VARIABILITY OF GEOTECHNICAL PROPERTIES OF MATERIALS WITHIN WAIRAKEI SUBSIDENCE BOWL, NEW ZEALAND

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ABSTRACT

At Wairakei geothermal field in New Zealand, subsidence has occurred since the onset of production in the 1950's. A good understanding of the stress-deformation behaviour of materials is important for understanding the phenomenon and being able to predict future subsidence. For many of the materials found in the Wairakei geothermal field simple, linear constitutive laws are sufficient for describing their stress-deformation behaviour. However, evidence shows that the stress-deformation behaviour of the formations responsible for majority of the subsidence at Wairakei is not represented well in this approach. In this work we have selected the Modified Cam-Clay model to describe the behaviour of these formations more accurately. The selection of the Modified Cam-Clay model is justified by considering the geotechnical properties of the materials using data obtained from published K_o triaxial tests, Atterberg limits and particle size analysis data. The consolidation and compression characteristics were also considered and data from samples taken at different depths were analysed using the Mohr–Coulomb model. The Modified Cam-Clay model was then employed in ABAQUS finite element analysis to reproduce the stress-deformation behaviour of a number of laboratory triaxial tests. This allowed the Modified Cam-Clay model parameters to be calibrated giving a clearer understanding of the nonlinear constitutive laws for the materials. The outcomes from this study form the basis for the characterisation of the stress-deformation behaviour and strength properties of materials for purposes of modelling subsidence at borehole WKM 15 within Wairakei subsidence bowl.

1. INTRODUCTION

The importance of understanding thermal geomechanical behaviour has increased enormously as a result of increased interest from a wide range of industries. Production of energy from geothermal systems, carbon sequestration and nuclear waste storage are all examples of applications that require a detailed understanding of thermo-hydro-mechanical (THM) behaviour of geological materials (Xiong *et al*. 2013; Tsang *et al*., 2004; Jing, 2003). The work presented in this paper is part of a larger project to analyse and predict THM behaviour in geothermal systems being utilized for energy production and in particular subsidence that may occur as a result. The interaction of thermo (T), hydro (H) and mechanical (M) phenomena can be complex. It occurs on a wide range of time scales and is dependent on not only the properties of the geothermal fluid flow but also the material properties of the geological units. Reliable estimates of behaviour of geological materials under these coupled thermo-hydro-mechanical conditions are essential for the accurate modelling of subsidence.

It is well known that the stress-strain volume change relationship of soils is dependent on a number of factors such as soil type, density, strain level and stress path (Duncan and Chang, 1970; Yudhbir and Varadarajan, 1975; Lade and Duncan, 1976). Therefore in order to determine an appropriate model for the stress-strain behaviour of a material some effort must be made to determine its geotechnical properties.

Geological materials within and above geothermal reservoirs often include a significant number of lithotypes. These geological materials evolve into clay formations due to hydrothermal alterations and a substantial increase in water content. This may lead to a reduction in mechanical properties such as strength and stiffness (Pinyol *et al*., 2007). Thus the mechanical properties can vary appreciably within a geothermal field. The variations may be caused by variability in the petrographic characteristics of the geomaterials (rock type, crystal content and mineral composition etc.). In addition, the variations in engineering index properties (e.g., density, porosity, and hardness) may also contribute to variations in strength and deformability.

In Section 2 the available geotechnical data is analysed for samples taken from the Wairakei subsidence bowl. The results of the analysis show that soft, clay material is found in several formations sampled which justifies using the Modified Cam-Clay (MCC) model to predict its nonlinear stress-deformation behaviour (Roscoe and Burland, 1968). While this soil model has been implemented in various commercial softwares such as PLAXIS and ABAQUS, there are no studies in literature comparing its predictions to field data for materials taken from geothermal systems. Section 3 discusses constitutive laws and gives a detailed description of the MCC model. A numerical modelling study using the model is presented in Section 4 with the results compared to K_o triaxial test results for samples from the Wairakei bowl. Through this process the parameters for the MCC model are estimated and it is demonstrated that the model predicts the stress-deformation behaviour well. This result shows that the MCC model is suitable for use within complex numerical simulations of the subsidence bowl at Wairakei. These simulations will form an important part of monitoring and predicting future subsidence at Wairakei. Brief descriptions of the Wairakei subsidence bowl and investigations of it are summarised in the following sections.

