Can One Use the Dynamic Cone Penetrometer to Predict the Allowable Bearing Pressure?

K. A. Mogotsi¹, F. H. van der Merwe²

¹GaGE Consulting, Johannesburg, Gauteng, Keabetswe@gageconsulting.co.za
²GaGE Consulting, Johannesburg, Gauteng, Frans@gageconsulting.co.za

Abstract

It is a common question asked by the structural engineer to the geotechnical engineer whether one can determine allowable bearing pressure from a set of Dynamic Cone Penetrometer (DCP) results. This often comes as a shock to the geotechnical engineer as the DCP test wouldn’t be applicable in most applications. The DCP was originally designed in South Africa by Kleyn (1975) and was originally intended to be used for pavement applications as an indicator test.

The test is done by driving a cone into the ground by means of an 8 kg standard mass falling through a constant distance of 575mm. The penetration depth is recorded after every 5 blows. A number of methods have been developed to estimate soil properties from the penetration rate. This paper discusses the DCP as a tool to predict the allowable bearing pressure.

Keywords: In-situ testing, Dynamic Cone Penetrometer, allowable bearing pressure estimation, economic testing.

1 Introduction

The objective of a subsurface investigation is to determine the engineering properties of the soils on which the foundations will be placed. Dynamic Cone Penetration (DCP) test is one of the most inexpensive field testing methods and is used worldwide in conjunction with various empirical correlations. Since its development, the DCP has been widely used as a simple, but effective means of determining the in-situ stiffness of subgrade materials, and can be used to determine the load bearing capacity of the soil. This paper discusses and compares means of establishing the allowable bearing pressure (ABP) from DCP readings.

2 Methodology and testing

In a DCP test, an 8 kg free fall hammer is lifted and dropped through a height of 575mm as shown in Figure 1. The distance of penetration of the cone tip is then recorded after every 5 blows and the cycle is repeated.
The DCP may also be referred to as a Dynamic Probe Light (DPL) which slightly departs from the European standard ISO/DIS 22476-2:2002. Both the DCP and DPL have similar energy inputs as shown in Table 1.

Table 1. Comparison of DCP and DPL

<table>
<thead>
<tr>
<th>Device</th>
<th>Hammer mass (kg)</th>
<th>Drop height (mm)</th>
<th>Cone angle (º)</th>
<th>Cone Diameter (mm)</th>
<th>Energy (J)</th>
<th>Diameter of rods (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>DCP</td>
<td>8</td>
<td>575</td>
<td>60</td>
<td>20</td>
<td>45</td>
<td>16</td>
</tr>
<tr>
<td>DPL</td>
<td>10</td>
<td>500</td>
<td>90</td>
<td>35.7</td>
<td>49</td>
<td>22</td>
</tr>
</tbody>
</table>

It should be noted from Table 1 that the cone diameter is significantly larger than the rod diameter for the DPL when compared to the DCP. The DPL will therefore have reduced friction acting on the rods if any friction is present. Therefore, the DCP should not ideally be undertaken over layer depths exceeding 1m, in one go (Paige-Green, 2009). Extraction of the DCP rod by 300mm and subsequent redrive of the rod can be considered to establish friction although the cone is not disposable and therefore not entirely correct.

A typical DCP logging sheet is shown in Figure 2.
Dynamic Cone Penetration recording sheet

<table>
<thead>
<tr>
<th>Date</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
</tr>
<tr>
<td>Blow</td>
</tr>
<tr>
<td>5</td>
</tr>
<tr>
<td>10</td>
</tr>
<tr>
<td>15</td>
</tr>
<tr>
<td>20</td>
</tr>
<tr>
<td>25</td>
</tr>
<tr>
<td>30</td>
</tr>
<tr>
<td>35</td>
</tr>
<tr>
<td>40</td>
</tr>
<tr>
<td>45</td>
</tr>
<tr>
<td>50</td>
</tr>
<tr>
<td>55</td>
</tr>
<tr>
<td>60</td>
</tr>
<tr>
<td>65</td>
</tr>
<tr>
<td>70</td>
</tr>
<tr>
<td>75</td>
</tr>
<tr>
<td>80</td>
</tr>
<tr>
<td>85</td>
</tr>
<tr>
<td>90</td>
</tr>
<tr>
<td>95</td>
</tr>
<tr>
<td>100</td>
</tr>
</tbody>
</table>

