Humidity;
- Test for different amelioration times (0, 6, 12, 18 and 24 hours).

The Amelioration process to be undertaken for the UCS samples is as follows:

Stage 1: The test portion will be mixed with half the total lime needed and with 80% of target water. Test portion will be subjected to half compaction and allowed to cure as per the target amelioration periods.

Stage 2: The cured test portion will be broken up by hand and rubber mallet then the remaining half of the lime mixed with the remaining 20% water. This will bring it up to the total needed to achieve target percentage. Test portions will then be compacted to perform UCS testing.

4.3.1 Classification Testing

Emerald and Blacksoil soil samples were sourced from the Capricornia Highway by the Emerald Geotechnical staff on 11 April 2011 and sent to Main Roads Testing Laboratory in Herston, Brisbane. Preliminary testing was completed on 19 May 2011.

Preliminary testing conducted prior to the stabilisation process included:

Moisture	-	Q102A	Moisture Content (Hygroscopic)
Grading	-	Q103A	Particle Size Distribution (Wet Sieving)
Atterberg Limits	-	Q104A	Five Point Liquid Limit (Cone Penetrometer)
	-	Q105	Plastic Limit and Plastic Index
	-	Q106	Linear Shrinkage
ADP	-	Q109A	Apparent Particle Density of Soil
Organic Content	-	Q120B	Organic Control (Loss on Ignition)
Sulphate Content	-	Q131A	Sulphate Content of Soil and Water
Lime Demand	-	Q133	Lime Demand Test
MDD	-	Q142A	Dry Density/Moisture Relationship – Standard Compaction
MDD with Lime	-	Q142A	Dry Density/Moisture Relationship – Standard Compaction
X-ray Diffraction	-		Powder X-ray diffraction

Samples for testing were prepared in accordance with Main Roads Test Method Q101. This involved breaking up aggregations of fines and soil clods, using the rotary cone splitter to split the bulk sample into smaller sub samples. The riffle was used to split the sub samples into the required sample weights. The samples were air dried or oven dried in accordance with the specified section of test method Q101.

4.3.1.1 Q102A (1993) – STANDARD MOISTURE CONTENT (OVEN DRYING)

This method describes the procedure for the determination of the moisture content of fine-grained, medium-grained and course-grained soils as a percentage of dry mass using a drying oven. This method is the standard method for determining moisture content and it may be used to establish relationships with subsidiary methods of moisture content measure.

A representative test portion with a mass of at least 100 grams was obtained in accordance with Test Method Q101. An empty container and lid was weighed and the mass recorded. The test portion was placed in the container where large aggregations were crumbled to assist drying. The container, lid and soil were weighed and the mass recorded.

The test portion in the container, without lid, was placed in the oven for drying at 105°C for 24 hours. The container was removed from the oven and covered with the lid replaced then allowed to cool until warm to touch. The container, lid and soil were weighed and the mass recorded.

The drying in the oven was repeated at 1 hour intervals and sample weighed until the difference between successive weighing was less than 1 percent of the total of the previous moisture loss.

4.3.1.2 Q103A (1996) – PARTICLE SIZE DISTRIBUTION (WET SIEVING)

This method describes the procedure for determining the particle size distribution of soil, down to the 75µm AS sieve. The method uses a dry sieving technique in association with washing the passing 2.36mm material prior to dry sieving. This method applies the principles of AS1289.3.6.1-1995: Determination of the Particle Size Distribution of a Soil – Standard Method of Analysis by Sieving.

A representative sample of 500g was obtained in accordance with Test Method Q101. The sample was placed into a tray and soaked in Calgon Solution for 2 hours. Calgon solution is a dispersing solution which helps break up the clay material into finer particles. After the 2 hours of soaking, the material was washed over a 75 µm sieve. The retained material was put back into the tray and put in the oven to dry the material. The oven dried sample was sieved on the following sieve sizes: 9.5mm, 4.75mm, 2.36mm, 0.425mm and 0.075mm. The weight of soil retained on each of the sieve sizes was recorded.

4.3.1.3 Q104A (2010) – LIQUID LIMIT (CONE PENETROMETER)

This method describes the procedure for the determination of the liquid limit of the portion of a soil passing the 0.425mm sieve using the cone penetrometer apparatus. This method is based on BS 1377-2: Methods of test for soils for civil engineering purposes – Part 2: Classification tests - Method 4.3: Cone Penetrometer Method.

The liquid limit is defined notionally as the moisture content at which the soil passes from the plastic to the liquid state, as determined by this procedure.

The soil sample was prepared in accordance with Test Method Q101. This involved sieving the soil over a 4.75mm and 425µm sieve. Any material retained on both of the sieves was ground with a mortar and pestle until the soil passed the 425µm sieve. These steps were repeated until a sample of approximately 300g passed the 425µm sieve. Distilled water was added to the soil in a mixing bowl until a stiff homogeneous paste was formed. Water was added until penetration of 9mm to 12mm was achieved with the Cone Penetrometer. Once this was achieved the sample was stored in an air tight container and allowed to cure for a minimum of 12 hours.

After the soil had cured it was returned to the mixing bowl and remixed for a further 3 minutes. For each test the penetrometer shaft was released to allow the cone to penetrate the sample for 5 seconds. A total of five penetration values were recorded with measures between 10 and 14.5mm, 14.5 and 16.5mm and greater than 16.5mm.

When the penetrations were achieved within these ranges the reading was recorded to the nearest 0.1mm and the sample returned to the mixing bowl and remixed for a further 30 seconds. The penetration test was repeated on the sample and when the 2 values were within 1mm of each other the two values were averaged and this value used for the penetration of the sample. A representative sample was taken from each of the 5 tests to determine the moisture content as detailed in Method Q102.

4.3.1.4 Q105 (2010) – LIQUID LIMIT AND PLASTICITY INDEX

This method describes the procedure for the determination of the plastic limit of the portion of a soil passing the 0.425mm sieve. The plastic limit is notionally defined as the moisture content at which the soil passes from the semi-solid to the plastic state as determined in accordance with this standard procedure.

The plasticity index is defined as the numerical difference between the liquid limit and plastic limit and is a measure of the range over which the soil is notionally considered to be in the plastic state.

The weighted plasticity index (WPI) is defined as the product of the plasticity index (PI) of a soil and the percentage of the soil finer than 0.425mm.

This method is based on AS 1289.3.2.1: Soil classification tests – Determination of the plastic limit of a soil – Standard method and AS 1289.3.3.1: Soil classification tests – Calculation of the plasticity index of a soil.

When determining plastic limit, the sample is initially prepared to an oven or air dried condition, then brought to moisture content higher than the liquid limit before commencing the test. Rolling of soil threads between the hands is not permitted and the definition of the endpoint where the soil thread reaches the plastic limit differs from the source method. For the plasticity index, the calculation is based on a liquid limit determined using a cone penetrometer.

A sample of approximately 8 grams was rolled into a ball and then rolled between the fingers and a glass plate. The soil sample was rolled slowly reduced the thread diameter. When the thread crumbled prior to reaching 3mm then the soil was considered to be at the plastic limit. Rolling of the threads continued until an approximate weight of 10grams of soil reached the plastic limit. This process was repeated to obtain a second sample. The samples were then oven dried in accordance with Test Method Q102 to determine the moisture content. The plasticity index was calculated by finding the difference between the liquid limit and the plastic limit.

4.3.1.5 Q106 (1989) – LINEAR SHRINKAGE

This method describes the procedure for the determination of the linear shrinkage of that portion of a soil passing the 0.425mm AS sieve. The linear shrinkage is defined as the percentage decreases in the longitudinal dimension of a soil bar when it is dried out from the liquid limit to the oven dry state.

The Linear Shrinkage sample was taken during the Liquid Limit test when the penetration was between 14.5 and 16.5mm. The soil was put into the mould ensuring air bubbles were removed. Excess soil was then removed from the surface using not more than 3 half strokes of the spatula each way. The sample was then allowed to sit on top of the oven for a few hours prior to being put into a 45-50°C oven for two hours and then a 105-110°C oven overnight.

4.3.1.6 Q109A (1996) – APPARENT PARTICLE DENSITY OF SOIL

This method describes the procedure for the determination of the apparent particle density of a soil. The apparent particle densities of the coarse fraction and the fine fraction are determined using separate procedures and a weighted average of these values is then used to determine the apparent density of the soil.

The method applies the principles of AS1289.3.5.1-1995: Determination of the Soil Particle Density of a Soil – Standard Method. It differs from this Australian Standard in variations to the apparatus and techniques used and in the reporting interval.

4.3.1.7 Q120B (1991) – 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 organic content of a sample (which may include undecomposed organic matter such as particles of grass, sticks etc.) by igniting the sample at 500°C in a furnace and calculating the resultant percentage mass loss.

A 50g oven dried sub-sample passing the 2.36mm sieve is prepared using a mortar and pestle in accordance with test method Q101, Section 4.4 - Fine Fraction Sub-samples. A 5 gram representative sample was placed in the crucible and weighed prior to being put in a furnace for 2 hours. The crucible was cooled in the desiccator and weighed; recording the mass to the nearest 0.001g. The crucible was then returned to the furnace for another hour and the procedure repeated until a constant mass is attained. This test is carried out for numerous subsamples until two samples do not differ by more than 0.45%.

Organic content interferes with the hydration process as well as competing for available stabilising agent – water paste. Table 3 of DTMRs Testing Protocol for Lime Stabilisation (DTMR 2007) specifies that organic content should be less than 1%.

4.3.1.8 Q131A (2010) – SULPHATE CONTENT OF SOIL AND WATER

This test shall be performed in accordance with AS 1289.4.2.1: Soil chemical tests – Determination of the sulphate content of a natural soil and the sulphate content of the groundwater - Normal method.

A 100g sample was obtained by sieving the soil over a 9.5mm sieve. The 100g sample is then pulverized so that it passes the 425µm sieve. An approximate 10g sub-sample was obtained by riffling the sample through a 7mm divider. The 10g sample was then placed in a glass bottle and dried at 75-80°C until a constant mass is achieved. After cooling, the soil was transferred to an extraction bottle and 150mL of distilled water added. The suspension was turned to acid state by using indicator paper and adding hydrochloric acid then agitated for 30 minutes in a shaker. After agitation the suspension was filtered through filter paper on the Buchner funnel using suction.

The extraction bottle and soil were washed and the washings transferred to a filter funnel. The extract was transferred to a 500mL beaker and the filter flask washed with distilled water which was also added to the beaker to make a total volume of approximately 250mL. The extract was then tested with indicator paper and if necessary made slightly acidic then brought to boiling point and stirred whilst adding 25mL of barium chloride solution drop by drop. The suspension was covered and kept hot for at least 1 hour.

After 1 hour the suspension was allowed to settle and a few drops of barium chloride solution was added to clear the supernatant liquid and to ensure that there has been complete precipitation of the barium sulphate.

The solution is then kept hot again but not boiling for 1 hour. The precipitate was transferred to a filter paper in a glass funnel and filter. The precipitate was then washed several times with hot distilled water until the washings are free from chloride then heated to approximately 800°C. After heating, the mass of the precipitate is calculated then the sulphate present in the original soil sample is calculated.

Sulphate can interfere with pozzolanic reactions due to the formation of very expansive hydrates which if formed after compaction, can result in heave. Table 3 of DTMRs Testing Protocol for Lime Stabilisation (DTMR 2007) specifies that sulphate content of less than 0.3%.

4.3.1.9 Q133 (1998) – LIME DEMAND TEST

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, and is used as a starting estimate of an optimum in design lime content. The lime demand test as performed by this procedure has been shown to provide lime contents that correspond well with optimum lime contents for long effective stabilisation.

This method was developed in-house by DTMR and is based on the procedure described in A Quick Test to Determine Lime Requirements for Soil Stabilisation: Highway Research Record No. 139 (Eades & Grim 1996).

The lime demand test was conducted using the oven dried hydrated quicklime.

A sub-sample of 500 grams of air dried material, passing a 2.36mm screening sieve was prepared in accordance with test method Q101. The hygroscopic moisture content of a test portion of approximately 100 grams was obtained in accordance with Method Q102A. The mass of the hydrated quicklime and soil was calculated for lime increments of 1 to 10% and 100%. The hydrated lime for each test portion was then put into individual beakers with 75mL of distilled water. The suspensions were mixed using a magnetic stirrer and then the beaker covered with a watch glass and allowed to stand for 2 hours.

The pH meter was calibrated using 6.88 and 9.23 buffer solutions then the pH of each of the soil-lime mixture were recorded starting at the lowest lime content until the three highest pH values did not vary by more than 0.05 pH units.

4.3.1.10 Q142A (2010) – DRY DENSITY – MOISTURE RELATIONSHIP (STANDARD COMPACTION)

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/m^3). Compaction is conducted over a range of moisture contents so as 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 37.5mm sieve. Material that passes the 19.0mm sieve is compacted in a 105mm diameter mould. Material that has any rock retained on the 19.0mm sieve but no more than 35% of rock retained on the 37.5mm sieve is compacted in a 152mm diameter mould.

This method is based on AS 1289.5.1.1: Soil compaction and density tests - Determination of the dry density/moisture content relation of a soil using standard compactive effort. It differs from this standard in the coarseness of material permitted, provision for the addition of stabilizing agents, and gathering the oversize data necessary to correct reference density results in cases where a nuclear gauge is used to measure insitu density and reporting requirements.

The bulk sample was prepared in accordance with Section 3 of Test Method Q101 to prepare six portions weighing approximately 2.5Kg each. The sample materials were black soil and contained no stones or aggregates and 100% of the material passed the 9.5mm sieve.

The test method requires the optimum moisture content (OMC) to be straddled with at least 2 samples, one being drier and one wetter. Due to this requirement four soil samples were prepared with moisture contents (MC) of 6%, 8%, 10% and 12%. The additional samples would be available for testing if one of the tests failed or if the OMC has not been straddled. Additional water (within reason) could be added to the extra samples to vary the moisture content. The samples were bagged and left to cure for 7 days.

Testing was conducted on the 4 prepared samples in Mould Type A (105mm diameter, 115.5mm high) using a standard rammer to compact the soil in 3 equal layers with 25 blows to each layer. After the collar was removed from the mould, excess soil was trimmed off the top and the mass of the mould and soil recorded. The soil was then removed from the mould and placed into a metal tray and weighted prior to being put into a 105°C oven.

Once the moisture content was determined in accordance with Test Method 102A the OMC can be calculated.

4.3.1.11 Q142A (2010) – DENSITY – MOISTURE RELATIONSHIP (LIME ADDED)

The dry density/moisture relationship for the lime stabilised soil was determined in a similar fashion to that of the insitu soil sample, except that Test Method Q135B was used for the addition of the stabilising agent and curing of the samples.

The hygroscopic moisture content of the soil was tested and used to calculate the required mass of the hydrated lime for the test portions. The soil-lime samples were prepared with lime at Lime Demand (LD) and 2% above and below LD. Laboratory testing was conducted using hydrated lime. Each soil-lime mixture was compacted in 40mm layers using one-half the standard compaction effort. The samples were bagged and placed in a temperature controlled room (95% relative humidity at 23±2°C) for the nominated amelioration time period. The remaining testing was then carried out in accordance with that of the insitu soil sample.

4.3.1.12 Q115 (2010) – UNCONFINED COMPRESSIVE STRENGTH OF COMPACTED MATERIAL

This method describes the procedure to determine the unconfined compressive strength (UCS) of compacted specimens of soils and crushed rock which have been either modified or stabilised with a stabilising agent or are in their natural state. The method has particular application as a laboratory design procedure but can also be used to test field-moulded specimens in order to check field processes.

In the laboratory, test specimens are prepared by compacting passing 19.0mm material by standard or modified compactive effort to a nominated dry density and moisture content, as detailed in Q145A.

This method was developed in-house by DTMR using techniques evolved through internal Departmental research investigations.

Fifteen (15) representative samples of 9kg were obtained in accordance with Test Method Q101 to perform UCS testing for a range of different amelioration periods used in the initial curing of the samples. It is proposed to target the Lime Demand percentage, in accordance with Q133, as well as 2% either side of Target Lime Demand.

UCS testing was performed in accordance with the above information and cured for 28 days @ 23 degrees and 95% humidity.

4.3.1.13 FERRIC OXIDE CONTENT

A sample of each soil was sent to Australian Laboratory Services Pty Ltd, Brisbane to determine the percentage of ferric oxide in each soil.

Ferric oxide can interfere with pozzolanic reactions. Recent DTMR experience has indicated that contents in the range of 10-13% have a deleterious impact on stabilised materials. Table 3 of DTMRs Testing Protocol for Lime Stabilisation (DTMR 2007) specifies that advise should be sort for ferric oxide content values are greater than 2%.

4.3.1.14 X-RAY DIFFRACTION

A sample of each soil was sent to Geochempet Services, Maleny for quantitative X-ray diffraction analysis to determine mineralogy of the samples. The analysis was investigated using Queensland University of Technology (QUT) facilities.

4.3.1.15 AMELIORATION PROCESS

It is proposed to perform testing of amelioration periods for both soil groups at 0, 6, 14, 18 and 24 hour timeframes between mixing stages.

The amelioration process to be undertaken for the UCS samples was as follows:

Stage 1: The test portion will be mixed with half the total lime needed and with 80% of target water. Test portion will be subjected to half compaction and allowed to cure as per the target amelioration periods.

Stage 2: The cured test portion will be broken up by hand and rubber mallet then the remaining half of the lime mixed with the remaining 20% water. This will bring it up to the total needed to achieve target percentage. Test portions will then be compacted to perform UCS testing.

If, on adding the remaining lime and water in Stage 2 mixing, it is seen that the sampled looks dry additional water can be added which will be recorded.

4.4 Results

4.4.1 Classification Testing

Laboratory testing was undertaken on representative soils at the DTMR Materials Services in Herston, Brisbane.

4.4.1.1 MOISTURE CONTENT

The moisture content was performed in accordance with QTMR test method Q102A. The test results showed that the moisture content for the Emerald black soil and Barcaldine black soil sample was 9.4% and 14.1% respectively. A copy of test results can be found in Appendix C for the Emerald black soil and Appendix D for the Barcaldine black soil.

4.4.1.2 PARTICLE SIZE DISTRIBUTION

The Particle Size Distribution (PSD) for sieve sizes 9.5mm to 0.075mm and less than 0.075mm were performed in accordance with QTMR test method Q103A. PSD results for the Emerald black soil show that 77% of the material passed the 75µm sieve, whilst for the Barcaldine black soil 78% of the material passed the 75µm sieve. PSD results can be found in Appendix C for the Emerald black soil and Appendix D for the Barcaldine black soil. Figure 4.1 below is a graphical representation of the test results.

