University of Southern Queensland

Faculty of Health, Engineering & Sciences

Engineering Correlations for the Characterisation of Reactive Soil Behaviour for Use in Road Design

A dissertation submitted by

Peter William Reynolds

in fulfillment of the requirements of

Courses ENG4111 and ENG4112 Research Project

towards the degree of

Bachelor of Engineering (Civil)

Submitted: October 2013

Abstract

In the field of road design and construction, expansive or reactive soils are problematic materials. Clay minerals within reactive soils are subject to large volume changes when exposed to water, and conversely when they are exposed to prolonged periods of drying.

The surface movements resulting from the wetting or drying of a reactive soil can cause distress to structures that are founded on them. This creates safety and serviceability issues for road users and high maintenance costs to the road authorities and the community.

Whilst there is a body of data and published information on some of the relationships between certain material parameters, a definitive engineer's guide on the correlations of engineering parameter for expansive soils within Queensland does not appear to exist.

Currently, the primary reference for site classification in respect to the degree of reactivity is the Australian Standard AS2870 – Residential Slabs and Footings. Within this standard, methods are provided to enable an estimation of the range of vertical movement due to swelling and shrinkage. These estimates are based on the Shrink Swell Index (I_{ss}), which is determined by as simple soil test method on an undisturbed soil sample taken from the site of investigation.

In situations where it may be difficult to obtain undisturbed soil samples for shrink swell testing, using an approximation of the relationship between other test parameters and the Shrink Swell Index, it may be possible to determine the site classification and estimate the amount of potential heave or shrinkage, as an alternative to the Shrink Swell Index.

Historical data was gathered from reported site investigations carried out on Queensland state road projects from 1995 to 2012. From this data, the relationships between the measures and indices from some of the most commonly used laboratory methods for characterising reactive soils were examined.

Some useful relationships between various parameters were identified that will assist engineers in simplifying the identification and classification process.

University of Southern Queensland

Faculty of Health, Engineering & Sciences

Limitations of Use

Certification of Dissertation

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

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

PETER REYNOLDS
Q9421608
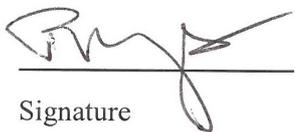
_____
Signature

23-10-2013
_____
Date

Acknowledgments

The author would like to thank the following people for their assistance: to Dr Kazem Ghabraie (University of Southern Queensland) for his technical guidance, feedback and support during this project; to Mr Siva Sivakumar (Department of Transport and Main Roads) for his assistance in the selection of this topic of research and for his permission in allowing access to the data resources of the Department of Transport and Main Roads; to my colleague Mr Jeremy Kirjan, for his encouragement and support; and to my family, for their support and patience in what has been a long wait for the end of this chapter.

Peter Reynolds
University of Southern Queensland
October 2013

Table of Contents

List of Figures

List of Tables

1 Introduction

In the field of road design and construction, expansive or reactive soils are problematic materials. The clay minerals within reactive soils are subject to large volume changes when exposed to water, and conversely when they are exposed to prolonged periods of drying. For some clay minerals, such as montmorillonite, the volume change due to the absorption or removal of water can be as much as 30%. This can result in resulting surface movements of up to (and sometimes greater than) 75mm[1], causing distress to the structures founded on them resulting in safety and serviceability issues for road users and high maintenance costs to the road authorities and the community.

Expansive soils are widely distributed over almost all geographical locations in the world. In Queensland, expansive or reactive soils are referred to by soil scientists as "Cracking Clays" or, more commonly, as "Black Soils" (Dept of Main Roads Qld, 2000). The distribution of these Cracking Clays by land area covers approximately one third of the state. Figure 1 below illustrates the extent of these types of soils within Queensland, based on geological soil mapping.

It is important to properly characterise the properties of expansive soils prior to construction to minimise their impact on the long term performance of infrastructure.

In the analysis and testing of expansive soils, there are various measures and indices used by road designers, and pavement/geotechnical engineers to predict behaviour during the service life of road infrastructure such as pavements, embankments and culverts. Parameters such as CBR Swell, Shrink Swell Index, soil suction, weighted plasticity index, swelling pressure, weighted linear shrinkage, as well as clay content, cation exchange capacity are regularly used.

---

[1] AS2870 - 1996
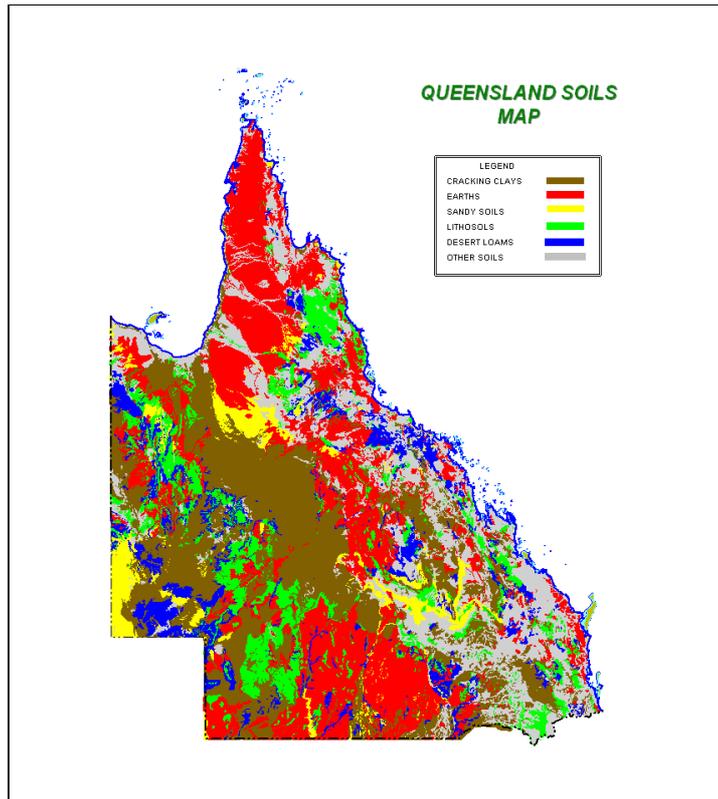
Figure 1 - Distribution of Cracking Clays within Queensland (Main Roads 2000)

Whilst there is a body of data and published information on some of the relationships between certain material parameters, a definitive engineer's guide on the correlations of engineering parameter for Queensland soils and climatic conditions does not appear to exist. Whilst it is acknowledged that to comprehensively characterise the expansive behaviour of all Queensland soil types is beyond the scope of this project, it is hoped that some useful relationships may be identified during the course of the research.

1.1   Aims and Objectives of the Project

The aim of this project is to identify and quantify the relationships that exist between key measures of reactive soil behaviour. The study will focus on the most common methods of laboratory testing used for identifying and characterising reactive soils within Australia for the purposes of the field of road construction. Historical data gathered from investigations and testing undertaken since the mid 1990s, by the Department of Transport and Main Roads, Queensland, will be used to carry out parametric studies. Any linkages or correlations that may exist

between the key indicators of reactive soil behaviour will be identified and quantified for future use as guides in the design of roads using these materials.

The objectives of this project can be summarised as follows:

- To understand the nature of expansive soils and their properties
- To identify by research, the key measures of reactive soil behaviour used by engineers in road construction.
- To investigate and quantify any relationships that may exist between the key measures.
- To establish a ranking of reliability for the key measures of reactive soil behaviour.
- To develop guidelines to assist road designers in the identification and characterisation of reactive soils during the site investigation phase of road construction projects.

2 Impacts on Road Performance

The key elements of a road that are at most risk of damage due to reactive soils are primarily structures on shallow foundations, such as culverts, and pavement layers. The use of reactive soil as fill materials in embankment construction also poses a significant risk for long term performance, if adequate controls are not implemented.

The following section of this report will illustrate by case history, the damaging effects of reactive soil, if the designs do not adequately account for the properties of these materials.

2.1 Distress on Drainage Structures – Case History

During a routine inspection of drainage structures in western Queensland in 2011, the Department of Main Roads Queensland identified major distress to a concrete culvert on the Kennedy Developmental Road, south west of Hughendon, in north western Queensland. The structure had undergone severe cracking to the

abutment, wing and pier walls[2]. Figure 2 below shows the extent of the distress on the culvert.

The distress to the culvert had rendered the structure unsafe, and resulted in its replacement. Actions required to replace the culvert included the construction of a temporary side track, demolition of the existing culvert, removal of the reactive soil to a depth below the zone of influence of the reactive soil, and a replacement of this soil with non-reactive/stable fill. A new culvert base was constructed complete with new concrete culvert cells. It was estimated that the final costs of the replacement works were in the order of $500000.

Based on testing of soil samples taken from the site, it was determined that the cause of the cracking to the structure was due to the stresses imposed on the structure as a result of the shrinking and swelling of the foundation soil as a result of seasonal moisture variations.

Figure 2 - Image of damaged abutment wall of culvert at East Warianna Creek

2.2 Distress on Road Embankment – Case History

Another example of the problems reactive soil can cause to road construction is when they are used as embankment fill. In this example, the approaches to a railway overbridge on the Bruce Highway at Yandina in Queensland, were constructed in 1997-1998[3]. The embankments (maximum height 9m) were

---

[2] MR2469 Department of Transport and Main Roads (2011)

[3] MR1923 Qld Dept. of Main Roads (2001)

constructed from local soil materials sourced from a site near to the railway crossing.

Shortly after the completion of construction, longitudinal cracks were observed on the outside of the edge of the concrete pavement wearing course. These cracks were sealed with polystyrene bitumen, however, the cracking continued. Due to concerns about the possible major damage to the concrete pavement, a site investigation was carried out to determine the cause of the cracking. Samples of the embankment were tested in the laboratory and found to have high shrink swell properties. A review of the construction records found that the embankment fill had not been placed in accordance with the road authority's technical guidelines on construction of expansive clay embankments.

The cause of the cracking was found to be as a result of the shrinkage of the clayey soils within the embankment fill due to a prolonged dry period post construction. It was proposed that the steep embankment batters (1V:1.5H) enabled the drying to occur more rapidly than wound normally be expected. In addition, the flanks of the embankment were not constructed of material suitable for protecting the embankment core from fluctuating moisture contents, due to climatic variations.

The adopted remediation action taken to repair the embankment was to flatten the existing batter slopes using non-expansive soil (1V:2.5H). The estimated total cost of the final repairs to the embankment was approximately $750000.

As the above two case studies illustrate, the cost to the community of underperformance of engineering structures is significant. It is important to properly identify and characterize reactive soil sites and to carefully quantify their potential for shrink-swell movement.

Figure 3 - Location of distressed embankments on Bruce Highway, Yandina

3 Background and Literature Review

3.1 Properties of Reactive Soils

Expansiveness or reactivity is a property of a soil to undergo large volume changes when subject to the actions of wetting and drying. Consideration of the mechanism of interaction between water and reactive soils show that the three most important components are mineralogy, the change in moisture content or suction due to climatic conditions (atmospheric changes), and the stress applied to the soil.

Fine grained soils (eg clayey soils) are known to have a greater potential for reactivity than coarse grained soils (eg soils and gravels). For fine grained soils, the type and proportion and type of clay mineral present influences the potential for reactivity. For example, soils with a high proportion of clay mineral montmorillonite are known to exhibit the very high amount of reactive behaviour.

Another factor which influences expansive behaviour is the potential for change in the moisture content of a soil, known as soil suction. Moisture conditions beneath the ground surface are described by the actual moisture content and the water pressure within the void space; the pore water pressure. Where there is a deficit of water within the soil, this usually gives rise to a negative pore water pressure,

6

known as soil suction. The drier the soil, the greater the suction, and potential for absorption of moisture. [4]

The depth within a soil at which a stable or equilibrium water content can be found is known as the Active Depth or Zone of Seasonal Fluctuations. Soil above the Active Depth is subject to changes in water content and soil suctions, due to climatic conditions; specifically the seasonal cycles of wetting and drying, which influence the degree of reactivity of the soil.

In addition to these intrinsic properties, the magnitude of the stresses applied to the soil by overburden or, for example, embankment loading, also governs the degree to which the soil can react. Problems of volumetric movement and swelling pressure on elements of road infrastructure only arise when the expansiveness is unable to be suppressed.

3.2 Mineralogy and Mechanics of Volume Change

The type of clay mineral is largely responsible for determining the intrinsic expansiveness of the soil. Kaolinitic clays are relatively non-expansive whilst the more expansive clays are smectite clays, also known as montmorillonite clays. The formula for the chemical composition of montmorillonite is $Al_4Si_8O_{20}(OH)_4.nH_2O$

The structure of the clay particles consists mainly of three (3) layers, octahedral sheet, usually occupied by aluminium or magnesium (gibbsite) sheets sandwiched between two sheets of tetrahedral silicon (silica) sheets to give a two (2) to one (1) lattice structure[5]. This network typically allows water to enter to the centre of the clay particle and be retained for long periods of time (Figure 4). The swelling occurs due to the poor electrostatic bonds between the silica sheets, enabling

---

[4] Nelson et al. (1992)

[5] Chen (1988)

osmotic pressures to build up and allow for water molecules to be absorbed between the sheets[6].
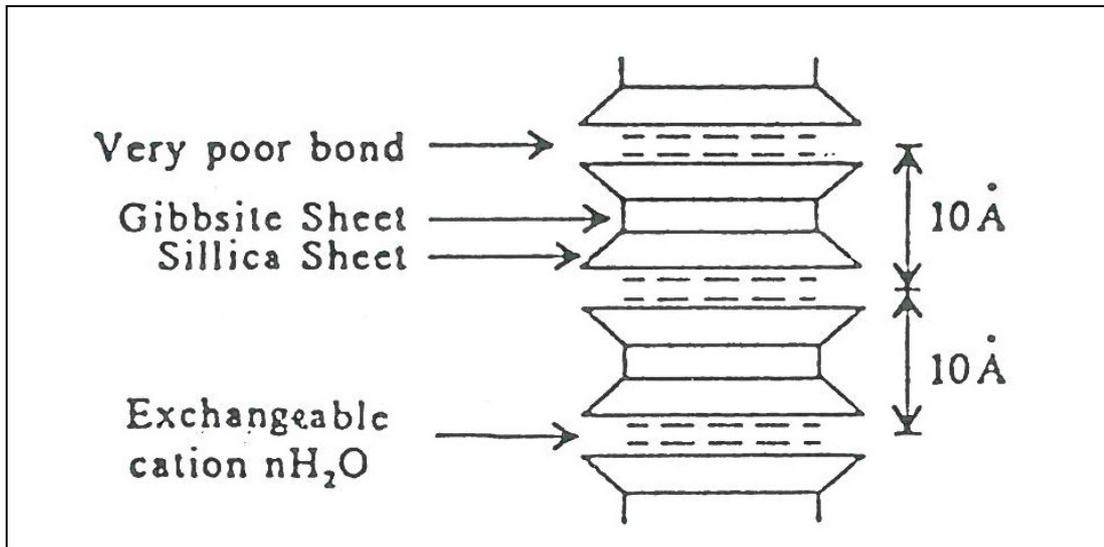
Figure 4 - Diagram of the structure of a montmorillonite layer (Al-Omari 2000)

By comparison, kaolinite, which exhibits little expansive properties, comprises a one to one lattice structure Figure 5 and has a much lower affinity to absorb or retain water molecules within its lattice.
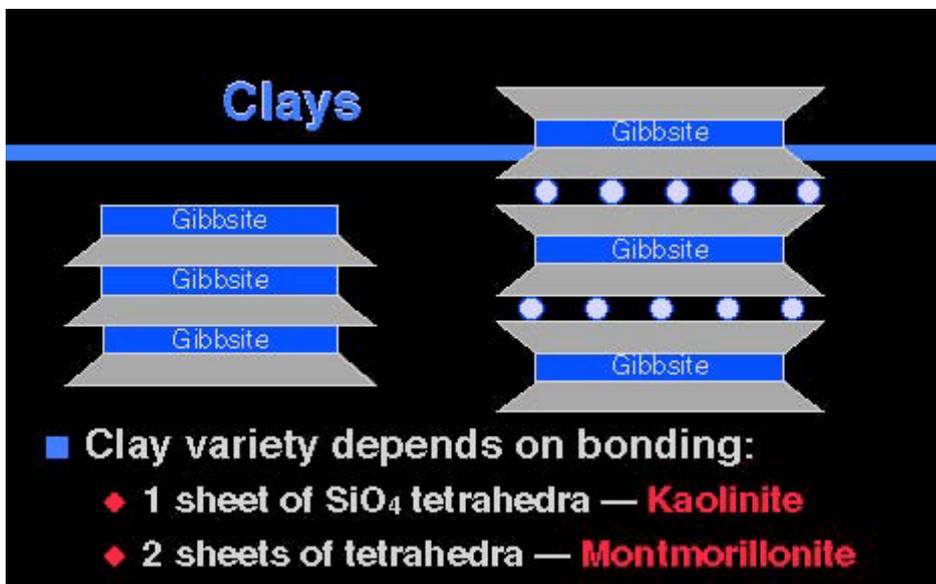
Figure 5 - Comparison of mineralogical structures of kaolinite and montmorillonite (Farris)

---

[6] Al-Omari et al (2000)

This retention is responsible for the movement associated with expansive materials in road construction.

