University of Southern Queensland

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Studying the effects of non-uniform stress distributions on soil heave

A dissertation submitted by

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Abstract

Soil movement is a leading cause of structural damage annually globally and is constantly rising. The main cause of this movement is through swell from reactive soils by changes in moisture content. All soils experience a volume change when subjected to moisture changes, the severity of these volume changes depends on many factors relative to the soil. The severity of these changes are most commonly measured using Atterberg limits and the plastic index but this only gives a magnitude of possible swell and alone is not a reliable indicator as it can produce false negatives.

More reliable tests for swell are generally more involved with time and resources and this increases if predictions of swell for site conditions are required. Most of these methods also make a number of assumptions and simplifications but also generally fail to accurately predict actual soil swells. Accuracy can generally be improved by increasing the number of tests therefore decreasing some assumptions and generalisations but in practice this is difficult to do. For this reason most research and development in this area has focused on testing a wider range of soils and comparing to known correlations or applying easily tested parameters to simplified swell models but no significant gain in prediction accuracy has been achieved in over 30 years.

This study explores the behaviour of soil swell when lateral stain and movement of soil is not restrained aver a non-uniform stress distribution. Behaviour, parameters and predictions from these test are compared with those from standard swell overburden oedometer testing. It was found that the swell behaviour of the soil under a load that has unrestrained lateral movement from under a loaded area to an unloaded area was significantly different to those experience in oedometer and triaxial testing. Significant focus is placed on the movement of soil from under the loaded area and the problems this may cause with current prediction models.

From the results obtained in this study it is possible that at least under certain condition soil does not exhibit behaviour that current prediction models are based upon and the assumptions they use. Recommendation of further study into changes in soil behaviour under a range of different conditions is made. More research efforts should be made on complex modelling of soil movement especially in areas of high damage rates for simulation purposes and design improvements.

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.

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David Petersen

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Nomencleture

Δm	=	Change in mass under loaded area, (% relative to initial mass)
ρ	=	Moist sample density, (kg/m^3)
ρ_d	=	Dry sample density, (kg/m^3)
σ	=	Applied load, (kPa)
S_i	=	Initial sample saturation
S_f	=	Final sample saturation
d_1	=	Diameter of loaded area, (mm)
d_2	=	Diameter of sample, (mm)
e	=	Sample void ratio
e_f	=	Final sample void ratio
e_i	=	Initial sample void ratio
e_{M^*}	=	Assumed sample void ratio at maximum swell
h	=	Height of sample, (mm)
h_i	=	Height of sample just before water addition, (mm)
h_s	=	Height of prepared sample, (mm)
m_{df}	=	Measured final dry mass under load, (g)
m_{di}	=	Initial dry mass under load, (g)
m_f	=	Measured final mass under load, (g)
m_i	=	Initial mass under load, (g)
t	=	Time since addition of water to sample, (min)

Chapter 1

Introduction

1.1 Outline of the Research

For this study the assumption used in one of the most common direct tests was investigated. Oedometer tests are used to measure one-dimensional vertical displacement and pressure of a soil sample allowed free access to water. When these values are used in models they often grossly overestimate vertical displacement and need to be factored. Al-Shamrani & Dhowian (2003) investigated including a lateral restraint factor to predictions made from oedometer tests and found that applying this factor gave mostly satisfactory results compared to field measurements. This requires triaxial test data for the soil sample or an assumption of the factor value. If triaxial data is available then oedometer tests are redundant and it was found that the factor value changes with depth although not significantly. This value also changes based on the oedometer test used and is not consistent for all soil types.

Performing two-dimensional analysis using an oedometer, the effects of non-uniform loading on swell prediction was compared to standard one-dimensional test results. The two parameters of interest from these tests are the swell index and maximum swell pressure. The free swell value can also be determined from the tests and is used for comparison but is not used in the selected prediction model. The most extreme non-uniform case of loaded area and area with no load was tested and is where it was believed that lateral displacement of soil is at a maximum.

It was the intention to identify areas where current models or test can be changed to include more parameters without significantly increasing time or difficulty to performing the tests and models. The parameter of focus was lateral restraint or pressure and the effects on a soil sample under a loaded area. The effects were only studied for a single soil sample under a single set of initial conditions.

1.2 Background information

Soil heave is a major contributor of damage caused to structurers every year in many locations worldwide. In 1973 it was reported that \$2.3 billion of damage was caused by expansive soils annually (Chen 1988) in USA and in 2006 it was estimated that the cost of repair to damage was \$10 billion (Phanikumar 2006). This makes expansive soil a more costly destructive event than any other natural disaster annually. With the ever increasing demand for structures this cost is expected to continue rising unless a more accurate understanding of heave can be obtained so suitable construction designs can be developed.

Heave or shrinkage is the result of a change in moisture content in the soil The amount of swell or shrinkage is not the same for all soils and will vary for different initial conditions. There are many different methods currently available for use to predict the swell of a soil. Each of these methods have limitations to availability and accuracy and so no single method is regarded generally more suitable for use.

Because of the number of parameters available in soil testing there are also numerous different models available to predict the swell potential of soils. Some use correlations found from parameters that are relatively quick and easy to test for and measured heave. These methods are referred to as indirect. Other methods use swelling measured under test conditions or the measurement of parameters directly related to swelling like soil suction to predict the swell potential. The reliability of predictions made from these models depends both on the type of model selected and the tests used to obtain the required parameters for the model.

Cost, time and resource requirements of performing some of these tests can be excessive to produce results that are reliably accurate. Because of this quicker tests are generally used to give an indication of the magnitude of swelling potential in favour of more accurate modelling. Quicker and cheaper tests are generally more favourable because in many cases the value of the structure being constructed is not significant enough to warrant a precise knowledge of soil movement and critical failure can be avoided by knowing an estimate of potential.

Assumptions are also made to both reduce difficulty and time associated with testing. The homogeneity of a layer of soil is most often assumed to reduce the need of testing multiple heights of a single core sample or increasing the number of core samples required within a site. The condition of the soil being constructed on is the same as what was tested is another assumption generally made that is not the case. Though some of the assumptions made save required resources they may result in inadequate solutions that do not stop damage occurring.

Increasing or assessing the ability of models to accurately predict changes in vertical displacement has been the focus of many recent research articles. This is coupled with research on the methods used to test parameters for use in models. Difficulty with many current testing methods and models to accurately make predictions can be attributed to the many different types of expansive soils and that they do not behave the same way under the same conditions. Many studies have found factors or correlations that apply to a single soil or small group of soils. This leads to inaccuracy of predictions by using models or assumptions that may not relate to the soil that was tested that may have been right for a different soil.

1.3 Soil behaviour

The key soil behaviour being explored is related to changes in moisture content within the soil. All soils undergo volume changes with changes in moisture content and the magnitude depends on a number of factors. Currently only a small number of these factors are considered in any one method used to describe expansiveness of a soil. These limited approaches are used to simplify methods used to classify soils but leave the behaviour of a soil undergoing moisture changes subject to a large number of unknowns. These unknowns in how a soil will behaviour under certain conditions greatly hinder the accuracy of predicted parameters needed in construction.

Another important measure of a soils behaviour is the moisture content the physical behaviour changes. Soils experience three physical states depending on moisture content which are solid, plastic and liquid. A standard measure of a soils expansiveness is the plastic index which is the range of moisture content where a soil will exhibit plastic behaviour. This relates to the amount of water a soil can absorb (Chen 1988) though is not always reliable or an accurate measure.

In general an increase in moisture content will result in a swell or heaving of soil while reduction results in shrinkage for consolidated clays. The two key measurements of this behaviour usually sought for construction purposes are vertical displacement and swelling pressure. In almost all cases the lateral properties and movement of soils is not considered. Very little is considered in the way of behaviour of soil at lateral boundaries of changes in surcharge pressures and the effects this has on parameters of surrounding soils.

1.4 Scope and limitations

Because the aim of the research is to only indicate possible effects of non-uniform loading on the vertical displacement of a loaded area and only a single scenario was tested with a single clay sample, no general relationship can be derived from the study. It was the intention that results could indicate the need for further research in specific areas to improve the accuracy soil swell modelling using less resource intensive testing methods. The testing procedure used to produce these results is not a standard accepted test and was developed for the use in this research and would require further development to both improve accuracy of results and increase the range of scenarios that can be tested. Recommendations made in study were based on results of this single comparison with support of accepted theories and require further investigation and validation.

1.5 Overview of chapters

This dissertation contains information on the design of testing procedure, testing data and analysis of oedometer swell overburden tests on a soil sample. The contents of this dissertation are; an introduction on the background of the study, literature review of past and recent methods used in soil prediction, the methodology adopted for testing and prediction, data collection from testing, analysis of raw data and prediction results and conclusion of findings. A brief outline of each chapter is included below.

1.5.1 Chapter 1 - Introduction

This chapter outlines the direction of research and gives a brief background to why this research is required. The scope and limitations of the project are detailed along with an overview of the dissertation structure with chapter summary.

1.5.2 Chapter 2 - Literature review

A comprehensive review of a number of testing methods used to gather parameters to predict swell and some of the different models that are used. Current research developments are also reviewed with a focus on lateral restraint and strain of reactive soils along with new model developments.

1.5.3 Chapter 3 - Methodology

The approach used for testing and prediction modelling is used is explained. Testing equipment is detailed including design of custom equipment. Techniques used to analyse and compare planned data is outlined based on research goals and expectations.

1.5.4 Chapter 4 - Test results and observations

Details of tests performed and the data collected is presented along with any observations during the test. Initial conditions of samples and points of interest in the recorded data are noted.

1.5.5 Chapter 5 - Analysis

A detailed analysis of results is by comparing the raw data collected, extrapolated parameters and model predictions using the extrapolated parameters. Data is also assessed for consistency between tests to check for abnormalities in results that may change results.

1.5.6 Chapter 6 - Discussion

Results from the analysis are investigated and relationships between differences observed in the tests related to relevant theory or expectations. Differences between observations of approximated factors and assumptions used in other studies to establish agreement or need for further investigation.

1.5.7 Chapter 7 - Conclusion

Based on the available data conclusions were drawn about what other parameters should be considered to improve swell prediction models and about how effective modified oedometer tests could be in facilitating this. The possible long term effects of lateral movement behaviour of soil could have is also summarised.

Chapter 2

Literature review

Over multiple decades engineers have been developing models to predict the behaviour of reactive soils during periods of moisture change. Particular interest in the construction sector is the vertical displacement during hydration and the pressures associated with this change. Over this time there have been many theories about why such large volume changes occur during hydration leading to a number of different models being developed that require a particular set of parameters believed to describe the vertical strain. Some of these models have been shown to perform to a high degree of accuracy when extensive tests are performed to obtain the most accurate parameters, however time and resources are generally not available to test to such high standards.

2.1 Swell estimation methods

There are two branches for estimating swell, indirect and direct. These methods generally differ in the level of precision they provide, universal applicability and cost to perform. The method used greatly depends on the availability of a suitable model, cost benefit of analysis or required standards but there are fundamental difference in the two categories that influence the use of them in prediction.

2.1.1 Indirect swell estimation

Indirect methods are usually the quicker but less precise way to estimate swell potential. Soil properties such as chemical composition and Atterberg limits are used to give an estimation of swell percentage based on swell from soils with correlating attributes. Because of this, indirect methods are tailored to specific locations or soil types due to large variability of soils between geographic locations (Al-Shamrani & Dhowian 2003). Therefore to achieve greater levels of accuracy and certainty in predictions using indirect methods a procedure based on relevant soil properties needs to be developed for the soil type, taking account of variations in the area. This still means predictions are based on an averaged result which can be prone to outliers with no indication until structural failure. There is also no comprehensive database of soils properties and their related swell to be able to draw wide scale correlations in general or for soil groups.

The presence of outliers within indirect models has been seen in many studies before. Al-Shamrani & Dhowian (2003) tested a soil that based upon mineral composition should have low swell values but the soil properties indicated large swell potential. This is in conflict with the generalisation that Yitagesu, van der Meer, van der Werff & Zigterman (2009) found a high correlation between soil spectrometer analysis and properties indicating swell potential. This shows that general correlations can still have gross estimation errors if not properly checked with multiple methods and in conflicting cases may need greater investigation.