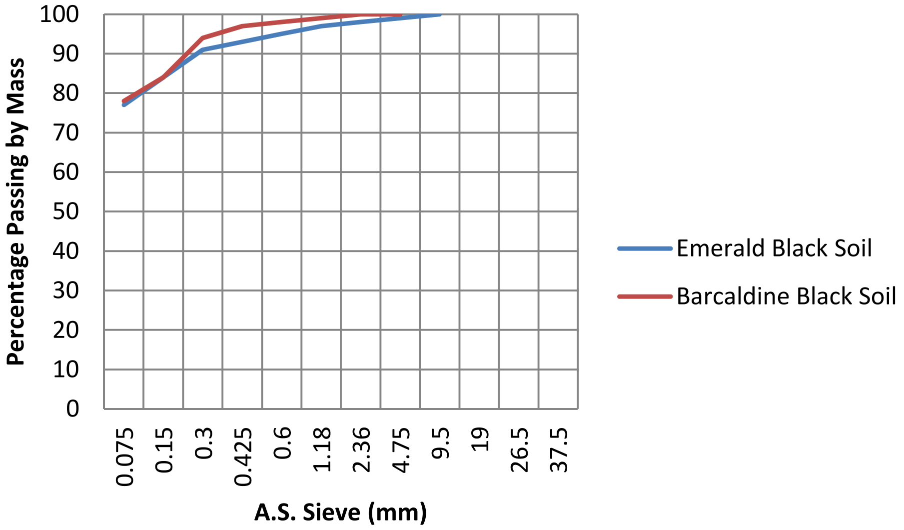

Figure 4.1: Grading Limits for Emerald and Barcaldine Black Soil

4.4.1.3 ATTERBERG LIMITS

Liquid Limit (LL), Plasticity Index (PI) and Linear Shrinkage (LS) tests were performed in accordance with QTMR test methods, Q104A, Q105 and Q106 respectively. Atterberg Limit test results are provided in Appendix C for the Emerald black soil and Appendix D for the Barcaldine black soil.

Test results for both black soil groups are shown in Table 4-1.

Table 4-1 Atterberg Limits for Emerald and Barcaldine Black Soils

Soil Group	Liquid Limit (%)	Plastic Index (%)	Linear Shrinkage (%)	PI x Passing 0.425mm sieve	LS x Passing 0.425mm sieve
Emerald	61.0	37.0	20.0	3441	1860
Barcaldine	64.2	40.2	18.8	3879	1814

4.4.1.4 APPARENT PARTICLE DENSITY OF SOIL

The apparent particle density (ADP) was performed in accordance with QTMR test method Q109A. The test results showed that the ADP for the Emerald black soil and Barcaldine black soil sample was 2.693t/m^3 and 2.723t/m^3 respectively.

A copy of test results can be found in Appendix C for the Emerald black soil and Appendix D for the Barcaldine black soil.

4.4.1.5 DRY DENSITY – MOISTURE RELATIONSHIP (STANDARD COMPACTION)

The Dry Density – Moisture Relationship values were calculated in accordance with QTMR test method Q142A. The resultant Maximum Dry Density (MDD) and Optimum Moisture Content (OMC) for the Emerald insitu black soil and with 4% hydrated lime added is shown in Table 4-2. The test results for the Barcaldine black soil sample is shown in Table 4-3.

A copy of test results can be found in Appendix C for the Emerald black soil and Appendix D for the Barcaldine black soil.

Table 4-2 Maximum Dry Density and Optimum Moisture Content for Emerald Black Soil

Addition of Lime (%)	**MDD (t/m^3)**	**OMC (%)**
0	1.542	25.0
4	1.530	23.2

Table 4-3 Maximum Dry Density and Optimum Moisture Content for Barcaldine Black Soil

Addition of Lime (%)	**MDD (t/m^3)**	**OMC (%)**
0	1.550	25.0
4	1.487	23.4

4.4.1.6 LIME DEMAND

The lime demand was performed in accordance with QTMR test method Q133. The test showed that the pH value became constant for both the Emerald and Barcaldine black soils with approximately 4% lime additive. A summary of the lime demand test results for the Emerald and Barcaldine soils are shown in Table 4-4 and Table 4-5 respectively.

A copy of test results can be found in Appendix C for the Emerald black soil and Appendix D for the Barcaldine black soil.

Table 4-4 Lime Demand for Emerald Soil

Lime Content (%)	0	1	2	3	**4**	5	6
pH	7.86	11.31	12.45	12.69	**12.76**	12.78	12.79

Table 4-5 Lime Demand for Barcaldine Soil

Lime Content (%)	0	1	2	3	**4**	5	6
pH	7.76	11.88	12.62	12.72	**12.76**	12.78	12.79

4.4.1.7 SULPHATE AND ORGANIC CONTENT

The Sulphate and Organic Content tests were performed in accordance with QTMR test methods Q131A and Q120B, respectively. The test report in Appendix C and D shows that the Sulphate Content and Organic Content of the Emerald black soil was 0.14% and 6.50%, respectively. Sulphate Content and Organic Content of the Barcaldine black soil was 2.7% and 6.85%, respectively.

The Barcaldine black soil test sample was above the recommended Queensland Main Roads Testing Protocol (DTMR 2007) value for Sulphate Content of 0.3% and above the recommended value of 1% for Organic Content.

4.4.1.8 FERRIC OXIDE CONTENT

The Ferric Oxide Content tests were performed by Australian Laboratory Service. The test report in Appendix C and D shows that the Ferric Oxide Content of the Emerald and Barcaldine black soil was 6.75% and 3.98%, respectively.

Both black soil test samples were above the recommended Queensland Main Roads Testing Protocol (DTMR 2007) value of 2%.

4.4.1.9 X-RAY DIFFRACTION

Quantitative X-ray diffraction analysis was performed by Geochempet Services to determine mineralogy the of the Emerald and Barcaldine black soil samples.

The results indicated that both soil samples comprised smectite dominated mixed layers of illite and amorphous content which is suspected to be mostly grossly disordered illite/smectite with minor unidentified minerals and organic matter. The illite and smectite clays are considered to be the most expansive clays and have a capacity for significant swelling and shrinkage upon changes in moisture content.

A summary of minerals found in each soil group is shown in Table 4-6.

Table 4-6 Summary of Minerals in Barcaldine and Emerald Black Soils

Mineral	Barcaldine Black Soil	Emerald Black Soil
Quartz	42.4%	46.5%
Albite	2.2%	2.9%
Calcite	0.7%	-
Gypsum	6.9%	-
Kaolinite	1.1%	2.1%
Mixed layer clay (Illite / Smectite)	17.1%	32.3%
Amorphous Matter	29.6% (probably mostly disordered illite/smectite clay)	16.1% (probably mostly disordered illite/smectite clay)

4.4.2 Unconfined Compressive Strength Results

UCS testing was performed by the DTMR Materials Section in Herston, Brisbane in June 2011 in accordance with test methods Q115 for UCS and Q102A for Moisture Content.

Lime demand was determined to be 4%, so testing was conducted with lime added to the natural insitu Emerald and Barcaldine black soil at 2%, 4% and 6%.

UCS testing was performed on amelioration periods of 0, 6, 14, 18 and 24 hours and cured for 28 days @ 23 degrees and 95% humidity.

The preparation and placement of the UCS samples was to target the density and moisture achieved from the Dry Density – Moisture Relationship test performed at Lime Demand (Test Method Q142A). The target density and moisture results were determined at 4% Hydrated Lime.

The target for the Barcaldine black soil was to be OMC of 23.4% and MDD of 1.487t/m^3 whilst the target for the Emerald black soil was OMC of 23.2% and MDD of 1.530t/m^3.

The UCS results for the varying amelioration periods and lime percentages for the Barcaldine black soil and Emerald black soil is shown in Table 4-7 and Table 4-8 respectively. The density and UCS strength shown in the results is the mean of three compacted samples.

Table 4-7 Amelioration Period - UCS Results for Barcaldine Balck Soil (June 2011)

Amelioration	0 Hours (MPa)	6 Hours (MPa)	14 Hours (MPa)	18 Hours (MPa)	24 Hours (MPa)
2%	0.6	0.7	0.7	0.6	0.7
Achieved Moisture %	23.7	23.8	23.3	23.8	23.5
Relative Compaction	99.8	99.8	100.1	99.7	100.0
4%	1.1	1.3	1.4	1.3	1.4
Achieved Moisture %	23.9	23.6	23.2	23.7	23.4
Relative Compaction	99.5	99.8	100.1	99.7	100.0
6%	1.6	1.6	2.0	1.7	1.7
Achieved Moisture %	24.0	23.7	23.2	24.1	23.4
Relative Compaction	99.5	99.7	100.2	99.4	99.9

Table 4-8 Amelioration Period - UCS Results for Emerald Black Soil (June 2011)

Amelioration	0 Hours (MPa)	6 Hours (MPa)	14 Hours (MPa)	18 Hours (MPa)	24 Hours (MPa)
2%	0.5	0.5	0.5	0.5	0.5
Achieved Moisture %	23.3	23.2	22.8	23.0	22.9
Relative Compaction	100.4	99.9	100.3	100.0	100.2
4%	1.0	1.3	1.4	1.2	1.4
Achieved Moisture %	23.4	23.0	22.8	23.2	22.9
Relative Compaction	99.7	100.1	100.3	99.9	100.2
6%	1.1	1.3	1.5	1.3	1.6
Achieved Moisture %	23.3	23.0	22.8	23.2	22.8
Relative Compaction	99.9	100.1	100.5	100.0	100.3

The resultant UCS for each soil group and varying lime contents is shown graphically in Figure 4.2.

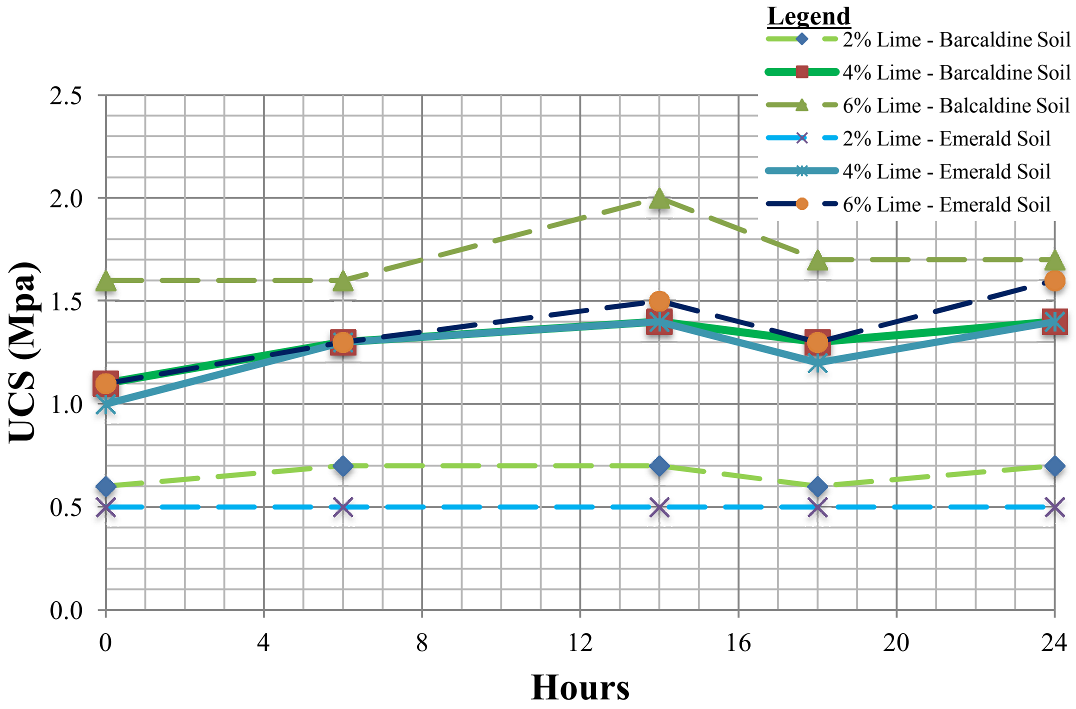

Figure 4.2: Amelioration Period – UCS Results

The material strength of both black soils at 14 hours using lime demand of 4% was equal to those at 24 hours. In fact all results at 14 hours were equal or in fact greater than 24 hours as was the case with 6% lime added to the Barcaldine black soil. The strength of material with 4% lime is 1.4MPa, just below the required 1.5MPa.

The Cunningham Highway lime stabilised subgrade trial exhibited UCS increases after construction. Based on these strength gains it can be expected that the UCS value at 14 hours will increase to be achieve the required strength of 1.5MPa. This indicates that an amelioration period of 14 hours is acceptable.

The results do show a reduction in strength at 18 hours and it was decided to re-test the UCS for 14, 18 and 24 hours. This will be discussed further in Section 4.5.

Changing the amelioration period to 14 hours has the significant benefit of running the first lime mixing pass late in an afternoon, leaving overnight and completing the second pass the next morning. This will allow construction to progress without delays to contractor and reduce the exposure time to weather.

4.5 Discussion

During classification testing both black soils were identified as having high sulphate, organic and ferric oxide contents. High levels of these elements can interfere with hydration and pozzolanic reactions but the UCS results tended to indicate that this wasn't an issue with these samples. Further investigation may be needed to confirm if these high levels contributed to the actual test results.

The UCS test results for varying amelioration periods indicated that 14 hours is an acceptable solution. The strength at 14 hours with Lime Demand at 4% was 1.4MPa which is just below the DTMR recommended strength of 1.5MPa. Whilst it is just below, this strength is the same strength obtained at 24 hours.

The test results showed a reduction in strength at 18 hours in all results, so it was decided to re-test both the black soils at 14, 18 and 24 hours and lime percentages of 4% and 6% to confirm if the reduction was an error in testing. This additional testing was conducted in September 2011.

Using the same target MDD and OMC determined at 4% hydrated lime for the Barcaldine and Emerald black soil from the original tests, the UCS results for the varying amelioration periods and lime percentages for the Barcaldine black soil and Emerald black soil is shown in Table 4-9 and Table 4-10 respectively.

The results from the re-testing showed increases in UCS for all results and the strength at 14 hours achieved the DTMR target of 1.5MPa but the decrease in strength at 18 hours was still present.

In consultation with DTMR staff, we believe that this reduction in strength at 18 hours could be where the hydration reaction is starting to complete. Further laboratory investigation using amelioration time periods immediately either side of 18 hours, at say 16 and 20 hours, may give clearer understanding to the cause of strength reduction.

Table 4-9 Amelioration Period - UCS Results for Barcaldine Black Soil (September 2011)

Amelioration	0 Hours (MPa)	6 Hours (MPa)	14 Hours (MPa)	18 Hours (MPa)	24 Hours (MPa)
2%	-	-	-	-	-
Achieved Moisture %	-	-	-	-	-
Relative Compaction	-	-	-	-	-
4%	-	-	1.5	1.2	1.3
Achieved Moisture %	-	-	22.9	22.9	23.0
Relative Compaction	-	-	100.4	100.4	100.5
6%	-	-	1.8	2.0	1.8
Achieved Moisture %	-	-	23.0	22.9	23.1
Relative Compaction	-	-	100.3	100.4	100.3

Table 4-10 Amelioration Period - UCS Results for Emerald Black Soil (September 2011)

Amelioration	0 Hours (MPa)	6 Hours (MPa)	14 Hours (MPa)	18 Hours (MPa)	24 Hours (MPa)
2%	-	-	-	-	-
Achieved Moisture %	-	-	-	-	-
Relative Compaction	-	-	-	-	-
4%	-	-	1.7	1.5	1.7
Achieved Moisture %	-	-	24.4	24.7	24.5
Relative Compaction	-	-	100.5	100.1	100.4
6%	-	-	1.8	1.6	1.8
Achieved Moisture %	-	-	24.2	24.8	22.8
Relative Compaction	-	-	100.6	100.2	100.3

Testing for this amelioration period has only been undertaken by laboratory work to date. Field trials should be conducted on a lime stabilised subgrade to verify that the 14 hour amelioration period is acceptable.

4.6 Chapter Summary

Two-stage mixing process is currently DTMRs standard practice. This process consists of mixing half the lime dosage to the subgrade in the first pass, moist curing for a period of 24 hours then mixing the remaining half of lime to the subgrade in the second pass.

The aim of this part of the research was to assess whether the amelioration period can be reduced to less than 24 hours whilst retaining the minimum strength requirement of 1.5Mpa. This investigation involved obtaining two samples of black soil using Emerald and Barcaldine soil groups, conducting soil classification testing, conducting UCS testing with cure for 28 days @ 23deg / 95% Humidity and testing for different amelioration times (0, 6, 14, 18 and 24 hours). These black soils were chosen as they are a dominant reactive soil throughout Queensland.

Classification testing indicated that Lime Demand for both black soils was 4% hydrated lime. The preparation and placement of the UCS samples was to target the maximum dry density (MDD) and optimum moisture content (OMC) achieved from the Dry Density – Moisture Relationship test performed at Lime Demand.

The target for the Barcaldine black soil was OMC of 23.4% and MDD of 1.487t/m^3 whilst the target for the Emerald black soil was OMC of 23.2% and MDD of 1.530t/m^3.

The UCS results for the varying amelioration periods and lime percentages for the Barcaldine black soil and Emerald black soil conducted in the laboratory showed that an amelioration period of 14 hours was an acceptable timeframe between lime mixing passes. This will be subject to further confirmation by conducting field testing to trial this amelioration period.

5 CONCLUSION AND FURTHER WORKS

5.1 Achievement of Project Aims

The following objectives have been addressed:

Unconfined Compressive Strength of Existing Lime Stabilised Subgrade
Chapter 2 provided background into the processes involved with lime stabilisation and Chapter 3 presented the findings of field sampling and laboratory testing to confirm that the unconfined compressive strength of a trial lime stabilised subgrade on a state controlled road with significant heavy vehicle use has increased in strength after 13 years operation.

Reduction in Amelioration Period
Chapter 2 provided background to amelioration and Chapter 4 presented the findings of laboratory testing of varying amelioration time periods and percentages of added lime to confirm that it is possible for the amelioration period to be reduced from 24 hours to 14 hours.

5.2 Conclusion

Queensland has a large road network which is made up of both rigid and flexible pavements. Natural soils across the State are dominated by reactive soils and when used as subgrade materials can influence both the durability and strength of the road pavement and reduce its design life. The Department of Transport and Main Roads (DTMR) and many Councils use various forms of stabilisation to improve the load bearing capacity and/or stability of poor subgrade materials to extend the design life to try and meet standard expectancy of at least 20 to 40 years. There has been a lot of research into lime stabilisation but problems still exist with lime stabilisation and research is ongoing.

Due to the failure of numerous projects in Queensland during the late 70's, doubt fell upon the effectiveness of lime stabilisation. Support has again grown into the use of lime stabilisation and it is now accepted that the addition of small quantities of lime promote soil modification. New lime stabilisation testing procedures have been adopted by DTMR to determine the optimum lime content. These procedures have been trialled on numerous lime stabilisation projects with success. Even though industry is now more comfortable with the lime stabilisation process there are still problems with construction processes and determining long term strength gain to achieve the required design life.

This research identified a couple of areas that required further work. These were confirming the long term strength of lime stabilised subgrade and assessing whether the amelioration period can be reduced from DTMRs standard time of 24 hours.