Figure 2. Typical logging sheet used by GaGE Consulting

3 Correlation between DCP and SPT

Lacroix and Horn (1973) proposed that nonstandard penetration resistance obtained from the DCP test, \( N_{30L} \), could be correlated with Standard Penetration Resistance, \( N_{30SB} \), for drive samples or a solid conical point, such as a static cone, which incorporated consideration of driving energy and distance of penetration. They reasoned that the energy required to drive the sampler or cone a given distance or “depth” (L) was directly proportional to the square of the outside diameter (D) and the distance of penetration, and inversely proportional to the energy per blow (Weight of hammer multiplied by the height of drop, WH).

The flowing equation was derived to compare DCP to SPT results:

\[
N_{30SB} = N_{30L} \left( \frac{WH}{48260} \right)^{\frac{1290}{OD^2}}
\]  

(1)

Where:

\( N_{30SB} \) = SPT-equivalent blow count, over 300mm,

\( N_{30L} \) = Measured blow count, over 300mm from DCP results,
W = Mass of DCP hammer (kg),
H = Fall-distance (mm),
OD = Outer diameter cone (mm),
ID = Inner diameter cone (mm),
48260 is the energy weight of the SPT test (760mm x 63.5kg),
1290mm is OD^2-ID^2 for the SPT test.

From the above equation, we can assume that the corrected SPT-N_{30SB} value will be equal to 30% of the N_{30L}.

\[ N_{30SB} = 0.3N_{30L} \]  
\[ (2) \]

4 Meyerhof’s Allowable Bearing Pressure (ABP)

The SPT test was developed in 1927 (Bowles, 1997), and has become one of the most popular in situ tests. According to Meyerhof (1956), the allowable bearing pressure of the soil may be obtained from SPT-N values.

The following equations where slightly adjusted by Bowels (1997) to determine the ABP of sandy soil:

\[ q_a = \frac{N_{30SB}}{F_1} k_d \quad \text{where } B \leq F_4, \]
\[ q_a = \frac{N_{30SB}}{F_2} \left( \frac{B+F_3}{B} \right) k_d \quad \text{where } B > F_4, \]
\[ k_d = 1 + 0.33 \frac{D}{B} \leq 1.33 \]  
\[ (3) \]

where:
\( q_a \) = allowable bearing pressure for \( \Delta H = 25\text{mm} \)
\( F = F\)-Factors as given in Table 2

If one reviews Terzaghi’s ultimate bearing capacity equations, the ultimate bearing capacity of a footing would increase if the footing size increases. These formulations are however based on conservative assumptions for the design of shallow foundations and the largest footing should not settle by more than 25mm and limited by the serviceability limit state. Therefore the 25mm is a maximum value and the formulas not intended to yield actual settlements.

Table 2. Meyerhof F factor values

<table>
<thead>
<tr>
<th>( N_{30SB} )</th>
<th>SI</th>
</tr>
</thead>
<tbody>
<tr>
<td>F1</td>
<td>0.05</td>
</tr>
<tr>
<td>F2</td>
<td>0.08</td>
</tr>
<tr>
<td>F3</td>
<td>0.30</td>
</tr>
<tr>
<td>F4</td>
<td>1.20</td>
</tr>
</tbody>
</table>
Figure 3 plots the ABP for a N30L at different foundation widths and embedment depths of 0m.

![Figure 3](image)

**Figure 3.** Allowable Bearing Pressure for a surface-loaded footing values at different N\textsubscript{30L} values

Although these formulations were not originally intended to be used for clayey materials, a factor of 6 to 21 times the N\textsubscript{30SB} can be used for clayey materials. The lower range will typically be for clays of low plasticity (SL-ML) and the upper range for clays of high plasticity (CH), (Aggour, 2002).