3.3 Soil Suction

A volume of soil is comprised of solid particles and void space. The void space can be filled with aid and water. When the void space of a soil is completed filled with water, it is said to be saturated. Conversely, when the void space is only partly filled with water, it is said to be in an unsaturated state.

The mechanics of the way in which the soil will behave in the unsaturated state, is governed by the inter-particle forces, the air and water pressures within the void space, and the surface tension arising from the interactions of the water and air within the voids. Figure 6 below shows a phase diagram of an unsaturated soil.
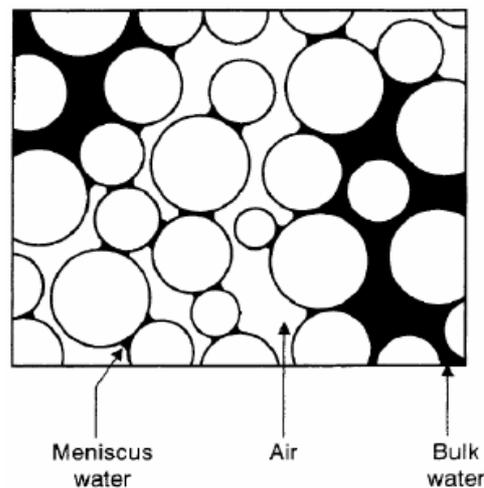
Figure 6 - Phase diagram showing soil particles and water in void space

The retention and movement of water in soils and its loss to the atmosphere is best described in terms of energy. The more strongly water is held by a soil, the greater is the energy required for it to be released. As such, if water is to be removed from a moist soil, work has to be done against adsorptive forces; the forces that cause the adhesion of a thin layer of water to the surface of soil particles. Conversely, when water is adsorbed by the soil, a negative amount of work is done. The potential energy required to cause the movement of soil water from a zone where

the free energy of water is high (eg standing water table) to one where the free energy is low (a dry soil), is referred to the total soil water potential.

The key components that comprise the total soil water potential are the Matric potential, the Osmotic potential, the potential due to Gravitational forces, and the Pressure potential, due to applied pressures or stresses on a soil.[7]

The Matric potential is the energy associated with the attraction of water by the particles within the soil; Osmotic or solute potential describes the energy associated with the attractive forces within a soil due to water containing salts. Gravitational potential relates to the energy associate with water at an elevation, and pressure potential describes the energy due to any applied stresses to the soil.

Figure 7 below shows graphically the soil water energy states.
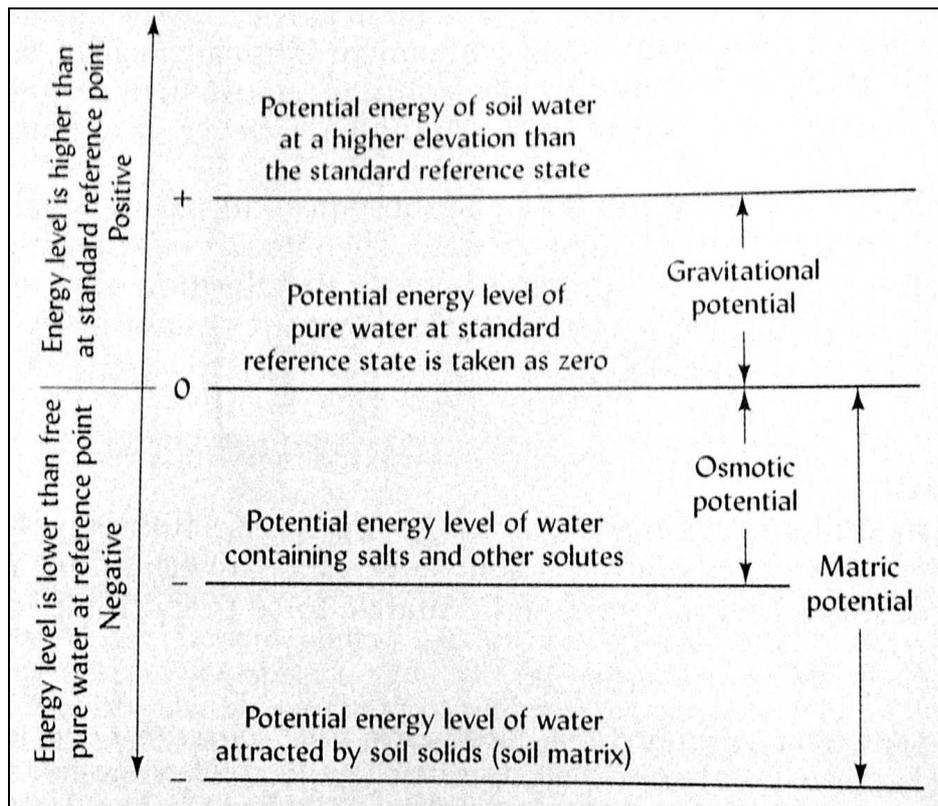
Figure 7 - Diagram of soil water potential energy states (Or et al 2005)

The total soil water potential can be expressed in the following equation:

---

[7] Or, Tuller and Wraith (2003)

$$\varphi_T = \varphi_M + \varphi_s + \varphi_P + \varphi_Z \quad \text{(Or, Tuller and Wraith)}$$

Where

φ_T = The total soil water potential, in centimetres

φ_M = The matric potential, in centimetres

φ_s = The osmotic (solute) potential, in centimetres

φ_P = The pressure potential, in centimetres

φ_Z = The gravitational potential, in centimetres

In practical terms, the primary forces acting on soil water held within a rigid soil matrix are: (i) matric forces resulting from interactions of the solid phase with the liquid and gaseous phases (matric potential) and (ii) osmotic forces (solute potential) owing to differences in chemical composition of soil solution.

As stated above, the matric potential is caused by a difference between the air and water pressures within the pore space within the soil.

This is expressed as the simplified equation:

$$\varphi_M = (u_a - u_w) \quad \text{(Or, Tuller and Wraith)}$$

Where

u_a = pore air pressure, in centimetres

u_w = pore water pressure, in centimetres

Generally, soil water potential is referred to as total suction terms, and is expressed as:

$$\varphi_T = (u_a - u_w) + \varphi_S \quad \text{(Or, Tuller and Wraith)}$$

Methods of expressing suctions - there are two units to express differences in energy levels of soil water:

pF Scale: The soil water potential (total suction) is measured in terms of the height of a column of water required to produce necessary suction or pressure difference at a particular soil moisture level. The normal range of the values of suction is wide. Suction is usually expressed as a logarithm of the height of water column (cm) to give the necessary suction, in units of picoFarads (pF); ie

$$pF\ index = \log_{10} \varphi_T \text{ (Or, Tuller and Wraith)}$$

Lopes and McManus (1996) published indicative values relating to specific soil states and are presented in Table 1 below.

Atmospheres or Bars: It is another common mean of expressing suction. One atmosphere (one bar) is the average air pressure at sea level. If the suction is very low as occurs in the case of a wet soil containing the maximum amount of water that it can hold, the pressure difference is of the order of about 0.01 atmospheres or 1 pF equivalent to a column of water 10 cm in height. Similarly, if the pressure difference is 0.1 atmosphere the pF will be 20. Soil moisture constants can be expressed in term of pF values. A soil that is saturated with water has PF 0 while an oven dry soil has a pF 7.

Suction (pF)	Soil State
6.5-7.0	Oven Dry
6.0	Air Dry
5.5	Shrinkage Limit
4.5	Wilting Point (Plants)
2.0-1.5	Swell Limit
1.0	Liquid Limit

Table 1 - Suction values corresponding to certain soil states (Lopez et al 1996)

3.4 Active Depth

For the problematic behaviour of reactive soils to manifest itself, the soil must be subject to prolonged periods of wetting or drying. If the moisture content within a soil is constant, there is stability. Conversely, where the strata are subject to seasonal fluctuations of soil moisture, the volumetric changes are observable.

Figure 8 shows a number of idealised water content profiles for a problematic reactive clay soil. Profile A demonstrates a water content profile in a uniform soil at an undeveloped site in a dry climate. Below some depth (Depth Z_s) an equilibrium water content exists. This is referred to as the Active Zone.
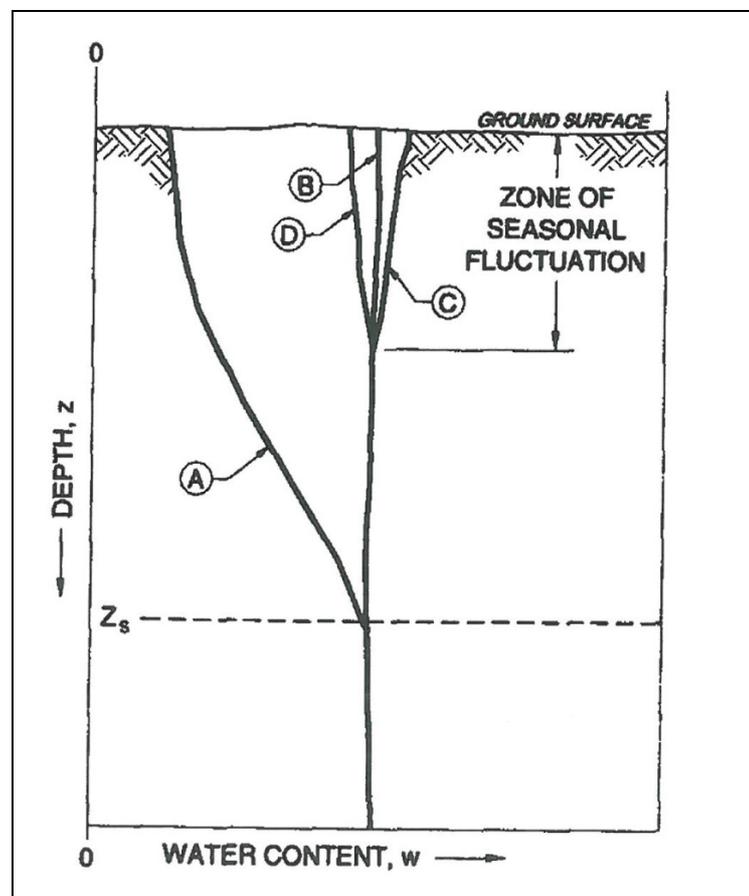
Figure 8 - Idealised water content profiles within active depth zone (Nelson et al 2001)

Above Depth Z_s, the water content decreases due to water losses from the ground surface, usually evaporation and also transpiration from vegetation. If a cover is placed on the ground surface that is large enough that edge effects can be neglected, surface water losses are eliminated, and the water content profile will come into equilibrium with the environment as shown, for example, by Profile B.

13

If the ground surface is subjected to temperature fluctuations such as due to summer and winter climates, the water contents in the zone affected by temperature changes will fluctuate about Profile B. Profile C would be typical of wet climatic conditions and Profile D would be typical for dry climatic conditions. The zone in which temperature effects occur, and depths below that in which climatic effects can change the water content define the zone of seasonal fluctuation. The depth of this zone would be less than or equal to the depth Z_s (Nelson et al. 2001).

3.5 Applied Stress

The applied stress depends on the structural design and the embankment geometry. The embankment material will act to suppress vertical movement. The amount of overburden required to suppress movement depends on the expansiveness of the soil, and the active depth, as described above.

4 Laboratory Methods for Measuring Reactivity

4.1 Laboratory Measurements

Below is a summary of the most commonly used laboratory methods in the determination of soil reactivity for roads (Pritchard et al. 2000)

- California Bearing Ratio - % Swell
- Particle Size Distribution using Hydrometer (% Finer that 2 um)
- Atterberg Limits (Liquid Limit, Plasticity Index, Linear Shrinkage)
- Weighted Plasticity Index
- Weighted Linear Shrinkage
- Shrink Swell Index
- Loaded Swell/Swelling Pressure
- Soil Suction – Filter Paper Method
- Cation Exchange Capacity

California Bearing Ratio (CBR) - % Swell

CBR tests were developed in 1952 to assess the strength of pavement subgrades (Lacey, 1998). California Bearing Ratio is defined as the ratio of force required to cause a circular plunger of 1932mm^2 area to penetrate the material for a specified distance expressed as a percentage of a standard force (Main Roads 2002). Test specimens are prepared from passing 19.0mm material using a compactive effort of 596 kJ/m^3. They are then tested either in a soaked or unsoaked condition.

The method allows for the determination of CBR Maximum Dry Density (MDD) and CBR Optimum Moisture Content (OMC) as well as the optional determination of swell and post penetration moisture content (Main Roads 2002).

This test can be used to simulate climatic conditions and estimate the potential swell behaviour of expansive clays in its natural state (Fox 2002).

The part of the test that is primarily used as a measure of reactivity is the percentage of swelling measured at the completion of the 4 day soaking period. This is performed by using a dial gauge mounted on a frame which, when placed on top of the CBR mould, aligns the dial gauge centrally over a measuring point. This measuring point is placed on top of the compacted sample, prior to the mould and soil being placed in the water bath for soaking.

An initial reading is taking on the dial gauge on the unsoaked compacted soil. At the end of the soaking period (generally 4 days, but for some tests, 10 days), another reading is taken at the measuring point on the soaked soil. The percentage of swelling is calculated as the ratio of the difference in vertical displacements between the dry and the soaked condition and the compacted height of the soil sample (gauge length).

Figure 9 - CBR test specimens with swell measurement apparatus (Walters 2008)

Particle Size Distribution using Hydrometer

Fine grained soils such as silts and clays and have particles smaller than 0.060mm. To determine the grain size distribution of a material <0.075mm sieve the Hydrometer method is commonly used. This test is important in determining the amount of clay present in a soil structure, therefore detailing the amount of plasticity and potential swell the material possesses.

Soil is mixed with water and a dispersing agent, stirred vigorously, and allowed to settle to the bottom of a measuring cylinder. As the soil particles settle out of suspension the specific gravity of the mixture reduces.

An hydrometer is then used to record the variation of specific gravity with time. By relating Stoke's Law, velocity of a free falling sphere to its diameter the test data is reduced to provide particle diameters and the percent weight of the sample finer than a particular particle size.

The hydrometer test is usually discontinued when the percentage of clay sized particles has been determined.

Figure 10 - Hydrometer Testing (Earl 2005)

Atterberg Limits

In the year 1911 Atterberg proposed the limits (liquid limit LL , plastic limit *PL* and shrinkage limit *SL*) of consistency in an effort to classify the soils and understand the correlation between the limits and engineering properties like compressibility, shear strength and permeability (Casagrande, 1932).

The limits represent the water holding capacity at different states of consistency. The limits are the more prominent procedures for gathering information on the expansive nature and mechanical behavior of clay soils (Williams, A, 1958).

Figure 6 shows a diagram of the different states of soil consistency with increasing water content, including the Atterberg Limits.
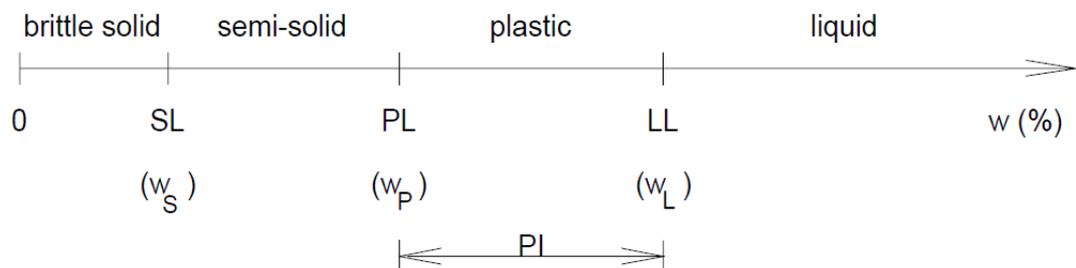
Figure 11 - Soil consistency with increasing water content (Sivakugan 2000)

A useful set of classification data for identifying the swell potential of expansive subgrades are the liquid limit (LL) and plasticity index (PI). The liquid limit is the water content at which a soil changes from the liquid state to a plastic state while the plastic limit is the water content at which a soil changes from the plastic state to a semisolid.

The **Plasticity Index** is derived from the plastic limit and liquid limit and is represented by the equation below:

$$PI = LL - PL \qquad (\%)\ (Head)$$

Where:

PI = Plasticity index   *(%)*

LL = Liquid Limit   *(%)*

PL = Plastic Limit   *(%)*

Liquid Limit (LL)

There are two methods to describe the liquid limit (LL) namely percussion cup method and fall cone method. In the percussion cup method, liquid limit is defined as the moisture content corresponding to a specified number of blows required to close a specified width of groove for a specified length (Casagrande, 1932 and 1958). In Queensland, this test is generally carried in accordance with Australian Standard AS1289.3.1.1

The principle of the determination of the liquid limit from the fall cone method is that the liquid limit corresponds to a value of the depth of soil penetration due to a steel cone of specified mass and dimensions.

It is important to note that this depth of penetration value will vary, depending on the test method used for the determination. For example, the Department of Transport and Main Roads Queensland method Q104A, specifies that the cone penetration at liquid limit is a value of 15.5mm. Alternatively, the penetration at liquid limit for the Australian Standards AS1289.3.9.1 is 20mm.

The liquid limit data considered in this study has been determined by both Casagande and fall cone methods, using Australian Standard and Transport and Main Roads methods.