1.1 Wairakei Subsidence bowl

The Wairakei geothermal system (including its co-joined neighbour Tauhara) is located to the North of Lake Taupo in the central North Island of New Zealand. Subsidence was detected soon after the operation of geothermal power plant at Wairakei began in 1958. Subsidence rates increased from the 1950s to a peak in 1970s, followed by a decline to much lower rates at present (Bromley *et al*., 2013; Currie, 2010; Allis *et al*., 2009). In the most profound subsidence area, the Wairakei subsidence bowl near the Eastern Borefield, the peak rate was 498mm/year in 1978. This has now reduced to a current rate of 58mm/year (Currie, 2010). The centre of the Wairakei subsidence bowl has dropped by a total of approximately 15.1m since the 1950s. The total area of the subsidence bowl covers approximately 1 km^2.

The geology and structure of the Wairakei-Tauhara geothermal field is described and reviewed in Rosenberg *et al*., (2009) and Bignall *et al*., (2010). According to the compressible sequence, the formations responsible for compaction in Wairakei bowl included upper layers of altered tuff breccia within Waiora Formation (230-330 m), sub-units within Huka Falls Formation (75-230 m) and decaying peat/vegetation at shallow depth (30-45m) (Bromley *et al*., 2010; Rosenberg *et al*., 2009). Some of these formations are very soft and have been used previously to model subsidence at Wairakei (Koros *et al*., 2015; Koros *et al*., 2014; Bromley *et al*., 2013; Wanninayake, *et al*., URS, 2010).

1.2 Previous studies of the Wairakei bowl

Subsidence levelling surveys have since been conducted routinely across the Wairakei-Tauhara fields and permanent continuous Global Positioning System (cGPS) installed at the sites revealed presence of anomalous subsidence (Currie, 2010). The location and approximate subsidence rates of known bowls were confirmed with the use of satellite-based ground deformation image techniques involving Differential Interferometric Synthetic Aperture Radar (DinSAR) (Samsonov *et al*., 2009; Hole *et al*., 2007). The understanding of geotechnical properties of geological units within Wairakei bowl involved laboratory tests by Read *et al*., (2003); Grant, (2000); Allis (1999); Kelsey, (1987) and Robertson, (1984). Although stiffness, void ratio, yielding and stress-strain behaviour were investigated through consolidation tests, these tests did not determine a number of other geotechnical material properties, cohesion and friction angle that we think are important for a full understanding of subsidence within the Wairakei bowl.

A comprehensive subsidence investigation program initiated by Contact Energy Ltd in 2006-2010 involved extensive geotechnical analyses which included compressibility measurements, bulk rock properties and petrology tests (X-ray, diffraction, smectite abundance and scanning electron microscopy) (Bromley *et al.*, 2010; Lynne *et al.*, 2011). The work by Pender *et al*., (2013) discusses the K_o triaxial compression tests at ambient temperature carried out on selected samples and observed yielding behaviour in some samples as reported previously by Pender, (2009a, 2009b). These investigations led to substantial increase in knowledge of the geotechnical properties of the recovered cores as described in the following section.

2. GEOTECHNICAL PROPERTIES

Data from a range of studies and tests can be used to determine an appropriate model for describing the stress-deformation behaviour of formations responsible for the subsidence at Wairakei. This section summarises the findings of a number of studies and tests that show that, clay-like material is present in several of the key formations at Wairakei which justifies the selection of the Modified Cam-Clay model for describing their stress-deformation behaviour. Practical limitations meant that it was not possible to carry out each test on every sample but when viewed collectively a good understanding of the range of materials can be obtained.

The grain size distributions from six different locations in the core samples are shown in Figure 1. The distributions have been plotted using the data reported by Opus (2009). The figure shows that five of the six locations have a grain composition that largely consisted of clayey silt (i.e. when the boundary particle size is $< 2\mu m$).