This investigation involved obtaining core samples of existing subgrade material from the Cunningham Highway lime stabilised subgrade trial site, testing the UCS of upper, middle and lower layers of the subgrade samples, analysing the UCS test results and comparing results to previous UCS data.

Test results of the upper, middle and lower sections of core samples ranged from 2.9MPa to 8.1MPa. The lowest strength value is still well above DTMRs required 1.5MPa and the average of these latest test results is above all previous test results indicating that the lime stabilised subgrade is still increasing in strength. However a lack of recent test results will necessitate the immediate need for further strength testing to verify that the subgrade strength is in fact still increasing in strength.

Capillary rise testing was conducted on the subgrade samples and confirmed that the material was impermeable to moisture which has the benefit of protecting the pavement gravels above. This will significantly maintain the pavements ability to retain strength and durability.

These results indicate that lime stabilised subgrade and road pavement are on target to reach the expected design life of at least 20 years and possibly well beyond. This research has a major benefit for DTMR on pavement rehabilitation life cycle costs. If pavements that are laid on lime stabilise subgrades are achieving expected design life then DTMR can benefit from significant saving if they do not have to rehabilitate or reconstruct road pavements prematurely.

There has been a significant amount of research conducted in the interaction of lime with clay soils but an area of research that appears deficient is the assessed of the amelioration time period between lime mixing passes when constructing the lime stabilised subgrade.

Two-stage mixing process is currently DTMRs standard practice. This process consists of mixing half the lime dosage to the subgrade in the first pass, moist curing for a period of 24 hours then mixing the remaining half of lime to the subgrade in the second pass. The amelioration time period has been set based on tradition and experience.

The aim of this part of the research was to assess whether the amelioration period can be reduced to less than 24 hours whilst retaining the minimum strength requirement of 1.5Mpa. This investigation involved obtaining two samples of black soil using Emerald and Barcaldine soil groups. Soil classification testing and unconfined compressive strength (UCS) testing were conducted with cure for 28 days @ 23deg / 95% Humidity for varying amelioration times and percentages of added lime.

Classification testing indicated that Lime Demand for both black soils was 4% hydrated lime. The preparation and placement of the UCS samples was to target the maximum dry density (MDD) and optimum moisture content (OMC) achieved from the Dry Density – Moisture Relationship test performed at Lime Demand.

The UCS results for the varying amelioration periods and lime percentages for the Barcaldine black soil and Emerald black soil conducted in the laboratory showed that an amelioration period of 14 hours was an acceptable timeframe between lime mixing passes. This will be subject to further confirmation by conducting field testing to trial this amelioration period.

5.3 Further Work

I would suggest that research should be continued into the determination of a strength gain relationship for lime stabilised black soils. This is especially the case on the Cunningham Highway lime stabilised subgrade trial site which has been in operation now for 13 years. Additional UCS testing should be conducted in the next 12 to 24 months to confirm the strength relationship of this trial site.

It would be my recommendation that more lime stabilised sites be tested and that testing should be carried out over a longer periods on all these sites. The larger database will identify potential issues with lime stabilisation and provide a greater understanding of possible anomalies in testing and construction methods.

Field trials should be conducted to confirm the acceptability of reducing the amelioration period between lime mixing passes to 14 hours. Further investigation should be carried out to determine what has caused the reduction in UCS at 18 hours amelioration. If it is confirmed that the reduction is as a result of hydration process completing, then there should be a greater acceptance towards reducing the standard amelioration period and not only DTMR but the entire industry will become more comfortable with using lime stabilisation on reactive soil subgrades.

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APPENDICES

Appendix A - Project Specification

University of Southern Queensland
FACULTY OF ENGINEERING AND SURVEYING
ENG4111 / ENG4112 Research Project
PROJECT SPECIFICATION

FOR: **PAUL WILSON**
TOPIC: EFFECTIVENESS OF LIME STABILISATION ON REACTIVE SOILS FOR MAIN ROADS
SUPERVISORS: Dr Soma Kathirgamalingham (USQ)
Mr Jothi (Rama) Ramanujam (DTMR)
PROJECT AIM: To analyse the effectiveness and success of lime stabilised pavements used on reactive soils in Southern Districts of the Department of Transport and Main Roads (DTMR).

PROGRAMME: (Issue A, 20 March 2011)

1. Carry out a literature review, identifying issues encountered in the design, construction and maintenance of lime stabilised roads on reactive soil sub-grades.

2. Investigate and document the problems that have been encountered from reactive soil sub-grade effects on the road pavements.

3. Analyse and evaluate field data and test results on various projects undertaken by DTMR to determine outcomes achieved with proposed pavement design and strength gain of lime stabilised reactive soils.

4. Analyse factors affecting stabilisation including stabiliser test procedures, dosage effects to soil properties, mixing and compaction.

5. Analyse the success and / or failures (causes and extent) on these selected roads.

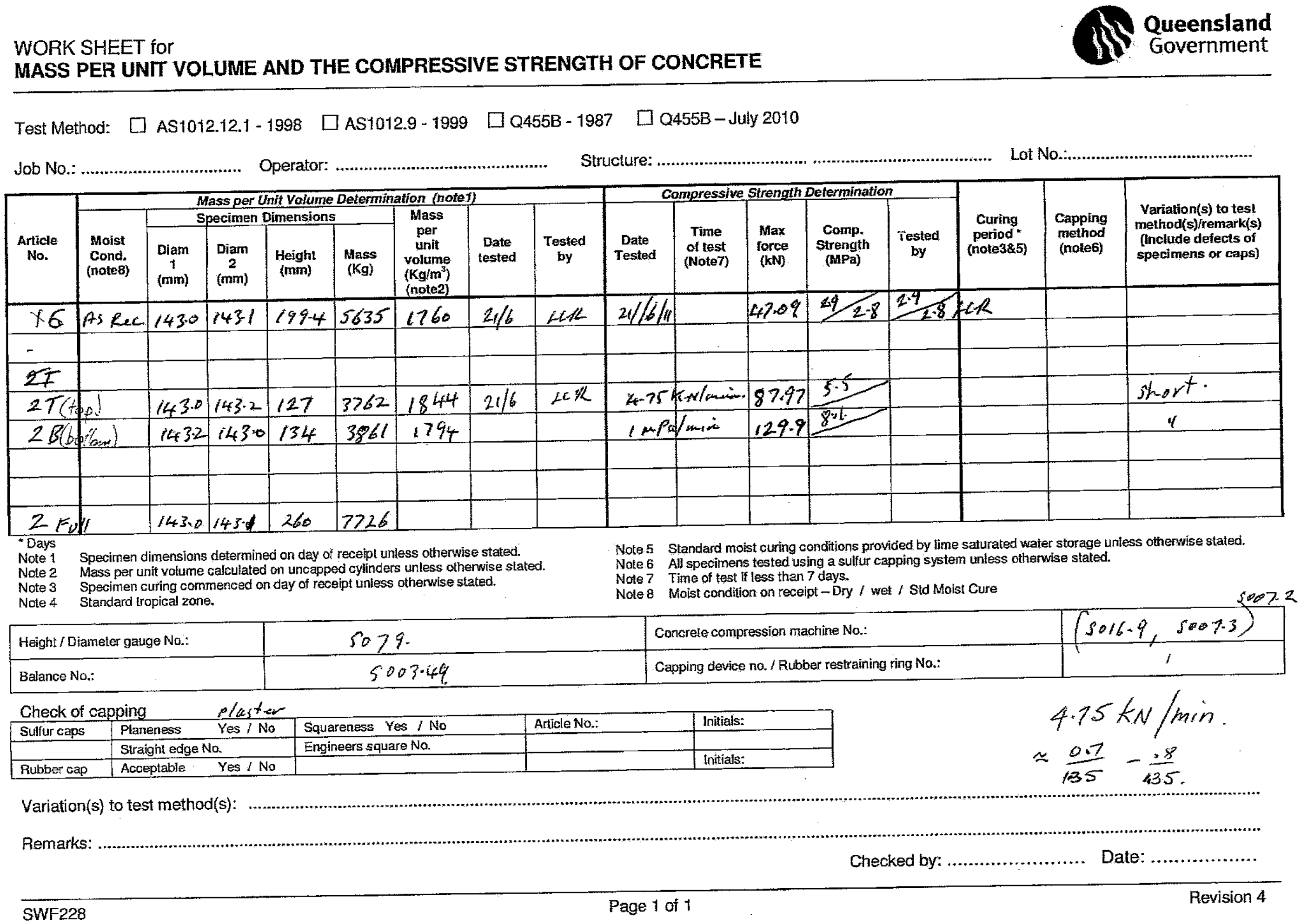
6. Analyse the cost effectiveness and comment on life expectancy of the selected roads.

7. Report findings to peer group via oral presentation at the Project Conference and submit the report in written format.

AGREED:
___________________ (Student) _______________,______________ (Supervisors)
Date ___ /___ / 2011 ___ /___ / 2011 ___ /___ / 2011

Examiner / Co-examiner: ____

Appendix B - UCS Test Record



Queensland Government

WORK SHEET for
MASS PER UNIT VOLUME AND THE COMPRESSIVE STRENGTH OF CONCRETE

Test Method: ☐ AS1012.12.1 - 1998 ☐ AS1012.9 - 1999 ☐ Q455B - 1987 ☐ Q455B – July 2010

Job No.: .... Operator: .... Structure: .... Lot No.:......................................

	Mass per Unit Volume Determination (note1)								Compressive Strength Determination							
		Specimen Dimensions														
Article No.	Moist Cond. (note8)	Diam 1 (mm)	Diam 2 (mm)	Height (mm)	Mass (Kg)	Mass per unit volume (Kg/m³) (note2)	Date tested	Tested by	Date Tested	Time of test (Note7)	Max force (kN)	Comp. Strength (MPa)	Tested by	Curing period * (note3&5)	Capping method (note6)	Variation(s) to test method(s)/remark(s) (Include defects of specimens or caps)
X6	As Rec	143.0	143.1	199.4	5635	1760	21/6	LCR	21/6/11		47.09	2.9/2.8	2.9/2.8 LCR			
-																
2T																
2T(top)		143.0	143.2	127	3762	1844	21/6	LCR	4.75 kN/min		87.97	5.5				short.
2B(bottom)		143.2	143.0	134	3861	1794			1 MPa/min		129.9	8.1				"
2 Full		143.0	143.1	260	7726											

* Days
Note 1 Specimen dimensions determined on day of receipt unless otherwise stated.
Note 2 Mass per unit volume calculated on uncapped cylinders unless otherwise stated.
Note 3 Specimen curing commenced on day of receipt unless otherwise stated.
Note 4 Standard tropical zone.
Note 5 Standard moist curing conditions provided by lime saturated water storage unless otherwise stated.
Note 6 All specimens tested using a sulfur capping system unless otherwise stated.
Note 7 Time of test if less than 7 days.
Note 8 Moist condition on receipt – Dry / wet / Std Moist Cure

Height / Diameter gauge No.:	5079.	Concrete compression machine No.:	S007.2. (S016.9, S007.3)
Balance No.:	S003.44	Capping device no. / Rubber restraining ring No.:	/

Check of capping plaster

Sulfur caps	Planeness Yes / No	Squareness Yes / No	Article No.:	Initials:
	Straight edge No.	Engineers square No.		
Rubber cap	Acceptable Yes / No			Initials:

4.75 kN/min.
≈ $\frac{0.7}{135}$ $-\frac{.8}{435}$.

Variation(s) to test method(s): ....

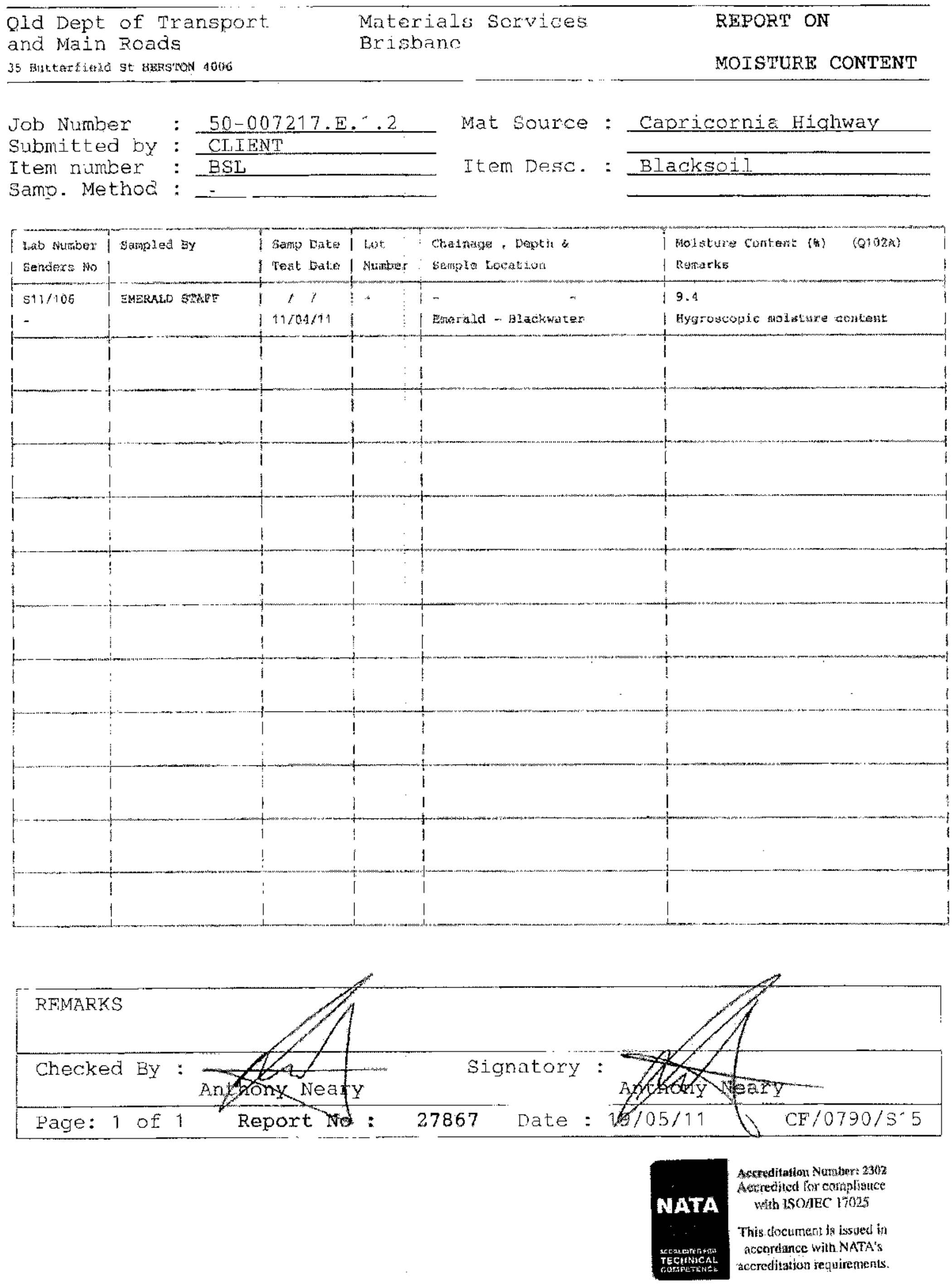
Remarks: ....

Checked by: .... Date: ....

SWF228 Page 1 of 1 Revision 4

Figure 5.1: Work Sheet for UCS of Core Samples from Cunningham Highway

Appendix C - Test Results for Emerald Soil



Qld Dept of Transport and Main Roads
35 Butterfield St HERSTON 4006

Materials Services
Brisbane

REPORT ON
MOISTURE CONTENT

Job Number : 50-007217.E.´.2
Submitted by : CLIENT
Item number : BSL
Samp. Method : -

Mat Source : Capricornia Highway
Item Desc. : Blacksoil

Lab Number Senders No	Sampled By	Samp Date Test Date	Lot Number	Chainage , Depth & Sample Location	Moisture Content (%) (Q102A) Remarks
S11/106 -	EMERALD STAFF	/ / 11/04/11	-	- - Emerald - Blackwater	9.4 Hygroscopic moisture content

REMARKS

Checked By : Anthony Neary

Signatory : Anthony Neary

Page: 1 of 1 Report No : 27867 Date : 19/05/11 CF/0790/S´5

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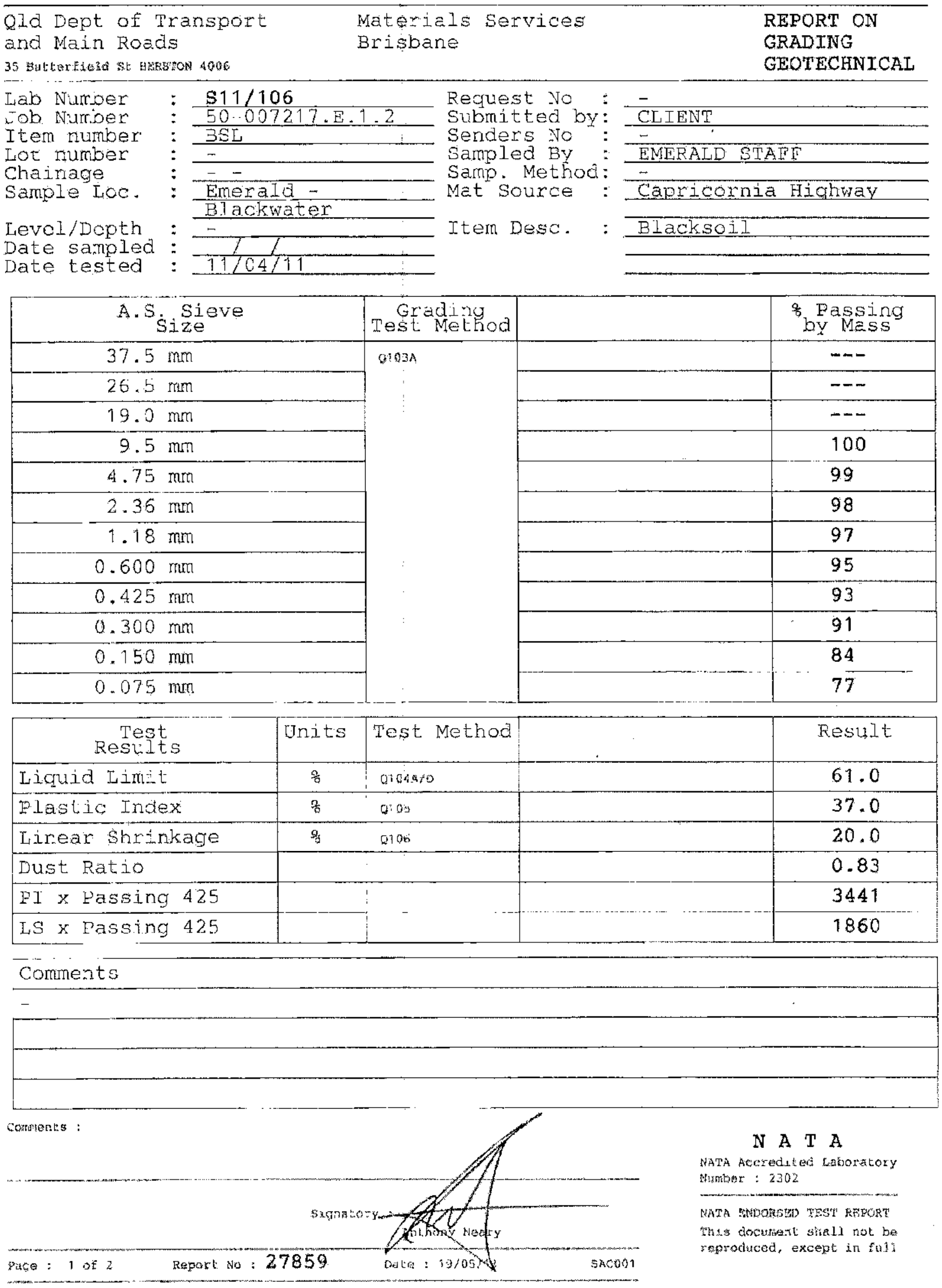
Figure 5.2: Report on Moisture Content for Emerald Soil