Figure 12 - Apparatus for Casagrande liquid test (Earl 2010)

Figure 13 - Apparatus for fall cone liquid limit test (TMR 2013)

Plastic Limit (PL)

Plastic limit is the water content at which the soil begins to crumble when rolled into 3 mm threads. This is the minimum water content at which the soil will deform plastically. This gives an accurate representation of the minimum moisture content a fine grained soil can contain before deformation occurs. This test can be determined by Australian Standard and Transport and Main Roads methods (AS1289.3.2.1 and Q105).

Figure 14 - Plastic limit test specimens adjacent to 3mm diameter guide rod (Earl 2010)

Shrinkage limit (SL) – Linear Shrinkage Test

The shrinkage limit is the water content dividing the semi-solid and the solid state of the soil. It is the water content at which further reduction in moisture content does not result into a decrease in volume of the soil mass. This test can be determined by Australian Standard and Transport and Main Roads methods (AS1289.3.4.1 and Q106). A sample of fine-grained soil, at approximately the liquid limit of the soil, is placed in a shallow trough-shaped mould of 150mm in length, 25mm wide and 15mm deep. The soil and mould are placed in a low drying temperature oven for 2 hours and then placed in a high temperature drying oven overnight.

The length of dried specimen is measured and the linear shrinkage is calculated as the ratio of the change in length due to drying and the original length.

Figure 15 - Soil specimen in linear shrinkage test mould (Main Roads 2008)

Loaded Swell/ Swelling Pressure Test

The loaded swell test provides an indication of the moisture content, soil strength and surface pressure required to suppress swell of a fully saturated soil. This test is performed on an undisturbed sample of soil in its natural (insitu) state.

The soil is trimmed in a steel ring so as to be confined laterally. It is placed in an oedometer apparatus and inundated with water. It is allowed to swell vertically until it achieves a constant value. This completes the Free Swell test, and from this, the percentage of swelling strain observed from the insitu moisture content to the saturated soil condition can be calculated.

Once swell is complete, a series of vertical pressures are applied to the specimen, consolidating it until it reaches the original height of the specimen. A graph of Voids Ratio (%) and. Pressure (kPa) can be plotted and from this, an estimation of the loading required to suppress expansion can be found. This loading value (in kPa) can then be related to a measurement of soil overburden. All loaded swell data used in this study has been determined using ASTM test method D4546 - 03.

Shrink Swell Index (Iss)

The shrink swell test is composed of companion core shrinkage and swelling tests, carried out on undisturbed soil samples from their initial field moisture contents. The vertically oriented sample is usually obtained from the ground, using a 50-mm-diameter thin-walled tube. The sample is extruded from the tube as a soil core and a suitable portion of the sample is selected for the preparation of a shrinkage core (70–100 mm long) and a swell core (20–25 mm long). Test samples must be from adjacent portions of the core to ensure that water content and both compositional and structural differences are minimized. The test is carried out in accordance with Australian Standard AS1289.7.1.1.

The shrinkage and swell tests are then conducted simultaneously, as follows.

<u>Shrinkage Test</u>—This component of the test is identical in procedure to the core shrinkage test, although fewer measurements are required as the shrink-swell index is based on the oven-dried state. A shrinkage core, 45–50 mm in diameter and a length of 1.5–2 diameters, is trimmed from the soil sample. Where possible, it is selected and trimmed to be free of major structural defects and loose material. Initial dimensions and mass are recorded. Small pins are added to each end as reference points to facilitate consistent measurements of sample length as drying proceeds.

The shrinkage core is firstly air-dried. Regular measurements of length and mass are taken until shrinkage ceases. The core is then oven-dried to a constant mass at 105–110∘C and final mass and dimensions are recorded. The data recorded enables initial and final water contents, axial strain to be calculated, and a graph of axial strain against water content to be plotted.

Throughout the drying process, the core is kept in a shallow tray, so that any crumbs that become loose during the test are not lost, as this would affect moisture content calculations.

<u>Swelling Test</u>—this involves a simplified oedometer test in which the sample (of measured mass) is installed in a steel ring, (of measured volume; usually around 20 mm high and 50 mm in diameter) and placed in a consolidation apparatus. A gauge to monitor the sample height is then zeroed under a nominal seating pressure of 5 kPa. A load of 25 kPa or the estimated in situ overburden pressure (whichever is greater) is then applied for 30 minutes to record any initial settlement or seating adjustment. This displacement is used to correct the initial sample height for determination of swelling strain.

After re-zeroing the displacement gage, the sample is inundated with distilled water and allowed to swell until the swelling increment, in a period of not less than 3 h duration, is not more than 5 % of the total recorded swell.

The initial water content is determined from the sample trimmings, and the final water content is measured from the extracted sample at the end of the test. In the sample preparation process, particular care is taken to ensure that the sample neatly fills the sample ring, as voids and recompacted or remolded portions will accommodate internal adjustments in the volume of the sample and hence, affect the realized vertical swell. Shrinkage strains and swell strains, measured in the respective tests, are then combined to give a shrink-swell index (I_{ss}). [8] This is given by the following equation, (as per AS1289.7.1.1):

$$I_{ss} = \frac{\in_{sh} + \dfrac{\in_{sw}}{2}}{1.8} \qquad in \ \%/pF$$

Where

$\in_{sh} = shrinkage\ strain\ in\ \%$

$\in_{sw} = swelling\ strain\ in\ \%$

The factor of 1.8 is a constant that is adopted for all Australian soils. This relates to the linearly varying part of soil suction changes corresponding to the changes in the axial strains during shrinking and swelling of the soil.

Figure 16 - Core shrinkage specimen

---

[8] Fitkus et al (2009)

Figure 17 - Apparatus (Oedometer) for swell tests

Soil Suction – Filter Paper Method

This method uses laboratory grade filter paper to measure the matrix suction properties of an undisturbed soil. The filter paper is initially calibrated for suction at different moisture contents.

Once the calibration curve is completed, the test specimen of soil is carefully dissected into a minimum of three layers. Three discs of filter paper (pre-weighed using a balance with an accuracy to 0.0001g) are sandwiched between the soil slices. The sample is tightly wrapped in plastic, ensuring the paper discs are fully in contact with soil only. The soil and filter paper is stored in a sealed container in a controlled temperature environment for 7 days. At the completion of the test duration, the filter papers are individually removed from between the soil slices and their wet mass is measured. The papers are then dried to a constant mass in a drying oven.

The moisture content for each filter paper is then determined, and the average of all three results is calculated. Using the calibration curve for the specified grade of filter paper, the soil suction value is determined from the average filter paper moisture content. The soil suction data used for this study has been determined using BRE Information Paper Method IP 4-93.[9]

---

[9] Crilly and Chandler (1998)
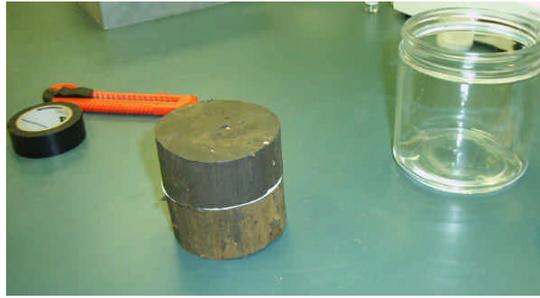
Figure 18 - Image showing filter paper and soil specimen for suction test (Bulut 2001)

Cation Exchange Capacity

This test method describes the procedures for measuring the soluble and bound cations as well as the cation exchange capacity (CEC) of fine-grained inorganic soils.

Clay minerals in fine-grained soils carry a negative surface charge that is balanced by bound cations near the mineral surface. These bound cations can be exchanged by other cations in the pore water, which are referred to as soluble cations. The cation exchange capacity is a measure of the negative surface charge on the mineral surface. The CEC generally is satisfied by calcium (Ca), sodium (Na), magnesium (Mg), and potassium (K), although other cations may be present depending on the environment in which the soil exists. This test method was developed from concepts described previously in Lavkulich (1981) and Rhoades (1982) In soils with appreciable gypsum or calcite, dissolution of these minerals will release Ca in solution that may affect the measurement.

In this test method, the soluble salts from the mineral surface are washed off with de-ionized water and then the concentration of soluble salts within the extract is measured. The bound cations of the clay are measured by using a solution containing an index ion that forces the existing cations in the bound layer into solution. The total concentrations of bound and soluble cations in this solution are measured.

The CEC is measured by displacing the index ion with another salt solution and measuring the amount of the displaced index ion.

It was found during the collection of the Cation Exchange Capacity data that the suppliers of the testing did not use methodology from well recognized suppliers of standards. Instead it was found that the suppliers used in-house methods based on conventional chemical analysis techniques.

5   Predictions of Surface Movement using AS2870

5.1   General

The Australian Standard AS2870 uses the following equation to predict the characteristic surface movement that a particular layer of soil may produce under seasonal moisture variations:

$$y_s = \frac{1}{100} \sum_{n=1}^{N} (I_{pt}.\overline{\Delta u}h)_n$$

Where

y_s = characteristic surface movement, in millimeters

I_{pt} = instability index in % picoFarads (pF)

$\overline{\Delta u}$ = soil suction change averaged over the thickness of the layer under consideration, in picoFarads (pF)

h = thickness of layer under consideration, in millimeters

N = number of soil layers within the design depth of suction change[10]

The key parameters within this equation are:
- the instability index (I_{pt})
- the soil suction change ($\overline{\Delta u}$)
- the depth of the clay layer (h)

A point worth noting in reference to the principle behind the determination of the reactivity for a site within then Residential Slabs and Footings Construction standard AS2870, was made by Brown et al. (2002). Specifically, it was observed that AS2870 does not refer to standards tests such as Atterberg limits or linear

---

[10] AS2870 Section 2.3(2011)

shrinkage to determine reactivity. Instead surface movement (y_s) is the primary characteristic used to classify reactive soils. As detailed above, this incorporates the use of soil suction data and the instability index derived from shrink-swell tests into the calculation of surface movement.[11]

Caunce (2010) observed that the AASHTO guidelines for assessing expansive soils utilise LL, PI and soil suction, whereas Brown (2002) quotes the Australian Standard, AS2870, stating that there are no clear tests to determine clay reactivity. In addition, Brown also postulates that movement is simply a function of mineralogy, proportion of clay, moisture change, loading and lateral restraint.

As such, from these statements and observations above, it can be concluded that the instability index accounts for a number of different properties relating to the reactivity of a soil, within the one single parameter.

5.2 Instability Index

As discussed above, the instability index (I_{pt}), in basic terms is a measure of the soil reactivity. This index value is a function of a number of factors including matric and osmotic suction and the stress state of the soil, and the moisture history[12]. This property can be best approximated by the Shrink Swell Index (I_{ss}).

Fityus (2005) reported on the results of a field trial by Cameron in 1989, from a site in South Australia – Gilles Plains. The mean observations of the ground surface movements taken over one wet and one dry cycle were compared against the predicted surface movements based on laboratory values of the shrink swell index of soil taken from the site. The study found that the predicted movements calculated using the shrink swell index were a good estimated of actual ground surface movement. Figure 19 below illustrates these findings.

---

[11] Caunce (2010)

[12] Cameron (1989)
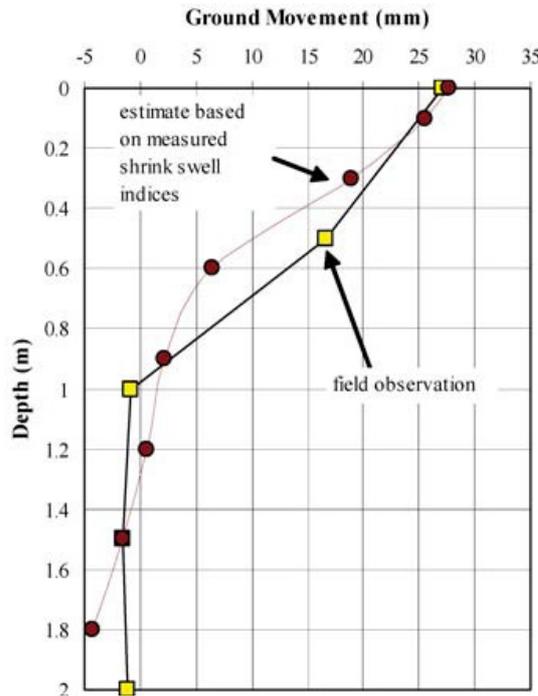
Figure 19 - Predicted surface movement compared to observed surface movement (Fityus 2005)

5.3 Changes in Soil Suction

Where there is a deficit of water within the soil, this usually gives rise to a negative pore water pressure, known as soil suction. Essentially, the drier the soil, the greater the suction, and potential for absorption of moisture.

Soil suction is a property of soil which is a function of the degree of saturation of the soil and the size of the void spaces within the soil structure. For fine grained soils such as clays, the void space within the soil structure is relatively small compared to that of a granular soil. As a result, when a clayey soil is partially saturated, it is difficult for water to escape from the void space.

5.4 Previous Assessments of Test Parameters with Shrink Swell Index

Earl (2005) reported on research into the relationships between the shrink swell index, Atterberg limits and linear shrinkage. Earl found a poor correlation existed between the shrink-swell index and linear shrinkage, from analysis of test data on clays from the Shepparton Formation in northern Victoria. However, Earl reported reasonably good correlations between the shrink-swell index and the plasticity

28

index weighted by the percentage clay fraction, and similarly with the shrink-swell index and the plasticity index weighted by the percentage clay fraction.

It should be noted that Earl used the a value of the coefficient of determination (R^2) > 0.8 as his criterion for determining if the strength of the relationships examined in his work would be useful for the purposes of estimation. Whilst this is considered a reasonable approach, this study has adopted the combined criteria of a coefficient of correlation (r) > 0.7 and a coefficient of determination of (R^2) > 0.5 for determining whether a relationship between parameters would be valid in the estimation of the shrink swell index/instability index.

Earl also referenced work by Cameron (1989) which identified a correlation between shrink swell index and linear shrinkage, with a coefficient of correlation of r= 0.76, for a broad range of soils.

In a similar study, Wan et al. (2002) reported a strong correlation between the shrink swell of a soil and liquid limit was reported for volcanic soils from Honolulu.

In research conducted by Earl (2005) a poor correlation was between the shrink-swell index and LS, however, reasonably good correlations were shown between the shrink-swell index and the PI factored by the clay fraction and similarly with the shrink-swell and the PL factored by the clay fraction.

6 Analysis of Historical Data

6.1 General

Based on a search of public records from Department of Transport and Main Roads Queensland, laboratory test data were extracted from geotechnical reports on site investigations carried out between 1992 and 2012. The geotechnical reports were filtered for their relevance to sites that were identified as having reactive soil. Data from a total of 108 technical reports was reviewed. From these reports a total of 33 reports were found to have relevant laboratory test

information. The data from these specific reports were extracted and compiled for the purposes of this research.

The reports detailed investigations throughout Queensland's state road network, ranging from locations from South East Queensland, to the Darling Downs, Central Queensland and North Western Queensland. A map of the sites and the relevant reports to which they relate is shown in Figure 20.

6.2 Method of Analysis

The objective of the analysis of the historical laboratory data is to determine if there are any relationships between the various parameters obtained from different laboratory techniques. As previously discussed, the basis for the classification of reactive sites by characteristic movement, as per AS2870, is the instability index (I_{pt}). This is best approximated by the Shrink Swell Index, (I_{ss}). Using the I_{ss} as the independent variable of the relationship, each test parameter was plotted in a scatter diagram. Using a linear regression analysis of the data, the coefficient of correlation (r), and the coefficient of determination (R^2) were determined for each set of data. These statistical parameters were used to determine (a) the strength of the relationship between (I_{ss}) and the other variables, and (b) measure of how well the calculated regression line represents the data. Table 2 below gives some guidance on the interpretation of strength of the relationships.

Figure 20 - Site locations referenced by TMR technical reports

Using the criteria for strength of relationship from Crewson above, the corresponding values of the coefficient of determination (R^2) are presented in Table 3.

This analysis was performed using the shrink swell index as the dependent variable and the other parameters were treated as independent variables.

Range of Coefficient of Correlation (r) Values	Strength of Relationship
0.9 to 1.0 (or -0.9 to -1.0)	Very High
0.7 to 0.9 (or -0.7 to -0.9)	High
0.5 to 0.7 (or -0.5 to -0.7)	Moderate
0.3 to 0.5 (or -0.3 to -0.5)	Low
0 to 0.3 (0 to -0.3)	None

Table 2 - Guidelines for interpreting strength of relationships (Crewson 2006)

Range of Coefficient of Correlation (r) Values	Strength of Relationship	Range of Coefficient of Determination (R^2) Values
0.9 to 1.0 (or -0.9 to -1.0)	Very High	0.81 to 1.0
0.7 to 0.9 (or -0.7 to -0.9)	High	0.49 to 0.81
0.5 to 0.7 (or -0.5 to -0.7)	Moderate	0.25 to 0.49
0.3 to 0.5 (or -0.3 to -0.5)	Low	0.09 to 0.25
0 to 0.3 (0 to -0.3)	None	0 to 0.09

Table 3 - Modified guidelines for interpreting strength of relationships (including R^2)

6.3   Sample Population Statistics

Table 4 below summarizes the size and distribution of the sample populations of data for each test parameter used in this study.