So with a combination of indirect method estimates only being accurate for the soils they were developed with and correlations not always holding for all soils, there is a vast amount of uncertainty when using these methods to predict soil swell. For smaller projects they may be a good reference point to indicate different levels of swell potential and therefore construction methods to minimise possible damage but for critical or high value structures more certainty about soil behaviour is paramount for safety and longevity of the structure.

2.1.2 Direct swell estimation

Direct methods of swell prediction are generally more complicated and take longer to test but can result in more accurate predictions without the risk of relying on correlations.

Currently though, like indirect methods there are no universally accepted models for estimating soil swell, though there are a number of tests and methods shown to give relatively accurate results. The need for more accurate, universally applicable swell test and prediction methods is constantly producing new theories on which parameters are of importance and how they relate to swell.

There are two main distinctions between types of direct methods for heave prediction. The first is through the direct measurement of one-dimensional soil swell in an oedometer during wetting of a sample and the second is through theoretical or semi theoretical equations in many different categories. The main methods of theoretical predictions according to Al-Shamrani & Dhowian (2003) and Adem & Vanapalli (2015) are consolidation theory, water content and suction based methods. Recently there has also been development in thermo-chemo-electro-mechanical framework (Lei, Wong, Fabbri, Limam & Cheng 2014, Loret, Hueckel & Gajo 2002) and in linking micro and macro structural (Mašín 2013, Alonso, Vaunat & Gens 1999) behaviour of expansive soils. Though these last two methods are mostly aimed at waste liners and have much more complex theories and testing methods they do describe the volume changes in a soil.

Despite the increased accuracy of predictions made by many of these models they usually contain assumptions and generalisations of soils. In general the simpler the tests required for the model the greater the assumptions and generalisations needed. The assumptions are consistent across almost all methods like the assumption of homogeneity within a layer, the generalisations however have been found to have less of an impact on results. The lateral restraint factor used in models making use of oedometer data to scale to triaxial data can be assumed to be one third but in reality this varies slightly for soil types and with depth of layer. This leads to a reduced degree of certainty in absence of a larger quantity or quality of data for some methods.

2.2 Prediction models

The ultimate goal and reason behind soil swell tests is to be able to predict accurately the swell that will be experienced in field conditions. Like methods of testing there are many theoretical methods of making these predictions, many of which are not limited to direct swell testing though the use of oedometers or triaxial test. Other methods use soil suction measurements or theoretical micro and macro structure analysis. Regardless of the method none have proven to be highly accurate for all cases when compared to site measurements over all depths of soil or have not been tested extensively enough to prove better fits than more commonly used models in general application.

2.2.1 Direct swell prediction models

Direct swell prediction models use parameters gathered from tests that either directly measure volume change of the soil or a variable found to directly relate to swell like suction. Presented below are a number of commonly used models with brief explanations of the required parameters and assumptions or generalisations involved to use these models.

Australian Standards

The model set out by Australian standard AS2870 is given in equation 2.1 (Standards Australia 2011) is used to predict surface soil movement by estimating the movement of soil layers within the depth of design.

$$y_s = \frac{1}{100} \sum_{n=1}^{N} \left(I_{pt} \overline{\Delta u} h \right)_n \tag{2.1}$$

where

y_s	=	characteristic surface movememnt, in mm
I_{pt}	=	instability index, in %/picofarads (pF)
$\overline{\Delta u}$	=	soil suction change averaged over the thickness of the layer under consideration (pF)
h	=	thinckness of layer under consideration, in mm
N	=	number of soil layers within the design depth of suction change

This model allows for theoretically very accurate predictions of swell given enough information. By using an infinite number of layers and having values of suction and instability index at every point it is likely that a high level of precision between estimations and observations could be achieved. It is very costly and resource intensive to take such measurements even for a single point. For this reason in the standard many general estimations and assumptions are made for all of the variables.

Because of difficulties in measuring soil suction estimations of maximum suction values are provided in the standard based on geographical location and assumed to vary linearly down to a set depth. The instability index is also difficult to measure so in absence of data this value can be substituted by a factored shrinkage index measured from shrinkage and swell tests set out by AS 1289 7.1.1. If this value is exchanged the assumption of a constant factor for the cracked zone which can be estimated by location and linear variation for the rest of the depth.

The model presented in AS 2870 can be accurate in theory with available information but it is generally unlikely that time and resources are available for such data. To allow for this the generalisation and assumptions allowed in the use of this model give a large amount of uncertainty in predictions and if the soil is an outlier on the generalisations it could lead to gross errors in prediction.

Suction and swell index

Fredlund (1995) developed a comprehensive model for predicting heave that uses data from one-dimensional swell tests and suction based tests. Compared to other models available that have been adapted or simplified this requires a larger amount of data for each soil layer. Equation 2.2 shows the model with all stress states included.

$$\Delta h_i = h_i \frac{C_s}{1+e_0} \log\left(\frac{(\sigma_v - u_a) \pm \Delta\sigma - u_{wf}}{(\sigma_v - u_a) + (u_a - u_w)_e}\right) \tag{2.2}$$

where

Δh_i	= the heave of the ith soil layer
h_i	= the height of the ith soil layer
C_s	= Swell index measured from oedometer tests
e_0	= the initial void ratio
σ_v	= original overburden pressure
u_a	= void air pressure (usually assumed to be equal to atmospheric)
u_w	= void water pressure
$(u_a - u_w)_e$	= matric suction equivalent
$\Delta\sigma$	= the change in total stress
u_{wf}	= estimated final pore water pressure

Like other models accuracy can be increased by increasing the number of layers used but the model also requires a larger number of inputs which either means more measurements need to be taken from more tests or more assumptions and generalisations need to be made. The swell index can be derived from one-dimensional swell tests, overburden pressure is based upon depth of layer and unit weight of soil and equivalent matric suction requires the conversion from a matric suction test measurement to pressure. Pore water pressure is measured based on the level of the water table is present within the layer while final pore water pressure is estimated based on expected water table levels and drainage. For precise estimation measurements need to be taken from multiple depths otherwise the matric suction profile needs to be assumed.

The contents of the logarithm can be simplified as the numerator is simply the final pressure applied to the soil layer and the denominator is the pressure exerted from swelling. This is consistent with a logical approach to the problem as equal applied pressure and swell pressure results in zero net height change, greater swelling pressure results in swell and greater applied pressure results in consolidation.

Swell index and swell pressure

Al-Shamrani & Dhowian (2003) use an equation from Johnson and Snethen that is a similar to that developed by Fredlund and uses the same principals. Equation 2.3 shows that it is of the same form as equation 2.2 except the log ratio is inversed changing the sign of the answer given. Inputs used can also be gained exclusively from one-dimensional swell tests.

$$\Delta H = \frac{C_s H}{1 + e_0} \log \left(\frac{p_s}{\sigma'_{vf}} \right) \tag{2.3}$$

where

ΔH = the heave of the soil layer

C_s = Swell index measured from oedometer tests

e_0 = the initial void ratio

p_s = the swell pressure

σ'_{vf} = final effective vertical stress

For this model only three measurements from the soil sample are required and all can be obtained during a standard oedometer swell test. Because of this the model is one of the simplest models using inputs measured directly from swell. The only input of significant difference from equation 2.2 is swell pressure is used instead of equivalent matric suction. The swell pressure is the pressure required to result in zero volume change and hence if applied pressure equals swell pressure the result of the logarithm is zero.

In studies conducted by Al-Shamrani & Dhowian (2003) it was found that for the soil studied this model gave very good fits to layers from 2 5 m using triaxial test data or oedometer data that was scaled by the average ratio between triaxial and oedometer predictions. The larger discrepancies in the top 1 meter for this case was believed to be due to an increase non expansive matter within the soil. This could also be cause by surface cracks but there was no mention of them in the study or any factoring for them. This highlights the problems with homogeneity assumptions and the factor applied to oedometer predictions was shown to be different to the generalised factor of one third and vary with depth. The field data the model was compared to was an open site so effectively uniform loading which is how samples were tested.

2.2.2 Indirect swell prediction models

Indirect or index models use soil correlations parameters not related to swell to estimate heave potential of soil. Atterberg limits are one of the more common parameters used in classifying reactivity of soil but other measurements such as relationships between dry density and moisture content have been used as well as different methods of classifying chemical composition like mineralogy or reflected spectra. Regardless of the parameters used, models are constructed by examining the values exhibited by a soil and measured swell from observations or testing and then making statistical correlations between them.

Atterberg limits

Atterberg limits are some of the easier properties of soil to measure and because transition between liquid limit and plastic limit depends on the amount of liquid a clay tries to absorb it can be related to swelling properties (Chen 1988). The plastic index is often used as one of the simple measures to indicate severity of swelling potential for a soil. Similar classifications of soil can be estimated by use of the shrinkage limit as a percentage. Though these do not give any predictions on actual displacement it is generally a good indication of what to expect from a soil in further testing if needed.

Mineralogy

There are many minerals that have been identified to be associated with high or low swell potential. By using x-ray diffraction to identify minerals in a soil sample estimations of a soils swell potential can be made. Though identifying minerals known to have large swelling potential like montmorillonite is a good indication of swelling potential of a soil the inverse is not always true. Al-Shamrani & Dhowian (2003) found that the shale being tested contained mostly kaolinite, a mineral know to be generally stable in the way of volume change but shale was found to be highly reactive. This means mineralogy is not a viable way to discount the possibility of large amounts of swell.

There are a number of other indexes that can be used like shrinkage limit and activity (Das 2010) that can be used in similar ways but all have similar problems. These methods only indicate the severity of possible swell and do not work in reverse, a value that does not indicate severe swell potential does not mean that there is no severe swell potential. These means that using any models developed by correlations of these indexes with swell behaviour holds a level of risk if predicting a soil that was not part of developing the model or any property that may cause a contradiction.

2.2.3 Thermo-chemo-eleco and micro/macro sructure models

In recent years there have been developments in models that account for changes at a particle level. These models generally try to model the changes in structure of soil particles as conditions change, like increase in moisture content.

2.2.4 Other swell prediction models

Recently Adem & Vanapalli (2015) reviewed a range of methods for predicting volume change of soil. This shows a broad range of models using different theories and technics to predict soil movement ranging from models using changes in water volume, stresses and strains in x, y and z directions to elastoplastic models making assumptions about reversible and non-reversible strains to models making use primarily of changes in water content. Some of the models presented here have been compared to observed data and cases but all at a point either over or under estimate movement even though there are points with good agreement. Of interest is also a study where four separate footings at a site were observed for movement and all had very different measurements with two different models only giving values and trends similar to the measurements. This shows the variation that can occur even within a small spatial difference and that modelling done may give good results at times for a specific place but be inaccurate at other time or places within the same area.

Only a small selection of models were presented for the purpose of indicating the breadth of models currently available. Though the large number, so far none have gained a universal acceptance so they each have limitations depending on the time and resources available for soil data analysis. Most models will give an accurate prediction for the soil layer depth the sample data was taken from even with some general factors giving small errors but using this data away from the location and depth of the sample can result in gross errors of prediction.

2.3 Soil testing Methods

There are many different types of tests that can be used to collect data for the use in prediction of soil swell since there is no universally accepted way of measuring swell. These methods often vary depending on the level of precision required or based on relevant standards for the country used in. Focusing on tests that are used to directly predict the swell of a soil there are a number of options including methods described in Australian and ASTM standards.

2.3.1 Oedometer based tests

Tests involving oedometers are most likely to be used for more accurate prediction of swell in construction because it is more accessible, cheaper and quicker than other methods like triaxial tests and suction based testing. One limitation of oedometer testing is that the soil conditions at the time of testing may not be the same at the time of construction (Day 1999). Al-Shamrani & Dhowian (2003) describe 3 common tests; free swell, constant volume and swell overburden with the swell overburden being the test that is suggest to represent light foundation sites the closest. The difficulty associated with the better representation is that multiple samples are required to be tested to acquire the needed data. Lambe & Whitman (1969) express that friction between the soil and confinement ring can have an effect on the results of tests and ASTM standard D4546-14 (2014) list that the tests only measure vertical deformation and not any lateral swell, that the swell is a representation of the most severe conditions and that any difference in chemical content of the wetting fluid could change heave or settlement. There are still many unknowns with regards to the accuracy and effectiveness to swell testing through the use of oedometers.