Qld Dept of Transport
and Main Roads
35 Butterfield St HERSTON 4006

Materials Services
Brisbane

REPORT ON
GRADING
GEOTECHNICAL

Lab Number	: **S11/106**	Request No	: -
Job Number	: 50-007217.E.1.2	Submitted by	: CLIENT
Item number	: BSL	Senders No	: -
Lot number	: -	Sampled By	: EMERALD STAFF
Chainage	: - -	Samp. Method	: -
Sample Loc.	: Emerald - Blackwater	Mat Source	: Capricornia Highway
Level/Depth	: -	Item Desc.	: Blacksoil
Date sampled	: / /		
Date tested	: 11/04/11		

A.S. Sieve Size	Grading Test Method		% Passing by Mass
37.5 mm	Q103A		---
26.5 mm			---
19.0 mm			---
9.5 mm			100
4.75 mm			99
2.36 mm			98
1.18 mm			97
0.600 mm			95
0.425 mm			93
0.300 mm			91
0.150 mm			84
0.075 mm			77

Test Results	Units	Test Method		Result
Liquid Limit	%	Q104A/D		61.0
Plastic Index	%	Q105		37.0
Linear Shrinkage	%	Q106		20.0
Dust Ratio				0.83
PI x Passing 425				3441
LS x Passing 425				1860

Comments
-

Comments :

Signatory Anthony Neary

Page : 1 of 2 Report No : 27859 Date : 19/05/11 SAC001

N A T A
NATA Accredited Laboratory
Number : 2302

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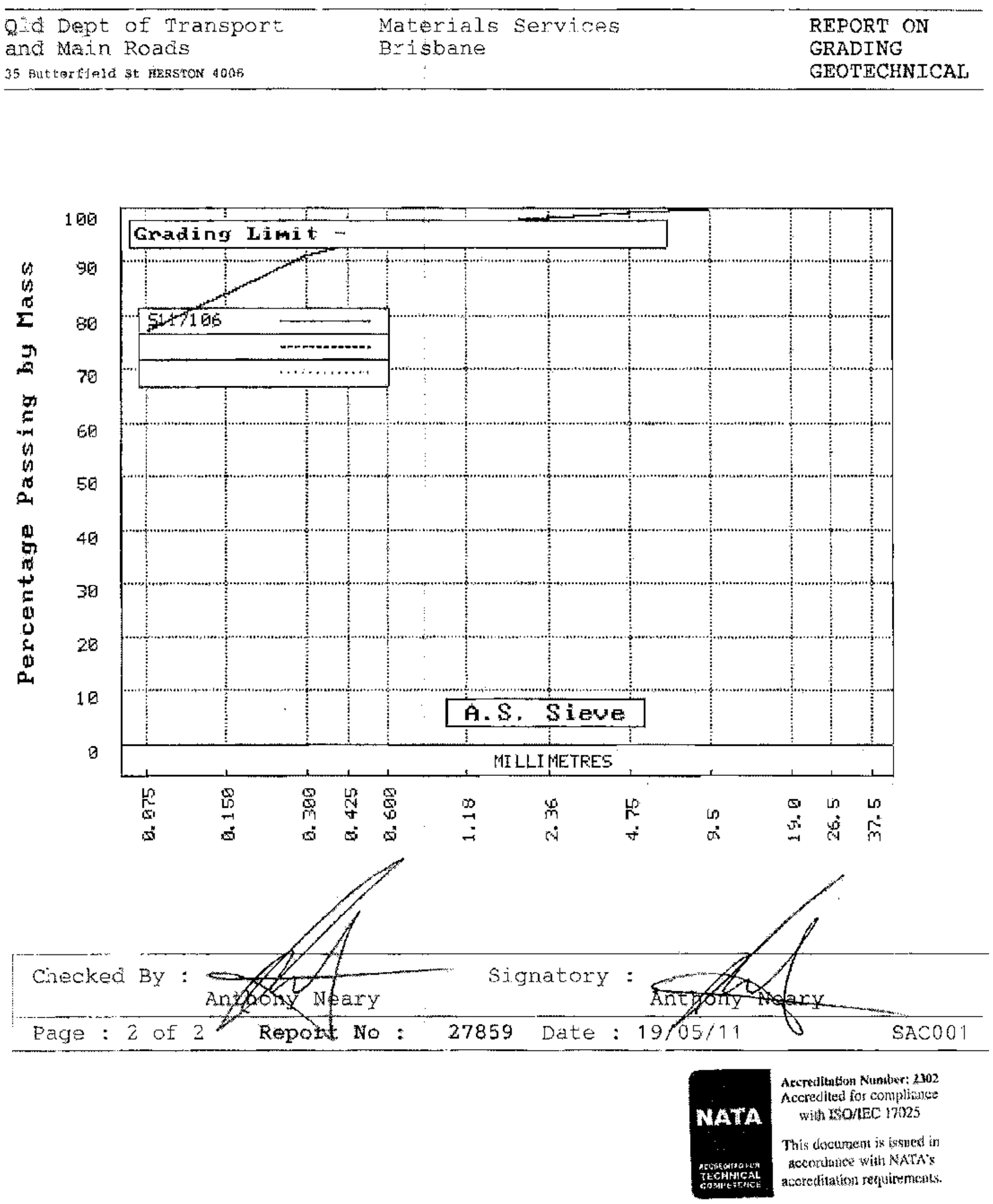
Figure 5.3: Report on Particle Size Distribution and Atterberg Limits for Emerald Soil



Qld Dept of Transport
and Main Roads
35 Butterfield St HERSTON 4006

Materials Services
Brisbane

REPORT ON
GRADING
GEOTECHNICAL

Checked By : Anthony Neary
Signatory : Anthony Neary
Page : 2 of 2 **Report No :** 27859 Date : 19/05/11 SAC001

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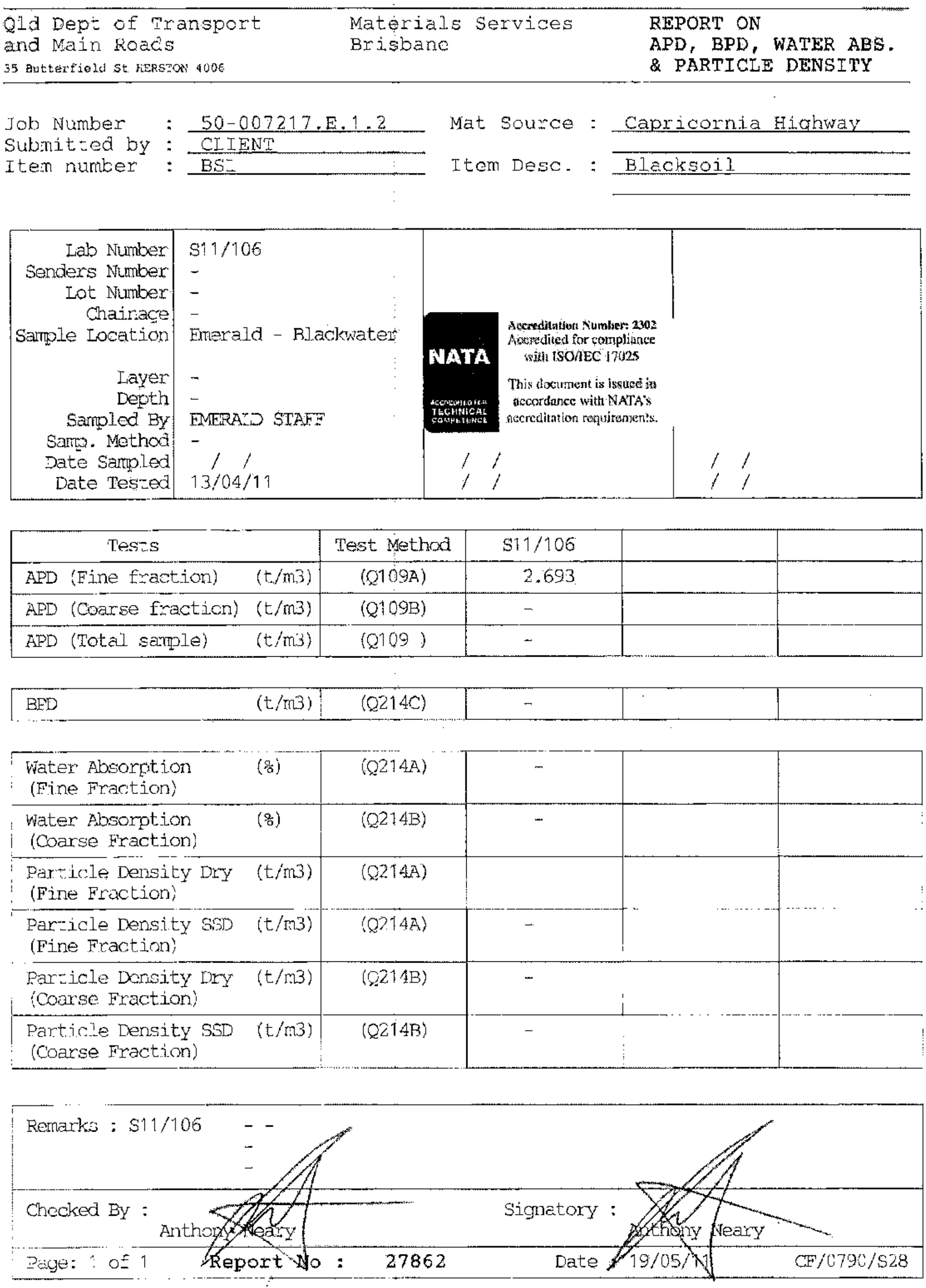
Figure 5.4: Report on Grading Limit for Emerald Soil



Qld Dept of Transport
and Main Roads
35 Butterfield St HERSTON 4006

Materials Services
Brisbane

REPORT ON
APD, BPD, WATER ABS.
& PARTICLE DENSITY

Job Number : 50-007217.E.1.2
Submitted by : CLIENT
Item number : BSI
Mat Source : Capricornia Highway
Item Desc. : Blacksoil

Lab Number	S11/106		
Senders Number	-		
Lot Number	-		
Chainage	-		
Sample Location	Emerald - Blackwater		
Layer	-		
Depth	-		
Sampled By	EMERALD STAFF		
Samp. Method	-		
Date Sampled	/ /	/ /	/ /
Date Tested	13/04/11	/ /	/ /

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Tests	Test Method	S11/106		
APD (Fine fraction) (t/m3)	(Q109A)	2.693		
APD (Coarse fraction) (t/m3)	(Q109B)	-		
APD (Total sample) (t/m3)	(Q109)	-		

BPD (t/m3)	(Q214C)	-		

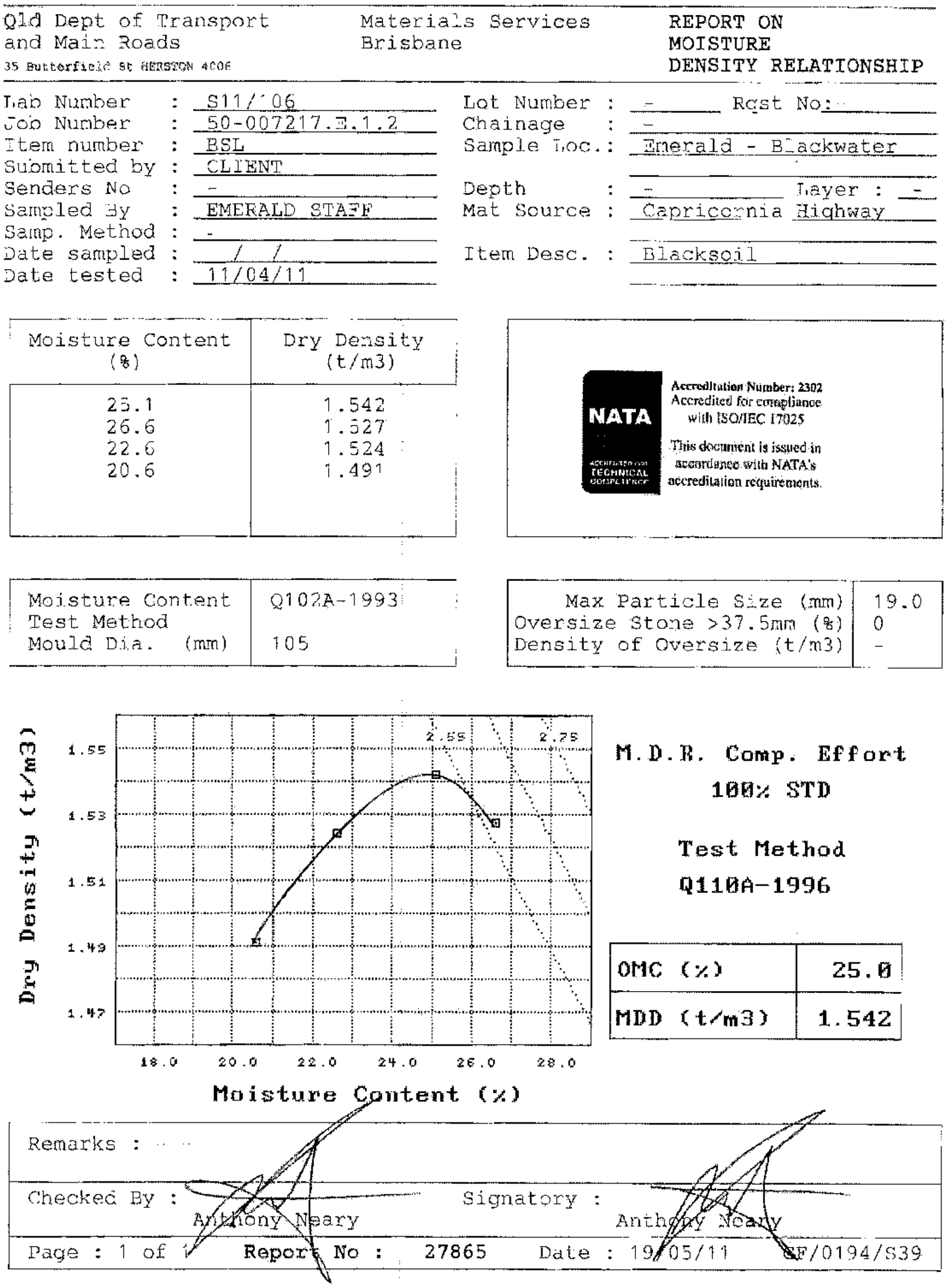
Water Absorption (%) (Fine Fraction)	(Q214A)	-		
Water Absorption (%) (Coarse Fraction)	(Q214B)	-		
Particle Density Dry (t/m3) (Fine Fraction)	(Q214A)			
Particle Density SSD (t/m3) (Fine Fraction)	(Q214A)	-		
Particle Density Dry (t/m3) (Coarse Fraction)	(Q214B)	-		
Particle Density SSD (t/m3) (Coarse Fraction)	(Q214B)	-		

Remarks : S11/106 - -
-
-

Checked By : Anthony Neary
Signatory : Anthony Neary

Page: 1 of 1 Report No : 27862 Date : 19/05/11 CF/0790/S28

Figure 5.5: Report on Apparent Particle Density of Emerald Soil



Qld Dept of Transport
and Main Roads
35 Butterfield St HERSTON 4006

Materials Services
Brisbane

REPORT ON
MOISTURE
DENSITY RELATIONSHIP

Lab Number : S11/106
Job Number : 50-007217.E.1.2
Item number : BSL
Submitted by : CLIENT
Senders No : -
Sampled By : EMERALD STAFF
Samp. Method : -
Date sampled : / /
Date tested : 11/04/11

Lot Number : - Rqst No:
Chainage : -
Sample Loc.: Emerald - Blackwater
Depth : - Layer : -
Mat Source : Capricornia Highway
Item Desc. : Blacksoil

Moisture Content (%)	Dry Density (t/m3)
25.1	1.542
26.6	1.527
22.6	1.524
20.6	1.491

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Moisture Content Test Method	Q102A-1993
Mould Dia. (mm)	105

Max Particle Size (mm)	19.0
Oversize Stone >37.5mm (%)	0
Density of Oversize (t/m3)	-