	<0.075mm	<0.002mm	LL (%)	PI (%)	LS (%)	WPI	WLS	Suction - Filter Paper (kPa)	Swelling Pressure (kPa)	Iss %/pF	CBR Swell - 4 Day Soak (%)	CBR Swell - 10 Day Soak (%)	CEC (mEq/100g)
N	183	55	209	208	209	209	188	29	21	100	56	13	10
MEAN	77.8	56.6	67.7	40.7	17.5	3710.7	1604.6	1776.7	321.1	3.0	3.8	4.5	39.2
MEDIAN	82.0	57.0	65.8	38.4	17.4	3443.2	1601.0	2008.0	140.0	2.7	3.3	4.0	28.5
SD	17.3	14.8	23.4	17.5	5.4	1808.5	600.9	937.5	371.2	2.0	3.0	3.0	23.3
CV (%)	22	26	35	43	31	49	37	53	116	68	79	66	59
MIN	16	16.6	19.6	4	2.6	346	225	4	49	0.1	-1.9	0.5	19
MAX	100	88	143	100	29.2	9900	2800	3354	1400	9	11.5	7.9	91

Table 4 - Summary of statistical properties of sample data

Despite the high number of test results extracted from the records, there were a high number of occurrences in which not all laboratory tests were performed on each soil sample. This had the consequence of reducing the number of sets of paired data able to be analyzed through linear regression. For example, in correlating shrink swell index with the soil suction parameter, only 8 pairs of data out of a possible 29 were considered valid, as being obtained from the same representative soil sample. As a result, it was not possible to analyse some relationships, as the sample population was considered too small to be meaningful.

6.4 Statistic Relationships

Table 5 below summarizes the results of the analysis of the relationships between the shrink swell index and the other key parameters. Figures 21 to 33 show the linear regression analysis plots for each pair of parameters compared.

7 Discussion of Results

7.1 General

The intention of this study was to apply a statistical approach to analyzing historical data from laboratory testing from site investigation activity, accumulated over a reasonable period of time, to include as many reactive soil areas within Queensland. Data was limited to that contained in the records of the Department of Transport and Main Roads – this was, however, considered a reasonable source of data, as Departmental records of laboratory test results from site investigations date back to the late 1960s.

In gathering the data, the main objective was to collect as many sets of data which included the Shrink Swell Index in addition to the other parameters of interest. The purpose of this was ultimately to enable a direct substitution of one of a number of test parameters used to characterize reactive soil properties for the Shrink Swell Index in the AS2870 equation for the determination of the characteristic surface movement,. For this reason, it was important to ensure that

reasonable sample populations of data could be gathered for data sets/pairs including the Shrink Swell Index as the primary parameter.

The limitation of this approach, however, was that the Australian standard test method for the Shrink Swell Index AS1289.7.1.1 was first published in 1992. Indeed, the inclusion of the characteristic surface movement equation in the Residential Slabs and Footings Standard 2870 was only first introduced in 1996. This means that prior to year of publication, the Shrink Swell test did not formally exist and consequently, there is no data older than 1992 of relevance to this study for Queensland or Australia-wide, for that matter. As a result, the period of time considered for data gathering for this study was from 1992 to 2013.

7.2   Correlation of Individual Parameters and Shrink Swell

A discussion of the correlation of each individual parameter with the shrink swell index is presented in following sections of the report. The criteria adopted for this study for assessing whether a particular relationship would be sufficiently reliable to enable the substitution of the shrink swell index with a paired parameter, are based on guidelines given by Crewson (2006), as discussed in Section 5.4. Specifically, where the coefficient of correlation (r) > 0.7 and the coefficient of determination of (R^2) > 0.5 for any particular correlation, the relationship is rated of sufficient reliability to be used in the estimation of the characteristic surface movement (y_s).

7.2.1   Correlation of Shrink Swell Index and % Passing 0.075mm

The test data used in for the regression analysis are presented in Appendix B. The data is shown plotted in Figure 21 with a regression curve fitted to the data sets.

The size of the sample population for the data is N=67, which appears reasonable to enable the regression to have validity.

The resulting trendline is a linear function with r = 0.543 and R^2 = 0.2557. Using the criteria for correlation in Table 3, it can be concluded that the data set shows the relationship between the shrink swell index and the percentage finer than 0.075mm is of low to moderate strength. As a result, the percentage finer than 0.075mm would not be a reliable parameter to use for the estimation of the characteristic surface movement (y_s).
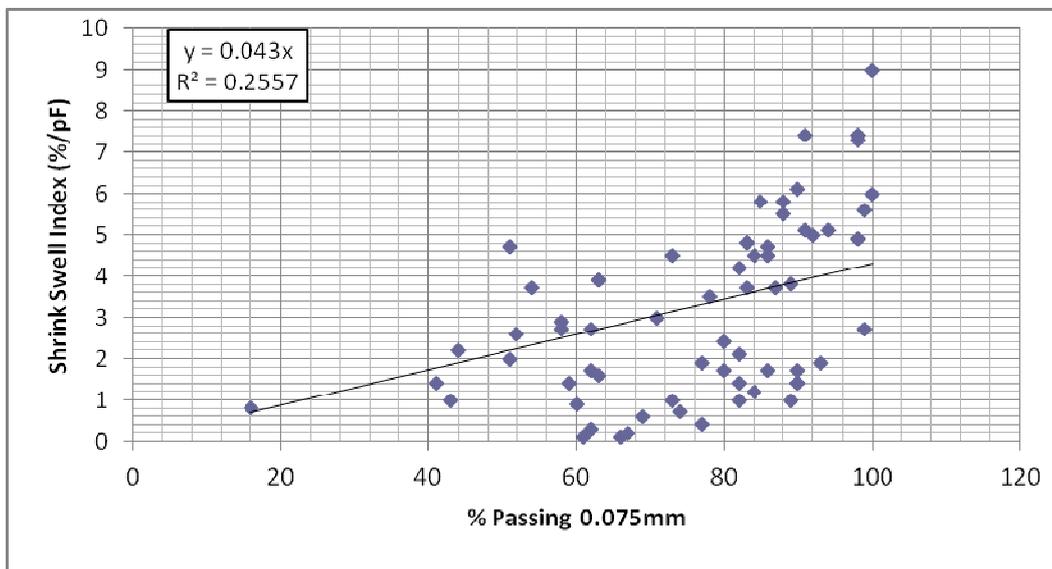
Figure 21 - Linear regression plot of % passing 0.075mm and shrink swell index

7.2.2 Correlation of Shrink Swell Index and % Passing 0.002mm

The test data used in for the regression analysis are presented in Appendix B. The data is shown plotted in Figure 22 with a regression curve fitted to the data sets.

The size of the sample population for the data is N=14, which appears too small to enable the regression to have validity. The following analysis must be viewed in the context of the small sample population.

The resulting trendline is a linear function with r = 0.417 and R^2 = 0.169. Using the criteria for correlation in Table 3, it can be concluded that the data set shows the relationship between the shrink swell index and the percentage finer than 0.002mm is of low strength. As a result, based on this data, the percentage finer than 0.002mm would not be a reliable parameter to use for the estimation of the characteristic surface movement (y_s). However, lack of data available for this correlation makes the outcome inconclusive.
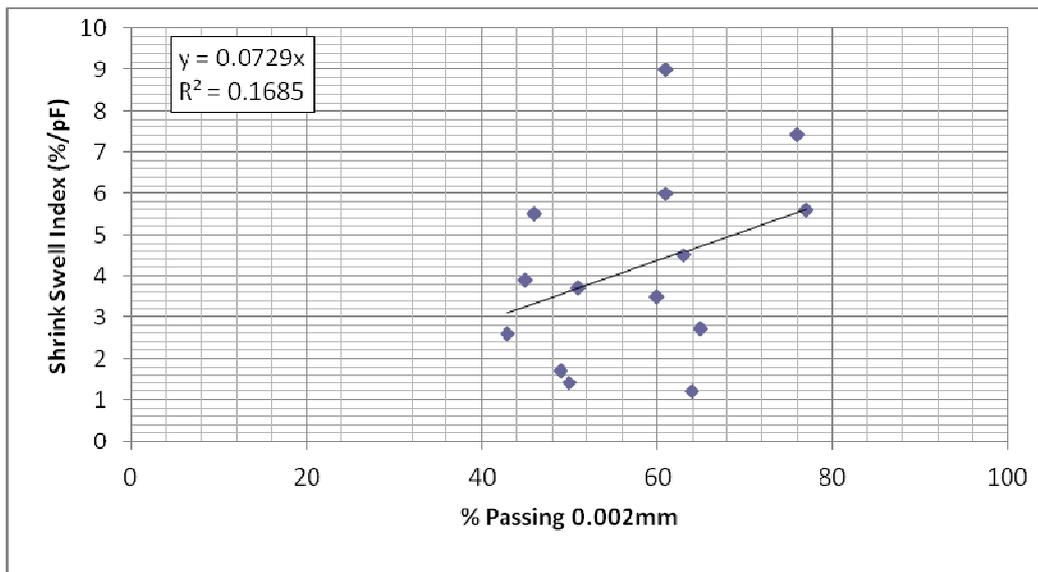
Figure 22 - Linear regression plot of % passing 0.002mm and shrink swell index

7.2.3   Correlation of Shrink Swell Index and Liquid Limit

The test data used in for the regression analysis are presented in Appendix B. The data is shown plotted in Figure 23 with a regression curve fitted to the data sets. The size of the sample population for the data is N=69, which appears sufficient to enable the regression to have validity.

The resulting trendline is a linear function with r = 0.791 and R^2 = 0.626. Using the criteria for correlation in Table 3, it can be concluded that the data set shows the relationship between the shrink swell index and the plasticity index is of high strength. As a result, based on this data, the liquid limit would be a reliable parameter to use for the estimation of the characteristic surface movement (y_s).

By using the linear relationship of the line of best fit below, the shrink swell index, can be estimated, and this estimated value can be used for the calculation of the characteristic surface movement, using the AS2870 equation as presented in Section 5.1.

$$Iss=0.0745(LL)-1.611$$
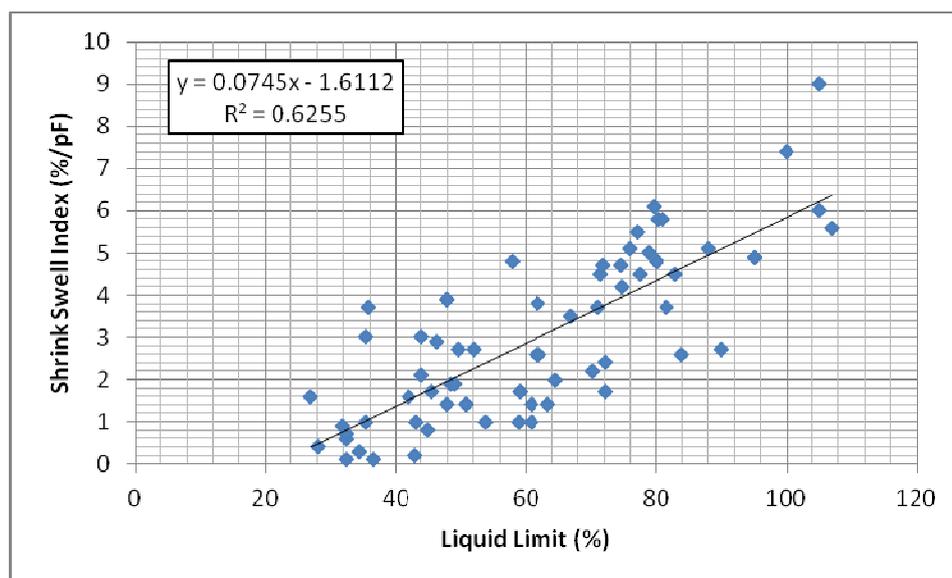
Figure 23 - Linear regression plot of liquid limit and shrink swell index

7.2.4   Correlation of Shrink Swell Index and Plasticity Index

The test data used in for the regression analysis are presented in Appendix B. The data is shown plotted in Figure 24 with a regression curve fitted to the data sets. The size of the sample population for the data is N=71, which appears sufficient to enable the regression to have validity.

The resulting trendline is a linear function with r = 0.719 and R^2 = 0.517. Using the criteria for correlation in Table 3, it can be concluded that the data set shows the relationship between the shrink swell index and the plasticity index is of high strength. As a result, based on this data, the plasticity index would be a reliable parameter to use for the estimation of the characteristic surface movement (y_s). By using the linear relationship of the line of best fit below, the shrink swell index, can be estimated, and this estimated value can be used for the calculation of

the characteristic surface movement, using the AS2870 equation as presented in Section 5.1.

$$I_{ss} = 0.0882(PI) - 2.637$$
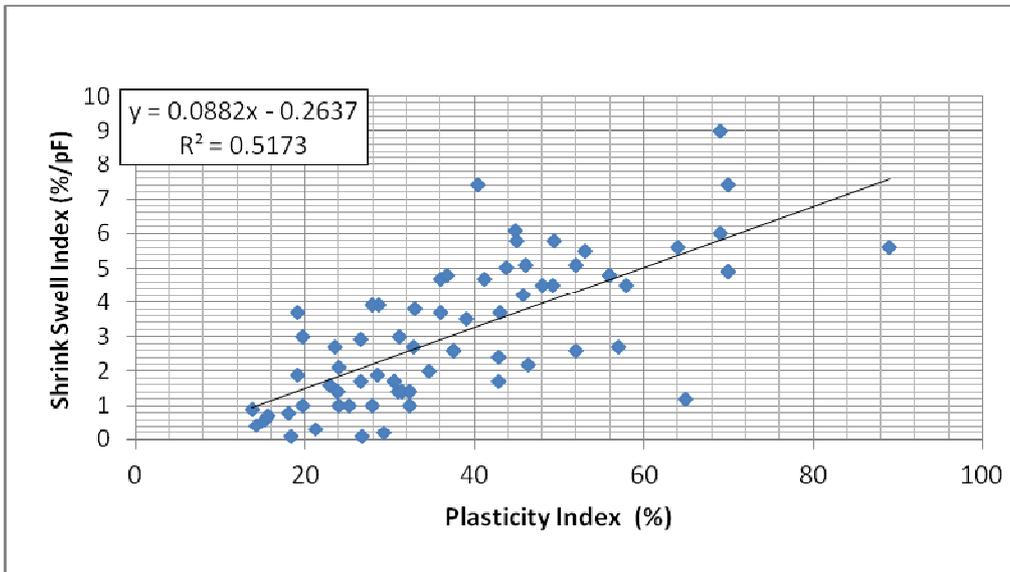
Figure 24 - Linear regression plot of plasticity index and shrink swell index

7.2.5 Correlation of Shrink Swell Index and Linear Shrinkage

The test data used in for the regression analysis are presented in Appendix B. The data is shown plotted in Figure 25 with a regression curve fitted to the data sets. The size of the sample population for the data is N=71, which appears sufficient to enable the regression to have validity.

The resulting trendline is a linear function with r = 0.662 and R^2 = 0.438. Using the criteria for correlation in Table 3, it can be concluded that the data set shows the relationship between the shrink swell index and the plasticity index is of moderate strength. As a result, based on this data, the plasticity index would not be a reliable parameter to use for the estimation of the characteristic surface movement (y_s).
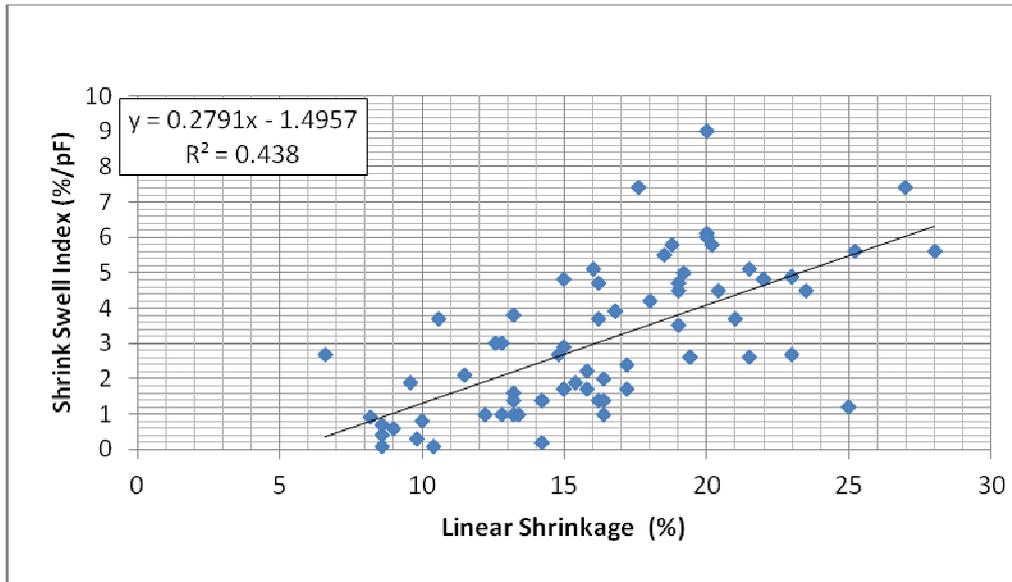
Figure 25 - Linear regression plot of linear shrinkage and shrink swell index

7.2.6 Correlation of Shrink Swell Index and Weighted Plasticity Index

The test data used in for the regression analysis are presented in Appendix B. The data is shown plotted in Figure 26 with a regression curve fitted to the data sets. The size of the sample population for the data is N=67, which appears sufficient to enable the regression to have validity.