5 **Correlation between DCP and CBR**

A number of methods to correlate DCP penetration values and CBR have been derived by various authors. Paige-Green (2009), suggests that the following can be used to estimate the CBR of in-situ materials from the DCP test as previously published by the Transvaal Roads Department if DN > 2 mm/blow

\[
\text{CBR} = 410 \text{DN}^{1.27}
\]  

(4)

Where:
DN = Cone penetration rate (mm/blow)

The US Army Corps of Engineers as taken from Kessler (2010) recommend that the CBR of in-situ materials can be derived from the following equation:

\[
\text{CBR} = \frac{292}{\text{DN}^{0.77}}
\]  

(5)

They went on to suggest that the above equation can be used on all soils except for CL material. CBR values less than 10% and CH soils.
The following equations were suggested for these exceptions:

For CL soils $\text{CBR}<10\%$:

$$\text{CBR} = \frac{1}{(0.017019 \times \text{DN})^2} \quad (6)$$

For CH soils:

$$\text{CBR} = \frac{1}{0.002871 \times \text{DN}} \quad (7)$$

Figure 4 shows the CBR value versus penetration and shows that the US Army Corps and Transvaal Roads Department gives almost exactly the same values for all soils except for CL and CH values. CH soils results in slightly higher CBR values and CL almost identical.

![Figure 4. CBR Value plot against Penetration depths](image)

### 6 DCP and Bearing Capacity correlations (Packard, 1973)

Packard (1973) discusses the procedures given as part of the design of concrete airport pavements through the plate load test. This Plate-load test consists of a reaction or dead load and a hydraulic jack with dial gauges. Tests were undertaken on various CBR materials and the bearing capacity (bearing value) measured at 2.5mm of deflection which corresponds to the CBR test deflection.

Similarly, Paige-Green (2009) suggested that bearing capacity (bearing value) can be estimated from the following equation:

$$\text{Bearing capacity} = 3426 \times \text{DN}^{-1.014} \quad (8)$$

Figure 6 plots the CBR versus bearing capacity as taken from Figure 5 at 2.5mm deflection and from the Paige-Green’s correlation.
Figure 5. Interpretation of soil classification at different CBR
ith bearing capacity (Packard, 1973)
Packard (1973) is derived from various plate load test undertaken on materials with various CBR values and the bearing capacity measured at 2.5mm (same as CBR test). This doesn’t imply that the values are ultimate or allowable values and therefore it is considered that at lower CBR values the bearing capacity measured at 2.5mm deflection is closer to an ultimate value whilst at higher CBR values further from the ultimate load with higher factors of safety (FoS), but nevertheless unknown. It is not known if Paige-Green’s (2009) formulation are ultimate or allowable or how the formulation was derived.

7 Comparison between Meyerhof and Packard

Figure 7 compares the ABP for $N_{30L}$ at $B = 0.75m$ (foundation width) from Packard (1973) and the values derived from Meyerhof for $N_{30L}$ at various widths.
From Figure 7 it is concluded that the Packard (1973) bearing capacity at 2.5mm of deflection is at a lower FoS at low N\textsubscript{30L} values and at higher FoS at higher N\textsubscript{30L} values. It is therefore recommended that the Meyerhof N\textsubscript{30L} values be used in sands and that 2 to 6 times the N\textsubscript{30L} be used for clayey materials, depending on the plasticity.

8 Limitations of the DCP to predict ABP

Investigations are normally undertaken to establish stiffness and ground profile to a 2B influence zone below a foundation (SAPEM, 2014). Therefore, if undertaken only over a 1m depth, a footing of maximum 0.5m, can be placed based on the ABP derived. If a DCP is undertaken as a test pit progresses to 3m, one can probably place a footing of maximum 1.5m, based on the ABP derived. If larger footings are required a deeper investigation would be required.

If one profile is in sand and saturated, the ABP should be halved and if the water table is within 2B below the footing consideration should be given to reducing the derived ABP (Craig, 2007).