ASTM D4546-14 specifies three different methods of testing for one-dimensional swell or collapse depending on the type of sample available for testing. Method A is primarily used for reconstituted samples compacted to site conditions where several samples are subjected to different loads and then allowed free water. This is the method Al-Shamrani & Dhowian (2003) referred to as swell overburden. Method B is intended for use on intact soil samples from a particular depth that are loaded to match in-situ vertical stress to estimate the swell of the soil when wetted from current condition to inundation. For prediction of swell at different depths, soil samples from those depths are required and to be loaded to the estimated stress resulting in a procedure similar to method A. Method C is used to measure load induced strains on a sample that has already gone through swell or settlement. The test starts the same as either method A or B but after settlement or swell is completed the load is increased and the load induced strain measured.

2.3.2 Triaxial based tests

Triaxial tests give similar data and can be performed in the same ways as oedometer tests (free swell, constant volume and swell overburden) but they allow for lateral movement of the sample. These test however require a lot more complex and expensive equipment along with a technician trained to use such equipment. The time required to perform triaxial test are also much long than that of one dimensional test adding to the cost to achieve greater accuracy in results. The accuracy is improved because lateral pressures can be simulated to that of the soil layer desired meaning that vertical and lateral strains are that which can be expected for the same volume of soil in the field.

2.3.3 Suction based tests

Soil suction test are generally very long and difficult to perform require very precise measurements and can easily contain errors if not performed by a trained technician. Suction is measured in picofarads (pF) or negative pressure (kPa) and increases as moisture content decreases (Day 1999). It is difficult though to measure the change in suction of a soil sample with respect to moisture content changes (Al-Shamrani & Dhowian 2003) and for this reason there a soil suction index has been devised using the measurement of suction at fixed moisture contents. Australian standards and ASTM outline different ways that this can be measured. The advantages of suction measurements are that they can be used to indicate more than just volume change in soil (ASTM standard D5298-10 2010).

The Australian standard AS1289.2.2.1 outlines procedure for determining total suction of a soil using a dewpoint microvoltmeter. A sample is prepared from a soil sample take from site and preserved. The prepared sample is placed into a sample cup and sealed into a sample chamber. A reading of microvolts is taken and then cooled for a specified time related to the microvolt reading and a dewpoint stable microvolt reading is recorded. Suction is then calculated using the chamber manufacturers calibration equation. This method is relatively quick as the test time is short and the longest period is waiting for the site sample to achieve temperature equilibrium before starting the test. The difficulties are having a place of controlled temperature and humidity and having the equipment accurately calibrated. The range measureable is also limited to the apparatus used.

ASTM International describe a filter paper method for measuring both total and matric suction. This method takes considerably more time and is subject to the skill of the person performing the test. The method involves preparing filter papers prepared and with known or measured suction-water content curves and either placing them in contact with a soil sample (matric suction) or suspended above a sample (total suction) inside a sealed container kept at a constant temperature for a minimum of seven days. The moisture content of the filter papers is then measured and suction can be taken from the filter paper suction-water content curves. This method though longer and is subject to the possibility of more errors is capable of measuring a wider range or suction values than the method described by AS1289.2.2.1.

2.3.4 Australian standards shrinkage index test

The guidelines in Australia for soil movement are in AS2870 which utilise a combination of indirect and direct method relying on an instability index, soil suction change and depth of soil layer. The instability index is a measure of percent vertical strain per unit change in suction that take into account applied stresses, lateral restrain and soil suction range. Because the measurement of soil suction is a long and complicated process the instability index can be approximated by a factored soil shrinkage index determined as per AS1289.7.1.1 (Standards Australia 2000).

The test for determining the soil shrinkage index is relatively short and simple making it more cost effective for smaller development and accessible without specialist technicians. The soil shrinkage index is a factored combination of the swell of a sample that has been loaded and then saturated and the shrinkage of a sample that has been fully dried.

Chapter 3

Methodology

With many available methods of testing and prediction there are many ways to design a wide range of tests with each method having merits for different reasons. Because for this study the aim is to indicate the effects on non-uniform loading and lateral movement of soil the methods used for testing need to be similar and comparable to those already used. For this reason and due to limited time and equipment availability testing was done using a manual oedometer shown in figure 3.1. There is however no conclusive evidence that the choices made are the best methods to achieve the aims of this study or the most accurate form of general measure.

Because time and resources did not permit no comparison of laboratory results could be made with those experienced in situ. Instead results of the two different tests were compared to general factors applied to one-dimensional tests and how predictions have differed in past studies. The test needs to be able to indicate if current factors and methods used account for lateral movement at points of different surcharge pressures.

3.1 Testing procedure

The outline of the method used for testing is the same as that in ASTM D4546 method A. There are two reasons for selecting such a method to test the swell potential; The first is that it best simulates actual site loading conditions (Al-Shamrani & Dhowian 2003) but the most important is that it is best suited for being able to examine relationships with

Figure 3.1: Manual oedometer used for tests

lateral displacement under different loads with the available resources. Different loads will be used to test and collect data for both the one-dimensional and two-dimensional tests on reconstituted clay samples with the same initial conditions.

The summarised procedure from ASTM D4546 is as follows:

1. Take measurements of oedometer parts. Cell ring inside diameter and mass, loading cap mass, etc.

2. Prepare sample to specified density and moisture content in cell ring.

3. Place cell into oedometer and record initial dial gauge reading.

4. Load cell in increments if possible and allow to consolidate for a minimum of 10 minutes but not longer than 30 minutes. Record total applied surcharge and consolidation if any.

5. Inundate sample with tap water ensuring top porous stone is covered by water.

6. Record swell (or consolidation) at time intervals of 0.5, 1, 2, 4, 8, 15, 30, 60, 120, 240 minutes and every 24 hours after start or as close as practical until swell is at a rate of less than 0.001 mm per hour. More recordings can be taken at certain points depending on sample behaviour.

7. Unload sample and drain quickly. Wipe excess water from sample ring and weigh.

8. Oven dry sample until constant mass and record dry mass of sample.

9. Calculate final saturation, void ratio, percentage swell of sample. Check that final dry mass agrees with prepared sample moisture content and mass.

This procedure can be followed for both the one-dimensional and two-dimensional test with a minor change when unloading the sample. Straight after unloading the sample use a thin walled sleeve the same inside diameter as the loaded area to cut and preserve the clay under the loaded area. The cut sample was weighed and then placed into a drying oven. Once dried the dry weight can be measured and the final void ratio, moisture content, saturation and reduction in mass under load can be calculated. Care will need to be taken during compaction of two-dimensional samples as the initial dry mass under the loaded area is assumed from the prepared moisture content and the assumption that the density is homogenous throughout.

3.1.1 Developing cell for two-dimensional test

For two-dimensional testing under the same conditions as one-dimensional tests a specially designed cell was needed. The design of the cell needed to take into consideration minimising the effects of lateral restraint of the soil around the loaded area by anything other than restraint given by the soil itself. The test also needed to be axisymmetric to maintain validity of only analysing the results in two dimensions. The restraints to the design of the cell were limited to an overall size that can fit in the oedometers available and the size of porous stones readily available. A sketch of the plans for the custom oedometer cell is shown in figure 3.2.

The overall dimensions were limited to 155 mm without needing to make alterations to the loading device of the oedometer. Limitations of available porous stones were 150mm diameter which is too big for the size limitations and 100 mm diameter which is too small to be a meaningful size increase to limit effects of lateral restraint so a 150 mm stone was cut to size. The sample ring inside diameter was selected to be 125 mm so that it could be constructed of stainless steel which is important to avoid oxidisation influencing results by either a change in size of the ring or contamination of the sample. It is also large enough so that when a 75 mm diameter load is applied that the lateral restraint should no significantly impact the loaded area. The outer cell being square in shape was

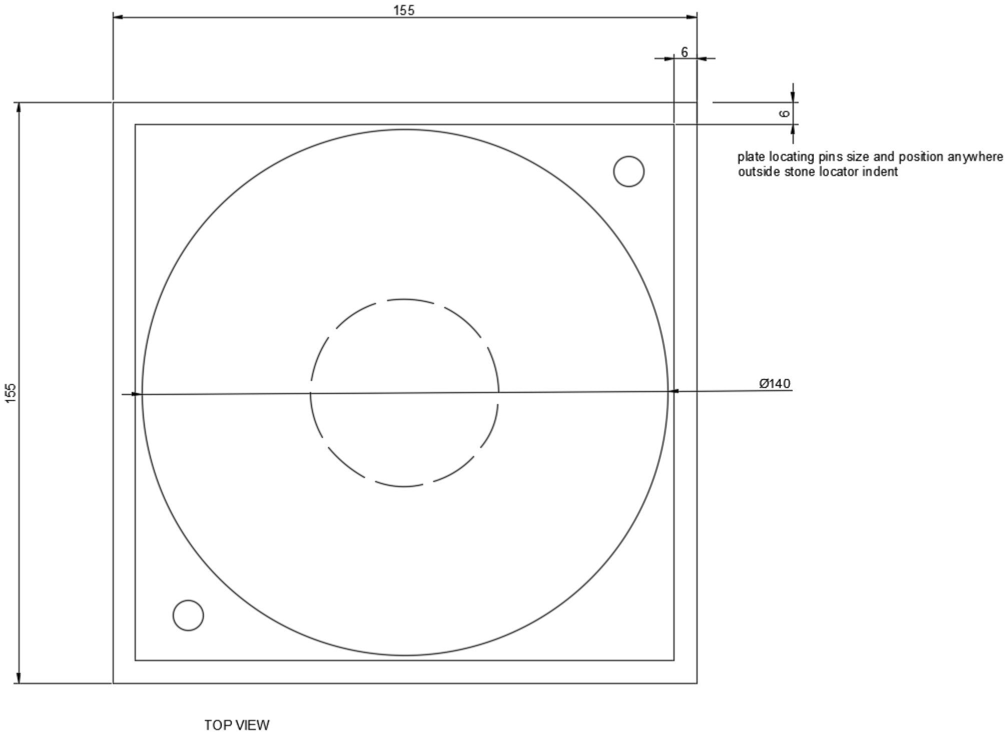


Figure 3.2: sketch of custom oedometer cell

simply for construction ease and has no impact on the tests. Figure 3.3 is an image of the completed device that was used.

3.2 Clay sample preparation

Samples were prepared in laboratory for each test as needed because results were not directly compared to any field measurements. Because tests were only directly compared with each other it needed to be ensured each sample had acceptably similar properties. All samples were reconstituted from the same package of a purchased clay powder to limit any variability that may be observed between packages. Before use the powder was oven dried to ensure a more accurate starting moisture content. The sample clay used was a kaolinite and quartz clay powder mix and was prepared to the properties detailed in table 3.1 where the specific gravity is the average value from the sample material.

The sample was prepared by adding the mass of water required to a known mass of the dried clay powder and mixed thoroughly by hand. Because of the low initial moisture content needed even mixture of moisture was difficult to obtain so only marginally more

Figure 3.3: Completed cell for two-dimensional testing

than what was required for a single sample was prepared each time to help ensure an even distribution of the moisture while maintaining the required overall initial moisture content. The sample was prepared by hand compaction in 10 layers and is trimmed to the height of the sample ring. The average height of the sample was measured and the total density checked to be in tolerable bounds of what was required. Extreme values of moisture content and dry density were required because kaolinite is known to have low swelling properties and a notable amount of swelling was required over a variety of loads.

Table 3.1: Clay sample target properties

G_s	2.72
ρ (kg/m^3)	1820
w	0.20

3.2.1 Constructing plots from test data

The most direct plot that can be produced from the collected data is swell percentage against log time to give the time-swell curve that can show when primary swell ends and secondary swell starts. This also gives an indication of the soil behaviour during swelling and visually identifies any points of interest in the data. For one-dimensional swell a period of quick approximately linear swell is expected on the semi-logarithmic scale preceded and followed by slower growth approaching no growth. Before swelling starts there is the possibility of some collapse and after maximum swell there can be some minor consolidation mostly caused by soil particle realignment after the rapid change.

To plot the strain - load graph the maximum swell is presented as percentage change in height from the original sample height and plotted as a point against the logarithm of applied pressure. When all points are plotted a linear regression line can be then used to extrapolate strains for pressures applied to that soil sample. The void ratio - load graph is constructed in the same way with points of maximum void ratio against the logarithm applied pressure with a linear regression line to obtain the swell index.