The resulting trendline is a linear function with r = 0.705 and R^2 = 0.523. Using the criteria for correlation in Table 3, it can be concluded that the data set shows the relationship between the shrink swell index and the plasticity index is of high strength. As a result, based on this data, the weighted plasticity index would be a reliable parameter to use for the estimation of the characteristic surface movement (y_s). By using the linear relationship of the line of best fit below, the shrink swell index, can be estimated, and this estimated value can be used for the calculation of the characteristic surface movement, using the AS2870 equation as presented in Section 5.1.

$$I_{ss} = 0.0009(WPI) - 1.274$$

39
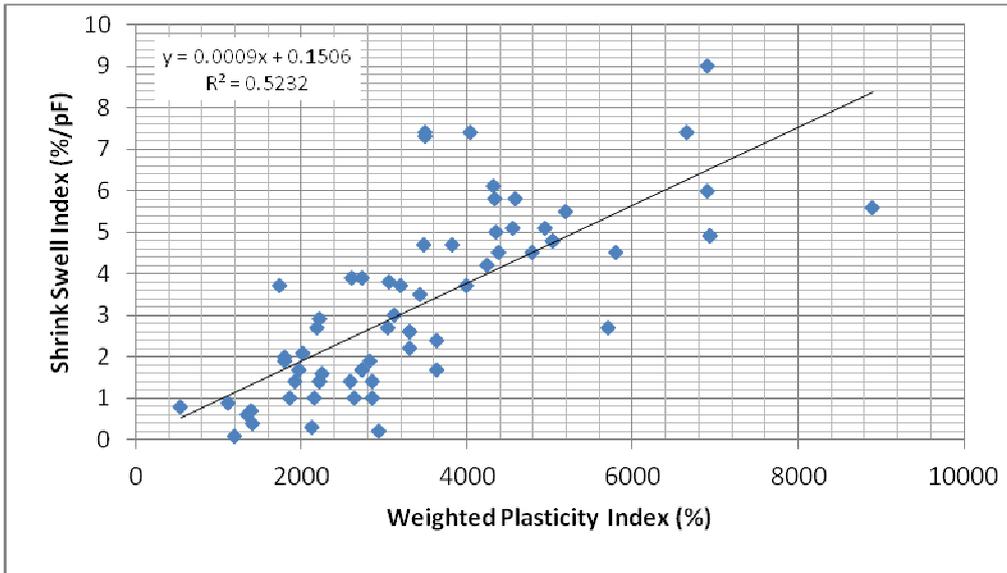
Figure 26 - Linear regression plot of weighted plasticity index and shrink swell index

7.2.7 Correlation of Shrink Swell Index and Weighted Linear Shrinkage

The test data used in the regression analysis is presented in Appendix B. The data is shown plotted in Figure 27 with a regression curve fitted to the data sets. The size of the sample population for the data is N=67, which appears sufficient to enable the regression to have validity.

The resulting trendline is a linear function with r = 0.679 and R^2 = 0.520. Using the criteria for correlation in Table 3, it can be concluded that the data set shows the relationship between the shrink swell index and the plasticity index is of moderate strength. As a result, based on this data, the weighted plasticity index would not be a reliable parameter to use for the estimation of the characteristic surface movement (y_s).

7.2.8 Correlation of Shrink Swell Index and CBR Swell

The test data used in for the regression analysis are presented in Appendix B. The data is shown plotted in Figure 28 with a regression curve fitted to the data sets. The size of the sample population for the data is N=26, which appears sufficient to identify any trend but may not be sufficient to be conclusive in defining the relationship. The following analysis must be viewed in the context of the relatively small sample population.
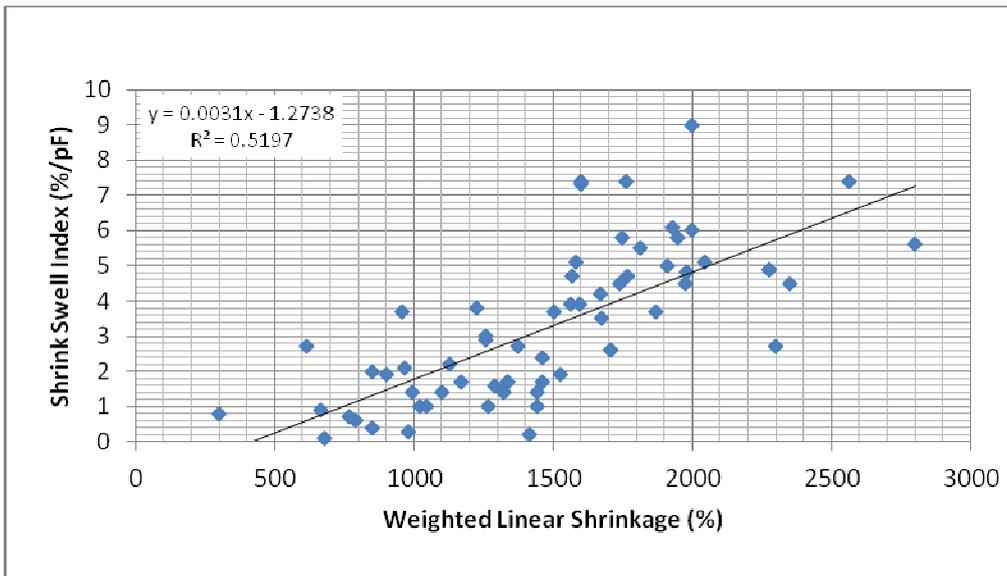
Figure 27 - Linear regression plot of weighted linear shrinkage plot and shrink swell index

In addition, the swelling measurements from tests where the soaked period was 10 days were included in the analysis with the data from tests where the soaking period was 4 days.

The resulting trendline is a linear function with r = 03821 and R^2 = 0.607. Using the criteria for correlation in Table 3, it can be concluded that the data set shows the relationship between the shrink swell index and the CBR Swell is of high strength. As a result, based on this data, the CBR Swell would be a reliable parameter to use for the estimation of the characteristic surface movement (y_s). By using the linear relationship of the line of best fit below, the shrink swell index, can be estimated, and this estimated value can be used for the calculation of the characteristic surface movement, using the AS2870 equation as presented in Section 5.1.

$$I_{ss} = 0.3321(CBRSwell) - 2.382$$

However, it is recommended that a larger sample population be used to confirm this correlation.

41
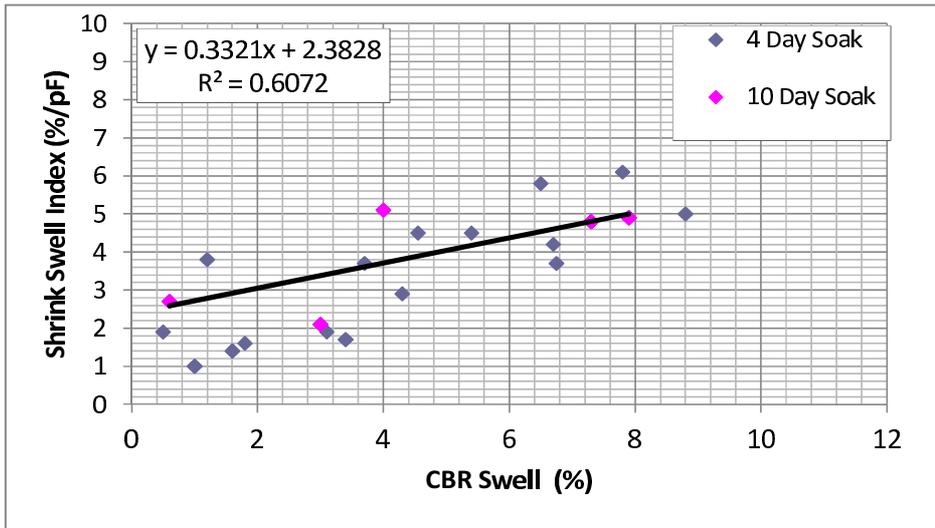
Figure 28 - Linear regression plot of CBR swell and shrink swell index

7.2.9 Correlation of Shrink Swell Index and Soil Suction

The test data used in for the regression analysis are presented in Appendix B. The data is shown plotted in Figure 29. The size of the sample population for the data is N=8, which is too small to enable the regression to have validity. As a result regression analysis was performed on this set of data. In order to assess the relationship between this data set, a larger sample population is required. The outcome for this data set is inconclusive.
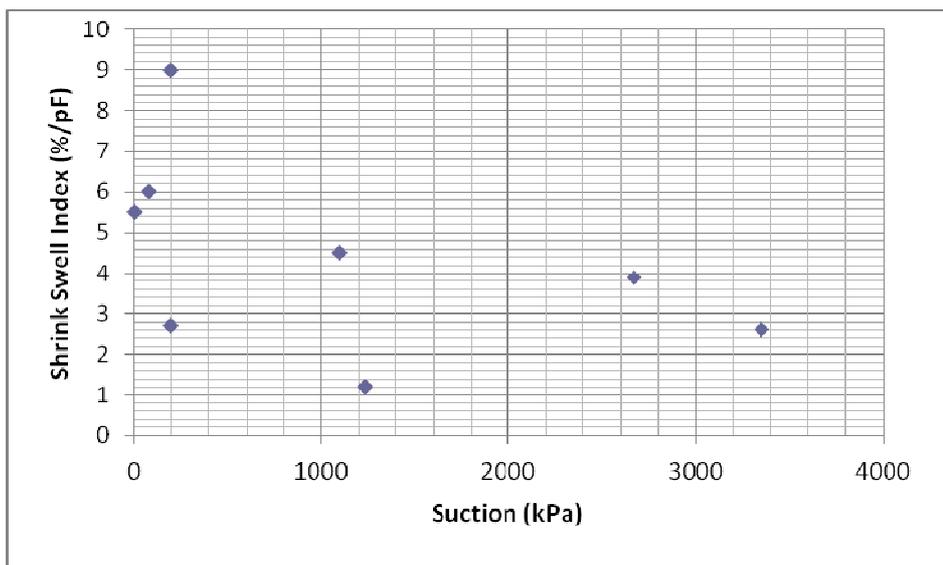
7.2.10 Correlation of Shrink Swell Index and Swelling Pressure

The test data used in for the regression analysis are presented in Appendix B. The data is shown plotted in Figure 30 with a regression curve fitted to the data sets. The size of the sample population for the data is N=19, which appears too small to enable the regression to have validity. The following analysis must be viewed in the context of the small sample population.

The resulting trendline is a linear function with r = 0.583 and R^2 = 0.340. Using the criteria for correlation in Table 3, it can be concluded that the data set shows the relationship between the shrink swell index and the swelling pressure is of low to moderate strength. As a result, based on this data, the swelling pressure would not be a reliable parameter to use for the estimation of the characteristic surface movement (y_s). However, lack of data available for this correlation makes the outcome inconclusive.
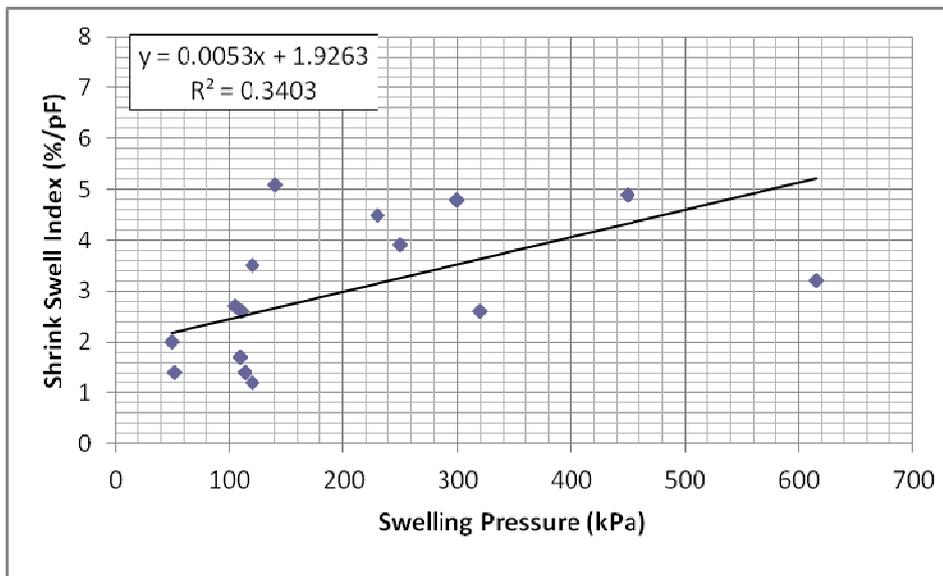
Figure 30 - Linear regression plot of swelling pressure and shrink swell index

7.2.11 Correlation between Shrink Swell Index and CEC

This relationship was unable to be investigated due to the very low data sets available from the historical Main Roads records. Only four valid data sets were able to be considered for analysis – clearly this sample population is insufficient. As a result the regression analysis between shrink swell index and cation exchange capacity (CEC) was not carried out, and is not presented in this report.

7.3 Correlation of Combined Parameter Functions and Shrink Swell Index

In the analysis above, only relationships between individual pairs of parameters were examined for potential correlation. However, it is also acknowledged that statistically valid relationships might also exist between combinations of parameters, as related by simple functions.

To explore this proposal in more detail, consideration was given to the method in which shrink swell index is calculated in AS1289.7.1.1.

As detailed in Section 4.1, the shrink swell index parameter is a summation between the strain measured in shrinkage and half of the swelling of the soil specimen, divided by a constant. Fundamentally, the shrink swell index is a function of the shrinkage strain and the swelling strain of the soil.

Linear Shrinkage and CBR Swell and Shrink Swell Index
Of the other parameters considered in this study, the linear shrinkage and the CBR Swell are direct measures of shrinkage strain and swelling strain respectively, despite the obvious differences in the methodology used to obtain these parameters.

On this basis, it was considered useful to investigate whether any relationship existed between the shrink swell index and some mathematical function between the linear shrinkage and the CBR Swell parameters.

7.3.1 Correlation of Shrink Swell Index and (LS + CBR Swell)

In an attempt to model the shrink swell index equation, it was decided to at first consider a correlation between the simple summation of the linear shrinkage and the CBR Swell values, and the shrink swell index. The results of this are shown in Figure 31 below, and the values are tabulated in Appendix B.
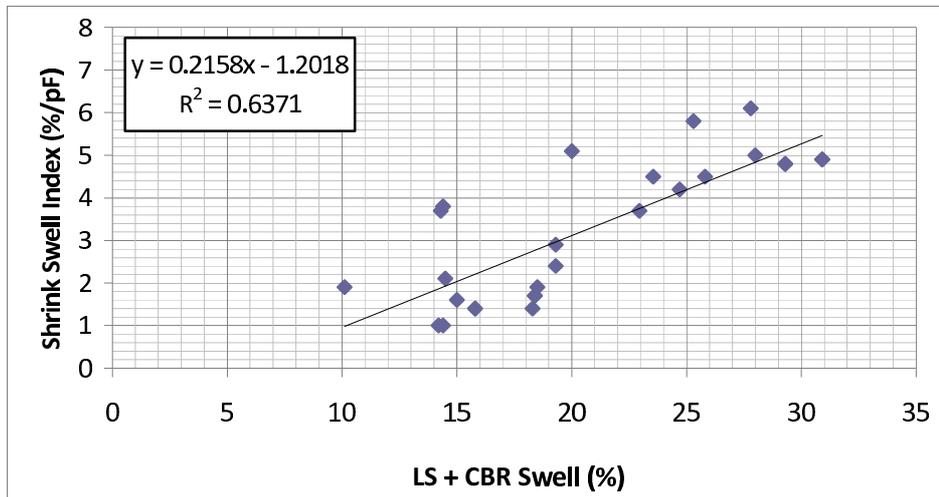
Figure 31 - Linear regression plot of linear shrinkage + CBR Swell and shrink swell index

It should be noted that the correlation is based on a modest sample population (ie N = 27). However, taking this into account, the correlation between the sum of linear shrinkage and CBR swell was assessed to be of high strength (r = 0.798, R^2 = 0.637).

As a result, based on this data, the combination of the linear shrinkage added to the CBR Swell would be a reliable parameter to use for the estimation of the characteristic surface movement (y_s). By using the linear relationship of the line of best fit below, the shrink swell index, can be estimated, and this estimated value can be used for the calculation of the characteristic surface movement, using the AS2870 equation as presented in Section 5.1.

$$I_{ss} = 0.216(LS + CBRSwell) - 1.202$$

However, it is recommended that a larger sample population be used to confirm this correlation.

7.3.2 Correlation of Shrink Swell Index and "Combined Reactivity Index"

Another combination of parameters was investigated for correlation with shrink swell index. The parameters of linear shrinkage and CBR Swell, measuring shrinkage and swelling strain were used in an arrangement similar to the equation used for the calculation of the shrink swell index. This new quantity postulated by the author, is defined by the equation below and is provisionally described by the author as the Combined Reactivity Index (CRI):

$$CRI = \frac{LS + (CBRSwell x 500)}{1.8}$$

The test data used in for the regression analysis are presented in Appendix B. The data is shown plotted in Figure 32 with a regression curve fitted to the data sets. The size of the sample population for the data is N=25. Despite the modest size of the data set, it appears sufficient to identify trend. However, the population size may not be sufficient to conclusively define the relationship. Hence following analysis must be viewed in this context.

The correlation between the CRI and the shrink swell index was assessed to be of high strength ($r = 0.887$, $R^2 = 0.787$). This relationship was observed to show the best correlation of all the parameters examined in this study.