The reader should take cognisance of the fact that the ABP would reduce further if the footing is not placed on level ground but adjacent to a slope. This paper does not discuss the anticipated reduction and is based on ABP on level ground.

The N\textsubscript{30L} value for clayey materials should be used with discrete adjustment since silts and clays may be stiffened or softened depending on an increase or decrease of their moisture contents. N\textsubscript{30L} values can be reduced by half for CL, ML, SL and SM materials if not originally saturated and expected to become saturated during the structures life (Sathawara, 2013).

Additionally, the above limitations shows the importance to undertake foundation indicator and moisture content tests to establish the unified soil classification (USCS) and saturation level.

N\textsubscript{30L} average values should additionally be derived continuously with statistical methods.

9 Conclusion and limitations of the methods

Conventionally values derived for pavement tests have been used to estimate the ABP, these do not take footing size into consideration and were derived from plate load tests on different CBR(%) materials. From this paper, it is shown that an equivalent N\textsubscript{30SB} value can be derived from the N\textsubscript{30L} by using Lacroix and Horn’s formulations. These equivalent values can be used in Meyerhof’s ABP formulations for sands and computes well to plate load test undertaken on sands with various CBR (%) values. Discrete judgement should however be used in deriving the ABP from DCP results and the limitations as discussed on Section 8 should be noted.

Acknowledgement

The Authors would like to express their deepest gratitude to Mr Fernando Pequenino (GaGE Consulting) for his assistance during the executions of the paper.

References

The Relationship Between the Bearing Capacity of a Screw Pile and the Total Work Done During Its Installation

A. V. Mdlalose¹, K. Mbanya²

¹Transnet Group Capital, Durban, Kwa-Zulu Natal, amdlalose0@gmail.com
²Aurecon, Pretoria, Gauteng, kamombanya@gmail.com

Abstract

A relationship between bearing capacity of screw piles and the work done per cubic metre of soil bored into has been investigated in this work using a small screw pile. The aim is to add to the body of existing knowledge and hopefully formulate a more reliable method or basis of relating bearing capacity to installation parameters. Results show a linear relationship exists between bearing capacity and work done per cubic metre of soil bored into by a screw pile and confirms the relationship observed by Silva et al. (2012). However, a relationship that best fits the data was observed to be nonlinear and was approximated by a quadratic function that shows that bearing capacity of a screw pile increases at a decreasing rate with increasing total work done and tends to a constant value.

Keywords: Bearing capacity, screw piles, work done, installation,

1 Introduction

The ability to accurately relate the bearing capacity of a screw pile or an auger to measurable installation parameters (i.e. torque and crowd force) has the benefit of ensuring that the required bearing capacity is achieved on site. The above issues have led to the desire to investigate for a new possible relationship between installation parameters and bearing capacity that would have a better accuracy and reliability. The relationship that was investigated in this study was that between the bearing capacity of a screw pile and the total work done per cubic metre of soil bored into.

The investigation was conducted at the University of the Witwatersrand, Johannesburg, as the final year research project in civil engineering. Seven experiments were conducted on a small screw pile. The experiment involved rotating the pile into a clayey soil compacted to varying density, whilst determining the work done by torque and crowd force, and finally performing a load test to determine the bearing capacity achieved. One screw pile was used for all experiments. The only parameter that was varied was the degree of compaction (density)
achieved and consequently the shear strength of the soil achieved. Soil laboratory tests were conducted to classify the soil and determine its characteristic properties. These were the sieve analysis, atterberg limits tests (liquid limit and plastic limit tests) and the direct shear box test. Compaction tests (modified proctor test) were conducted in order to determine the optimal dry density and to observe how the density varied with compaction effort in order to get a sense of how the compaction was to be varied to achieve increasing densities for the experiments.