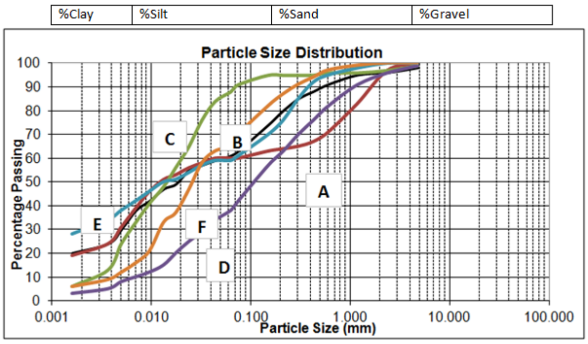

Figure 1: Particle size distribution of samples at A: Waiora (44%), B (82%) and C (90%): Huka Falls, D (57%): Oruanui and E (47%) and F (58%): Post Oruanui Formations at borehole WKM15. (Modified after Opus, 2009).

Two samples were also taken by Opus (2009) for which the Atterberg limits were determined. The results are shown in Table 1 and are within a typical range for clay properties presented in Table 2.

Table 1: Atterberg limits of the samples from borehole WKM 15. (After (Opus, 2009)).

Depth(m)	Liquid Limit W_L (%)	Plasticity Index PI(%)	Water content W_n(%)
151.25	78	34	63.5
250.8	75	47	30.9

Table 2: Typical plasticity description range (Reeves *et al*., 2006). W_P is the plastic limit.

Property	Range (%)	Plasticity Description
W_L	70-90	Very high
	50-70	High
W_P	> 35	Extreme plasticity
	17-35	Highly plastic
PI	> 25	High

Figure 2 shows the variation of water content and smectite content with depth obtained from Bromley *et al*., (2010). This indicates that the geological materials within the Wairakei bowl contain a considerable amount of smectite at various depths. Bromley, *et al*., (2010) have also shown that in general the main clay mineral is smectite. The soil materials with high plasticity indexes tend to be clay.

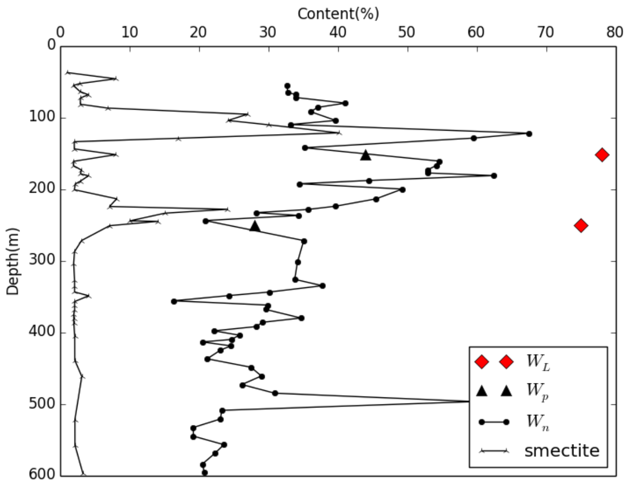

Figure 2: Variation of liquid limit, plastic limit, water content and smectite content with depth. (Modified after (Bromley *et al*., 2010)).

The K_o triaxial tests results given in (Pender, 2009a, 2009b) include plots of vertical effective stress versus axial strain and effective stress paths for a range of samples. The results show that the materials within the Wairakei bowl responded variably and but that the shallowest samples are largely normally consolidated as shown in Table 3. Between 1955 and 2009 pressure decline in the deep geothermal reservoir has propagated up to about 80m depth, but not any further because of low permeability mudstones and pressure support provided by shallow ground water. Below 80m depth pressure decline has increased the vertical effective stress causing historic "overconsolidation". Fine silts and clays

Table 3: Material at WKM 15. (After Pender, (2009b) and Bromley, *et al*., 2010).

Depth(m)	$\sigma'_{v(1955)}$ (kPa)	$\sigma'_{v(2009)}$ (kPa)	$stress\ ratio = \frac{\sigma'_{v(1955)}}{\sigma'_{v(2009)}}$
36.4-36.9	379	379	1
51.0-51.45	471	471	1
56.4-56.9	506	506	1
65.4-65.9	564	564	1
68.4-68.9	583	583	1
72.7-73.2	611	611	1
80.43-80.96	662	662	1
86.4-86.9	695	765	1
142.4-142.9	1073	1709	1

which are referred to as cohesive soils are significantly influenced by drainage conditions during testing and their history of deposition (i.e. normally consolidated or overconsolidated). The ratio of 1 in Table 3 means there has been no change (perhaps the drop in fluid pressure was not occurring in that depth range over that time). The stress change ratio describes the character of material response under given loading conditions. It is useful in establishing the initial yield surface of MCC model explained in Section 3.