3.2.2 Parameters Calculated from tests

There are two parameters required from the test results to allow for prediction of swelling and both can be extrapolated from the final results of the swell overburden tests. Both swell pressure and swell index are extrapolated from the plot of strain against the logarithm of confining pressure and void ratio against log swell pressure respectively. The swell pressure is found where the graph passes through zero swell. Another point of interest from this plot is that of the free swell strain where the confining load is effectively zero but because of the nature of logarithmic regression line this will be taken at a unit load. The swell index can be taken as the slope of the regression line of the points on the void ratio - logarithm confining pressure plot. These two parameters are specific to the condition of the sample soil and any difference in initial conditions of the soil will result in differences in these parameters.

For the two dimensional tests lateral the effect of lateral restraint can also be examined by measuring the movement of soil out of the area under the load. Using this a relationship between percentage change in mass under load and total load can be examined. Also because of the change in mass under the area of vertical movement being measured only initial and final void ratio calculations can be made without further assumptions.

3.3 Prediction models

A suitable model to use for prediction purposes based on the data obtained from testing is equation 2.3 developed by Fredlund from results of oedometer tests (Chen 1988). This model has proven to provide predictions with reasonable accuracy using one-dimensional test data when a lateral restraint factor is used (Al-Shamrani & Dhowian 2003). Using this model without any adjustment factors swell predictions from each test can be compared by using the parameters obtained to identify if there is a significant difference between predictions and if there is what factor would be required if any to adjust the models to produce similar results. This factor if observed can be compared to the factor of one third that is commonly accepted.

To check these results with the selected model two scenarios were used to test how increasing layer depths may impact results. The first prediction used small layer of 50 mm at surface level which best represents directly the two-dimensional test but expanding the results over a small layer. The second prediction will use a 500 mm layer which means average surcharge of the area outside the load deviates much more from what was experienced in the test. This was to check for the possibility that a significant improvement to models could be made by accounting for changes in lateral pressures.

Chapter 4

Test results and observations

Four tests for each the one and two-dimensional procedures were done at separate loadings. The results were recorded and the follow observations made for each of the tests. For all tests it was assumed the specific gravity was constant and did not vary from the value presented in table 3.1. As water was added to reconstitute the sample and assumed to be mixed evenly, the initial moisture content is also presented in table 3.1 but because of manual compaction there were some inconsistencies with density that fell within ± 0.1 % of the target value.

4.1 One-dimension 6 kPa load

Table 4.1 shows the intial conditions, table4.2 shows values throughout the test, table 4.3 shows final values at test termination and calcualted initial conditions and figure 4.1 shows the time-swell curve from the test.

Table 4.1: Initial sample conditions for one-dimension 6 kPa test

d_1 (mm)	75.00	ρ (kg/m^3)	1819.3	e_i	0.794
h_i (mm)	19.16	ρ_d (kg/m^3)	1516.1	h_s	19.16
m_i (g)	154.0	S_i	0.685		
m_{di} (g)	128.33	σ (kPa)	6.62		

Table 4.2: Measured and calculated changes during testing of one-dimension 6 kPa test

t (min)	0.5	1	2	4	8	15	30	60	120
h (mm)	19.19	19.25	19.35	19.49	19.74	20.17	20.75	21.40	21.76
Swell (%)	0.16	0.47	0.98	1.7	3.0	5.3	8.3	11.7	13.6
e	0.767	0.802	0.812	0.825	0.849	0.889	0.943	1.00	1.04

t (min)	300	1470	4485
h (mm)	21.89	21.92	21.91
Swell (%)	14.2	14.4	14.4
e	1.05	1.05	1.05

Table 4.3: Final sample conditions and maximum swell values of one-dimension 6 kPa test

m_f (g)	178.5	m_{df} (g)	128.0
S_f	1.00	w_i	0.197
e_M	1.05	Maximum swell (%)	14.4

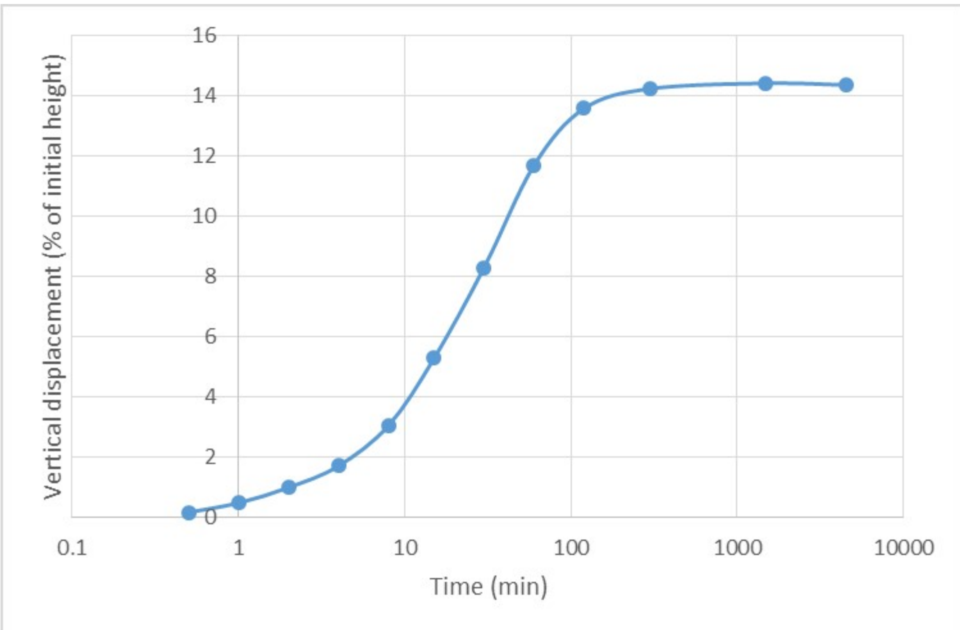


Figure 4.1: Time - swell curve of one-dimension 6 kPa test

Table 4.4: Initial sample conditions for one-dimension 23 kPa test

d_1 (mm)	75.00	ρ (kg/m^3)	1825.9	e_i	0.787
h_i (mm)	19.54	ρ_d (kg/m^3)	1521.6	h_s	19.55
m_i (g)	157.7	S_i	0.691		
m_{di} (g)	131.4	σ (kPa)	23.7		

Table 4.5: Measured and calculated changes during testing of one-dimension 23 kPa test

t (min)	0.5	1	2	4	8	15	30	60	120
h (mm)	19.55	19.58	19.64	19.74	19.93	20.27	20.63	20.84	20.94
Swell (%)	0.01	0.16	0.48	1.0	2.0	3.7	5.6	6.7	7.2
e	0.787	0.790	0.796	0.805	0.822	0.854	0.887	0.906	0.915

t (min)	240	1440	3015	4290	5890
h (mm)	21.02	21.08	21.08	21.08	21.07
Swell (%)	7.6	7.9	7.9	7.9	7.8
e	0.922	0.928	0.928	0.928	0.927

4.1.1 Observations during 6 kPa one-dimension test

Between the final two readings it is unknown if no change in height occurred like recorded or if a higher maximum swell was reached before some consolidation. From results of future tests it was assumed that if any more swelling occurred it would have been very minor. Swelling started occurring immediately upon addition of water. There were no indications of any issues that may affect the validity of the test during readings and time-swell behaviour is of expected appearance.

4.2 One-dimension 23 kPa load

Table 4.4 shows the intial conditions, table4.5 shows values throughout the test, table 4.6 shows final values at test termination and calcualted initial conditions and figure 4.2 shows the time-swell curve from the test.

Table 4.6: Final sample conditions and maximum swell values of one-dimension 23 kPa test

m_f (g)	175.3	m_{df} (g)	130.7
S_f	1.02	w_i	0.203
e_M	0.928	Maximum swell (%)	7.88

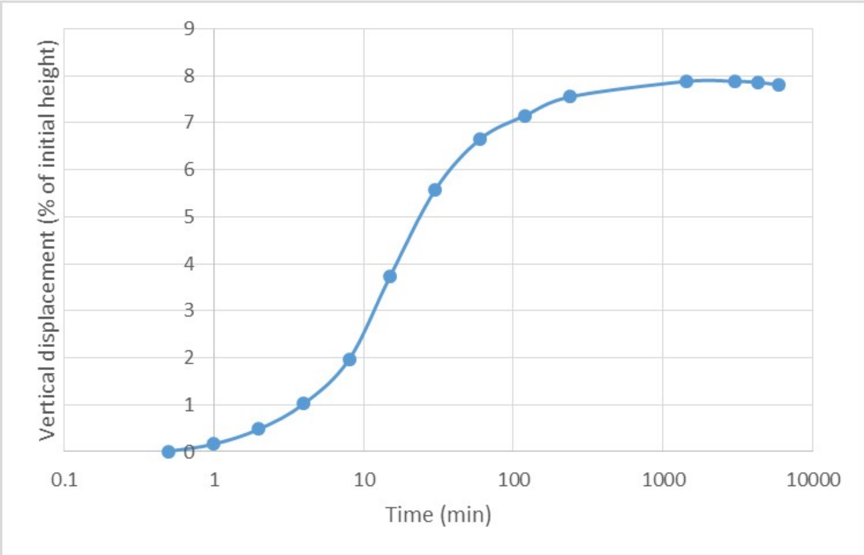


Figure 4.2: Time - swell curve of one-dimension 23 kPa test

4.2.1 Observations during 23 kPa one-dimension test

Between addition of water and the first reading a minor amount of collapse was observed before swelling started. An extra reading was taken between 24 and 72 hours because of the behaviour observed in the 6 kPa test and then a final reading taken approximately a further 24 hours later to ensure there was no severe collapse of the sample. All data recorded was within expected limits and trends. There was no indication of anything that would affect the validity of results obtained.

4.3 One-dimension 45 kPa load

Table 4.7 shows the intial conditions, table4.8 shows values throughout the test, table 4.9 shows final values at test termination and calcualted initial conditions and figure 4.3 shows the time-swell curve from the test.

Table 4.7: Initial sample conditions for one-dimension 45 kPa test

d_1 (mm)	75.00	ρ (kg/m^3)	1821.9	e_i	0.785
h_i (mm)	19.56	ρ_d (kg/m^3)	1518.2	h_s	19.63
m_i (g)	158.0	S_i	0.687		
m_{di} (g)	131.67	σ (kPa)	45.9		

Table 4.8: Measured and calculated changes during testing of one-dimension 45 kPa test

t (min)	0.5	1	2	4	8	15	30	60	111
h (mm)	19.54	19.57	19.64	19.75	19.97	20.19	20.35	20.42	20.47
Swell (%)	-0.09	0.08	0.42	0.98	2.1	3.2	4.0	4.4	4.7
e	0.783	0.786	0.792	0.802	0.823	0.843	0.857	0.864	0.868

t (min)	240	1340	3780	4315	5724
h (mm)	20.52	20.55	20.54	20.54	20.53
Swell (%)	4.9	5.1	5.0	5.0	5.0
e	0.873	0.876	0.875	0.874	0.874

Table 4.9: Final sample conditions and maximum swell values of one-dimension 45 kPa test

m_f (g)	173.5	m_{df} (g)	131.0
S_f	1.01	w_i	0.206
e_M	0.876	Maximum swell (%)	5.07

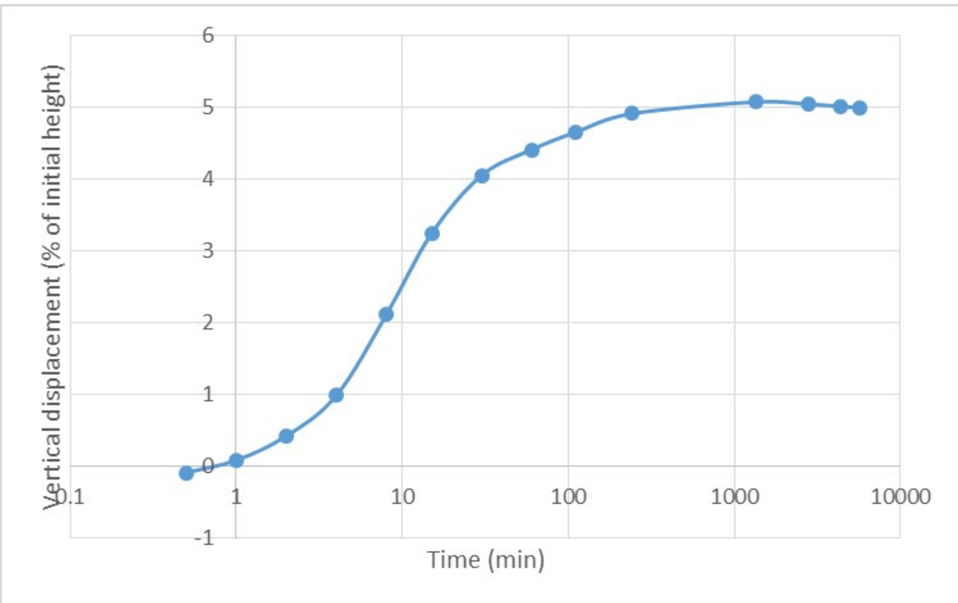


Figure 4.3: Time - swell curve of one-dimension 45 kPa test

Table 4.10: Initial sample conditions for one-dimension 90 kPa test

d_1 (mm)	75.00	ρ (kg/m^3)	1818.7	e_i	0.780
h_i (mm)	19.49	ρ_d (kg/m^3)	1514.2	h_s	19.67
m_i (g)	157.9	S_i	0.698		
m_{di} (g)	131.58	σ (kPa)	90.3		

Table 4.11: Measured and calculated changes during testing of one-dimension 90 kPa test

t (min)	0.5	1	2	4	8	15	30	60	120
h (mm)	19.49	19.51	19.55	19.61	19.71	19.89	20.01	20.04	20.06
Swell (%)	0.01	0.12	0.32	0.63	1.1	2.1	2.7	2.9	2.9
e	0.780	0.782	0.785	0.791	0.800	0.816	0.828	0.830	0.832

t (min)	225	1460	2914	4500	5826
h (mm)	20.06	20.08	20.08	20.08	20.08
Swell (%)	3.0	3.0	3.1	3.1	3.1
e	0.832	0.834	0.834	0.834	0.834

4.3.1 Observations during 45 kPa one-dimension test

An increase in collapse was observed compared to the 23 kPa test which was not unexpected. Behaviour of swell was consistent with those observed in previous tests with no indication that validity of results is suspect.