As a result, based on this data, the CRI would be a reliable parameter to use for the estimation of the characteristic surface movement (y_s). By using the linear relationship of the line of best fit below, the shrink swell index, can be estimated, and this estimated value can be used for the calculation of the characteristic surface movement, using the AS2870 equation as presented in Section 5.1.

$$I_{ss} = 0.0017(CRI) - 0.3081$$

The values used in the analysis are tabulated in Appendix B.
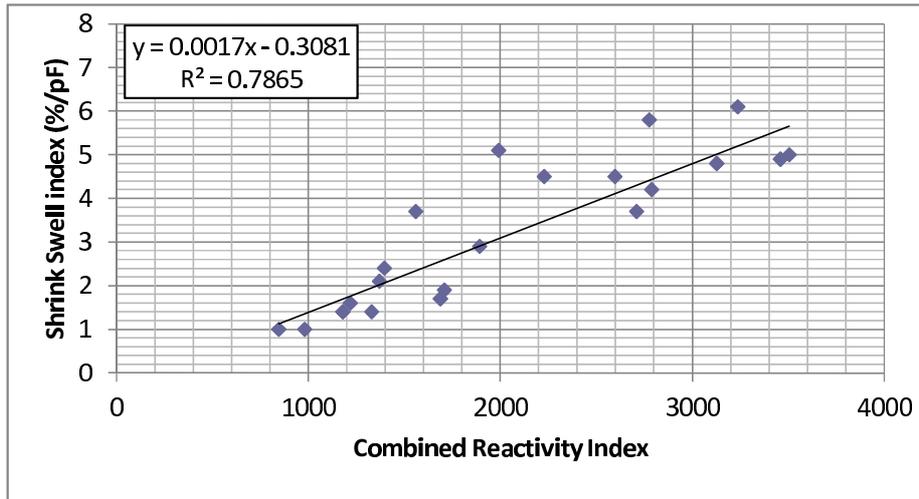
Figure 32 - Linear regression plot of CRI and shrink swell index

7.3.3 Correlation of Shrink Swell Index and (LS/CBR Swell) Ratio

A third combination of parameters was investigated for correlation with shrink swell index. The CBR Swell and the linear shrinkage were combined in a ratio; specifically CBR Swell/linear shrinkage.

The ratio of the CBR Swell to the Linear Shrinkage was calculated for each set of data in which there was a corresponding shrink swell index test result. The linear regression of the CBR Swell/LS ratio and the shrink swell index is shown in Figure 33, and the values are tabulated in Appendix B.
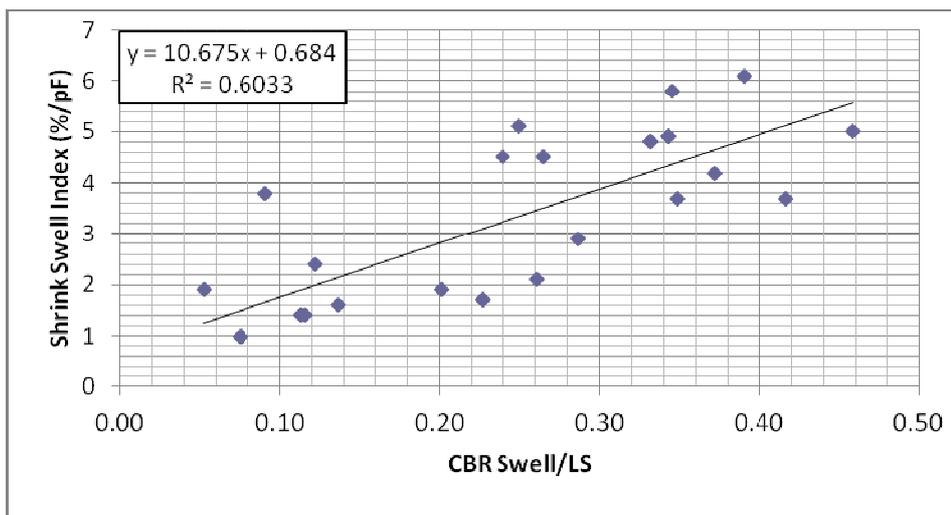
Figure 33 - Linear regression plot of ratio of CBR swell/LS and shrink swell index

47

Using Table 3, it can be seen that despite a relatively modest sample population (N=27), the parameters exhibit a high strength relationship (r=0.777, R^2=0.603). Thus as the ratio of CBR Swell and linear shrinkage correlates well with shrink swell index for soils over a range of locations, it can be can be cautiously classified as a useful parameter for the purposes of estimation.

By using the linear relationship of the line of best fit below, the shrink swell index, can be estimated, and this estimated value can be used for the calculation of the characteristic surface movement, using the AS2870 equation as presented in Section 5.1.

$$I_{ss} = 10.675(CBRSwell / LS) + 0.684$$

7.4   Summary of Correlation Analysis

Table 5 below summarises the results of the analysis including the factor and the constants of the regression lines. For the purposes of using the regression equations to calculate an estimation of the shrink swell index, only the parameters in which a high strength relationship was determined should be considered.

8   Conclusions

8.1 Major Outcomes and Key Findings

This report has introduced the main properties of expansive soils and highlighted some of the risks these materials pose to the serviceability and safety of road assets. It has provided an overview of the main approach for the design of structures on reactive soil sites (AS2870) within Queensland (and Australia) and the primary method for determining the magnitude of the potential surface movements likely to be encountered for such sites.

Parameter	No. of Data Pairs (N) Corrected for Outliers)	Coefficient of Correlation (r)	Coefficient of Determination (R^2)	Description of Relationship Strength	Factor	Constant
<0.075mm	67	0.543	0.256	Moderate-Low	0.0430	0
<0.002mm	14	0.417	0.169	Low	0.0729	0
LL	69	0.791	0.626	High	0.0745	-1.611
PI	71	0.719	0.517	High	0.0882	-2.637
WPI	67	0.705	0.523	High	0.0009	-1.274
LS	71	0.662	0.438	Moderate	0.2791	-1.496
WLS	67	0.679	0.520	Moderate	0.0031	0.151
CBR Swell	26	0.821	0.607	High	0.3321	2.382
Suction	8	NA	NA	NA	NA	NA
Loaded Swell/Swell Pressure	19	0.583	0.340	Moderate-Low	0.0053	1.926
CEC	4	NA	NA	NA	NA	NA
LS + CBR Swell	27	0.798	0.637	High	0.216	1.202
$\dfrac{LS + (CBRSwellx500)}{1.8}$	25	0.887	0.787	High	0.0017	-0.3081
CBR Swell/LS	27	0.777	0.603	High	10.675	0.684

Table 5 - Summary of correlation analysis of all parameters with shrink swell index

This study has also identified some of the laboratory test parameters used to identify and characterise soils with shrink/swell potential. Relationships between these parameters and the shrink swell index have been investigated to determine if alternatives to the shrink swell could potentially be used in the calculation of characteristic surface movement (y_s).

Test data gathered from site investigations throughout Queensland have been statistically analyzed to determine the relationships that exist between the various parameters and shrink swell index.

Of the relationships identified, it was found that the following parameters, determined by common laboratory tests, showed a high strength of correlation with the Shrink Swell Index:

- Liquid limit
- Plasticity Index
- Weighted Plasticity Index
- CBR Swell

It was also found that unique combinations of these standard test parameters exhibited high strength of correlation. Specifically:
- Linear Shrinkage + CBR Swell
- "Combined Reactivity Index"
- CBR Swell/Linear Shrinkage

It is the conclusion of this report that, based on the strength of the correlations, it would seem a reasonable approach for a designer or engineer to substitute these parameters for the Shrink Swell Index (and by association the Instability Index), in the calculation of the characteristic surface movement (y_s), as per AS2870 – Residential Slabs and Footings.

Some caution would be required in using these parameters, however, as the sample populations on which the correlations are based contain modest to small numbers of data.

8.2   Recommendations for Further Work

The source of data for this investigation was limited to the historical records of the Department of Transport and Main Roads, from 1992 to 2013. Despite the number of published reports available, and the large numbers of reactive soil sites investigated throughout Queensland over this period, the amount of laboratory test data available from this activity was found to be limited.

This was particularly the case for the less common laboratory tests considered, such as soil suction and cation exchange capacity.

To investigate these relationships in more detail, other sources of data need to be considered, such as local government authorities, commercial construction materials laboratories, and consultant engineering firms within Queensland.

9   List of References

Al-Omari, R.R and Oraibi, W.K. (2000), "Cyclic Behavior of Reinforced Expansive Clay", Soils and Foundations Vol. 40, No. 2, Japanese Geotechnical Society.

ASTM International (2003), Standard Test Method for One Dimensional Swell or Settlement Potential of Cohesive Soils, D4546 – 03, 100 Barr Harbor Drive, PO Box C700, West Conshohocken, PA 19428-2959, United States.

Bulut R., Lytton R.L. and Wray W.K. (2001), Expansive clay soils and vegetative influence on shallow foundations, Proceedings of Geo-Institute Shallow Foundation and Soil Properties, Committee Sessions at the ASCE 2001 Civil Engineering Conference

Casagrande, A. (1958), Notes of the design of the liquid limit device, Geotechnique, vol. 8, No. 2, pp 84-91.

Cameron D.A. (1989), Tests for Reactivity and Prediction of Ground Movement, Australian Civil Engineering Transactions, pp121-131

Caunce C.M. (2010) – Effective Road Pavement Design for Expansive Soils in Ipswich, University of Southern Queensland

Chen, F. H. (1988), Foundations on Expansive Soils, Elsevier Science Publishers

Clarke G.M. and Cooke D. (1998), A basic course in statistics. Arnold London, 4th edition

Crewson P. (2006), Applied Statistics Handbook, AcaStat Software

Crilly, M. and Chandler, R., (1993); A method of determining the state of desiccation in clay soils, BRE Information Paper, February 1993

Department of Transport and Main Roads, Queensland, (2011), Technical Report MR2469, East Warianna Creek Culvert Culvert Foundation Expansive Soil Investigation

Department of Transport and Main Roads, Queensland, (2011), Technical Report MR2470, Auchmar Culvert Culvert Foundation Expansive Soil Investigation

Department of Transport and Main Roads, Queensland, (2012), Materials Testing Manual: List of Test Methods, Queensland Volume 1

Earl D. (2005), To determine if there is a correlation between the shrink swell index and Atterberg limits for soils within the Shepparton Formation, University of Southern Queensland

Fox E., 2000, A climate based design depth of moisture change map of Queensland and the use of such maps to classify sites under AS 2870-1996, Australian Geomechanics, December 2000

Fredlund D.G., Rahardjo H., Fredlund M.D., (2012), Unsaturated Soil Mechanics in Engineering Practice, Wiley & Sons

Fityus, S.G., Cameron, D.A., and Walsh, P.F., (2009), The Shrink Swell Test, Geotechnical Testing Journal, Vol. 28, No. 1.

Gallagher A.G., (1991), Developments in the Measurement of Soil Suction, Queensland University of Technology.

Gourley GS, Newill D and Schreiner H.D, (1993), Expansive soils: Research Strategy, First International Symposium on Engineering Characteristics of Arid Soils, City University, London

Head, K. H., (1980), Manual of Soil Laboratory Testing, Volume 1: Soil Classification and Compaction Tests, Pentech Press, London.

Head, K. H., (1982), Manual of Soil Laboratory Testing, Volume 2: Permeability, Shear Strength and Compressibility Tests, Pentech Press, London.

Lopez, D and McManus, KJ. (1996), Soil Index Methods to Predict Ground Movements, 7th Australia New Zealand Conference on Geomechanics: Geomechanics in a Changing World: Conference Proceedings. Barton, ACT: Institution of Engineers, Australia, 1996: 921-924. National conference publication (Institution of Engineers, Australia) ; no. 96/07.

Lucian, C. (2012), Soil-Suction Measurements Using the Filter Paper Method to Evaluate Swelling Potential, International Journal of Scientific Engineering and Technology, Volume No.1, Issue No.4, pg : 49-54

Nelson, J. and Miller, D., (1992), Expansive Soils: Problems and Practice in Foundation and Pavement Engineering, 1992, Wiley & Sons

Nelson, J.D, Overton, D.D. and Durkee, D.B., (2001), Depth of Wetting and the Active Zone, pp95-109, Proceedings from ASCE Conference (2001)

Or, D., M. Tuller and J.M. Wraith, (2005), Soil Water Potential, In: Encyclopedia of Soils in the Environment, (ed. Hillel D.), Elsevier Science, Oxford, 270-277

Pritchard, R. and Wijeyakulasurily C. V.(2000), Drainage Structures on Expansive Soils in Western Queensland, Technical Note WQ37, Queensland Department of Main Roads(2000)

Punthutaecha, K and Puppala, J; Vanapalli,S and Inyang, H (2006), Volume Change Behaviors of Expansive Soils with Recycled Ashes and Fibers, Journal of Materials in Civil Engineering

Queensland Department of Main Roads, (1993), Technical Note 7 – Controlling Moisture in Pavements, Transport Technology

Queensland Department of Main Roads, (2002), Technical Report CR667, Gordonstone Creek Crossings Foundation Investigations

Queensland Department of Main Roads, (1999), Technical Report R3177, Bruce Highway Upgrade – Yandina to Cooroy (Cut J) – Geotechnical Investigation

Queensland Department of Main Roads, (1999), Technical Report MR1780, Thompson River Project, Culvert Site Classification

Queensland Department of Main Roads, (1999), Technical Report MR1795, Deep Creek and D'Aigular Highway Expansive Clay Testing

Queensland Department of Main Roads, (1999), Technical Report MR1798, Hughendon – Winton Road Culvert Bases – Expansive Property Results

Queensland Department of Main Roads, (2000), Technical Report MR1846, Wondalli Creek Base Slab – 122/17D/30

Queensland Department of Main Roads, (2000), Technical Report MR1892, Culvert Base Slab on Expansive Clay – Yaralla Deviation

Queensland Department of Main Roads, (2000), Technical Report MR1897, Culvert Foundation Investigation – Doctor's Creek

Queensland Department of Main Roads, (2000), Technical Report MR1898, Culvert Foundation Investigation – Lagoon Creek

Queensland Department of Main Roads, (2000), Technical Report MR1907, Culvert Foundation Investigation – Barkly Highway

Queensland Department of Main Roads, (2001), Technical Report MR1918, Macalister Culvert – Job Number 124/18C/16

Queensland Department of Main Roads, (2001), Technical Report MR1949, Glenorchy Straight – Additional Geotechnical Investigation

Queensland Department of Main Roads, (2002), Technical Report MR2052, Chippendale Creek Culvert Investigation

Queensland Department of Main Roads, (2004), Technical Report MR2140, Warrego Highway (Bowenville to Dalby) Culvert Investigation

Queensland Department of Main Roads, (2008), Technical Report MR2326, Site Investigation of Cracking at Cockatoo Creek Culverts

Queensland Department of Main Roads, (2008), Technical Report MR2334, Deep Creek Channel – Culvert Investigation

Queensland Department of Main Roads, (2009), Technical Report MR2366, Bribie Island Road – Expansive Subgrade Investigation

Queensland Department of Main Roads, (2009), Technical Report MR2367, Exit 54 Pacific (M1) Motorway at Coomera Supplementary Investigation

Queensland Department of Main Roads, (1999), Technical Report R3162, Bruce Highway Upgrade (Brisbane to Gympie) Additional Geotechnical Investigation

Queensland Department of Main Roads, (2002), Technical Report R3263, Gatton Bypass Package 2 (CH30000 – 39400) Job No. 114/18A/54.2

Queensland Department of Main Roads, (2006), Technical Report R3379, Mackay – Bucasia Road between Holts Road and Golflinks/Habana Roads (Ch.2.6km to Ch.3.3km) - Job No. 120/856/12

Queensland Department of Main Roads, (2006), Technical Report R3380, Final Report: Geotechnical Investigation for Linkfield Rd Overpass Duplication, Volume 1: Current Investigation

Queensland Department of Main Roads, (2007), Technical Report R3403, Geotechnical Investigation at Tamborine-Oxenford Deviation Link Road

Queensland Department of Main Roads, (2008), Technical Report R3419, Warrego Highway (Macalister-Warra) Culverts: Expansive Subgrade Investigation

Queensland Department of Main Roads, (2010), Technical Report R3460, Cunningham Highway/Amberley Interchange Preliminary Geotechnical Report

Queensland Department of Main Roads, (2010), Technical Report R3461, Cunningham Highway/Amberley Interchange Interpretive Preliminary Geotechnical Report

Ramanujam J (1999), Rehabilitation of Pavement on Expansive Soils, Workshop on low volume roads, Roma Bungil Cultural Community Centre, Roma Reeves, I, 2001, Risk Management – Materials [presented at Road System and Engineering Technology Forum 2001],

Snethen Dr D. R (1980), Expansive Soils in Highway Subgrades, U.S Department of transportation, United States of America Smith R.L (2004), Achieving the Goal of Management of Reactive Clays, Australian Institute of Building Surveyors Conference, Australia

Standards Australia, (1995), AS 1289.3.2.1-1995 Methods of testing soils for engineering purposes Method 3.2.1: Soil classification tests- Determination of the plastic limit of a soil – Standard method, Standards Australia, Homebush, NSW.