Preliminary screw plate bearing tests were also conducted to determine the capacity of the screw plate, so as to avoid loading the screw pile to failure, since one screw pile was to be used throughout the experiments. However, the critical buckling load of the screw pile shaft proved to be the limiting load as it was lower than that of the screw plate. 85% of the critical buckling load was used as the limiting maximum load to be applied to the screw pile if settlement equalling 10% of the diameter of the screw plate had not been achieved before this load was reached. If this happens the capacity of the pile will be unknown for that particular soil condition. The following limitations were also identified during the investigation:

- loss of strength per remoulding of the soil, therefore the soil properties were constantly changing;
- densities achieved are taken as uniform throughout the soil depth which is not true as the density increases linearly with depth;
- elastic shortening of the pile shaft not considered when measuring the settlement;
- all tests were done in one type of soil.

2 Experimental Procedure

This section gives details on the experimental plan: description of the screw pile used; apparatus used; experimental setup and procedure for installing the screw pile; and performing the load test. Soil laboratory tests carried out are also given, followed by an explanation of how each is relevant to the objectives of the investigation. Only one form of in situ test was conducted – this was the water content determination which was performed for each experiment to determine by how much the moisture content varied per experiment even though an attempt had been made to keep it constant by keeping the soil exposure environment the same.

2.1 Description of Screw Pile

A scaled model of a typical screw pile was used and a scale factor of 3 was chosen based on the practical limit of size of soil container available and weight of soil that could be handled. A typical screw pile has a shaft diameter of 60 mm, screw plate size of 240 mm in diameter and a pitch of 36 mm. The scaled model designed can be seen in Figure 1 (on the following page) with its scaled dimensions.
2.2 Experimental setup
The experimental setup is schematically shown in Figure 2. What follows now is a brief explanation of the experimental process.

Figure 2. Setup of the frame and pile 1 in the large Macklow Smith testing machine
2.2.1 Installation of screw pile
The screw pile was installed by applying a crowd force and torque from a pulley and rope system. The rope was wound up on the drum, passed through the two fixed pulleys on the supporting frame and the loading hooks were attached to the ends of the rope, see Figure 2 above. The loading hooks exerted forces on the two ends of the rope when loads were placed on them, which caused the rope on the drum to unwind, rotating the screw pile into the soil. During the process, the loading hooks dropped to the ground. In order to ensure the screw pile advanced into the soil, every time the loading hooks were allowed to drop, crowd force was applied by placing weights at the top of the shaft so as to make the screw pile press down on the soil, forcing it to bore into the soil every time it is rotated.

It has been recommended (Ruberti, 2015) that the rate of advancement of the screw pile into the ground be equal to the pitch of the screw plate per revolution. This is to ensure that little disturbance results in the soil as the screw pile advances.

The process of loading the loading hooks and allowing them to drop to the ground was repeated until a final embedment depth of 230 mm was achieved. This was set as the target depth for all the screw piles installed to ensure variability was not induced in the results obtained by different founding depths. The weights were applied to the hooks in increments of 5kg.

2.2.2 Parameters measured during installation
Since the aim of the investigation was to determine the relationship between bearing capacity and the total work done per cubic metre of soil bored into, the total work done in installing the pile had to be determined. To be able to evaluate the total work done per cubic metre of soil, the forces applied by a typical boring machine on a screw pile had to be identified. These have been identified, from literature consulted, as the torque which rotates the screw pile causing its plate to cut the soil, through shearing action. The crowd force and self-weight which cause the screw plate to press down on the soil and as the screw plate rotates, due to the torque, advance into the soil (Silva, et al., 2012). Therefore, the forces applied (torque, self-weight and crowd force) can be used in determining the total work done in boring the screw pile into the soil. Work done is the product of the force applied on an object multiplied by the displacement of the object in the direction of the applied force. Work is only done when the object moves in the direction of the force.

Therefore, in a boring machine (device) used to bore a screw pile into the soil, the total work done would comprise of the work done by torque, which is equal to the total torque applied to the screw pile multiplied by the total number of revolutions in boring the screw pile into its final position. Added to this the work done by the crowd force and self-weight, which is equal to the sum of the total crowd force and self-weight multiplied by the vertical displacement of the screw pile from its initial position to its final embedment depth. In terms of energy the three forces constitute to the total work done by torque, and the total work done by the fall, (i.e. change in potential energy), of the self-weight and crowd force.