M.D.R. Comp. Effort
100% STD

Test Method
Q110A-1996

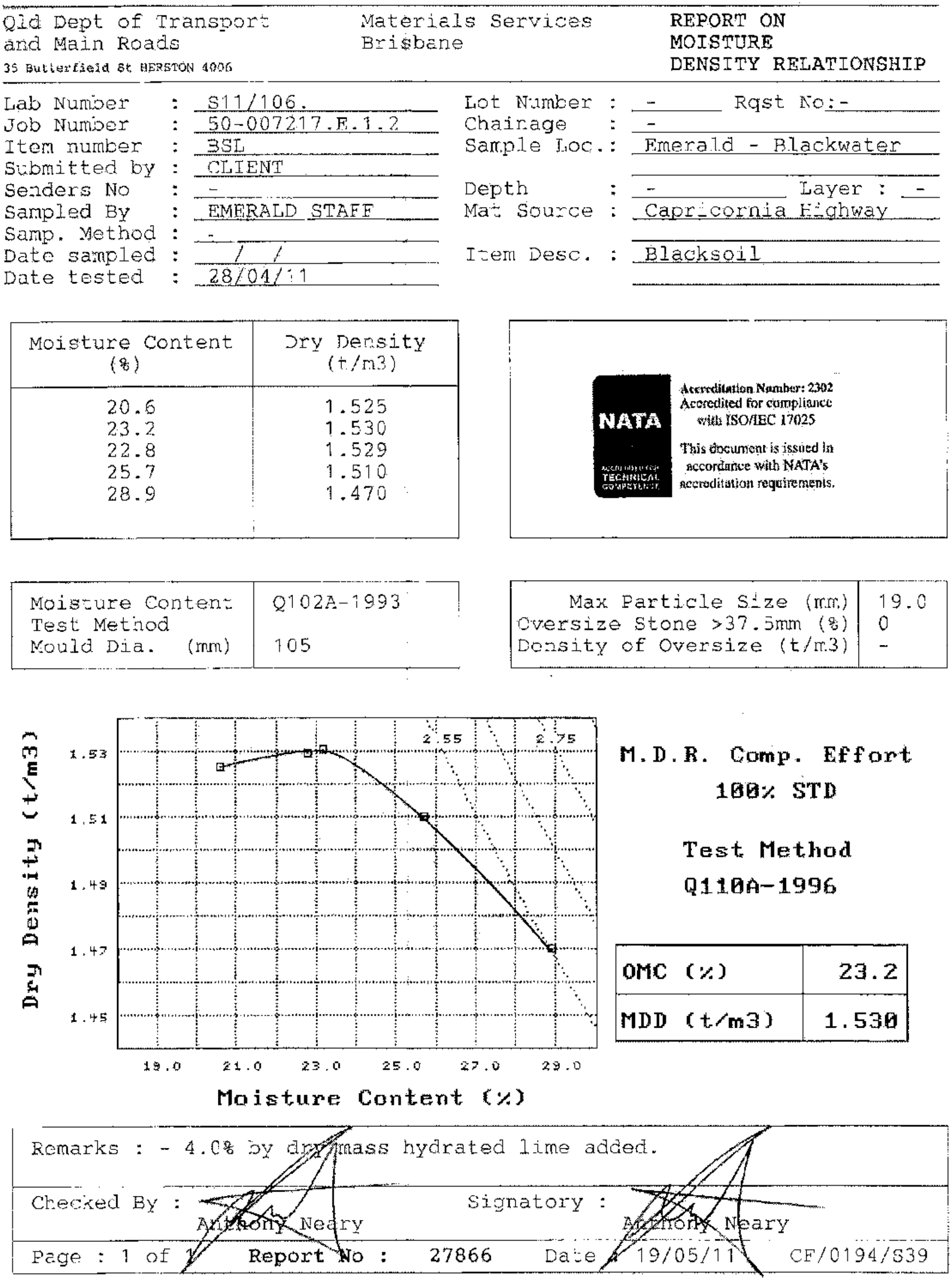
OMC (%)	25.0
MDD (t/m3)	1.542

Remarks :

Checked By : Anthony Neary
Signatory : Anthony Neary

Page : 1 of 1 **Report No :** 27865 Date : 19/05/11 SF/0194/S39

Figure 5.6: Report on Moisture Density Relationship (Standard Compaction) for Emerald Soil



Qld Dept of Transport
and Main Roads
35 Butterfield St HERSTON 4006

Materials Services
Brisbane

REPORT ON
MOISTURE
DENSITY RELATIONSHIP

Lab Number : S11/106.
Job Number : 50-007217.E.1.2
Item number : BSL
Submitted by : CLIENT
Senders No : -
Sampled By : EMERALD STAFF
Samp. Method : -
Date sampled : / /
Date tested : 28/04/11

Lot Number : - Rqst No:-
Chainage : -
Sample Loc.: Emerald - Blackwater
Depth : - Layer : -
Mat Source : Capricornia Highway
Item Desc. : Blacksoil

Moisture Content (%)	Dry Density (t/m3)
20.6	1.525
23.2	1.530
22.8	1.529
25.7	1.510
28.9	1.470

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Accreditation Number: 2302
Accredited for compliance with ISO/IEC 17025
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Moisture Content Test Method	Q102A-1993
Mould Dia. (mm)	105

Max Particle Size (mm)	19.0
Oversize Stone >37.5mm (%)	0
Density of Oversize (t/m3)	-

M.D.R. Comp. Effort
100% STD

Test Method
Q110A-1996

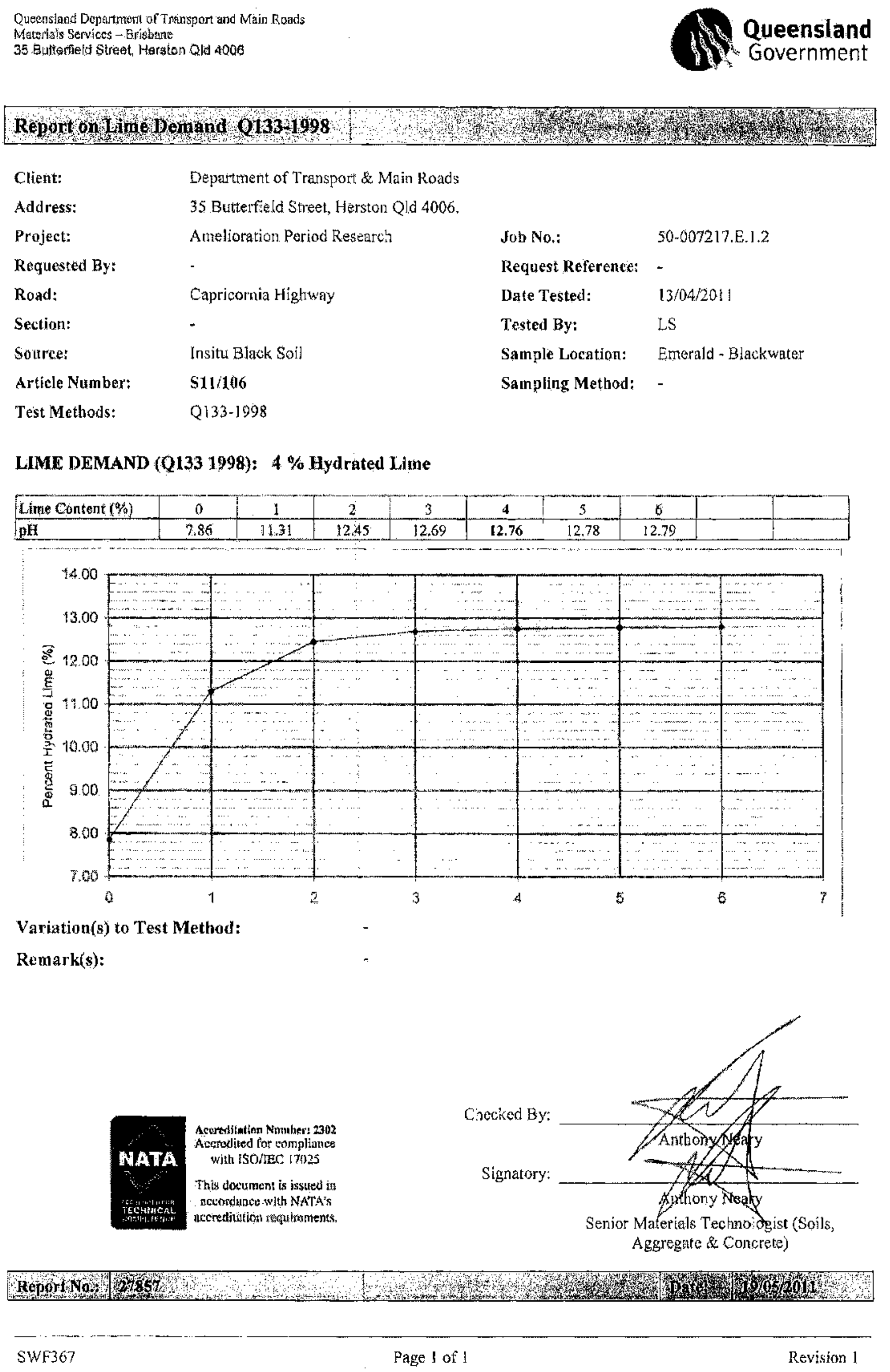
OMC (%)	23.2
MDD (t/m3)	1.530

Remarks : - 4.0% by dry mass hydrated lime added.

Checked By : Anthony Neary
Signatory : Anthony Neary

Page : 1 of 1 Report No : 27866 Date : 19/05/11 CF/0194/S39

Figure 5.7: Report on Moisture Density Relationship (with Lime @ LD) of Emerald Soil



Queensland Department of Transport and Main Roads
Materials Services – Brisbane
35 Butterfield Street, Herston Qld 4006

Queensland Government

Report on Lime Demand Q133-1998

Client:	Department of Transport & Main Roads		
Address:	35 Butterfield Street, Herston Qld 4006.		
Project:	Amelioration Period Research	**Job No.:**	50-007217.E.1.2
Requested By:	-	**Request Reference:**	-
Road:	Capricornia Highway	**Date Tested:**	13/04/2011
Section:	-	**Tested By:**	LS
Source:	Insitu Black Soil	**Sample Location:**	Emerald - Blackwater
Article Number:	**S11/106**	**Sampling Method:**	-
Test Methods:	Q133-1998		

LIME DEMAND (Q133 1998): 4 % Hydrated Lime

Lime Content (%)	0	1	2	3	4	5	6		
pH	7.86	11.31	12.45	12.69	12.76	12.78	12.79		

Variation(s) to Test Method: -

Remark(s): -

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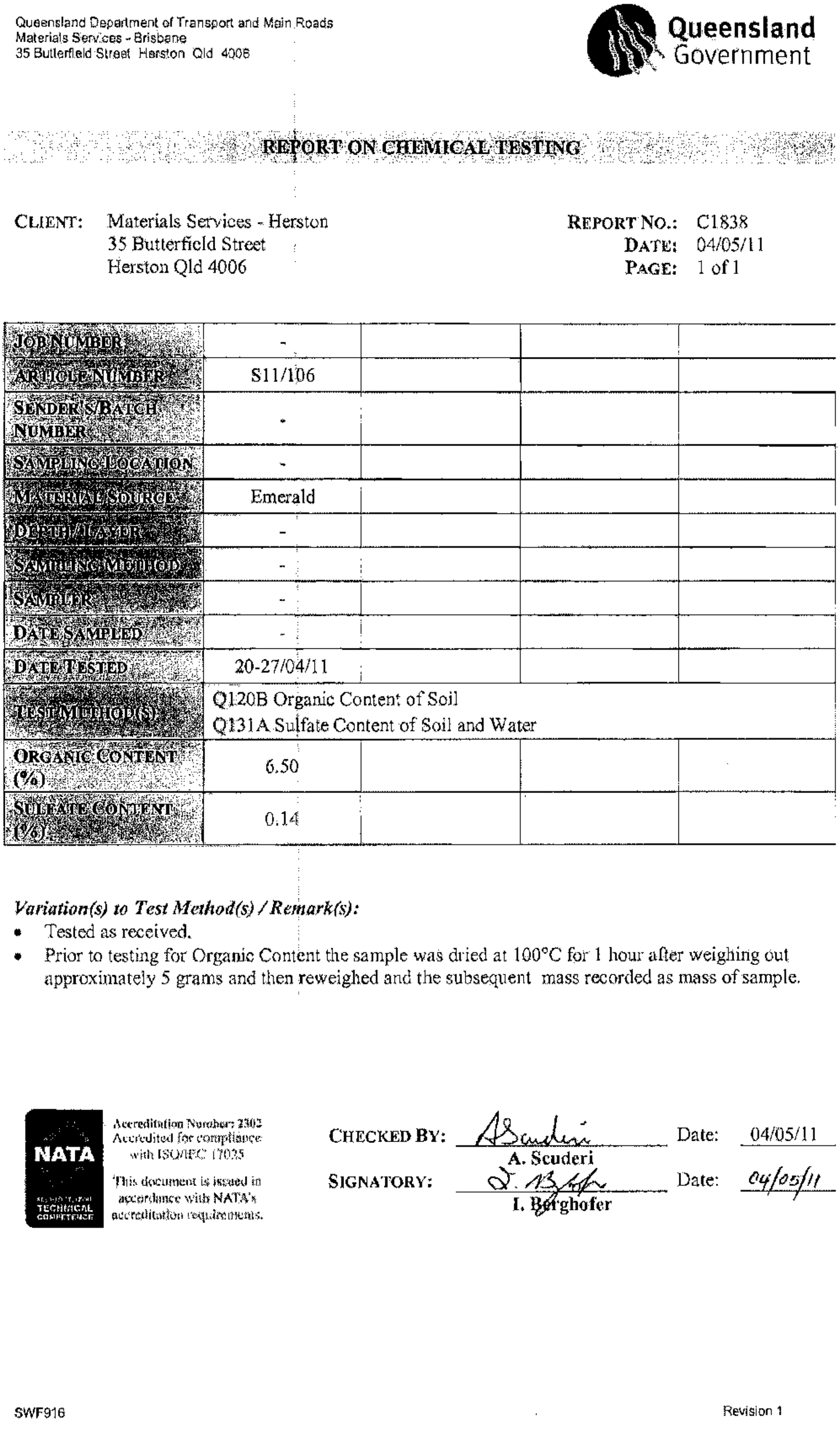
Checked By: Anthony Neary

Signatory: Anthony Neary
Senior Materials Technologist (Soils, Aggregate & Concrete)

Report No.:	**27857**	**Date:**	**19/05/2011**

SWF367 Page 1 of 1 Revision 1

Figure 5.8: Report on Lime Demand for Emerald Soil



Queensland Department of Transport and Main Roads
Materials Services - Brisbane
35 Butterfield Street Herston Qld 4006

Queensland Government

REPORT ON CHEMICAL TESTING

CLIENT: Materials Services - Herston
35 Butterfield Street
Herston Qld 4006

REPORT NO.: C1838
DATE: 04/05/11
PAGE: 1 of 1

JOB NUMBER	-			
ARTICLE NUMBER	S11/106			
SENDER'S/BATCH NUMBER	-			
SAMPLING LOCATION	-			
MATERIAL SOURCE	Emerald			
DEPTH/LAYER	-			
SAMPLING METHOD	-			
SAMPLER	-			
DATE SAMPLED	-			
DATE TESTED	20-27/04/11			
TEST METHOD(S)	Q120B Organic Content of Soil Q131A Sulfate Content of Soil and Water			
ORGANIC CONTENT (%)	6.50			
SULFATE CONTENT (%)	0.14			

Variation(s) to Test Method(s) / Remark(s):

- Tested as received.
- Prior to testing for Organic Content the sample was dried at 100°C for 1 hour after weighing out approximately 5 grams and then reweighed and the subsequent mass recorded as mass of sample.

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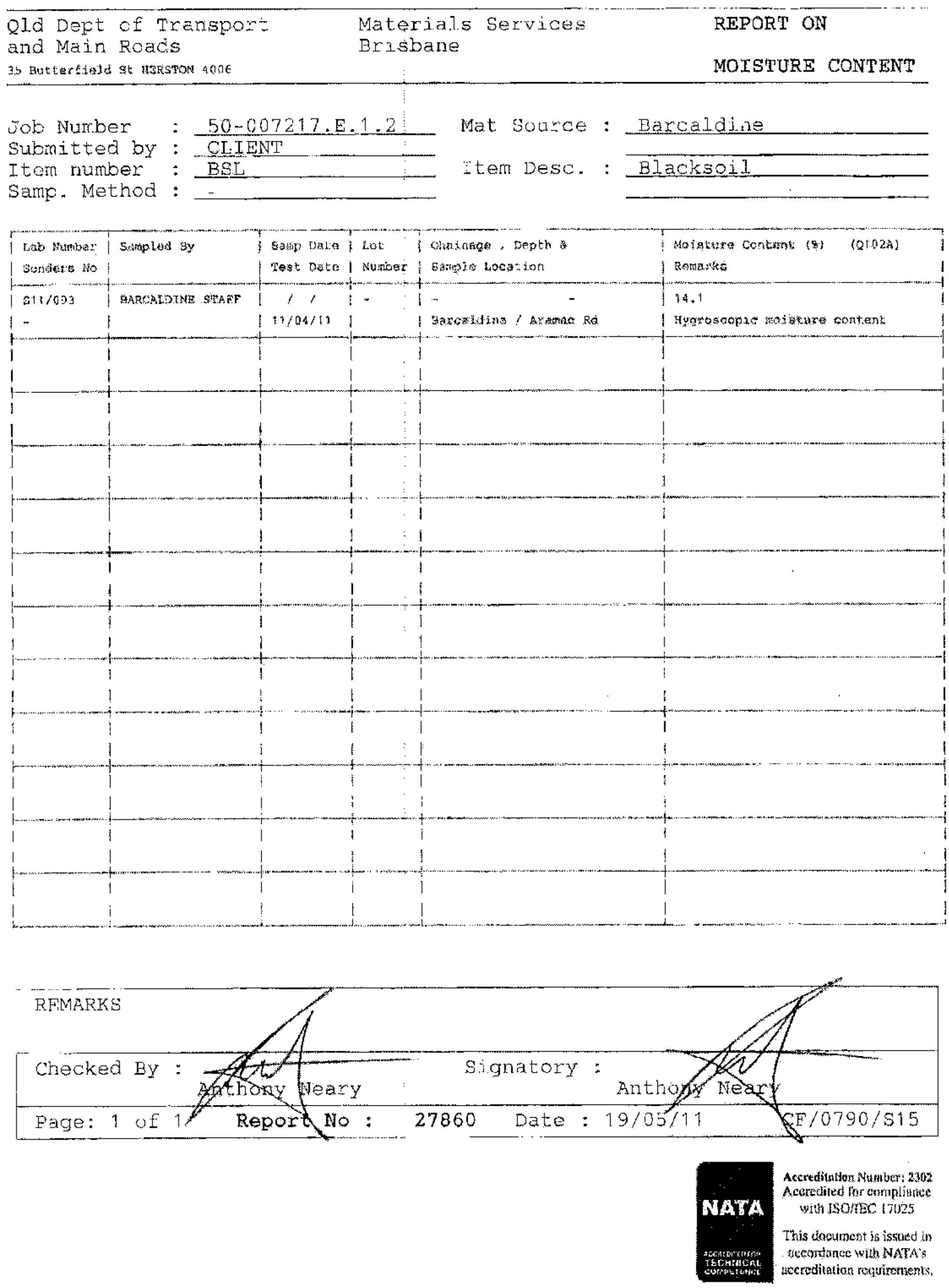
CHECKED BY: AScuderi Date: 04/05/11
A. Scuderi

SIGNATORY: Date: 04/05/11
I. Berghofer

SWF916 Revision 1

Figure 5.9: Report on Organic and Sulphate Content of Emerald Soil

Appendix D - Test Results for Barcaldine Soil



Qld Dept of Transport
and Main Roads
35 Butterfield St HERSTON 4006

Materials Services
Brisbane

REPORT ON
MOISTURE CONTENT

Job Number : 50-007217.E.1.2
Submitted by : CLIENT
Item number : BSL
Samp. Method : -

Mat Source : Barcaldine
Item Desc. : Blacksoil

Lab Number Senders No	Sampled By	Samp Date Test Date	Lot Number	Chainage , Depth & Sample Location	Moisture Content (%) (Q102A) Remarks
S11/093 -	BARCALDINE STAFF	/ / 11/04/11	-	- - Barcaldine / Aramac Rd	14.1 Hygroscopic moisture content