Standards Australia, (1995), AS 1289.3.3.1-1995 Methods of testing soils for engineering purposes Method 3.3.1: Soil classification tests- Calculation of the plasticity index of a soil, Standards Australia, Homebush, NSW.

Standards Australia, (1995), AS 1289.3.4.1-1995 Methods of testing soils for engineering purposes Method 3.4.1: Soil classification tests - Determination of the linear shrinkage index of a soil – Standard method,
Standards Australia, Homebush, NSW.

Standards Australia, (1995), AS 1289.7.1.1-2003 Methods of testing soils for engineering purposes Method 7.1.1: Soil reactivity tests- Determination of the shrinkage index of a soil – Shrink-swell index, Standards Australia, Homebush, NSW.

Standards Australia, (1996), AS 2870-1996 Residential slabs and footings - Construction, Standards Australia, Homebush, NSW.

Vanderstaay, A. G. And Reeves, I. N. (2000), Soils of Western Queensland, Technical Note WQ32 , Queensland Department of Main Roads (2000)

Walters K. A., (2008), Investigation of Construction Practices and Test Procedures for Road Pavements on Expansive Subgrades Volume I, University of Southern Queensland

Waters T (2002), Moisture in Pavements, Road System and Engineering Technology, Department of Main Roads

Watford Day, R. W. (1999), Geotechnical and foundation engineering design and construction, McGraw-Hill Companies, New York.

Yanagisawa E, Moroto, N and Mitachi, T (1998), Problematic Soils, Vol 1, Proceedings of the international symposium on problematic soils

Yang H P, Zheng J and Zhang R (2007), Addressing Expansive Soils, Institute of Special Soils, pg 64-69

Appendix A Project Specification

For	Peter Reynolds
Topic	Engineering Correlations for the Characterisation of Reactive Soil Behaviour For Use in Road Design
Supervisors	Dr Kazem Ghabraie
Enrolment	ENG 4111 – S1, X, 2013 ENG 4112 – S2, X, 2013
Project Aim	To identify and quantify the relationships that exist between key measures of reactive soil behaviour.
PROGRAMME	**Issue A -13th March 2013**
	1. Research the background information relating to the properties, identification and classification of reactive soils as they relate to road design and construction
	2. Identify sources of data containing published site classification information for road infrastructure projects within Queensland
	3. Obtain and extract relevant data from published sources and collate based on locality and geological formation, where possible.
	4. Investigate and analyse soil property parameters for any significant trends or correlations.
	5. Determine whether there is any specific parameters which are more reliable indicators of reactive soil behaviour and identify any limitations to the use of these parameters
	6. Rank soil property parameters in order of reliability and how economical they are to determine
	7. Develop a guideline for the investigation and identification of reactive soil, including a table of correlations of soil parameters

Appendix B Tabulated Test Data

Tabulated Data for Correlation of % passing 0.075mm sieve and Shrink Swell Index

Sample/Article Number	Depth (m)	0.075mm	Iss %/PF
GS00-447	0-1.3	82	4.2
GS00-444	1.3-2.1	92	5
BS00-1291/GS00-504	0-0.5	67	0.2
BS00-1292/GS00-505	0.5-1.0	62	0.3
BS00-1295/GS00-511	0.5-1.0	71	3
Performed by consultants	0.25-.7	62	1.7
Performed by consultants	0.7-2.1	16	0.8
Performed by consultants	0-1.2	77	0.4
Performed by consultants	0-1.2	60	0.9
Performed by consultants	0-2.0	66	0.1
Performed by consultants	0-2.0	69	0.6
Performed by consultants	0-2.0	74	0.7
BS99-0757	0.7-2.1	84	4.5
GS00-456, 00/1044	0.2-0.5	58	2.9
GS00-457, 00/1045	0.5-0.8	54	3.7
GS00-458, 00/1046	0.8-1.3	62	2.7
GS00-454, 00/1042	0.3-0.6	63	1.6
GS00-455, 00/1043	1.7-2.1	77	1.9
GS00-442, 00/964	0-0.4	90	6.1
GS00-443, 00/965	0.4-1.4	88	5.8
GS00-495,00/1075	0.3-0.6	73	1
GS00-496,00/1076	1.7-2.0	89	1
GS00-497,00/1077	0.25-0.65	87	3.7
GS00-498,00/1078	0.65-1.1	86	4.5
GS00-499,00/1079	1.7-2.0	86	4.7
GS00-500,00/1080	0.25-0.55	61	0.1
GS00-501,00/1081	0.7-1.0	89	3.8
GS00-502,00/1082	1.4-1.6	93	1.9
GS99/207.F/99-207	NA	86	1.7
GS99/208.F	NA	90	1.4
GS01/395	0-1.7	43	1
GS01/396	3.0-3.3	44	2.2
GS01/397	1.0-1.45	41	1.4
GSO4/418	0.5 - 0.8	63	3.9
GSO4/420	1.7 - 2.0	52	2.6
Performed by consultants	2.0 -2.4	85	5.8

Performed by consultants	4.0 - 4.4	51	4.7
Performed by consultants	0.0 - 2.5	98	7.4
Performed by consultants	NA	98	7.3
GS04-418	0.5-0.8	63	3.9
GS08-434	1.5	94	5.1
GS08-438	0.5	82	2.1
GS11-1353/CLON110772	0-1.3	59	1.4
GS11-1354/CLON110773	1.3-2.5	51	2
GS11-1471/11110313	2.5-2.95	58	2.7
GS99-207	NA	90	1.7
GS99-208	NA	90	1.4
GS06-061/062	0.5-0.9	98	7.4
603224/GS06-089/GS06-100	0-0.5	83	3.7
603228/GS06-093/GS06-102	1	99	5.6
603552/GS06-0110	1.0-1.4	91	5.1
603554/GS06-0113	1.5-1.9	91	7.4
GS07-525/526	1.5-2.9	84	1.2
GS07-528/529	3.0-3.9	73	4.5
GS07-536	0.5-0.9	78	3.5
GS10-072/129	0.3-3.1	98	4.9
GS10-078/079	0.2-0.9	83	4.8
GS01-212RA	NA	82	1
GS01-212RB	NA	82	1.4
GS01-211RA	NA	80	1.7
GS01-211RB	NA	80	2.4
GS06-809	3-3.45	100	6
GS06-789	1.0-1.45	88	5.5
GS06-793/809	2.5-2.9	100	9
GS06-798/836/837	1.0-1.45	99	2.7
GS10-129/072	0.3-3.1	98	4.9
GS10-078/79	0.2-0.9	83	4.8

Tabulated Data Used for Correlation between

% Finer than 0.002mm and Shrink Swell Index

Sample/Article Number	Depth (m)	0.002mm	Iss %/PF
GS99/207.F/99-207	NA	49	1.7
GS99/208.F	NA	50	1.4
GS04-418	0.5-0.8	45	3.9
GS04-420	1.7-2.0	43	2.6
603224/GS06-089/GS06-100	0-0.5	51	3.7
603228/GS06-093/GS06-102	1.0	77	5.6
603554/GS06-0113	1.5-1.9	76	7.4
GS07-525/526	1.5-2.9	64	1.2
GS07-528/529	3.0-3.9	63	4.5
GS07-536	0.5-0.9	60	3.5
GS06-809	3-3.45	61	6
GS06-789	1.0-1.45	46	5.5
GS06-793/809	2.5-2.9	61	9
GS06-798/836/837	1.0-1.45	65	2.7

Tabulated Data for Correlation between Liquid Limit and Shrink Swell Index

Sample/Article Number	Depth (m)	LL (%)	Iss %/PF
GS00-097.B	1.5-1.9	27	1.6
Performed by consultants	0-1.2	28.2	0.4
Performed by consultants	0-1.2	31.8	0.9
Performed by consultants	0-2.0	32.4	0.1
Performed by consultants	0-2.0	32.4	0.6
Performed by consultants	0-2.0	32.4	0.7
BS00-1292/GS00-505	0.5-1.0	34.4	0.3
GS00-096.B	1.0-1.4	35.4	1
GS00-094.B	0-0.4	35.4	3
GS00-457, 00/1045	0.5-0.8	35.8	3.7
GS00-500,00/1080	0.25-0.55	36.6	0.1
GS00-454, 00/1042	0.3-0.6	42	1.6
BS00-1291/GS00-504	0-0.5	43	0.2
GS01/395	0-1.7	43.2	1
GS08-438	0.5	44	2.1
BS00-1295/GS00-511	0.5-1.0	44	3
Performed by consultants	0.7-2.1	45	0.8
Performed by consultants	0.25-.7	45.6	1.7
GS00-456, 00/1044	0.2-0.5	46.4	2.9
GS01/397	1.0-1.45	48	1.4
GSO4/418	0.5 - 0.8	48	3.9
GS04-418	0.5-0.8	48	3.9
GS00-455, 00/1043	1.7-2.1	48.6	1.9
GS00-502,00/1082	1.4-1.6	49	1.9
GS11-1471/11110313	2.5-2.95	49.6	2.7
GS99/208.F	NA	50.8	1.4
GS99-208	NA	50.8	1.4
GS00-458, 00/1046	0.8-1.3	52	2.7
GS00-495,00/1075	0.3-0.6	53.8	1
GS07-630	0.9	58	4.8
GS00-496,00/1076	1.7-2.0	58.8	1
GS99/207.F/99-207	NA	59	1.7
GS99-207	NA	59	1.7
GS01-212RA	NA	60.8	1
GS01-212RB	NA	60.8	1.4
GS04-420	1.7-2.0	61.6	2.6
GS00-501,00/1081	0.7-1.0	61.8	3.8
GSO4/420	1.7 - 2.0	62	2.6
GS11-1353/CLON110772	0-1.3	63.4	1.4
GS11-1354/CLON110773	1.3-2.5	64.6	2
GS07-536	0.5-0.9	67	3.5
GS01/396	3.0-3.3	70.2	2.2
603224/GS06-089/GS06-100	0-0.5	71	3.7
BS99-0757	0.7-2.1	71.4	4.5
Performed by consultants	4.0 - 4.4	71.8	4.7
GS01-211RA	NA	72.2	1.7
GS01-211RB	NA	72.2	2.4

GS00-499,00/1079	1.7-2.0	74.6	4.7
GS00-447	0-1.3	74.8	4.2
GS08-434	1.5	76	5.1
GS06-789	1.0-1.45	77	5.5
GS00-498,00/1078	0.65-1.1	77.4	4.5
GS00-444	1.3-2.1	78.8	5
GS00-442, 00/964	0-0.4	79.6	6.1
GS10-078/079	0.2-0.9	80	4.8
GS10-078/79	0.2-0.9	80	4.8
Performed by consultants	2.0 -2.4	80.2	5.8
GS00-443, 00/965	0.4-1.4	81	5.8
GS00-497,00/1077	0.25-0.65	81.6	3.7
GS07-528/529	3.0-3.9	83	4.5
GS07-538/539	1.5-1.9	84	2.6
603552/GS06-0110	1.0-1.4	88	5.1
GS06-798/836/837	1.0-1.45	90	2.7
GS10-072/129	0.3-3.1	95	4.9
GS10-129/072	0.3-3.1	95	4.9
603554/GS06-0113	1.5-1.9	100	7.4
GS06-809	3-3.45	105	6
GS06-793/809	2.5-2.9	105	9
Performed by consultants	NA	107	5.6

Tabulated Data for Correlation between Plasticity Index, Linear Shrinkage
and Shrink Swell Index

Sample/Article Number	Depth (m)	PI (%)	LS (%)	Iss %/PF
GS00-447	0-1.3	45.8	18	4.2
GS00-444	1.3-2.1	43.8	19.2	5
GS00-094.B	0-0.4	19.8	12.8	3
GS00-096.B	1.0-1.4	19.8	12.8	1
BS00-1291/GS00-504	0-0.5	29.4	14.2	0.2
BS00-1292/GS00-505	0.5-1.0	21.2	9.8	0.3
BS00-1295/GS00-511	0.5-1.0	31.1	12.6	3
Performed by consultants	0.25-.7	26.6	15.8	1.7
Performed by consultants	0.7-2.1	18	10	0.8
Performed by consultants	0-1.2	14.2	8.6	0.4
Performed by consultants	0-1.2	13.8	8.2	0.9
Performed by consultants	0-2.0	26.8	8.6	0.1
Performed by consultants	0-2.0	15.4	9	0.6
Performed by consultants	0-2.0	15.6	8.6	0.7
BS99-0757	0.7-2.1	49.2	20.4	4.5
GS00-456, 00/1044	0.2-0.5	26.6	15	2.9
GS00-457, 00/1045	0.5-0.8	19.2	10.6	3.7
GS00-458, 00/1046	0.8-1.3	32.8	14.8	2.7
GS00-454, 00/1042	0.3-0.6	23	13.2	1.6
GS00-455, 00/1043	1.7-2.1	28.6	15.4	1.9
GS00-442, 00/964	0-0.4	44.8	20	6.1
GS00-443, 00/965	0.4-1.4	49.4	18.8	5.8
GS00-495,00/1075	0.3-0.6	24	13.2	1
GS00-496,00/1076	1.7-2.0	28	13.4	1
GS00-497,00/1077	0.25-0.65	43	16.2	3.7
GS00-498,00/1078	0.65-1.1	48	19	4.5
GS00-499,00/1079	1.7-2.0	41.2	19	4.7
GS00-500,00/1080	0.25-0.55	18.4	10.4	0.1
GS00-501,00/1081	0.7-1.0	33	13.2	3.8
GS00-502,00/1082	1.4-1.6	19.2	9.6	1.9
GS99/207.F/99-207	NA	30.6	15	1.7
GS99/208.F	NA	23.8	14.2	1.4
Performed by consultants	NA	64	25.2	5.6
GS01/395	0-1.7	25.2	12.2	1
GS01/396	3.0-3.3	46.4	15.8	2.2

GS01/397	1.0-1.45	31	13.2	1.4
GSO4/418	0.5 - 0.8	28.8	16.8	3.9
GSO4/420	1.7 - 2.0	37.6	19.4	2.6
Performed by consultants	2.0 -2.4	45.0	20.2	5.8
Performed by consultants	4.0 - 4.4	36.0	16.2	4.7
GS04-418	0.5-0.8	28	16.8	3.9
GS04-420	1.7-2.0	37.6	19.4	2.6
GS08-434	1.5	46	16	5.1
GS08-438	0.5	24	11.5	2.1
GS11-1353/CLON110772	0-1.3	31.4	16.2	1.4
GS11-1354/CLON110773	1.3-2.5	34.6	16.4	2
GS11-1471/11110313	2.5-2.95	23.6	6.6	2.7
GS99-207	NA	30.6	15	1.7
GS99-208	NA	23.8	14.2	1.4
GS06-061/062	0.5-0.9	40.4	17.6	7.4
603224/GS06-089/GS06-100	0-0.5	36	21	3.7
603228/GS06-093/GS06-102	1	89	28	5.6
603552/GS06-0110	1.0-1.4	52	21.5	5.1
603554/GS06-0113	1.5-1.9	70	27	7.4
GS07-525/526	1.5-2.9	65	25	1.2
GS07-528/529	3.0-3.9	58	23.5	4.5
GS07-536	0.5-0.9	39	19	3.5
GS07-538/539	1.5-1.9	52	21.5	2.6
GS10-072/129	0.3-3.1	70	23	4.9
GS10-078/079	0.2-0.9	56	22	4.8
GS01-212RA	NA	32.4	16.4	1
GS01-212RB	NA	32.4	16.4	1.4
GS01-211RA	NA	42.8	17.2	1.7
GS01-211RB	NA	42.8	17.2	2.4
GS07-630	0.9	36.8	15	4.8
GS06-809	3-3.45	69	20	6
GS06-789	1.0-1.45	53	18.5	5.5
GS06-793/809	2.5-2.9	69	20	9
GS06-798/836/837	1.0-1.45	57	23	2.7
GS10-129/072	0.3-3.1	70	23	4.9
GS10-078/79	0.2-0.9	56	22	4.8

Tabulated Data for the Correlation of Weighted Plasticity Index, Weighted Linear Shrinkage and Shrink Swell Index

Sample/Article Number	Depth (m)	WLS	WPI	Iss %/PF
GS00-447	0-1.3	680	1196	0.1
GS00-444	1.3-2.1	1416	2931	0.2
BS00-1291/GS00-504	0-0.5	979	2118	0.3
BS00-1292/GS00-505	0.5-1.0	851	1406	0.4
BS00-1295/GS00-511	0.5-1.0	792	1355	0.6
Performed by consultants	0.25-.7	765	1388	0.7
Performed by consultants	0.7-2.1	300	540	0.8
Performed by consultants	0-1.2	664	1118	0.9
Performed by consultants	0-1.2	1022	1858	1
Performed by consultants	0-2.0	1266	2632	1
Performed by consultants	0-2.0	1046	2160	1
BS99-0757	0.7-2.1	1443.2	2851.2	1
GS00-457, 00/1045	0.5-0.8	1323	2218	1.4
GS00-458, 00/1046	0.8-1.3	1102	2588	1.4
GS00-454, 00/1042	0.3-0.6	993	1925	1.4
GS00-455, 00/1043	1.7-2.1	1323	2218	1.4
GS00-442, 00/964	0-0.4	1443.2	2851.2	1.4
GS00-443, 00/965	0.4-1.4	1292	2252	1.6
GS00-495,00/1075	0.3-0.6	1169	1969	1.7
GS00-496,00/1076	1.7-2.0	1338	2730	1.7
GS00-497,00/1077	0.25-0.65	1338	2730	1.7
GS00-498,00/1078	0.65-1.1	1462	3638	1.7
GS00-499,00/1079	1.7-2.0	1525	2831	1.9
GS00-500,00/1080	0.25-0.55	902	1805	1.9
GS00-501,00/1081	0.7-1.0	850	1792	2
GS00-502,00/1082	1.4-1.6	966	2016	2.1
GS99/207.F/99-207	NA	1128	3313	2.2
GS99/208.F	NA	1462	3638	2.4
GS01/395	0-1.7	1707	3309	2.6
GS01/396	3.0-3.3	1707.2	3308.8	2.6
GS01/397	1.0-1.45	1372	3041	2.7
GSO4/418	0.5 - 0.8	614	2195	2.7
GSO4/420	1.7 - 2.0	2300	5700	2.7
Performed by consultants	2.0 -2.4	1256	2226	2.9
Performed by consultants	4.0 - 4.4	1259	3117	3