With the forces required to determine the total work done per cubic metre of soil bored into identified, the screw pile and supporting frame were constructed in a way that would allow for the ease of measuring. For every drop of the two loading hooks, the following measurements and/or recordings were made:
- height through which the loading hooks fell from;
- depth of penetration;
- crowd weights applied;
- weights applied on the loading hooks (both loading hooks were loaded with equal weights)
The self-weight of the screw pile, including the fixed pulley drum along its shaft, and the loading hooks were also measured and included in the calculations done for determining the total work done.

The above measurements and recordings were used together with the equations and relationships given below in determining:

- work due to torque;
- work done by the crowd force and self-weight of the screw pile;
- total work done;
- total work done per cubic metre of soil;
- torque applied on the drum.

The total work done, $W$, is equal to the work done by the self-weight, $w$, of the screw pile and crowd force, $c$, $W(w + c)$, in moving it from its initial position into its final embedment depth plus the work done by torque, $W_T$. The total work done was then given by:

$$W = F_{(W + C)} \Delta h + 2F_T L_{rope}. \tag{1}$$

Eqn. 1 above was partially derived with reference to the information in the article written by Silva et al. (2012). The total work done per cubic meter of soil, $W_Y$, given by:

$$W_Y = \frac{w}{\pi r_p^2 \Delta h}. \tag{2}$$

Where:

- $W$ = Total work done [N.m]
- $F_{(W+C)}$ = Self-weight and crowd force [N]
- $\Delta h$ = Advancement of the screw pile into the soil [m]
- $F_T$ = Force the rope exerts on the drum fixed onto the screw pile shaft [N]
- $L_{rope}$ = Length of rope unwound, from the drum fixed onto the screw pile shaft, as the loading hooks are allowed to fall to the ground [m].
- $W_Y$ = Total work done per cubic metre of soil [N.m/m$^3$]
- $r_p$ = Radius of screw plate [m].

The above equation (Eqn. 2) gives the energy required to bore a screw pile into the soil, expressed in Newton metres i.e. Joules per cubic metre of soil. Torque applied on drum is given by, $T$ in [Nm]:

$$T = 2r_d F. \tag{3}$$

Where:

- $F$ = Force applied to the rope per side [N]
- $r_d$ = Radius of fixed drum on screw pile shaft [m]

Figure 3 below depicts the forces that are applied to the screw pile, that are used in determining the total work done per cubic metre of soil using the above equations (Eqn. 1 to Eqn.3).
2.3 Load testing
Once the installation process was complete, with measurements and calculations done, load test was carried out in accordance with ASTM D1143 as outlined in Hubbell Power Systems Manual (2014). In brief this involved setting up the screw pile under the Macklow Smith and loading it using this machine at increments of 0.3 kN and reading settlements from a dial gauge placed on the screw pile. The setup of the test has been schematically drawn in Figure 4.

2.4 Soil type
Clayey soil was used for the investigation. This was the only type of soil used and its moisture content was kept constant throughout the experiment, the soil was virtually dry (air-dry). To do this the soil was kept indoors. The reason for keeping the two parameters, type of soil and moisture content, constant was to limit any sources of variability in the investigation, since
clayey soil behaves differently when dry (low moisture content) from when it has a higher moisture content. This happens because the shear strength of cohesive soils, i.e. clays, is a function of two parameters cohesion and friction. When the soil is dry, only friction contributes to the shear strength of the soil and when the soil has higher moisture content, cohesion also contributes to the strength. Under this condition of low moisture content, cohesion was assumed to be zero as the soil showed no signs of cohesion; for example, it was not sticky when handled. The degree of compaction of the soil was, however, varied for each screw pile installation. This meant that this was an independent variable and was the only parameter that was varied to observe the relationship between the bearing capacity of the screw pile and the total work done per cubic metre of soil bored into. The relationship was investigated by plotting a graph of bearing capacity versus total work done per cubic metre of soil bored into and each point on the graph came from the 7 screw pile installations in the clayey soil, compacted to varying degrees per installation. The degree of compaction varied by 10 blows, starting from 10 blows for the first test to 70 blows for the seventh test.