REMARKS

Checked By : Anthony Neary
Signatory : Anthony Neary

Page: 1 of 1 **Report No :** 27860 Date : 19/05/11 CF/0790/S15

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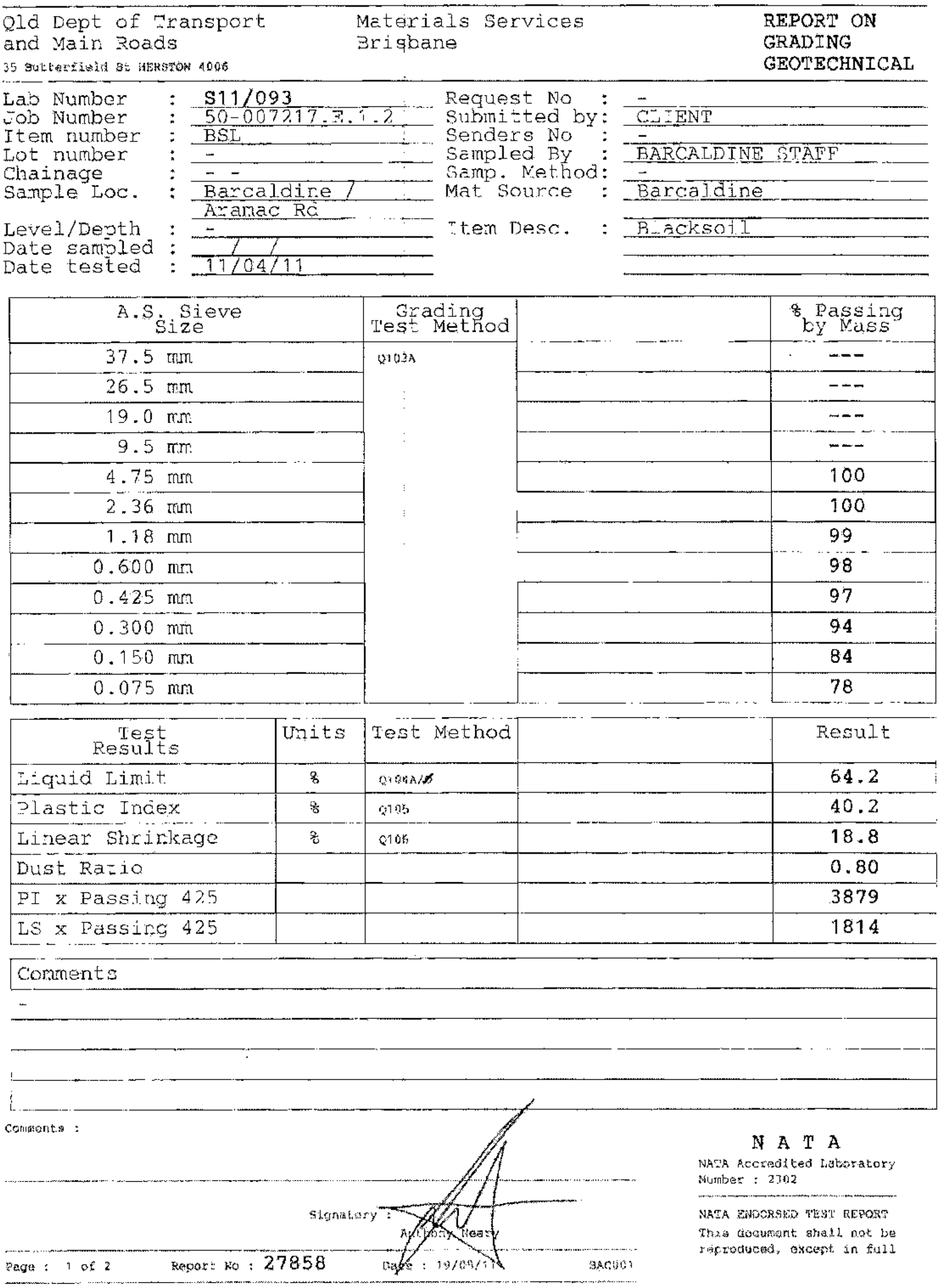
Figure 5.10: Report on Moisture Content for Barcaldine Soil



Qld Dept of Transport
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35 Butterfield St HERSTON 4006

Materials Services
Brisbane

REPORT ON
GRADING
GEOTECHNICAL

Lab Number : S11/093
Job Number : 50-007217.E.1.2
Item number : BSL
Lot number : -
Chainage : - -
Sample Loc. : Barcaldine / Aramac Rd
Level/Depth : -
Date sampled : / /
Date tested : 11/04/11

Request No : -
Submitted by: CLIENT
Senders No : -
Sampled By : BARCALDINE STAFF
Samp. Method: -
Mat Source : Barcaldine
Item Desc. : Blacksoil

A.S. Sieve Size	Grading Test Method		% Passing by Mass
37.5 mm	Q103A		---
26.5 mm			---
19.0 mm			---
9.5 mm			---
4.75 mm			100
2.36 mm			100
1.18 mm			99
0.600 mm			98
0.425 mm			97
0.300 mm			94
0.150 mm			84
0.075 mm			78

Test Results	Units	Test Method		Result
Liquid Limit	%	Q104A/[illegible]		64.2
Plastic Index	%	Q105		40.2
Linear Shrinkage	%	Q106		18.8
Dust Ratio				0.80
PI x Passing 425				3879
LS x Passing 425				1814

Comments
-

Comments :

Signatory :
Anthony Neary

Page : 1 of 2 Report No : 27858 Date : 19/05/11 BAC001

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NATA Accredited Laboratory
Number : 2302

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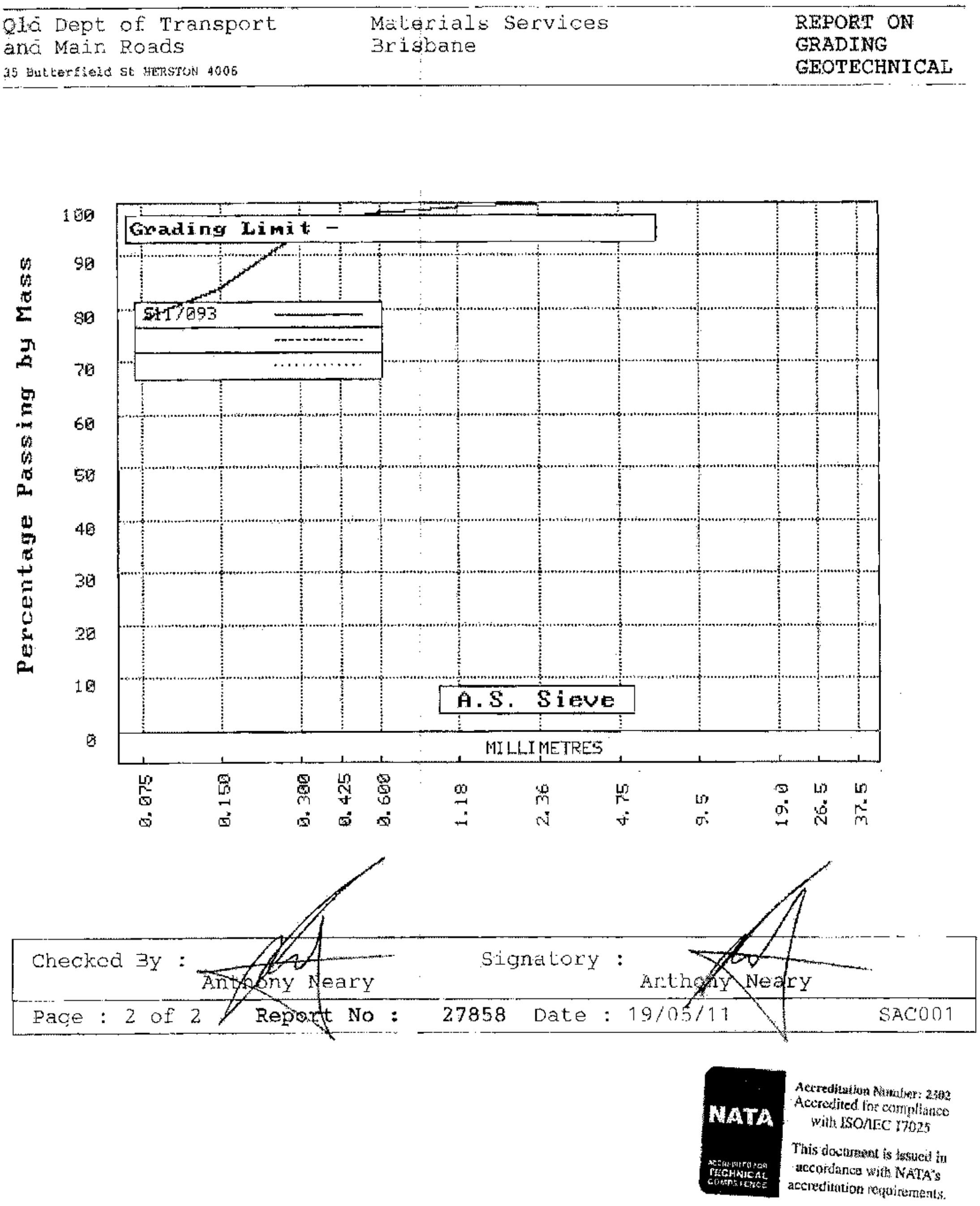
Figure 5.11: Report on Particle Size Distribution and Atterberg Limits of Barcaldine Soil



Qld Dept of Transport
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35 Butterfield St HERSTON 4006

Materials Services
Brisbane

REPORT ON
GRADING
GEOTECHNICAL

Checked By : Anthony Neary
Signatory : Anthony Neary

Page : 2 of 2 Report No : 27858 Date : 19/05/11 SAC001

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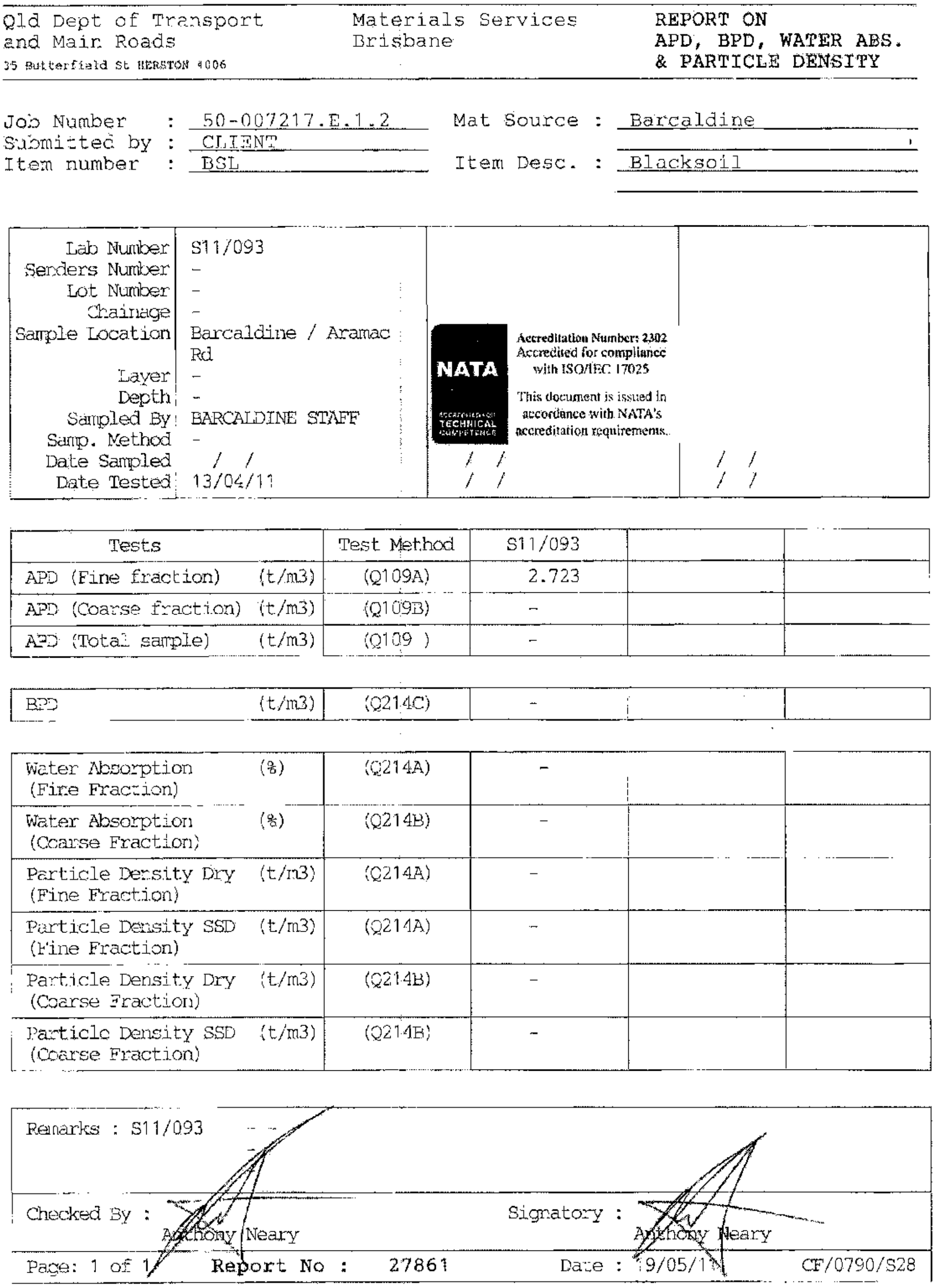
Figure 5.12: Report on Grading Limits of Barcaldine Soil



Qld Dept of Transport
and Main Roads
35 Butterfield St HERSTON 4006

Materials Services
Brisbane

REPORT ON
APD, BPD, WATER ABS.
& PARTICLE DENSITY

Job Number : 50-007217.E.1.2
Submitted by : CLIENT
Item number : BSL

Mat Source : Barcaldine
Item Desc. : Blacksoil

Lab Number	S11/093		
Senders Number	-		
Lot Number	-		
Chainage	-		
Sample Location	Barcaldine / Aramac Rd	Accreditation Number: 2302 Accredited for compliance with ISO/IEC 17025 This document is issued in accordance with NATA's accreditation requirements.	
Layer	-		
Depth	-		
Sampled By	BARCALDINE STAFF		
Samp. Method	-		
Date Sampled	/ /	/ /	/ /
Date Tested	13/04/11	/ /	/ /

Tests	Test Method	S11/093		
APD (Fine fraction) (t/m3)	(Q109A)	2.723		
APD (Coarse fraction) (t/m3)	(Q109B)	-		
APD (Total sample) (t/m3)	(Q109)	-		
BPD (t/m3)	(Q214C)	-		
Water Absorption (%) (Fine Fraction)	(Q214A)	-		
Water Absorption (%) (Coarse Fraction)	(Q214B)	-		
Particle Density Dry (t/m3) (Fine Fraction)	(Q214A)	-		
Particle Density SSD (t/m3) (Fine Fraction)	(Q214A)	-		
Particle Density Dry (t/m3) (Coarse Fraction)	(Q214B)	-		
Particle Density SSD (t/m3) (Coarse Fraction)	(Q214B)	-		

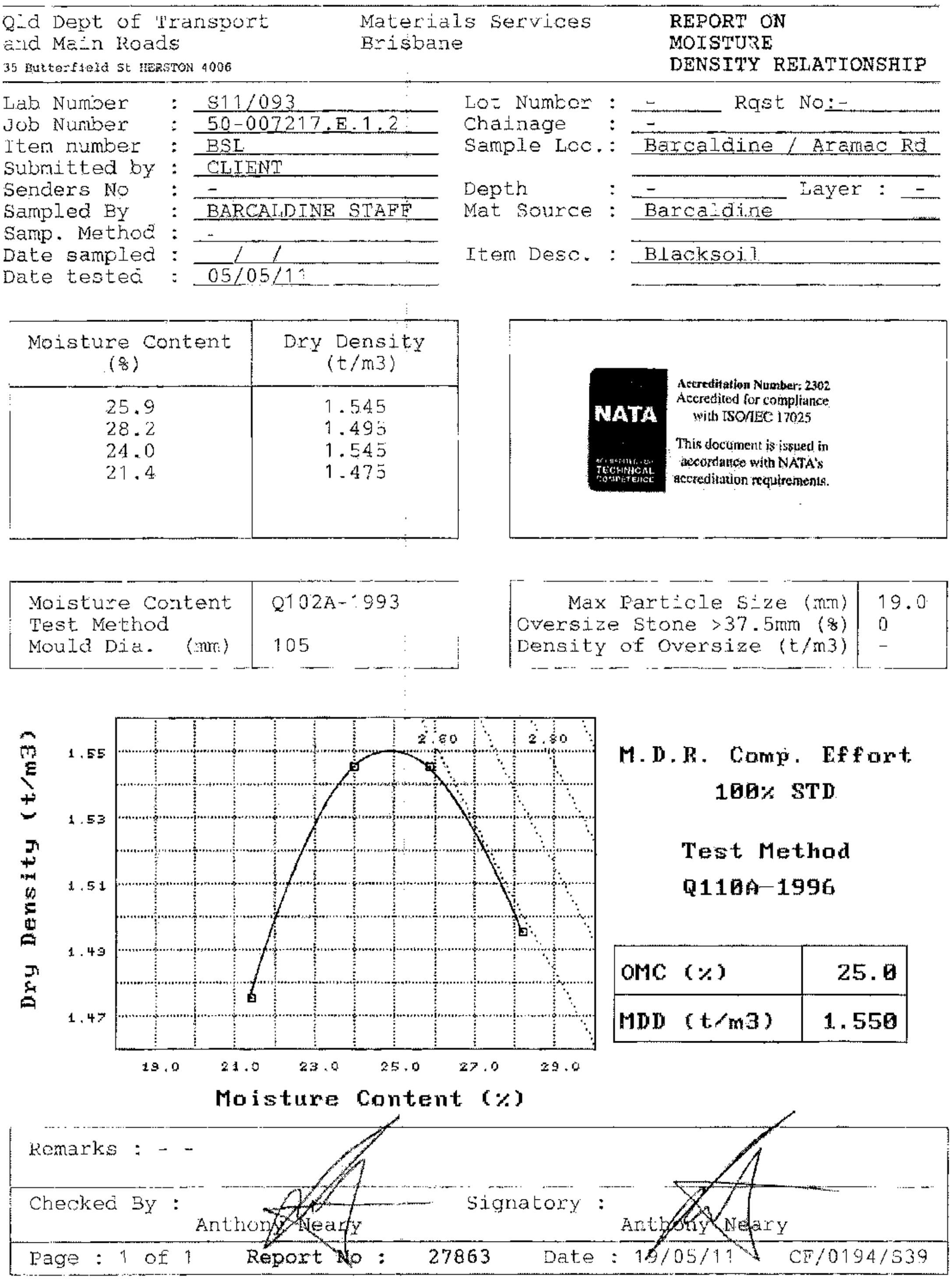
Remarks : S11/093 - -

Checked By : Anthony Neary

Signatory : Anthony Neary

Page: 1 of 1 **Report No : 27861** Date : 19/05/11 CF/0790/S28

Figure 5.13: Report on Apparent Particle Density of Barcaldine Soil



Qld Dept of Transport and Main Roads
35 Butterfield St HERSTON 4006

Materials Services
Brisbane

REPORT ON MOISTURE DENSITY RELATIONSHIP

Lab Number : S11/093
Job Number : 50-007217.E.1.2
Item number : BSL
Submitted by : CLIENT
Senders No : -
Sampled By : BARCALDINE STAFF
Samp. Method : -
Date sampled : / /
Date tested : 05/05/11