The strength of soil material can be determined if shear strength parameters cohesion (c') and friction angle ($\emptyset'$) and effective stresses are known (*see* Figure 3). Normally consolidated clay materials are cohesionless ($c' = 0$) (Shrof and Shah, 2003) and tend to compress more when sheared. The effective friction angles $\emptyset'$ presented in Table 4 are mainly a function of clay mineral content and mineralogy of its composition. Different values of effective friction angle $\emptyset'$ may result from the difference in particle clay size of soil and effective normal stress at which friction angle was measured. Typical values of $\emptyset'$ for soft clay, stiff clay and shale constituents are in the range of (25° to 35°), (20° to 35°) and (15° to 35°) respectively (Terzaghi *et al*., 1996). An example of a failure envelope for intact samples of normally consolidated material within Wairakei bowl is shown in Figure 3. These samples were identified based on their failure response type in Table 4. The cohesion intercept is small hence stress circles are at failure envelope in Figure 3 and correspond to a normally consolidated condition. The mobilized angle of friction defined by tangent to Mohr circles passes through origin in Figure 3 and is a measure of strength mobilized for soil material to carry the applied stress. The mobilized friction angle at failure in this study is related to Modified Cam-Clay constitutive model parameters explained in Section 3.

Table 4: Material response at Wairakei bowl (Pender, 2009a and b).

Formation Type	**Material Response**	$\emptyset'$
Post Oruanui	Yielding	33°
Oruanui	Yielding/ softening/ Failure	17°-32°
Upper Huka Falls	Failure/softening/ stiffening	17°-29°
Middle Huka Falls	Yielding	10°-20°

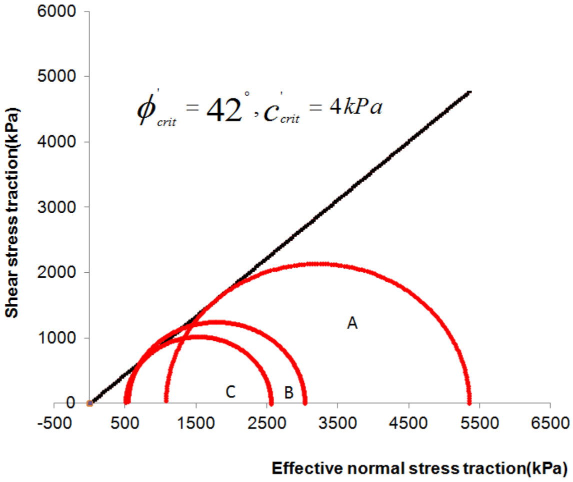

Figure 3: Failure circles and strength envelop for drained conditions on samples from A and C: Oruanui Formation (depth=72.7m), B: Upper Huka Falls (depth=80.43m) at WKM 15.

3. CONSTITUTIVE MODEL DESCRIPTION

When a material is stressed during geothermal fluid extraction, significant irreversible volume changes can occur resulting in an increase in the effective stress, σ' and a reduction in pore pressure. While the overall effect this has on the geological material is affected by drainage conditions, it has been shown by Terzaghi, (1936, 1943) and Biot (1941, 1956) that for most practical cases the effective stress tensor component σ'_{ij} is equal to intergranular stress and can be determined from the expression:

$$\sigma'_{ij} = \sigma_{ij} - \alpha P_p \delta_{ij} \qquad (1a)$$

Here σ_{ij} is total stress tensor, P_p is the pore-fluid pressure, i and j represent Cartesian coordinates directions x, y and z, and δ_{ij} is Kronecker delta, where

$$\delta_{ij} = \begin{cases} 1, & \text{if } i = j \\ 0, & \text{if } i \neq j \end{cases}$$

Biot's coefficient α (between 0 and 1) describes the relative contribution of total stress and pore pressure to the deformation of rock. For these materials $\alpha = 1$. For 1-D vertical stress compaction Equation 1a reduces to:

$$\sigma'_{zz} = \sigma_{zz} - P_p \qquad (1b)$$

Realistic prediction of this stress is vital for geotechnical engineering problems such as geothermal subsidence.

Generally, stress-strain response of soil, clay and mud consists of: pre-yielding quasi-elastic behaviour for stress conditions, a work hardening plastic behaviour and either a well-defined or narrow region of yielding along a boundary. To be able to predict the stress-strain response for a particular material, a constitutive model must be selected. Several methods for modelling the stress-strain response of soil and mud have been suggested and are discussed below.