4.4 One-dimension 90 kPa load

Table 4.10 shows the intial conditions, table4.11 shows values throughout the test, table 4.12 shows final values at test termination and calcualted initial conditions and figure 4.4 shows the time-swell curve from the test.

Table 4.12: Final sample conditions and maximum swell values of one-dimension 90 kPa test

m_f (g)	173.2	m_{df} (g)	131.7
S_f	1.03	w_i	0.199
e_M	0.834	Maximum swell (%)	3.07

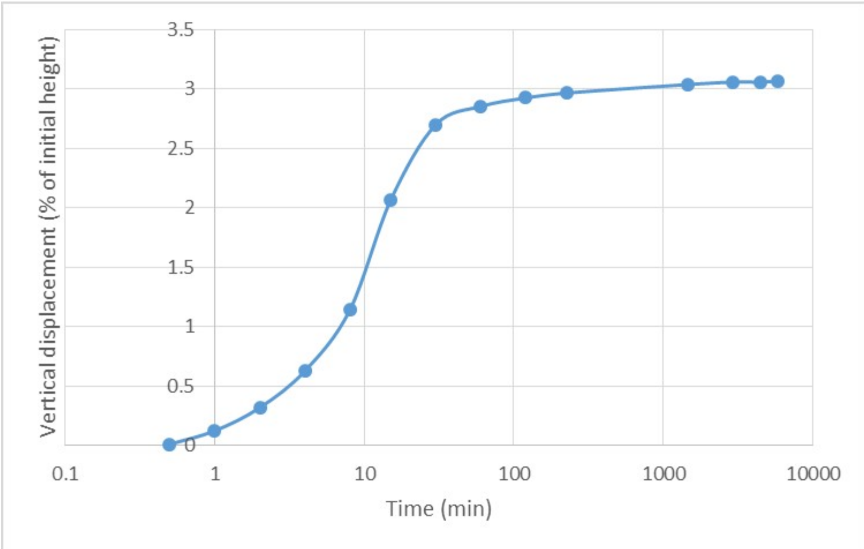


Figure 4.4: Time - swell curve of one-dimension 90 kPa test

4.4.1 Observations during 90 kPa one-dimension test

There was less collapse at the beginning of this test than that seen in the 45 kPa test when it was expected to see equal or more. The slight consolidation after maximum swell that was seen in the previous tests also was not observed and at 5800 minutes insignificant swell was still being observed. Though there were very minor changes in the behaviour the general swell behaviour remains as expected and there were no reasons to reject the validity of the results.

4.5 General observations of one-dimensional tests

Some general observations of the tests are that when removing the specimen after completion of the test some clay remained on the porous stones. This would explain some of the lower observed dry mass. This could have been resolved by placing filter paper between the sample and the stones but the lost mass can be considered minor and the after test results were just to show there was no gross error in the manual preparation of the sample.

Table 4.13: Initial sample conditions for two-dimension 6 kPa test

d_1 (mm)	75.00	h_i (mm)	20.34	σ (kPa)	6.62
d_2 (mm)	124.7	h_s	20.34	e_i	0.766
m_i (g)	166.11	ρ (kg/m^3)	1848.5	S_i	0.711
m_{di} (g)	138.42	ρ_d (kg/m^3)	1540.4		

After oven drying there was noticeable shrinkage of all the samples as they were smaller than the confining ring but no visible shrinkage cracks appeared. All final saturation values were above 1 which could be due to draw in of water when removing the load because water was not removed before doing so. This value isnt important to the study though and was just a check to indicate the sample was at saturation and no further significant swell could have happened.

It should be noted that the tests were not performed by an experienced professional and were completed by an undergraduate student with limited previous experience with the equipment. Though all tests were carried out with care to maintain consistency between tests, the accuracy prescribed in ASTM D4546-14 may not have been achieved. The level precisions observed was believed to be sufficient for comparisons done in this study.

4.6 Two-dimension 6 kPa load

Table 4.13 shows the intial conditions, table4.14 shows values throughout the test, table 4.15 shows final values at test termination and calcualted initial conditions and figure 4.5 shows the time-swell curve from the test.

4.6.1 Observations during 6 kPa two-dimension test

The two-dimensional test started the same as would be expected of a one-dimensional test but after the swell peaking there was significant consolidation. The value of saturation calculated for this test was much lower than seen in other tests but it can also be seen that the sample was still consolidating though at a thought to be insignificant rate. For this test the initial density was higher than the target range so results could be slightly skewed.

Table 4.14: Measured and calculated changes during testing of two-dimension 6 kPa test

t (min)	0.5	1	2	4	8	15	30	60	90
h (mm)	20.40	20.47	20.59	20.80	21.17	21.65	22.19	22.64	22.73
Swell (%)	0.32	0.64	1.2	2.3	4.1	6.4	9.1	11.3	11.8
e	0.771	0.777	0.787	0.806	0.838	0.879	0.926	0.965	0.973

t (min)	120	180	225	1368	2798	4244	5682
h (mm)	22.73	22.70	22.67	22.49	22.44	22.41	22.39
Swell (%)	11.8	11.6	11.4	10.6	10.3	10.2	10.1
e	0.973	N/A	N/A	N/A	N/A	N/A	N/A

Table 4.15: Final sample conditions and maximum swell values of two-dimension 6 kPa test

m_f (g)	176.2	m_{df} (g)	126.9
e_f	1.12	e_{M^*}	0.973
Maximum swell (%)	11.77	Final swell (%)	10.09
S_f	0.94	Δm (%)	-8.32

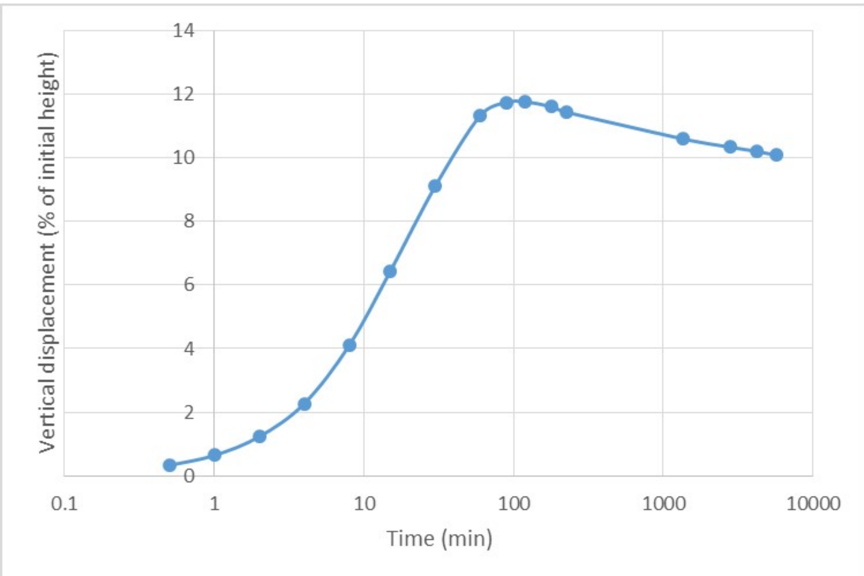


Figure 4.5: Time - swell curve of two-dimension 6 kPa test

Table 4.16: Initial sample conditions for two-dimension 12 kPa test

d_1 (mm)	75.00	h_i (mm)	20.49	σ (kPa)	12.6
d_2 (mm)	124.7	h_s	20.50	e_i	0.795
m_i (g)	164.63	ρ (kg/m^3)	1817.7	S_i	0.684
m_{di} (g)	137.19	ρ_d (kg/m^3)	1514.8		

Table 4.17: Measured and calculated changes during testing of two-dimension 12 kPa test

t (min)	0.5	1	2	4	8	15	30	60	120
h (mm)	20.56	20.62	20.70	20.86	21.15	21.65	22.23	22.50	22.38
Swell (%)	0.34	0.61	1.0	1.8	3.2	5.7	8.5	9.8	9.2
e	0.801	0.806	0.814	0.827	0.852	0.897	0.947	0.971	N/A

t (min)	180	230	1338	3112	4218	5658
h (mm)	22.15	22.03	21.61	21.53	21.50	21.47
Swell (%)	8.1	7.5	5.4	5.0	4.9	4.8
e	N/A	N/A	N/A	N/A	N/A	N/A

4.7 Two-dimension 12 kPa load

Table 4.16 shows the intial conditions, table4.17 shows values throughout the test, table 4.18 shows final values at test termination and calcualted initial conditions and figure 4.6 shows the time-swell curve from the test.

Table 4.18: Final sample conditions and maximum swell values of two-dimension 12 kPa test

m_f (g)	175.5	m_{df} (g)	124.7
e_f	1.07	e_{M^*}	0.971
Maximum swell (%)	9.81	Final swell (%)	4.78
S_f	1.03	Δm (%)	-9.10

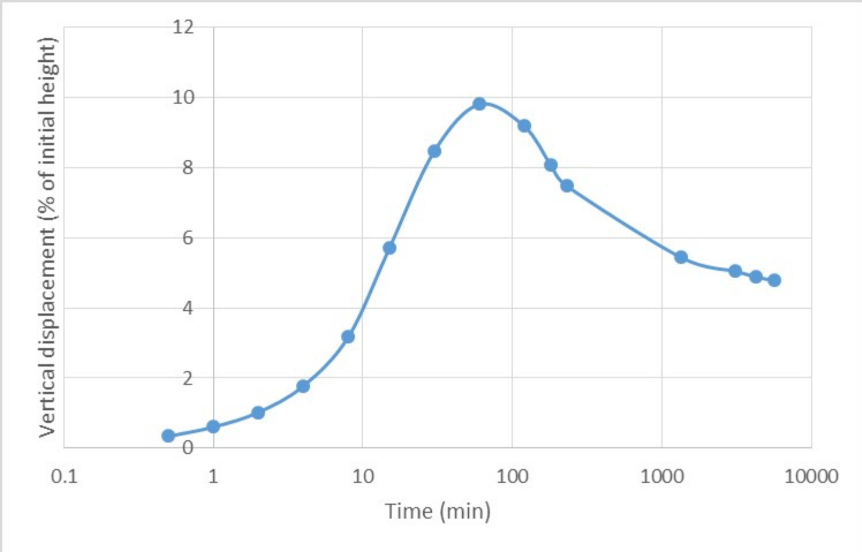


Figure 4.6: Time - swell curve of two-dimension 12 kPa test

Table 4.19: Initial sample conditions for two-dimension 23 kPa test

d_1 (mm)	75.00	h_i (mm)	20.33	σ (kPa)	23.7
d_2 (mm)	124.7	h_s	20.37	e_i	0.791
m_i (g)	163.72	ρ (kg/m^3)	1819.3	S_i	0.688
m_{di} (g)	136.43	ρ_d (kg/m^3)	1516.1		

4.7.1 Observations during 12 kPa two-dimension test

Swelling of the sample started as soon as water was added and peak swell was observed in a relatively short period of time. After peak swell rapid consolidation occurred before stabilising after a period. Minor consolidation was still occurring at termination but not deemed to be at a significant level.