Lot Number : - Rqst No: -
Chainage : -
Sample Loc. : Barcaldine / Aramac Rd
Depth : - Layer : -
Mat Source : Barcaldine
Item Desc. : Blacksoil

Moisture Content (%)	Dry Density (t/m3)
25.9	1.545
28.2	1.495
24.0	1.545
21.4	1.475

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Moisture Content Test Method	Q102A-1993
Mould Dia. (mm)	105

Max Particle Size (mm)	19.0
Oversize Stone >37.5mm (%)	0
Density of Oversize (t/m3)	-

M.D.R. Comp. Effort
100% STD

Test Method
Q110A-1996

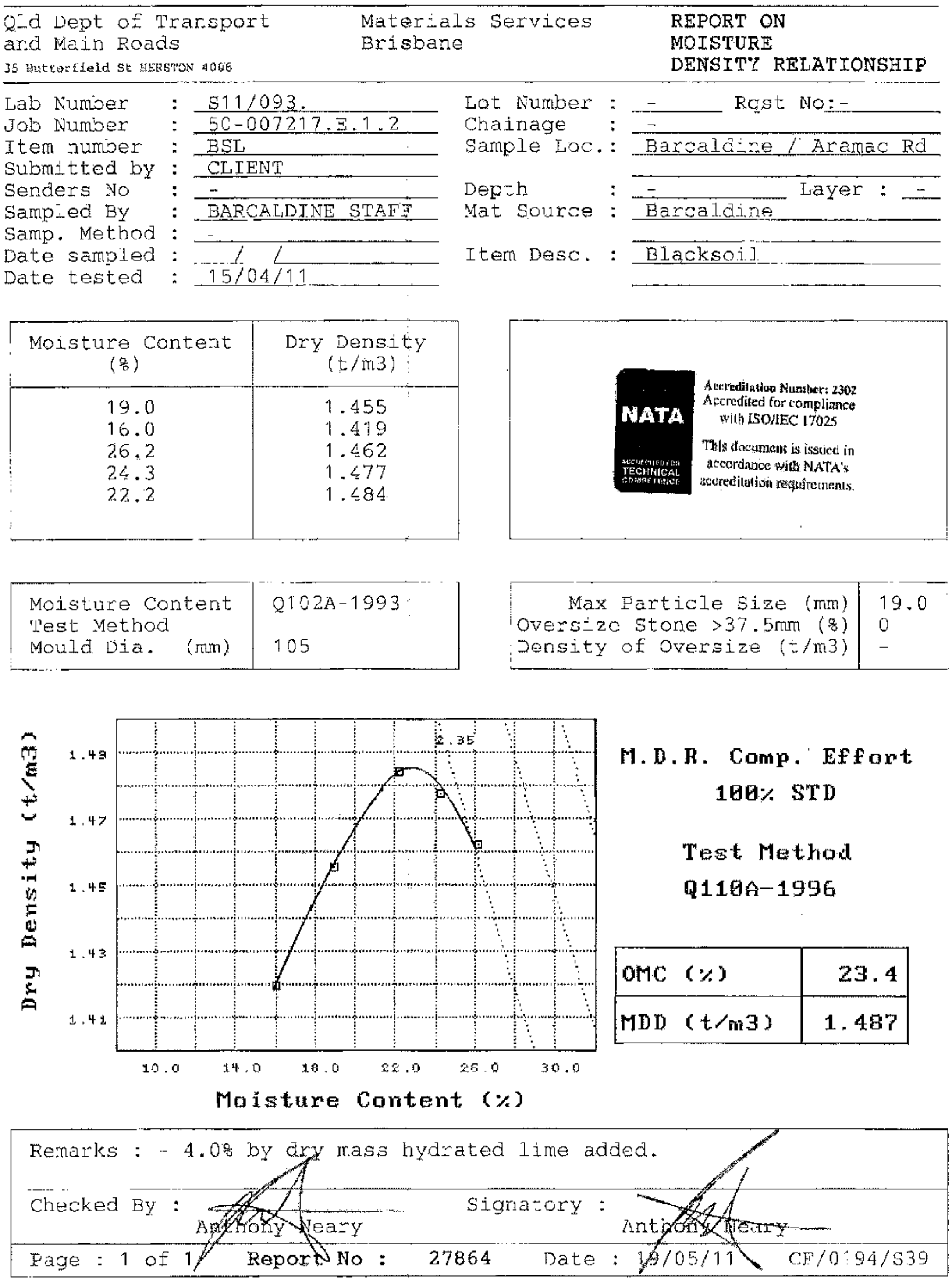
OMC (%)	25.0
MDD (t/m3)	1.550

Remarks : - -

Checked By : Anthony Neary
Signatory : Anthony Neary

Page : 1 of 1 Report No : 27863 Date : 19/05/11 CF/0194/S39

Figure 5.14: Report on Moisture Density Relationship (Standard Compaction) of Barcaldine Soil



Qld Dept of Transport
and Main Roads
35 Butterfield St HERSTON 4006

Materials Services
Brisbane

REPORT ON MOISTURE DENSITY RELATIONSHIP

Lab Number : S11/093.
Job Number : 5C-007217.E.1.2
Item number : BSL
Submitted by : CLIENT
Senders No : -
Sampled By : BARCALDINE STAFF
Samp. Method : -
Date sampled : / /
Date tested : 15/04/11

Lot Number : - Rqst No:-
Chainage : -
Sample Loc.: Barcaldine / Aramac Rd
Depth : - Layer : -
Mat Source : Barcaldine
Item Desc. : Blacksoil

Moisture Content (%)	Dry Density (t/m3)
19.0	1.455
16.0	1.419
26.2	1.462
24.3	1.477
22.2	1.484

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Moisture Content Test Method	Q102A-1993
Mould Dia. (mm)	105

Max Particle Size (mm)	19.0
Oversize Stone >37.5mm (%)	0
Density of Oversize (t/m3)	-

M.D.R. Comp. Effort
100% STD

Test Method
Q110A-1996

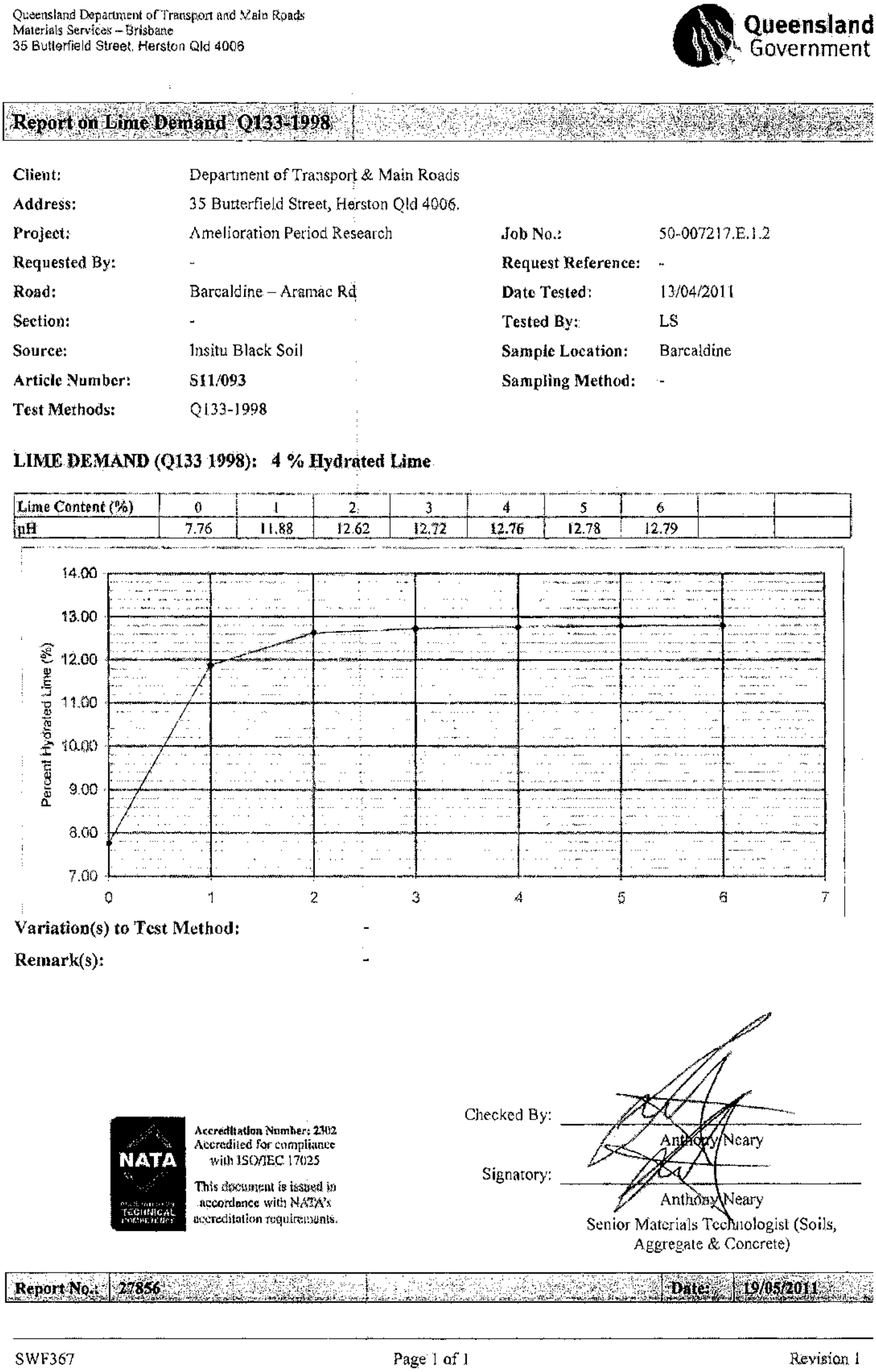
OMC (%)	23.4
MDD (t/m3)	1.487

Remarks : - 4.0% by dry mass hydrated lime added.

Checked By : Anthony Neary
Signatory : Anthony Neary

Page : 1 of 1 Report No : 27864 Date : 19/05/11 CF/0194/S39

Figure 5.15: Report on Moisture Density Relationship (with Lime @ LD) for Barcaldine Soil



Queensland Department of Transport and Main Roads
Materials Services – Brisbane
35 Butterfield Street, Herston Qld 4006

Queensland Government

Report on Lime Demand Q133-1998

Client:	Department of Transport & Main Roads		
Address:	35 Butterfield Street, Herston Qld 4006.		
Project:	Amelioration Period Research	**Job No.:**	50-007217.E.1.2
Requested By:	-	**Request Reference:**	-
Road:	Barcaldine – Aramac Rd	**Date Tested:**	13/04/2011
Section:	-	**Tested By:**	LS
Source:	Insitu Black Soil	**Sample Location:**	Barcaldine
Article Number:	S11/093	**Sampling Method:**	-
Test Methods:	Q133-1998		

LIME DEMAND (Q133 1998): 4 % Hydrated Lime

Lime Content (%)	0	1	2	3	4	5	6		
pH	7.76	11.88	12.62	12.72	12.76	12.78	12.79		

Variation(s) to Test Method: -

Remark(s): -

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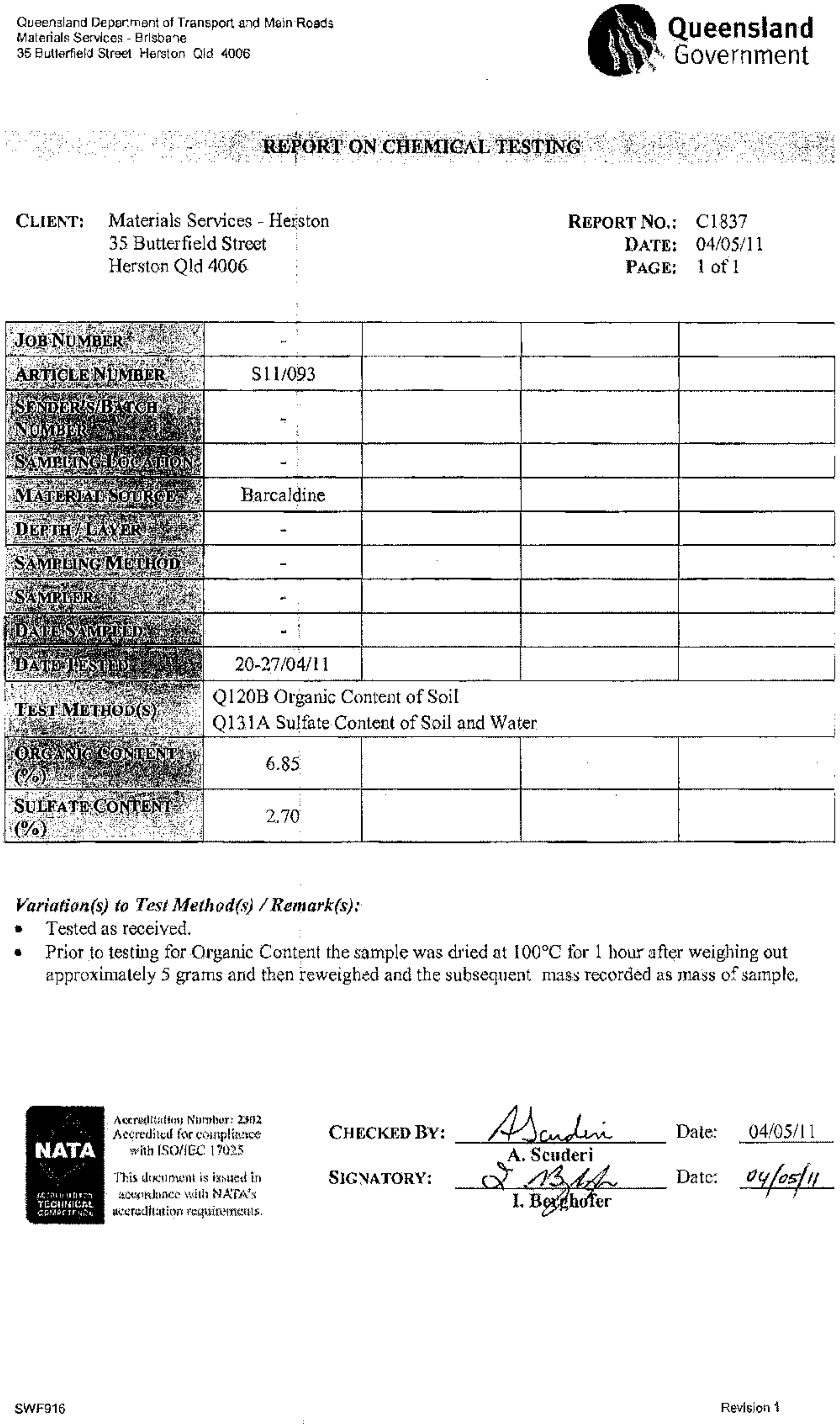
Checked By: Anthony Neary

Signatory: Anthony Neary
Senior Materials Technologist (Soils, Aggregate & Concrete)

Report No.:	27856		**Date:**	19/05/2011

SWF367 Page 1 of 1 Revision 1

Figure 5.16: Report on Lime Demand for Barcaldine Soil



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Materials Services - Brisbane
35 Butterfield Street Herston Qld 4006

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REPORT ON CHEMICAL TESTING

CLIENT: Materials Services - Herston
35 Butterfield Street
Herston Qld 4006

REPORT NO.: C1837
DATE: 04/05/11
PAGE: 1 of 1

JOB NUMBER	-			
ARTICLE NUMBER	S11/093			
SENDER'S/BATCH NUMBER	-			
SAMPLING LOCATION	-			
MATERIAL SOURCE	Barcaldine			
DEPTH / LAYER	-			
SAMPLING METHOD	-			
SAMPLER	-			
DATE SAMPLED	-			
DATE TESTED	20-27/04/11			
TEST METHOD(S)	Q120B Organic Content of Soil Q131A Sulfate Content of Soil and Water			
ORGANIC CONTENT (%)	6.85			
SULFATE CONTENT (%)	2.70			

Variation(s) to Test Method(s) / Remark(s):

- Tested as received.
- Prior to testing for Organic Content the sample was dried at 100°C for 1 hour after weighing out approximately 5 grams and then reweighed and the subsequent mass recorded as mass of sample.