Zienkiewicz and Naylor (1971) applied a critical state model to identify the yielding and represent work hardening of soil. Smith (1970, 1971) and Smith and Kay (1971) neglected pre-yielding elastic response and used the Modified Cam-Clay model to analyse the plane strain, drained behaviour of a pressurized thick cylinder of clay. Pinyol *et al*. (2007) considered a mechanical behaviour of soft clays using Modified Cam-Clay model to investigate the decaying structure of clay due to loading, wetting and drying.

This study proposes to describe the behaviour of the soft formations responsible for the subsidence at Wairakei using the Modified Cam-Clay (MCC) by Roscoe and his co-workers (Roscoe and Burland, 1968). The MCC has proven to be accurate in predicting the behaviour of soft clays under quasi-static and loading conditions (Wroth, 1975; Wood, 1990). Furthermore, MCC is defined by a few parameters which can be obtained from conventional laboratory tests (Schofield and Wroth, 1968; Atkinson and Bransby, 1978). The model is based on the Critical State concept which is widely accepted for simulating clay behaviour (Schofield and Wroth, 1968; Wood, 1990) and has been developed for isotropic clay materials. For a constitutive model to be useful, it should be simple and reflect the physical behaviour of materials. Model parameters should be determined easily from conventional tests and accurate prediction of stress-strain behaviour near failure.

3.1 Modified Cam-Clay Model in Triaxial Stress Conditions

A triaxial test is carried out in a cell and is so named because three principal stresses are applied to the soil sample (*see* Figure 4). Two of the principal stresses are applied to the sample by a water pressure inside the confining cell and are equal. The third principal stress is applied by a loading ram through the top of the cell and therefore may be different to the other two principal stresses.

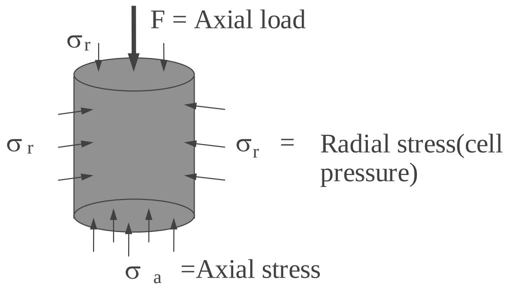

Figure 4: Triaxial stress state in a cylindrical test specimen.

The Modified Cam-Clay model (MCC) was developed based on triaxial compression tests carried out on isotropically consolidated samples. The model was described in terms of two stress variables, the mean effective stress p' and deviatoric stress (shear stress) q. Due to axial symmetry with $\sigma_2' = \sigma_3'$ (where σ_2' and σ_3' are intermediate and minor effective principal stresses, respectively), p' can be expressed as:

$$p' = \frac{(\sigma_1' + 2\sigma_3')}{3} \quad (2)$$

where σ_1' is the major effective principal stress. The deviator stress during pure shear is defined as:

$$q = \sigma_1' - \sigma_3' \quad (3)$$

For the triaxial state of stress,

$$d\varepsilon_v^p = d\varepsilon_1^p + 2d\varepsilon_3^p \quad (4a)$$

$$d\varepsilon_s^p = \frac{2}{3}\left(d\varepsilon_1^p - d\varepsilon_3^p\right) \quad (4b)$$

Here $d\varepsilon_v^p$ is plastic volumetric strain increment and $d\varepsilon_s^p$ is plastic shear strain.

In the plastic analysis, it is assumed that the associated flow rule holds for soils, which implies that the yield locus and the plastic potential coincide. The yield function and plastic potential may be represented by an ellipse in the $p' - q$ plane shown in Figure 5 as:

$$q = M[p'(p_c' - p')]^{0.5} \quad (5)$$

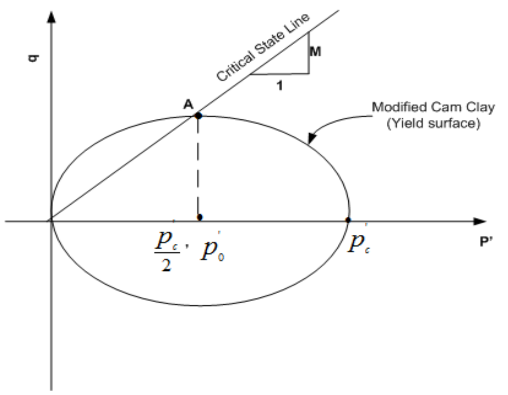

Figure 5: Yield surface for Modified Cam Clay model in the (p'-q) plane. (After Wood (1990)).

where M is a parameter whose value depends on the soil type and is determined from triaxial tests and p_c' is preconsolidation pressure that controls the size of the yield surface. This parameter is used in the definition of hardening behaviour of the soil.