4.8 Two-dimension 23 kPa load

Table 4.19 shows the intial conditions, table4.20 shows values throughout the test, table 4.21 shows final values at test termination and calcualted initial conditions and figure 4.7 shows the time-swell curve from the test.

Table 4.20: Measured and calculated changes during testing of two-dimension 23 kPa test

t (min)	0.5	1	2	4	8	15	30	60	90
h (mm)	20.33	20.39	20.49	20.63	20.83	21.25	21.59	21.66	21.52
Swell (%)	0.03	0.32	0.80	1.5	2.5	4.5	6.2	6.6	5.9
e	0.791	0.796	0.805	0.817	0.834	0.871	0.902	0.908	N/A

t (min)	124	180	309	1635	3186	4535	5860
h (mm)	21.36	21.12	20.81	20.49	20.44	20.41	20.39
Swell (%)	5.1	3.9	2.4	0.78	0.54	0.40	0.30
e	N/A	N/A	N/A	N/A	N/A	N/A	N/A

Table 4.21: Final sample conditions and maximum swell values of two-dimension 23 kPa test

m_f (g)	170.3	m_{df} (g)	121.9
e_f	1.01	e_{M^*}	0.908
Maximum swell (%)	6.57	Final swell (%)	0.295
S_f	1.07	Δm (%)	-10.7

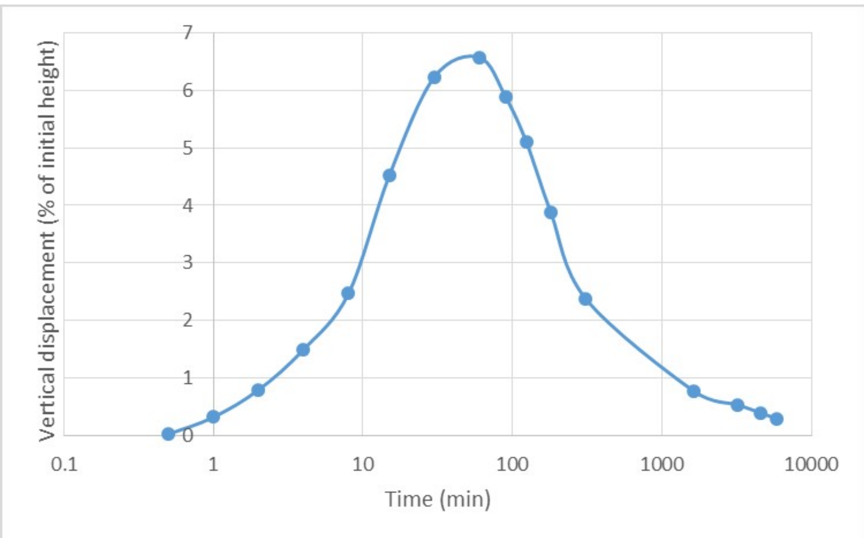


Figure 4.7: Time - swell curve of two-dimension 23 kPa test

Table 4.22: Initial sample conditions for two-dimension 45 kPa test

d_1 (mm)	75.00	h_i (mm)	20.26	σ (kPa)	45.9
d_2 (mm)	124.7	h_s	20.33	e_i	0.787
m_i (g)	163.54	ρ (kg/m^3)	1820.9	S_i	0.691
m_{di} (g)	136.28	ρ_d (kg/m^3)	1517.4		

Table 4.23: Measured and calculated changes during testing of two-dimension 45 kPa test

t (min)	0.5	1	2	4	8	15	30	60	90
h (mm)	20.24	20.56	20.33	20.44	20.62	20.93	21.10	20.97	20.82
Swell (%)	-0.12	-0.03	0.31	0.85	1.8	3.3	4.1	3.5	2.7
e	0.784	0.786	0.792	0.802	0.818	0.846	0.861	N/A	N/A

+

t (min)	120	196	1335	5656
h (mm)	20.63	20.22	18.53$^+$	N/A
Swell (%)	1.8	-0.21	-8.6$^+$	N/A
e	N/A	N/A	N/A	N/A

Observed change in height was great than this value as consolidation exceeded the range of the dial gauge.

4.8.1 Observations during 23 kPa two-dimension test

Swelling occurred immediately after the addition of water to a peak followed by rapid consolidation. At termination the sample under the load had almost returned to the initial height. Minor consolidation was still occurring at termination but not deemed to be at a significant level.

4.9 Two-dimension 45 kPa load

Table 4.22 shows the intial conditions, table4.23 shows values throughout the test, table 4.24 shows final values at test termination and calcualted initial conditions and figure 4.8 shows the time-swell curve from the test.

Table 4.24: Final sample conditions and maximum swell values of two-dimension 45 kPa test

m_f (g)	133.6	m_{df} (g)	94.8
e_f	N/A	e_{M^*}	0.861
Maximum swell (%)	4.14	Final swell (%)	N/A
S_f	N/A	Δm (%)	-30.4

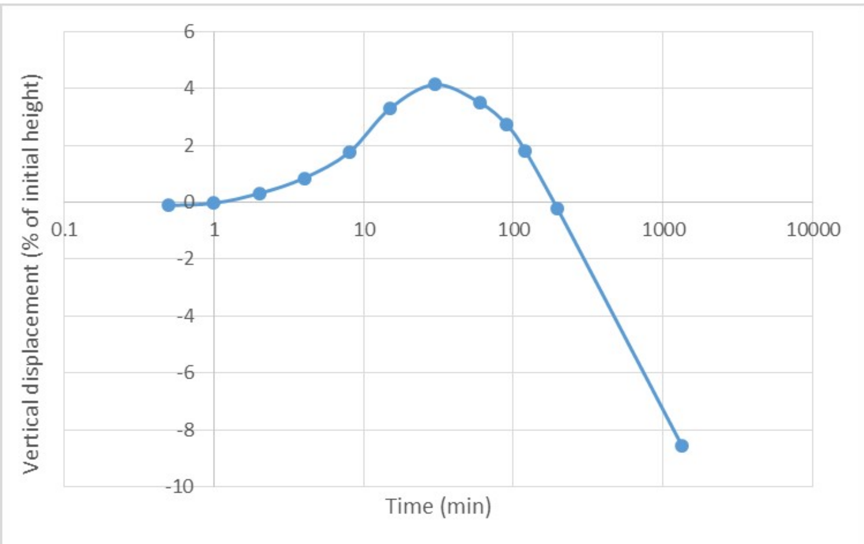


Figure 4.8: Time - swell curve of two-dimension45 kPa test

(a) Free movement of air from the sample

(b) Heaving of sample after initial swell

Figure 4.9: Behaviour of a clay sample during two-dimensional test

4.9.1 Observations during 45 kPa two-dimension test

Collapse was observed when water was added to the sample before swell occurred. Much larger than expected consolidation happened after peak swell to the extent that the dial gauge was not set up to fully measure it. The test was left to run the same length of time seen in the previous tests and assumed consolidation reached an insignificant or no rate of change so that final mass under load could be obtained.

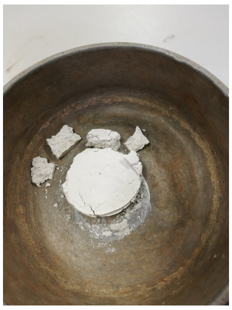
4.10 General observations of two-dimensional tests

Because part of the sample was not covered or restrained by anything it was observed in all samples that there were point where air trapped in soil was unobstructed in leaving the soil as saturation occurred shown in figure 4.9a. It appeared that heaving was occurring outside the loaded area and the swell was larger than that under the loaded area as can be seen in figure 4.9b. It would also appear that it is possible swelling was still occurring during consolidation as the final void ratio in all tests is larger than the void ratio at maximum swell, under normal consolidation this should have reduced.

After oven drying very large shrinkage and cracking was observed in all samples appearing to be worse for the samples with greater loads. Figure 4.10a shows a sample after drying still contained by the sample ring and after it was removed the sample crumbled due to the extent of the shrinkage cracks shown in figure 4.10b. As the load increased so did the change in mass from under the loaded area which was expected.

It should be noted that this adaptation on oedometer testing was developed without

(a) Shrinkage of sample after oven dry



(b) Extent of shrinkage cracks

Figure 4.10: Behaviour of a clay sample during two-dimensional test

knowledge of the behaviour of the clay being tested. It was not expected to have such extreme consolidation after swelling and as a result actual void ratios at peak swell were not able to be measured. The assumption that the whole sample only swells vertically in this initial period and loss of mass from under the load is minimal in likely to not be correct but no better estimation is available.

Chapter 5

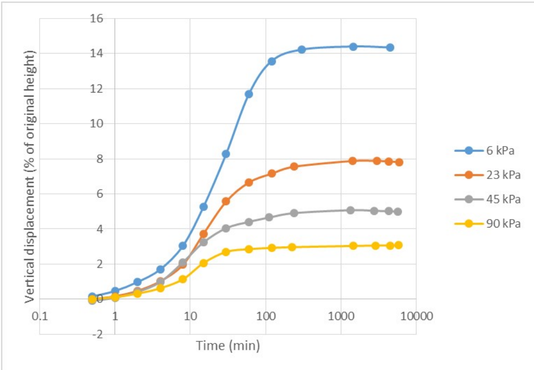
Analysis

From the results of test there were both a number of expected and unexpected outcomes. The main aim of collecting this data was to directly compare data obtained from both tests and the swell parameters swell index and swell pressure. Another aim was to compare profiles two-dimensionally which was done by measuring the migration of clay from under the load. A secondary aim was to use prediction models with the obtained parameters and compare values over a simulated layer of soil between each test type and then by applying a general lateral restraint factor to the one-dimensional model. Because of the unexpected results especially from the two-dimensional tests some of the analysis required assumptions to be made and then considerations into other soil behaviours.

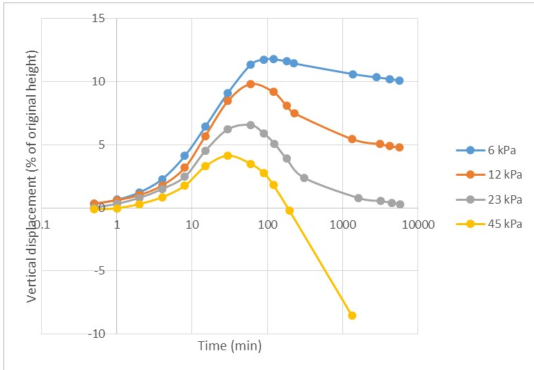
Because of unexpected observations some of the data from two-dimensional tests was not used for all analysis. Where data from the 6 kPa test did not match expected trends it was omitted if the other data points alone produced better fits due to the initial conditions being outside what was considered tolerable. The amount of data for the 45 kPa test was also limited due to consolidation exceeding the original height of the sample meaning that data from the 6 kPa test was required due to a lack of other data to use.

5.1 Direct result comparison

Comparing the direct results of the tests it was observed in figure 5.1 that the general behaviour of larger loads produce lower swell for both test types which is in line with what was expected. It was not predicted that such extreme consolidation would occur after maximum swells were observed which limited data that could be used without making assumptions for void ratios at points of maximum swell.



(a) All 1D time-swell curves



(b) All 2D time-swell curves

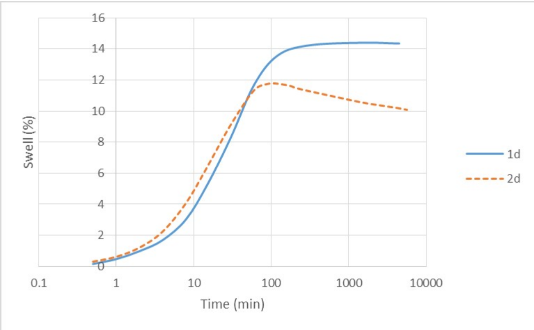
Figure 5.1: Time swell plots of tests

Directly comparing results of tests with the same applied load in figure 5.2 it can be seen that the maximum swell of the two-dimensional case is lower than that of the one-dimensional which was also expected due to allowing for lateral movement. The point of maximum two-dimensional swell approximately coincides with that of the primary one-dimensional swell. To this point the shape and position of the curves are approximately equal which gives some validation to the assumption of no lateral movement until after maximum swell to calculate void ratios.