Performed by consultants	0.0 - 2.5	1672	3432	3.5
Performed by consultants	NA	957	1734	3.7
GS04-418	0.5-0.8	1503	3999	3.7
GS04-420	1.7-2.0	1869	3204	3.7
GS08-434	1.5	1224	3059	3.8
GS08-438	0.5	1596	2736	3.9
GS11-1353/CLON110772	0-1.3	1562.4	2604	3.9
GS11-1354/CLON110773	1.3-2.5	1669	4246	4.2
GS11-1471/11110313	2.5-2.95	1975	4782	4.5
GS99-207	NA	1737	4387	4.5
GS99-208	NA	2350	5800	4.5
GS06-061/062	0.5-0.9	1765	3827	4.7
603224/GS06-089/GS06-100	0-0.5	1569	3486	4.7
603228/GS06-093/GS06-102	1	1980	5040	4.8
603552/GS06-0110	1.0-1.4	1980	5040	4.8
603554/GS06-0113	1.5-1.9	2277	6930	4.9
GS07-525/526	1.5-2.9	2277	6930	4.9
GS07-528/529	3.0-3.9	1910	4358	5
GS07-536	0.5-0.9	1584	4550	5.1
GS10-072/129	0.3-3.1	2043	4940	5.1
GS10-078/079	0.2-0.9	1813	5194	5.5
GS01-212RA	NA	2800	8900	5.6
GS01-212RB	NA	1747	4589	5.8
GS01-211RA	NA	1946	4335	5.8
GS01-211RB	NA	2000	6900	6
GS06-809	3-3.45	1928	4319	6.1
GS06-789	1.0-1.45	1600	3500	7.3
GS06-793/809	2.5-2.9	1600	3500	7.4
GS06-798/836/837	1.0-1.45	1760	4040	7.4
GS10-129/072	0.3-3.1	2565	6650	7.4
GS10-078/79	0.2-0.9	2000	6900	9

Tabulated Data for Correlation of CBR Swell and Shrink Swell Index

Sample/Article Number	Depth (m)	CBR Swell at OMC (%) (10 Day Soak)	CBR Swell at OMC (%) (4 Day Soak)	Iss %/PF
GS00-447	0-1.3		6.7	4.2
GS00-444	1.3-2.1		8.8	5
BS99-0757	0.7-2.1		5.4	4.5
GS00-456, 00/1044	0.2-0.5		4.3	2.9
GS00-457, 00/1045	0.5-0.8		3.7	3.7
GS00-454, 00/1042	0.3-0.6		1.8	1.6
GS00-455, 00/1043	1.7-2.1		3.1	1.9
GS00-442, 00/964	0-0.4		7.8	6.1
GS00-443, 00/965	0.4-1.4		6.5	5.8
GS00-495,00/1075	0.3-0.6		1	1
GS00-496,00/1076	1.7-2.0		1	1
GS00-497,00/1077	0.25-0.65		6.75	3.7
GS00-498,00/1078	0.65-1.1		4.55	4.5
GS00-501,00/1081	0.7-1.0		1.2	3.8
GS00-502,00/1082	1.4-1.6		0.5	1.9
GS99/207.F/99-207	NA		3.4	1.7
GS99/208.F	NA		1.6	1.4
GS08-434	1.5	4	**4**	5.1
GS08-438	0.5	3	**3**	2.1
GS99-207	NA		3.4	1.7
GS99-208	NA		1.6	1.4
GS10-072/129	0.3-3.1	7.9	**7.9**	4.9
GS10-078/079	0.2-0.9	7.3	**7.3**	4.8
GS06-798/836/837	1.0-1.45	0.6	**0.6**	2.7
GS10-129/072	0.3-3.1	7.9	**7.9**	4.9
GS10-078/79	0.2-0.9	7.3	**7.3**	4.8

Tabulated Data for the Correlation of Soil Suction and Shrink Swell Index

Sample/Article Number	Depth (m)	Soil Suction (Matrix) Filter Paper (kPa)	Iss %/PF
GS04-418	0.5-0.8	2668	3.9
GS04-420	1.7-2.0	3354	2.6
GS07-525/526	1.5-2.9	1235	1.2
GS07-528/529	3.0-3.9	1101	4.5
GS06-809	3-3.45	77.4	6.0
GS06-789	1.0-1.45	4.0	5.5
GS06-793/809	2.5-2.9	195.9	9.0
GS06-798/836/837	1.0-1.45	195.9	2.7

Tabulated Data for the Correlation of Swelling Pressure and Shrink Swell Index

Sample/Article Number	Depth (m)	Swelling Pressure (kPa)	Iss %/PF
GS99/207.F/99-207	NA	109.2	1.7
GS99/208.F	NA	113.9	1.4
GS99-421	NA	615	3.2
GS04-418	0.5-0.8	250	3.9
GS04-420	1.7-2.0	320	2.6
GS08-434	1.5	140	5.1
GS11-1353/CLON110772	0-1.3	52	1.4
GS11-1354/CLON110773	1.3-2.5	49	2
GS11-1471/11110313	2.5-2.95	105	2.7
GS99-207	NA	109	1.7
GS99-208	NA	114	1.4
GS07-525/526	1.5-2.9	120	1.2
GS07-528/529	3.0-3.9	230	4.5
GS07-536	0.5-0.9	120	3.5
GS07-538/539	1.5-1.9	110	2.6
GS10-072/129	0.3-3.1	450	4.9
GS10-078/079	0.2-0.9	300	4.8
GS10-129/072	0.3-3.1	450	4.9
GS10-078/79	0.2-0.9	300	4.8

Tabulated Data for Correlation of CBR Swell/LS Ratio and Shrink Swell Index

Sample/Article Number	LS (%)	CBR Swell at OMC (%) (4&10 Day Soak)	CBR Swell/LS	Iss %/PF
GS00-495,00/1075	13.2	1	0.08	1
GS00-496,00/1076	13.4	1	0.07	1
GS99/208.F	14.2	1.6	0.11	1.4
GS99-208	14.2	1.6	0.11	1.4
GS01-212RB	16.4	1.9	0.12	1.4
GS00-454, 00/1042	13.2	1.8	0.14	1.6
GS99/207.F/99-207	15	3.4	0.23	1.7
GS99-207	15	3.4	0.23	1.7
GS00-455, 00/1043	15.4	3.1	0.20	1.9
GS00-502,00/1082	9.6	0.5	0.05	1.9
GS08-438	11.5	3	0.26	2.1
GS01-211RB	17.2	2.1	0.12	2.4
GS00-456, 00/1044	15	4.3	0.29	2.9
GS00-457, 00/1045	10.6	3.7	0.35	3.7
GS00-497,00/1077	16.2	6.75	0.42	3.7
GS00-501,00/1081	13.2	1.2	0.09	3.8
GS00-447	18	6.7	0.37	4.2
BS99-0757	20.4	5.4	0.26	4.5
GS00-498,00/1078	19	4.55	0.24	4.5
GS10-078/079	22	7.3	0.33	4.8
GS10-078/79	22	7.3	0.33	4.8
GS10-072/129	23	7.9	0.34	4.9
GS10-129/072	23	7.9	0.34	4.9
GS00-444	19.2	8.8	0.46	5
GS08-434	16	4	0.25	5.1
GS00-443, 00/965	18.8	6.5	0.35	5.8
GS00-442, 00/964	20	7.8	0.39	6.1

Appendix C Project Aims and Objectives

The aim of this project is to identify and quantify the relationships that exist between key measures of reactive soil behaviour. A study of the existing methods used for identifying and characterising reactive soils within the field of road construction, will be conducted. Historical data gathered from investigations and testing undertaken since 2000, by the Department of Transport and Main Roads, Queensland, and other local road authorities, will be used to carry out parametric studies. Any linkages or correlations that may exist between the key indicators of reactive soil behaviour will be identified and quantified for future use as guides in the design of roads using these materials.

The objectives of this project can be summarised as follows:

- To understand the nature of expansive soils and their properties
- To identify by research, the key measures of reactive soil behaviour used by engineers in road construction.
- To investigate and quantify any relationships that may exist between the key measures.
- To establish a ranking of reliability for the key measures of reactive soil behaviour.
- To develop guidelines to assist road designers in the identification and characterisation of reactive soils during the site investigation phase of road construction projects.
- To develop a simple model to estimate the loads and pressures required to suppress volume change within an expansive soil foundation or embankment

Appendix D Project Methodology

Project Methodology The specific objectives as listed earlier in Appendix 2 will now be considered in terms of how each task will be undertaken, the tasks involved in each project and why the decision was made to carry out the work this way.

To Understand the Nature and Properties of Expansive Soils. This will be achieved by conducting a literature review of both print and web-based sources, from Australia and Overseas. This will ensure that a wide variety of information is obtained.

Within Australia and internationally, the subject of reactive soils, their effect on the built environment is well documented. Along with Australia, countries such as South Africa and North America have large zones of arid and semi arid land, where generally, these soils are at their most troublesome.

Researching the background information on reactive soils will help to give the author an understanding of their behaviour.

To Identify By Research The Key Measures Of Reactive Soil Behaviour. This will also be achieved by research well by reviewing the methods that have been adopted, by road engineers within Queensland and overseas, to identify and characterise expansive soils.

This information should provide guidance on the effectiveness of each method which will allow the author to rank the different methods and determine which method will be the most effective tool for use in the investigation and design processes for the road engineer.

Local shires and engineering bodies will also be investigated to determine what investigation tools have been adopted for their site investigation activities. By identifying

parameters that provide the most optimum characterisation of these soils, more cost effective testing and investigation programs can be implemented. This is a particularly worthwhile goal in the current state and national economic environments.

Investigate And Quantify Any Relationships That May Exist Between The Key Measures. This will be done by gathering data from test results obtained from site investigations carried out for state and local shire road projects. Test data will be sourced from Transport and Main Roads soil laboratories or Main Roads databases, as well as other publically available information from local shire authorities. The data will be analysed to determine whether any significant relationships exist between the various key parameters.

Establish A Ranking Of Reliability For The Key Measures Of Reactive Soil Behaviour. By analysing the outcomes of the data analysis, a review of each method will be conducted and a ranking will be given to each test method in terms of the meaningfulness of the reported values, the time required for the test, the ability of industry to perform the test and the skill of the operator required to perform the test.

Develop Guidelines To Assist Engineers In Conducting Site Investigations The intent of this objective is to provide a list of the minimum testing requirements to conduct an investigation to characterise the expansive properties of a site. In reference to the Australian Standard for the Residential Slabs and Footings AS2870, complementary information can be developed based on any identified relationships between parameters as well as the ranking score, which will assist the engineer in customising a testing program for site investigations to manage risk and provide economy.

Present project results in the required oral and written formats

This project as part of the course ENG 4112 will be submitted as a written dissertation and also presented at the University of Southern Queensland's 2013 Professional Practice Seminar.

Appendix E Assessment of Consequential Effects

Environmental, Sustainability and Consequential Safety Effects This project is being undertaken with the support of The Department of Transport Main Roads and it is the intent to develop guidelines for the improved identification and characterisation of reactive soil sites.

In this way, this project aims to minimise the effects of reactive soils, and therefore reduce the level of failures of roads constructed on these problematic materials. This will result in more efficient use of financial and natural resources, by reducing the need for maintenance works and rehabilitation. As a consequence, it is intended that any decrease in maintenance and rework will increase the sustainability of infrastructure in the future.

The main activities for this project involve information and data gathering, which are considered to represent no environmental risk.

Immediate safety concerns have been addressed in Appendix C. Identified risks are and will be managed and control methods implemented.

Ethical Effects The ethical responsibility for the work is fully accepted by the author and at no time will the author compromise the tenets of the Engineers Australia Code of Ethics, as detailed below:

Source: http://www.engineersaustralia.org.au. Code of Ethics 2010.

"As engineering practitioners, we use our knowledge and skills for the benefit of the community to create engineering solutions for a sustainable future. In doing so, we strive to serve the community ahead of other personal or sectional interests."

Improved understanding of expansive soils and their properties will be to the benefit of the community. It will enable to road planners and designers to make more informed decisions on the best solutions for the construction of roads on problematic materials.

"Demonstrate integrity"

Publication of the project will promote the honour, integrity and dignity of the engineering profession.

"Practise competently"

The author's area of education and current vocational specialisation is in the area civil engineering including high focus on construction materials and geotechnical testing. Careful consideration and diligence will be sought throughout the duration of the project. Professional advice from members with greater competence will be sought in situations when specialised skills or knowledge are required.

"Exercise leadership"

The author will ensure all actions and results throughout the project are fair, honest and in the best interest of the community, client, employers and colleagues.

"Promote sustainability"

Development and delivery of the project is supported by Department of Transport and Main Roads, which has protocol and guidelines which ensure that all work completed is in the best interest of the state of Queensland. As an employee of this organisation, the author has undertaken appropriate workplace training and has the awareness of the Department's focus on provide the best service for the community, now and for future generations.

Appendix F Risk Assessment

Risk Assessment In all projects it is essential that all risks are identified, assessed and controlled. This is done by safe workplace procedures and a controlled work environment.

The Project has the major tasks involved:

1. Office Work Research

2. Office Work Reporting

A summary of the Hazard Identification, Potential Risks and Control Methods is presented in Table A6-1.

Specific Activity	Hazard and Risks	C	L	Risk Level	Controls/treatments	C	L	Risk Level
Research and Reporting	Sitting at a desk for prolonged periods – potential back and neck injuries	Mod	P	High	Observe correct posture Use lumbar support on chair Take regular breaks incorporating stretches and standing	Mod	U	Medium
Research and Reporting	Viewing a computer screen for prolonged periods – potential eye strain and headaches	Mod	P	High	Limit computer use to no more than 1 hour at a time. Ensure adequate breaks are taken away from the computer (at least 20 mins) During periods of use, focus to infinity at regular intervals	Mod	U	Medium

C = Consequence. L = Likelihood

Risk Matrix					
Likelihood	**Consequence**				
	Insignificant	*Minor*	*Moderate*	*Major*	*Severe*
Almost certain	MEDIUM	HIGH	HIGH	EXTREME	EXTREME
Likely	MEDIUM	MEDIUM	HIGH	HIGH	EXTREME
Possible	LOW	MEDIUM	HIGH	HIGH	HIGH
Unlikely	LOW	LOW	MEDIUM	MEDIUM	HIGH
Rare	LOW	LOW	MEDIUM	MEDIUM	HIGH

Table F-1 Risk Assessment Matrix.

Appendix G Resource Analysis

Resource Analysis

To efficiently complete a project it is essential that prior investigations into the availability of resources, costs involved in using these resources and feasibility of these resources are completed before the project can be commenced. There are very few resources needed to complete this project, however the ones that are required are critical items and without them the project would not be able to be completed. All items can be sourced readily by the author, or from Department of Transport and Main Roads. The timing of external resource dependant tasks can be managed to negate any impact on the delivery time of the project.

The resources likely to be required are shown in Table A6 below:

Resource	Possible Problems	Control Measures
Transport and Main Roads Library and technical reports archive	Documents unavailable or delays in delivery	Ensure adequate lead times in making library borrowing requests
Reports and test data from Local Authorities	Unavailability of key staff contacts and not able to obtain permission to access data	Alternative sources of data may need to be found.
Computer Printing and Internet	Some restrictions on access at TMR.	Some minor printing costs may be required. Most internet access will be from private account – not likely to cause delays

Table G-1 Resources Required

Appendix H Project Timeline

Task	Description	Predecessors	Resources	Start	End	Mar	April	May	Jun	Jul	Aug	Sep	Oct
1	Specification	-	PR	Mar	Mar	█							
2	Background	1	PR, TMR	Apr	May		█	█					
3	Literature Review	1	PR, TMR, Other	Apr	May		█	█					
4	Appreciation	3	PR, TRM, Other	May	May			█					
5	Data Collection	3	PR	Jun	Jul				█	█			
6	Data Analysis	5	PR	Jul	Aug					█	█		
7	Review of Results	5,6	PR	Aug	Aug						█		
8	Develop Guidelines	3,5,6	PR	Aug	Aug						█		
9	Conclusions	3,7,8	PR	Sep	Sep							█	
10	Reporting	6,7,8,9	PR	Sep	Oct							█	█

PR – Peter Reynolds, TMR – Transport and Main Roads, Other – Local Road Authorities.

Table H-1 – Project Timeline