Roscoe and Burland (1968) derived an associated plastic flow rule given by:

$$\frac{d\varepsilon_v^p}{d\varepsilon_s^p} = \frac{M^2 - \eta^2}{2\eta} \quad (6)$$

where $\eta = \frac{q}{p'}$ is the stress ratio. Note that $\eta = M$ when $q = q_f$ and $p' = p_f'$ at failure.

The parameter, M in (5) is defined as the stress ratio at the critical state, $(\frac{q_f}{p_f'})$, where q_f and p_f' are the mean effective stress and the shear stress (i.e. shear strength) at failure respectively. The critical state parameter is constant for the MCC model with isotropic plasticity.When anisotropic plasticity is considered, M may not be constant and its value linked to the three principal stresses (Wroth, 1984; Wood, 1990).

Roscoe and Burland (1968) and Wood (1990) explain that the critical state parameter M can be evaluated using Mohr-Coulomb failure criterion. Based on the MCC model, M for a triaxial compression test on isotropic consolidated samples and can be related to the corresponding effective friction angle, $\emptyset'$ as follows at failure $\frac{q_f}{p_f'} = M$,

$$M = \frac{6sin\emptyset'}{3 - sin\emptyset'} \quad (7)$$

In reference to Figure 5, the critical state line (CSL) has the following relation at failure in the $p' - q$ plane:

$$q_f = Mp_f' \quad (8)$$

Equation (8) represents the failure criterion used in the Modified Cam-Clay Model. It bears the same meaning as Mohr-Coulomb failure criterion expressed as:

$$\tau_f = c' + \sigma' tan\emptyset' \quad (9)$$

here τ_f is shear stress at failure, σ' is effective normal stress on the failure plane and c' is cohesion of the soil and is assumed to be zero for soft clays.

We used the MCC model implemented in ABAQUS for geomaterials, called the ‘Clay plasticity’ model (see ABAQUS, (2002) for details). The ABAQUS model is based on the yield surface presented below:

$$\frac{1}{\beta^2}\left(\frac{p'}{a} - 1\right)^2 + \left(\frac{q}{Ma}\right)^2 - 1 = 0 \quad (10)$$

where p' the mean effective stress, q is the deviatoric stress, β is a constant used to modify the shape of the yield surface and a is a hardening parameter $(a = \frac{p_0'}{(1+\beta)})$ (defined as a point on the p'-axis at which the yield surface intersects the critical state line in Figure 5). Equation (10) reduces to (5) in the case $\beta = 1$. Other parameters for Tabular hardening model as explained in Pogacnik *et al.*, (2015) include initial yield stress, p_y' and final effective stress, p_f' together with their corresponding plastic strains. These parameters control the post yield behaviour of the material.

4. NUMERICAL MODELLING

This study considers simulation of a simple triaxial test similar to that carried out in the laboratory experiments by Pender (2009a). Wanninayake *et al.*,(URS, 2010), also did an exercise of modelling one of Pender’s tests in order to get MCC parameters for PLAXIS simulation of Wairakei subsidence bowl. The constitutive behaviour of the specimen was modelled with Modified Cam-Clay plasticity provided in ABAQUS. The details of the simulations performed along with Modified Cam-Clay parameters are presented in the following section.

4.1 Model dimensions, Properties and Boundary Conditions

The geometry of the model is as presented in Figure 6. An axisymmetric soil specimen is fixed at the bottom and the top surface has a downward vertical motion (for compression).

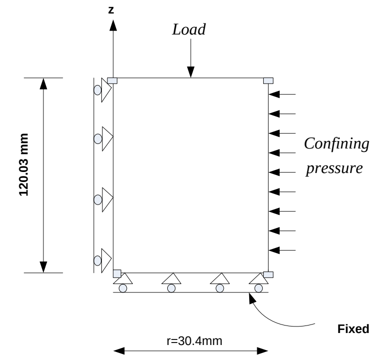

Figure 6: Triaxial consolidation: specimen dimensions and boundary conditions applied during simulation.