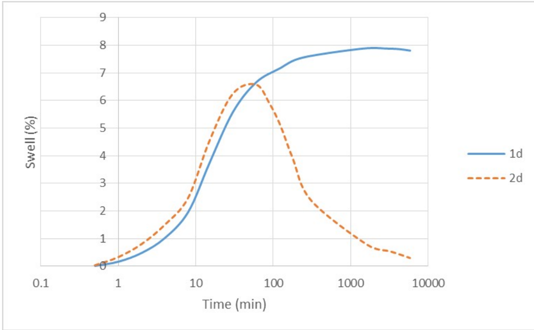
Figure 5.2a may be an indication that the initial conditions of the 6 kPa two-dimensional sample was indeed significantly different as the point of maximum swell does not intersect with the one-dimensional curve. Gradients of primary swell do not appear to be significantly changed by the removal of fixed lateral restraint. An average relative difference in maximum two-dimensional swell of 17.5% was observed but with only 2 reliable data points no trend can be extrapolated reliably.

5.2 Swell index and swell pressure

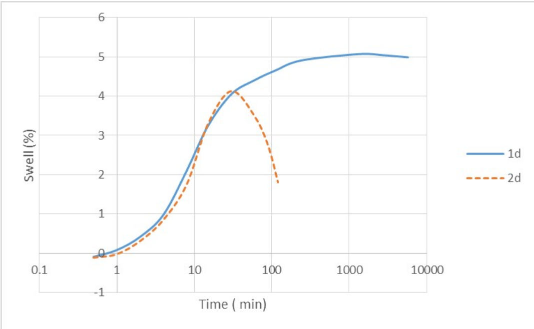
From the test data three points of interest were used to extrapolate the needed parameters to perform swell predictions. The one-dimensional test where all four data points were used at the point where maximum swell was observed, the two-dimensional test at



(a) 6 kPa load



(b) 23 kPa load



(c) 45 kPa load

Figure 5.2: Comparison of 1D and 2D time-swell curves

maximum observed swell where the 6 kPa result was excluded leaving three points of data to used and the final recorded two-dimensional swell where the 6 kPa data point was included so that there was a minimum of three points to use. Figure 5.3 and 5.4 show the required plots to extrapolate the parameters which are presented in table 5.1.

Table 5.1: Calculated swell parameters

Point of interest	**Swell index**	**Swell pressure (kPa)**	**Free swell (%)**
1-D maximum swell	0.0366	160.0	22.3
2-D maximum swell	0.0370	114.0	20.7
2-D final swell	0.0376	24.2	24.5

It is noted that across all points of interest the swell index obtained are similar meaning that across the tests the measured swell behaviours were similar. There are however large difference in calculated swell pressure particularly with the final swell two-dimensional measurements. Free swell values only varied by 4% across the points. The point for the two-dimensional 6 kPa test for maximum swell was also plotted in figures 5.3 and 5.4 and found if included in the calculation of regression lines a significant change and reduction

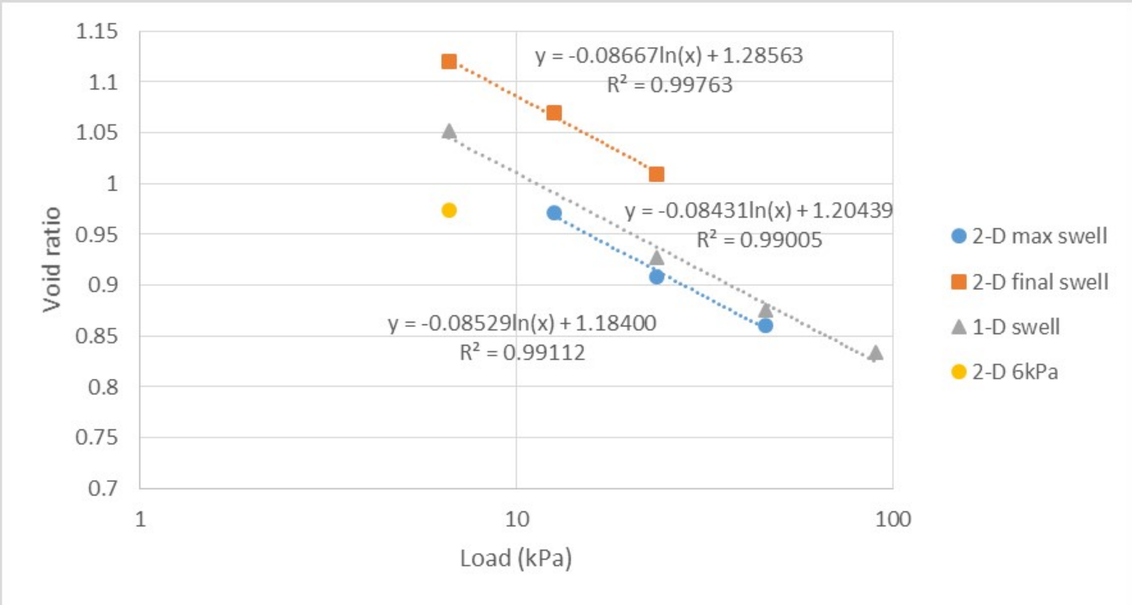


Figure 5.3: Void ratio against Load for the three points of interest from tesing

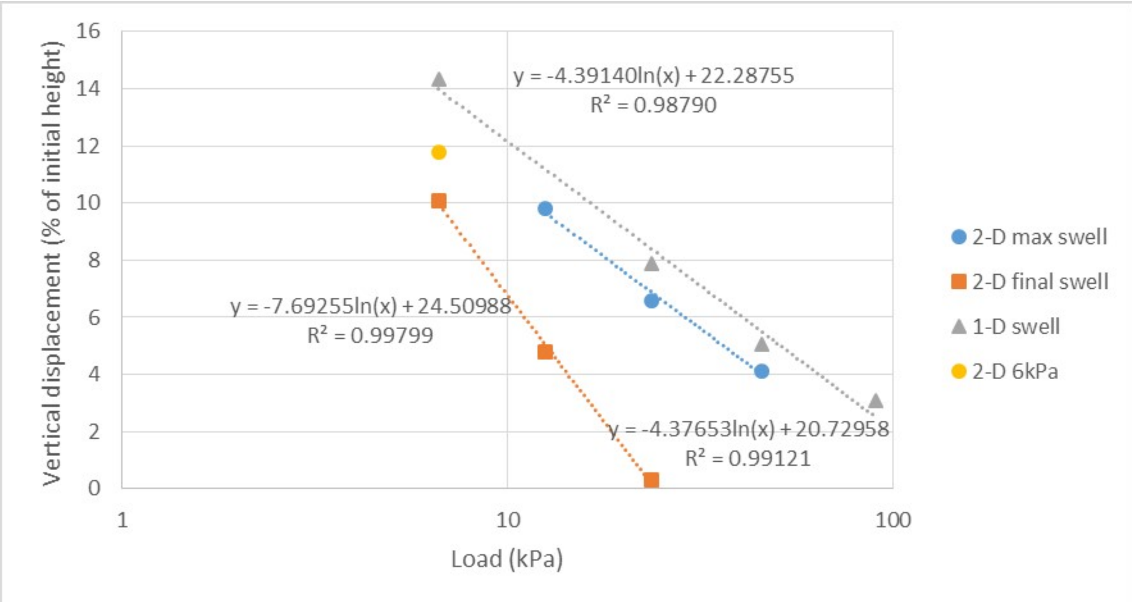


Figure 5.4: Swell against load for the three points of interest from testing

of fit was observed especially for calculation of the swell index.

5.3 Lateral movement

The lateral movement of soil measured at completion of the two-dimensional was found to have a purely linear increase with increased applied load shown in figure 5.5. For this the 45 kPa data was excluded because it severely impacted the fit for the rest of the data and was also past the calculated swell pressure for the final swell. It should be noted that even though consolidation was measured after the point of maximum swell there was still an increase in measured void ratio from the assumed maximum swell value. This indicates that swelling may have continued but was confined to lateral strains.

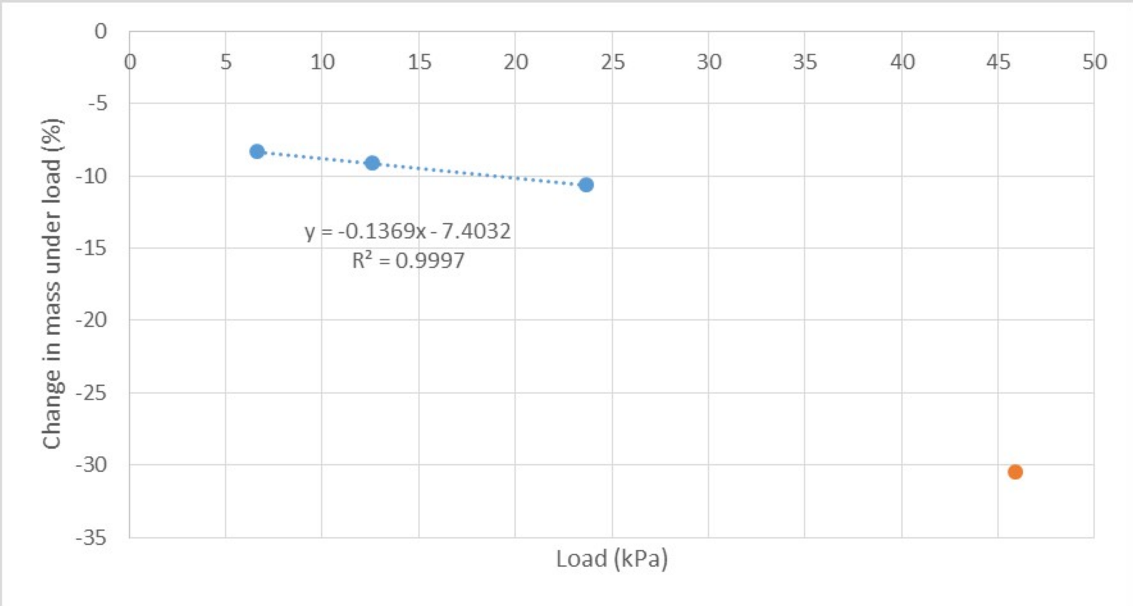


Figure 5.5: Percentage of mass lost from under loaded area

5.4 Predictions

Using equation 2.3 predictions of for swell over a 100 mm and 500 mm layer were made both by assuming a whole layer and by splitting the layer into 2 parts. A 10 kPa load was used applied at the soil surface and predictions using parameters from the 3 points of interest were done. Results of the predictions are presented in table 5.2.

Table 5.2: Swell predictions made using parameters from tests

Layer description	1-D max (mm)	2-D max (mm)	2-D final (mm)
100 mm 1 layer	2.39	2.11	0.730
100 mm 2 layers	2.39	2.12	0.731
500 mm 1 layer	10.7	9.30	2.36
500 mm 2 layers	10.8	9.35	2.41

From these predictions the lateral restraint factor for two-dimensional maximum and final swell was calculated. Table 5.3 presents the factors to apply to the prediction equation using one-dimensional parameters to achieve the two-dimensional predictions.

Table 5.3: Lateral restraint factors

Layer description	2-D max	2-D final
100 mm 1 layer	0.883	0.305
100 mm 2 layers	0.883	0.305
500 mm 1 layer	0.868	0.220
500 mm 2 layers	0.869	0.224

Chapter 6

Discussion

From the test results and analysis there were observations of both expected and unexpected soil behaviour based from previous literature. There were also tests and behaviours observed that have not be covered or observed in previous studies that may help to explain some of the errors in swell estimation from current prediction models. Because of the limited tests performed and only a single soil type being tested no generalisations or statistically significant comparisons can be made about general soil behaviour, only the behaviour of tested sample can be discussed. Because of known abnormalities of soils behaving outside of expectations for known general property behaviours and that the tested sample was manufactured to respond in an expansive manner when mineralogy would suggest a stable soil, applying any of the findings from this study generally without any further study carries a large risk.