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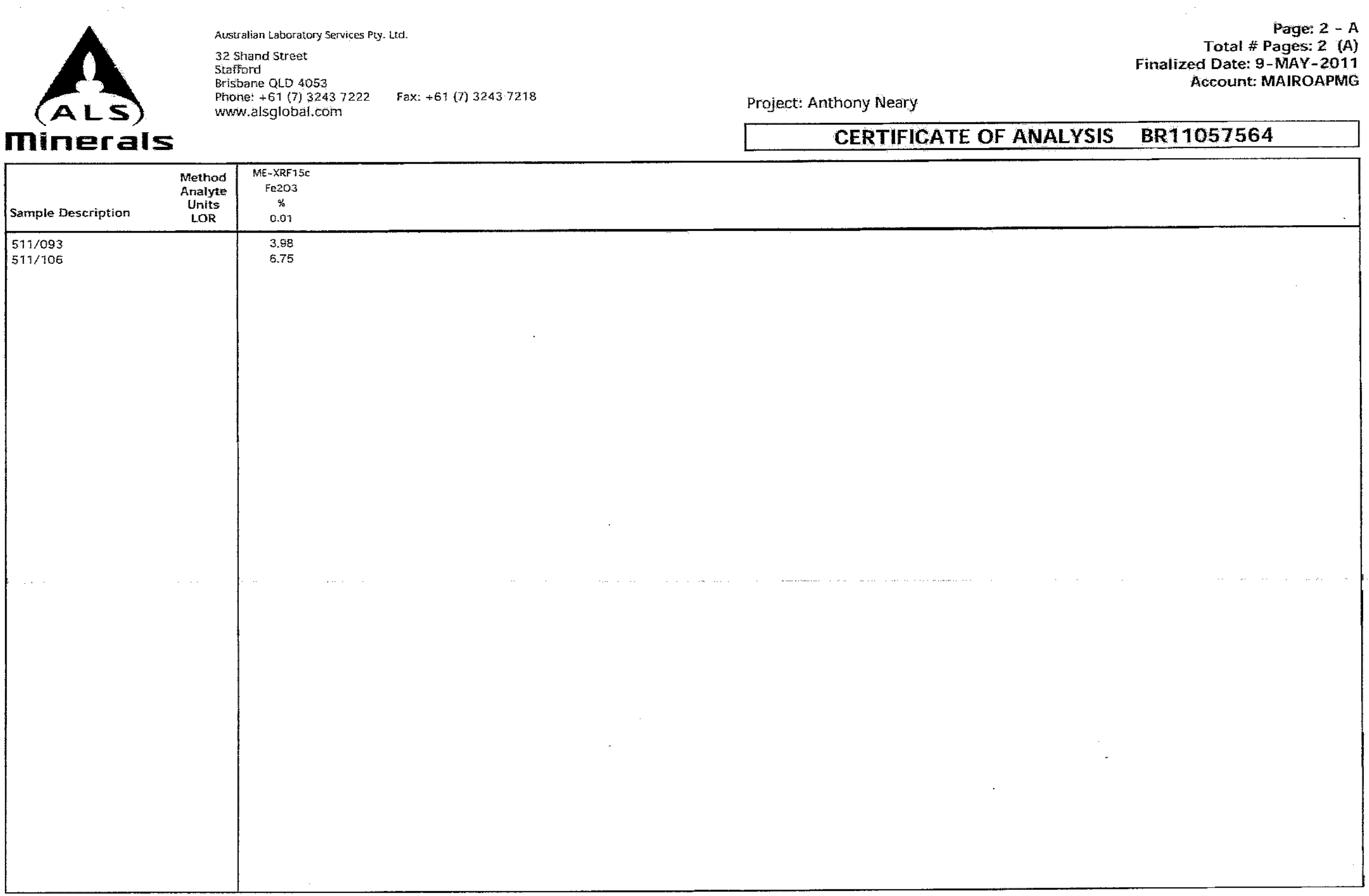
CHECKED BY: A Scuderi
A. Scuderi
Date: 04/05/11

SIGNATORY:
I. Berghofer
Date: 04/05/11

SWF916

Revision 1

Figure 5.17: Report on Organic and Sulphate Content of Barcaldine Soil



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Page: 2 - A
Total # Pages: 2 (A)
Finalized Date: 9-MAY-2011
Account: MAIROAPMG

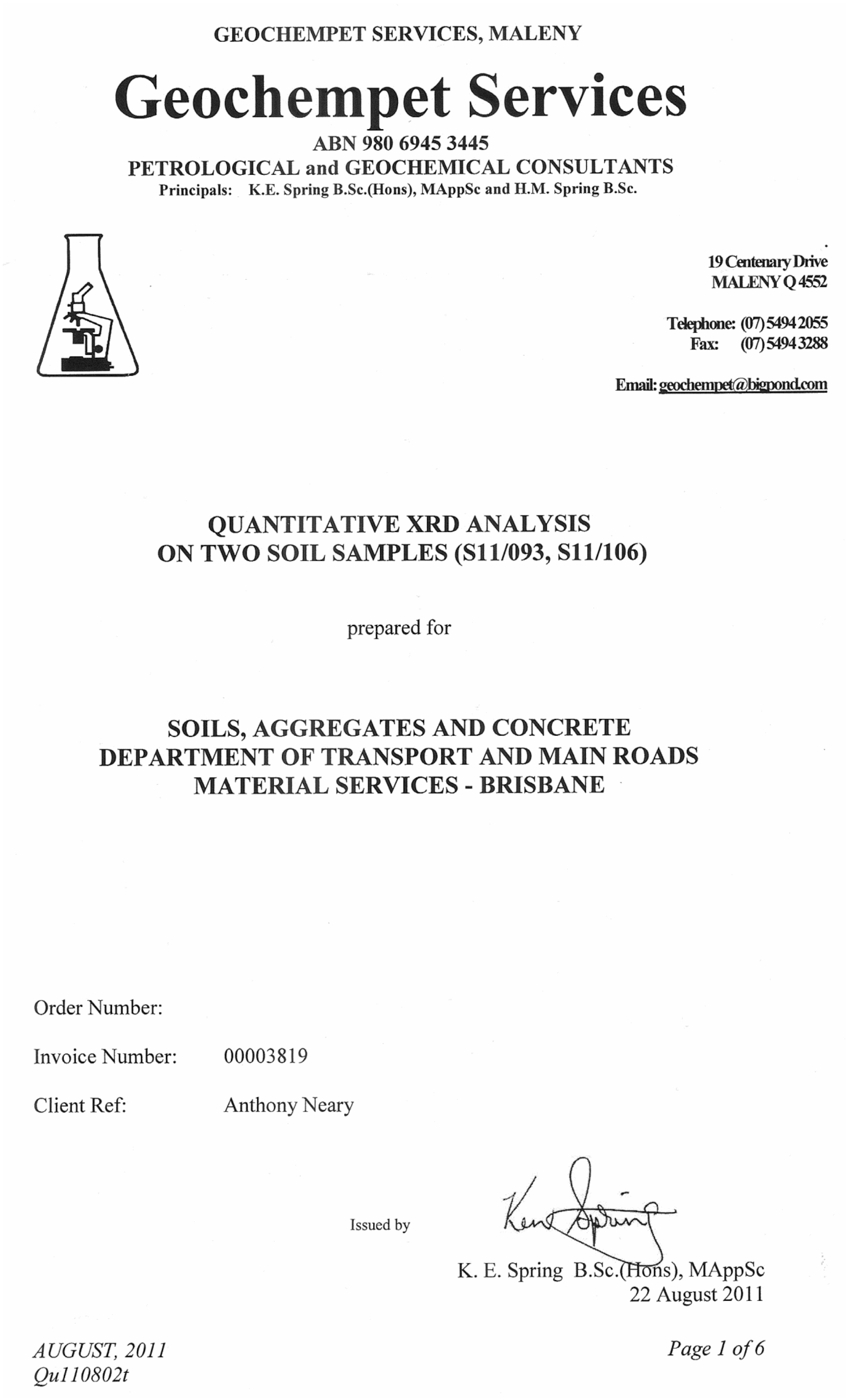
Project: Anthony Neary

CERTIFICATE OF ANALYSIS BR11057564

Sample Description	Method Analyte Units LOR	ME-XRF15c Fe2O3 % 0.01
511/093		3.98
511/106		6.75

Figure 5.18: Report on Iron Content of Emerald and Barcaldine Soils

Appendix E - XRD Report



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Geochempet Services

ABN 980 6945 3445
PETROLOGICAL and GEOCHEMICAL CONSULTANTS
Principals: K.E. Spring B.Sc.(Hons), MAppSc and H.M. Spring B.Sc.

19 Centenary Drive
MALENY Q 4552

Telephone: (07) 5494 2055
Fax: (07) 5494 3288

Email: geochempet@bigpond.com

QUANTITATIVE XRD ANALYSIS ON TWO SOIL SAMPLES (S11/093, S11/106)

prepared for

SOILS, AGGREGATES AND CONCRETE
DEPARTMENT OF TRANSPORT AND MAIN ROADS
MATERIAL SERVICES - BRISBANE

Order Number:

Invoice Number: 00003819

Client Ref: Anthony Neary

Issued by Ken Spring

K. E. Spring B.Sc.(Hons), MAppSc
22 August 2011

AUGUST, 2011
Qu110802t

Page 1 of 6

QUANTITATIVE X-RAY DIFFRACTION ANALYSES OF TWO SOIL SAMPLES (S11/093, S11/106)

Introduction

Two broadly similar light brownish-grey soil samples were provided by Department of Transport and Main Roads on 28/07/11, and received at Geochempet Service via Petrographic International on 02/08/11 and were sub-sampled for XRD analysis. No further details on the soil samples were provided.

Work Requested

Quantitative X-ray diffraction analysis to determine mineralogical of the soil samples.

Method

A small sub-sample of the two supplied soil samples were investigated by quantitative X-ray diffraction analysis using QUT facilities. The sample was despatched on 02/08/11 and then crushed at QUT. Some of this powder was dispersed in water. The fine fraction (in suspension) was transferred to a low background plate. This preparation is used to extract the fine (clay dominant) fraction as an aid to identifying the clays present in the sample. The ratios of the clays present in the extract may vary from those present in the bulk sample. The clay film specimens were treated with ethylene glycol (which expands some clays) and were re-examined to determine the effect of the treatment. The powder data was collected with a Panalytical vertical diffractometer, using copper radiation.

A portion of the crushed sample was weighed and sufficient internal standard (zinc oxide) was added to give 10 wt% in the analysed sample. This specimen was micronized in a McCrone mill using agate beads and ethanol as a fluid. The ethanol was evaporated overnight in a dry cabinet at 50C. The prepared sample was mounted in a conventional XRD powder holder. A step-scanned diffraction pattern was collected using a Panalytical vertical diffractometer, using copper radiation. The powder X-ray diffraction data was analysed using Jade (V9.0, Materials Data Inc.) for phase identification and SiroQuant (V3.0, Sietronics Pty. Ltd.) for quantitative analysis using a Rietveld based technique that models the diffraction pattern using the available crystal structure models of the phases present to obtain phase abundances. The Rietveld technique normalizes the phase abundances. The known concentration of the internal standard allows the concentration of the other modelled phases to be back-calculated on an absolute scale. The amorphous/unknown concentration is found by difference of the sum of absolute concentrations from 100wt%. The meaning of the amorphous/unknown concentration is open to interpretation – it could represent the non-diffracting component, any unidentified phases and/or any short-coming of the models used for the respective phases modelled. The crystal models used are the ones available that seem closest to the phases in the sample.

Powder XRD diffraction is phase analysis and is not chemical analysis. XRD and chemical analysis results may be able to be resolved if other information is available such as the chemical formula of the phases present or the polymorph (variant of crystal structure). The closest matches in the reference database were used in the modelling, but other members of the respective mineral groups may the present in the samples.

Results

The results indicated are in normalized weight percent and powder and clay XRD patterns are shown in Figures 1 and 2:

	S11/093	S11/106
quartz	42.4%	46.5%
albite	2.2%	2.9%
calcite	0.7%	
gypsum	6.9%	
kaolinite	1.1%	2.1%
mixed layer clay (illite/dominant smectite)	17.1%	32.3%
amorphous matter	29.6% (probably mostly disordered illite/smectite clay)	16.1% (probably mostly disordered illite/smectite clay)

- quantitative X-ray diffraction analysis is difficult because of inherent problems posed by overlapping diffraction peaks, enhancement or suppression of peaks if unintended preferred orientation occurs in the diffraction mount, problems of modelling the best available reference pattern against a mineral which may differ somewhat because of solid solution differences or degrees of ordering which can vary widely in nature
- determining composition by XRD is prone to misidentification of some minerals because of the complex overlap of diffraction patterns. It is quite possible for a minor diffraction peak of one mineral species to occur close to or upon the position of a major peak from a minor mineral.
- combinations of more than three minerals makes identification of the mineral assemblage extremely difficult (ie. prone to misidentification of minerals) because the variable mineralogical compositions model poorly using this technique
- XRD determines composition in weight % and is calculated from a very small sub-sample of the original rock sample.

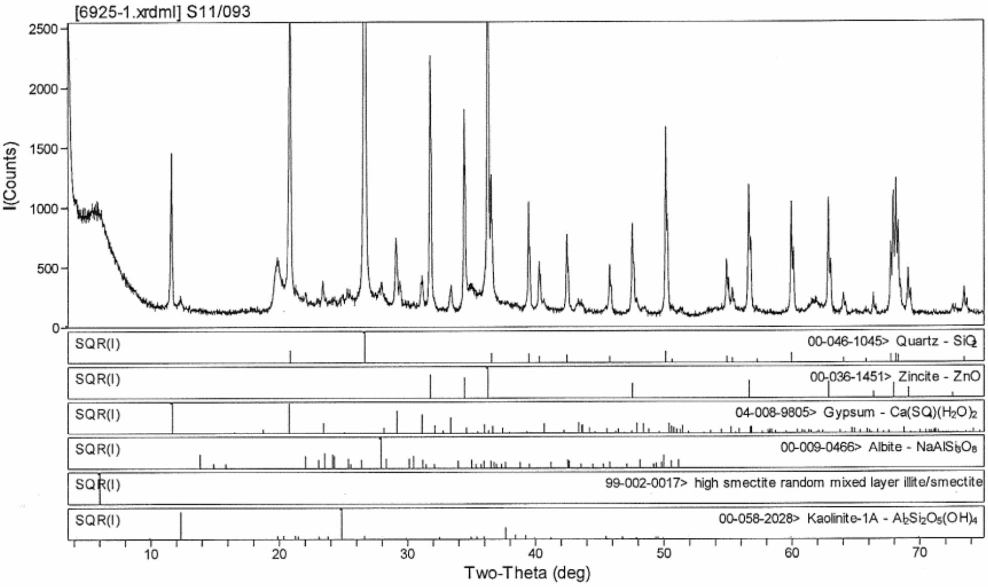
Comments

The supplied soil samples seem to be gravelly.

Components regarded as essentially benign within the supplied samples amount to about 45.3% and 49.4% respectively (quartz, albite and calcite). The calcite (0.7%) is only present in Sample S11/093.

Components regarded as undesirable because they are soft silicate minerals and clays amount to a total of about 54.7% and 50.5% respectively: specifically they comprise smectite dominated mixed layer illite and amorphous content which is suspected to be mostly grossly disordered illite/smectite along with minor unidentified minerals and organic matter. The illite and smectite clays are considered to be the most expansive clays and have a capacity for significant swelling and shrinking upon changes in moisture content. Gypsum (6.9%) is also present in Sample S11/093).

In short, the soil samples will probably have a capacity for significant swelling and shrinking upon changes in moisture content.



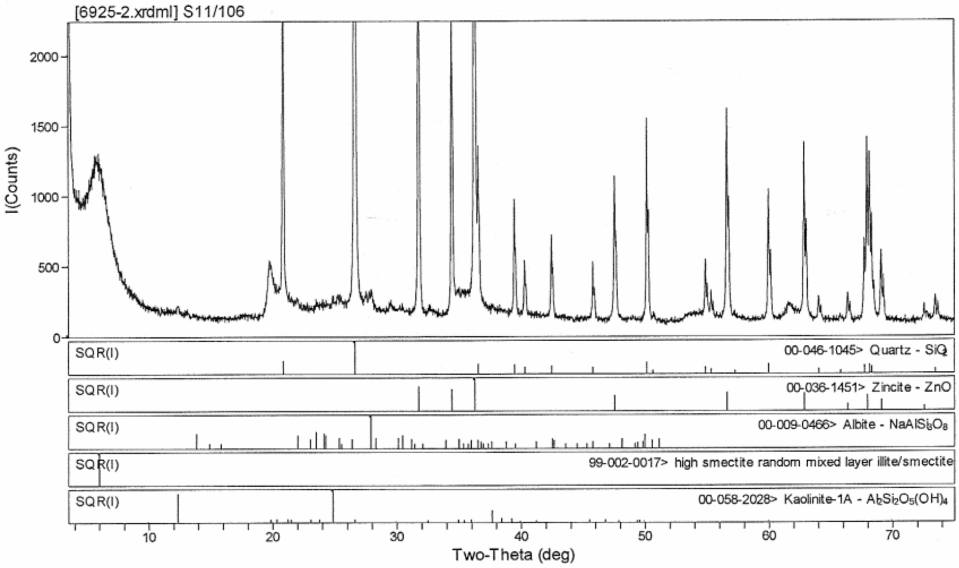
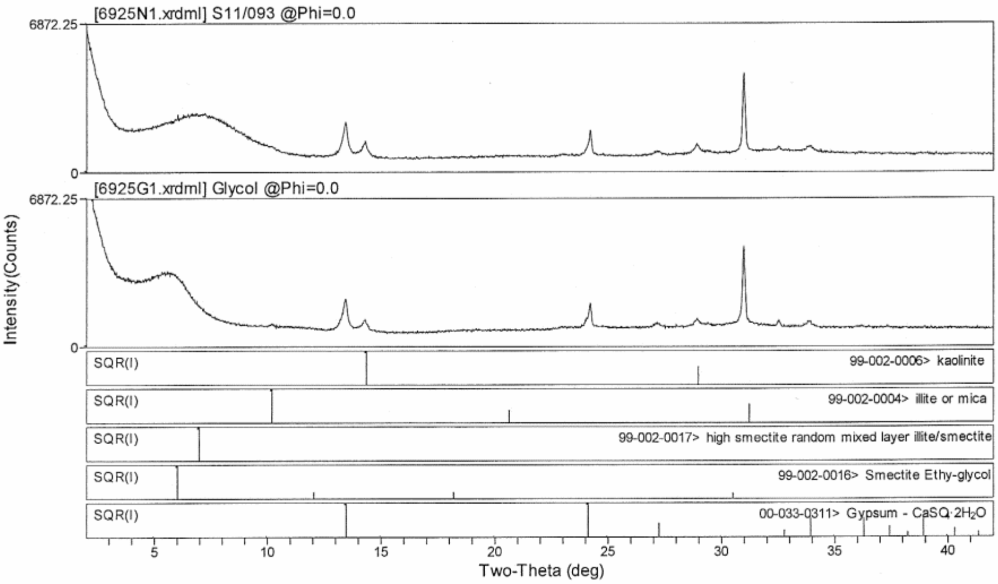


Figure 1. Powder XRD patterns for samples S11/093 and S11/106.



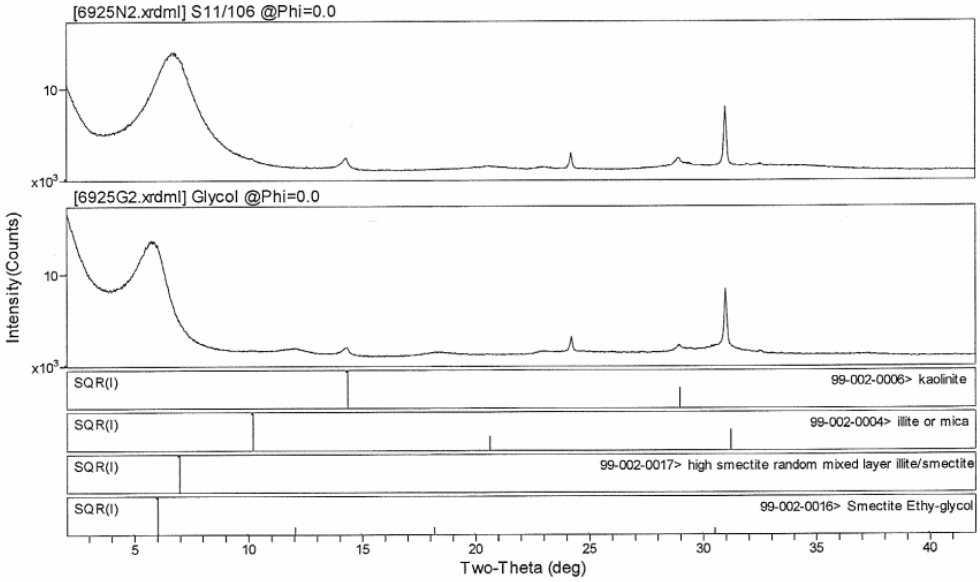


Figure 2. Clay XRD patterns for samples S11/093 and S11/106.