A perfect drainage is assumed so that the pore pressures, P_p throughout the specimen are constant. Analyses here were meant to simulate drained triaxial tests, which was effected through Python scripting with pure displacement elements in ABAQUS.

The material properties of the specimen WKM15UDT007b were derived from Pender (2009a, 2009b) and Bromley *et al.*, (2010). The properties for Modified Cam Clay model with porous elasticity for this particular specimen are shown in Table 5 and Table 6 respectively.

As the specimen is compressed, the elastic-plastic response of the specimen consists of two distinct behaviours. Elastically, the increased compressive hydrostatic effective stress on the material causes a stiffening response. When the material yields, inelastic deformation occurs resulting in a softer behaviour.

Table 5: Tabular hardening Cam-Clay parameters

Plasticity Parameter	**Value**
Stress ratio, M	1.1
Wet yield surface size, β	1.0
Flow stress ratio, K	1.0
Initial volumetric plastic strain, $\varepsilon^{pl}_{vol(y)}$	0.0
Final volumetric plastic strain, $\varepsilon^{pl}_{vol(f)}$	0.3
Initial yield stress, p'_y (Pa)	4.3e6
Final stress, p'_f(Pa)	4.8e6

Table 6: Elasticity material parameters.

Property	**Value**
Young's modulus E(Pa)	151e6
Poisson's ratio ν	0.23
Biot coefficient α	1

Ultimately, the stress state in some region of the specimen reaches critical state, where the material response becomes perfectly plastic. When this region is sufficiently developed, a limit state is attained and specimen's resistance to further compression no longer increases. The analysis in this study is intended to track the response of the material from initial loading to this limit.

4.2 Results and Discussion

At the start of a soil analysis with initial stresses, ABAQUS checks that stress specified does not violate the initial yield surface. Our initial stress state for this particular sample lay within the yield surface.

In the second step of analysis, the top surface of the model moves down for compression case. The material response is shown in Figure 7. From this figure, it can be seen that the material yielded progressively as the displacement increased until a critical state was reached. The experimental results for sample WKM15UDT007b (Pender, 2009a, 2009b) are also given in the figure which show that the MCC model does a very good job of predicting the material's behaviour.

5. CONCLUSIONS

We have presented the geotechnical properties of a range of the materials found within Wairakei subsidence bowl. Due to practical limitations in sampling and testing, the properties cannot be determined for all materials present. However, from the data available there is very strong evidence of the presence of clay material within the Wairakei subsidence bowl which contains significantly mechanically weaker engineering properties than the studied samples. The evidence also shows that there is sufficient justification for using the Modified Cam-Clay (MCC) model for predicting stress-strain behaviour of the material.

Numerical simulations of the simple triaxial test using the MCC model show that it correctly predicts the stress-strain behaviour of samples taken from the Wairakei subsidence bowl and that this approach can be used to obtain the model parameters required for subsidence modelling. It is the subject of future work to apply the calibrated material properties (from triaxial lab data) in a subsidence model of the Wairakei field.

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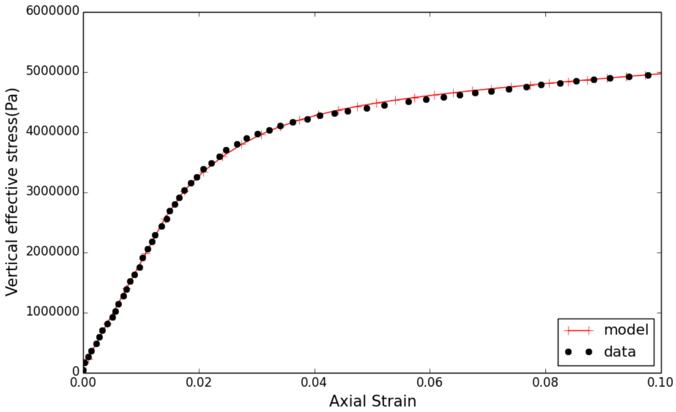

Figure 7: Prediction of consolidated drained triaxial behaviour of material at depth 72m of Oruanui Formation within Wairakei subsidence bowl using Modified Cam Clay model compared with experimental results.

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