6.1 Swelling behaviour

The results from the standard swell overburden tests indicated standard expansive soil behaviour. All of the data collected from these test were considered error free and to a quality suitable for use in a prediction model and comparison with other testing methods. A negligible amount of collapse was observed immediately after addition of water for some of the tests, particularly as the confining load was increased. This was not considered to be unusual as this possible behaviour was covered mentioned in the standard used for testing procedure.

The behaviour of swelling during the axisymmetric testing was expected to behave similar to triaxial tests where the time swell curve is much lower and shallower. With axisymmetric design three-dimensional swell was allowed but giving simplicity of analysing in two dimensions. Instead a behaviour very similar to the one-dimensional tests was observed up to a maximum swell when consolidation occurred due to lateral movement of the soil. It is unclear the extent of lateral movement up to the point of maximum swell though this could be the reason for a reduced maximum swell and so more investigation of behaviour up to this point is needed. It would also be of interest to check if other soils exhibit the same behaviour under these conditions.

The relatively large consolidation observed after the point of maximum swell could be caused by a number of possible effects. It is possible that the reduction in dry density of the clay coupled with saturation possibly past the liquid limit made the sample fail in bearing capacity. The heaving observed does show a likeness to heaving explained by general shear failure under bearing. Another possibility could be that in this case there is a higher vertical stress than lateral confining swell allowing for higher rates of lateral swell compared to vertical swell. After pressure caused by the primary swell is completed the pressure from secondary swell is not enough to maintain the load but because of low lateral pressures consisting almost entirely of lateral swell pressure from surrounding soil. This would require more complex analysis of changes in stress applied from soil swell during the process of swell both under and outside of the loaded area.

It is likely that a number of factors in combination lead to this behaviour and more investigation is required. If the assumption of negligible lateral swell until after maximum swell is proven then higher void ratios at final measurements are explained by continued lateral swell otherwise it would have been expected that the void ratio would have decreased from consolidation. It is clear however that the assumption of one-dimensional swell near the soil surface for soil at the edge of a loaded area does not accurately describe the behaviour of the soil. This also may cause issues with simply superimposing swell results from different soil layers to a total swell if all layers do not experience swell at the same time e.g. slow or uneven ground infiltration or rising water table.

6.2 Swell parameters

Values of swell data collected from oedometer tests can vary greatly depending on the testing method chosen. Al-Shamrani & Dhowian (2003) tested the same soil sample under three different methods and then using reconstituted samples in one of the methods and a triaxial test and found shockingly large difference in the swell index and swell pressures obtained from these tests. These differences are believed to be caused by friction from the sidewall of the oedometer cell. From the results of the swell index between the points of interest there is less than a 3 % difference from the value obtained by one-dimensional testing. This indicates that for this case the friction involved in the swell overburden method and the most extreme case of allowed lateral movement during swelling doesnt impact the value of swell index.

The difference in calculated swell pressure is much greater though and is probably a much greater indication of the effects of lateral restraint. There was a significant drop in maximum swell pressures between the two tests but more concerning was the 85 % reduction in final swell pressure. There are a number of possible reasons this value is much lower but it is hypothesised that the low lateral restraint is the main factor. The saturated clay possibly acting more like a fluid flows laterally until increased density and overburden pressure equals the pressure under the loaded area. More testing would be required to explore this relationship and behaviour.

Concerning is that the maximum swell pressure of the two-dimensional tests was 71.3 % of the one dimensional swell pressure. This is far higher than that expected from triaxial tests producing a swell pressure of a third of those in one-dimensional tests. This could be due to the interaction of the swell of surrounding soil where are triaxial test would be set at a constant lateral pressure but more tests are required to verify results.

Although free swell is not a required parameter for the prediction model it is worth noting that all three points of interest estimate the free swell to within 4 % of each other at a unit load. This gives another reasonable indication that all three points of interest in some form give an indication of the swelling potential of the sample.

6.3 Displacement of soil

Probably the most concerning information gathered from these tests is the movement of soil from under the loaded area. The behaviour of this movement compounds the problems associated with reactive soils. In the tested case the free lateral movement only reduced maximum swell by 17 % relative to one-dimensional swell and the swell pressure by 29 %. This indicates that in conditions where lateral restraint is at a minimum the maximum swell and swell pressure though reduced significantly they are not reduced to levels predicted from triaxial tests.

After maximum swell and pressures were experienced significant decreases in mass under load, swell and pressure were recorded which increased with the increase of load. This presents a problem as both the mass and volume decreased under the loaded area it would be unlikely that the mass would return under the loaded area during drying and shrinkage. This would compound issues that result from any shrinkage predicted by linear shrinkage tests. This behaviour was not tested in the study but could be an issue that needs further exploration. This effect was great magnified once loading exceeded the swell pressure calculated from the final test readings where approximately doubling the swell pressure tripled the percentage of soil moved from under the load.

From the limited data it does appear there is a linear relationship with applied load and mass moved from under the loaded area. More testing should be done with a focus on identifying for what region the linear relationship holds as theoretically the line should intersect zero movement from under a zero load which is not the case using the linear relationship of the available data. There are also however a large number of parameters that were fixed for these tests that would be expected to impact these values. Some of these variations are; increasing the surcharge to the area outside the load, increasing the loaded area, changing the shape of the loaded area (no longer would be a two-dimensional test) and increasing the distance to the fixed lateral restraint as there is no evidence to suggest that the distance used during testing had no impact on the loaded area.

6.4 Predictions

All of the parameters used for predictions and the model itself appears to be robust over the depths tested with even predicting for a 500 mm layer in a single calculation shows less than 0.1 mm difference in all points when compared to using 2 layers. There is however a very significant difference between the predicted values for each point especially for the two-dimensional final swell. Table 6.1 shows that that even though the model uses a layer 5 times greater than the height of the sample there is very little difference between the prediction and the test result. This raises questions about why the test swell is much greater than swells expected from predictions using the parameters from the test swells.

Table 6.1: Test swells versus predictions

	1-D (mm)	2-D max (mm)	2-D final (mm)
12.6 kPa test H = 20 mm	N/A	2.01	0.980
10 kPa prediction H = 100 mm	2.39	2.12	0.731

From table 5.3 it can be seen that the predictions made using the two-dimensional final swell would require a factor close to 0.3 applied to the one-dimensional prediction which is close to the assumed general factor of 0.33. This requires some investigation as maximum swell experienced during testing and predicted is much greater than that of final values. If behaviour in the field is similar to what was experienced during testing than the general lateral restraint factor may not give accurate predictions of the maximum experienced swell.

6.5 Future research

Because of the current lack of knowledge in soil behaviour and simple ways to measure and predict it there is a vast range of research that is still needed. One of the most common already applied methods of extra research involves performing detailed analysis of soil parameters and results from many different current methods used in predicting reactiveness and swell of a soil to build on correlations of converting simple to measure parameters with swell predictions. This also includes identifying outliers to current correlations and exploring the conditions or combination of parameters that identify such outliers. This applies to the testing performed with this report simply to identify if more soil types behave in the same way that was found.

Research specific to finding made in this study would include further study into the behaviour of soil particularly in regions where lateral restraint is lower than the swell pressure of a soil. From the findings made in this study it is apparent that in particular where very little lateral restraint is applied the assumption of swell behaviour and those measured in current tests may not be indicative of actual soil behaviour.

Of particular interest in this area would be to further explore the relationship of lateral pressure at the boarder of the loaded area and the swelling behaviour under the load. If a relationship can be found and then related to a parameter from a simpler one-dimensional test the prediction models can be updated by adding an adjustment for the depth of a soil layer in terms of the average lateral pressure. Locating points where swell behaviour is more like one-dimensional or symmetric three-dimensional if these occur at a depth shallower than that which would cause a pressure greater than the swell pressure.

Because of the concerns of the migration of soil from under the loaded area cyclic wetting and drying needs to be tested to check the impact caused from shrinkage as the sample dries. It would also be of interest to check if the swelling behaviour changes after any number of cycles or if further lateral soil migration could be a problem for each cycle. This behaviour is a known problem particularly for light foundations but there seems to be a lack of information on the specifics like prediction of loss of mass and positive or negative pressures because of this.

There seems to be no literature describing how changes in initial conditions of a sample effect the swell parameters used in current models. This information would be of particular used because with current tests taking up to a week to complete site conditions could have changed making the tested sample not valid and there is currently no way to adjust calculated parameters to initial condition changes. There is also currently no information available on rates of swell over time or proportions of swell for moisture contents below saturation. If equations of lateral and vertical swell, change in parameters for change in soil conditions (dry density, moisture content and void ratio) and soil movement could be related to soil parameters, internal conditions, boundary conditions and pore water flow then it may be possible to construct a numerical finite element model to track complex movements of soil during swell where pressures are likely to be concentrated.

To construct such a numerical model would require a vast amount of testing for a large scope of changes in a single soil parameter with no confidence that the relationships found for a single soil can be applied to any other soil without performing the same array of tests on that soil. This however would be very beneficial to improving construction designs to limit critical soil movements and reinforcing areas of high stress and hence reducing the cost of damage caused from expansive soils annually. There are still many unknowns in how soil behaves under complex conditions even with maintaining the assumption of a homogenous substance and this is evident with the annual increase in damages that are caused.

Chapter 7

Conclusion

The behaviour of soil across the boundary of non-uniform load distribution is more complex and different to the behaviours applied to soil from one-dimensional and uniform loaded triaxial tests. This behaviour across boundary of a change in pressures has not been explored or included in the modelling of soil expansiveness but may have large impacts on the properties of soil in these regions. There has also been very little progress in development of accuracy in models to provide better modelling of soil behaviour, in particular expected swell.

It has been shown through this study that the swell and movement behaviour of soils can exhibit characteristics that are not accounted for within current prediction models with only two parameters. It has also been shown in previous research that different testing methods can produce vastly different values for these parameters with no indication to which methods produces results for higher modelling accuracy. It is evident that current knowledge of soil behaviour is not to a high enough standard due to the large annual repair cost of damage caused by soil movement.

From the tests performed a number of difference were found with the parameters from the two-dimensional test and assumptions applied to results from standard one-dimensional tests. The swell index and free swell notably appeared mostly unaffected by the lateral boundary while swelling behaviour and swell pressure not only had significant difference to the values obtained in the standard test but also to the assumptions made and applied to standard tests to adjust them to triaxial data. There was also found to be a significant change in mass under the loaded area for the case tested which is not explained or factored by any of the current methods.

Triaxial tests allow for a lateral pressure to be applied but the sample is still confined and from results obtained this may not be a realistic representation of swell behaviour. More testing allowing for different levels of lateral pressure and restraint is required to observe the changes and possible trends in soil behaviour under a loaded area. These test also need to be performed on a number of soil samples to check for general or specific soil behaviours in these cases. There are also a number of other factors that need to be understood like distance to a fixed lateral boundary and the effects on a loaded area and the size of the loaded area and the effect this has on percentage of soil moved from under the load. The shape of the loaded area is another parameter of interest but the test would need to be redesigned for a three-dimensional purpose.

This is just a start on the need to perform much more complex analysis of soils to properly understand the way they behave under different conditions. Even if complex analysis can only capture the behaviour of area specific this may still be of a large benefit especially for infrastructure like pipe and road networks built in large areas of a single type of reactive soil. If detailed simulations can be performed with accurate predictions of soil movement and pressure exerted then better design decisions can be made and even new construction methods used to limit unwanted behaviours of control behaviour.

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Appendix A

Project Specification

ENG 4111/2 Research Project

Project Specification

For:	**David Petersen**
Topic:	Studying the effecs of non-uniform stress distribution on soil heave
Supervisors:	Kazem Ghabraie
Sponsorship:	Faculty of Health, Engineering & Sciences
Project Aim:	To compare soil swell prediction from one-dimensional swell tests and a two-dimensional axially symmetric non-uniform loaded soil swell test using swell overburden test methods and pressure based prediction approach.

Program: Issue A, 13/03/2016

1. Research current methods used to predict soil heave.
2. Design two-dimensional axisymmetric test so that it is comparable to standard tests.
3. Prepare testing samples that are similar within reason for swell comparisons.
4. Perform swell overburden tests for different loads and collect data.
5. Extrapolate data to obtain maximum theoretical pressure to contain soil swell to zero and analyse and compare swell data.

As time and resources permit:

1. Compare results with Australian standards methods of swell prediction.
2. Explore relationship between two-dimensional heave profile and loads.

Agreed:

Student Name:	David Petersen
Date:	13/03/2016
Supervisor Name:	Kazem Ghabraie
Date:	13/03/2016