3 Results

3.1 Introduction
This section presents and discusses the results of the laboratory tests and experiments (screw pile installation and load tests) that were conducted. Seven screw pile installations were made and for each installation a load test was carried out. The test was carried out to the ultimate capacity of the screw pile, which is defined as the capacity of the screw pile at a settlement equalling 10% of the screw plate diameter (Ruberti, 2015).

3.2 Laboratory results
3.2.1 Sieve analysis and atterberg limits tests results
The soil was classified according to the USCS, using the sieve analysis and atterberg limits test results, as fine clayey soil with an even grain distribution and with a fines content of 7 %. The grain distribution curve can be seen in Figure 5 below. From the grain distribution curve and plasticity chart the soil is defined as SP-SC, i.e. poorly graded sand-clay mixture or clayey sand with little fines.

![Grain Size Distribution Curve](image)

Figure 5. Grain size distribution curve for the soil sample

Screw piles can be installed in a wide range of soil types and conditions; however, they are limited to installation in soils that have the largest grain size less than about 60 % of the screw plate diameter and will not advance correctly in medium gravel, gravel and cobble deposits (Clayton, 2005). Therefore, the soil used in the investigation is satisfactory for installing a
screw pile, since from the grain size distribution curve, shown in Figure 5 above, the maximum grain size is 6.75 mm which is less than 60% of the screw plate diameter (48 mm). However, even though this condition was met it was observed that the screw pile failed to advance correctly in all the installations, it advanced into the soil at an angle of 2–5 degrees to the vertical.

3.2.2 Direct shearbox test results
Direct shearbox tests were conducted with an aim of obtaining the strength properties of the soil to be used for the investigation. The soil samples were unsaturated and compacted to varying degrees. Table 1 below shows the strength parameters for soil samples with different densities (degrees of compaction). The friction was noted to be high, the high moisture content could have contributed since the soil was not completely dry.

Table 1. Direct shear box test results

<table>
<thead>
<tr>
<th>Sample</th>
<th>Blows</th>
<th>Density (kg/m^3)</th>
<th>Effective friction angle (degrees)</th>
<th>Cohesion coefficient</th>
</tr>
</thead>
<tbody>
<tr>
<td>4</td>
<td>10</td>
<td>1633</td>
<td>42.5</td>
<td>0</td>
</tr>
<tr>
<td>5</td>
<td>30</td>
<td>1703</td>
<td>43.3</td>
<td>0</td>
</tr>
<tr>
<td>7</td>
<td>50</td>
<td>1722</td>
<td>44.6</td>
<td>0</td>
</tr>
</tbody>
</table>

3.3 Screw pile installation results
An empirical approach to the investigation has been followed as there were no means of assessing the actual interactions between the screw pile and the soil, i.e. how the screw plate cuts into the soil, magnitude of stresses it induces in the soil and their orientation. Therefore the analysis is limited to graphically analysing the data for any possible relationships that might exist between the investigated parameters.

3.3.1 Relationship between the bearing capacity of a screw pile and the total work done per cubic metre of soil bored into
The aim of the investigation was to investigate the relationship between the bearing capacity of a screw pile and the total work done per cubic metre of soil bored into. Figure 6 shows the observed relationship. A linear relationship has been observed between the bearing capacity and the total work done per cubic metre of soil bored into. The R^2 value (R^2 = 0.94) shows a strong correlation between these two parameters and it can be observed from the graph in Figure 6 that the data points plot close to the fitted regression line. A stronger correlation is shown by the nonlinear relationship given by the quadratic trendline as it has a higher R^2 value of 0.96. The relationship between the total work done per cubic metre of soil bored into and bearing capacity according to the quadratic trendline is that the bearing capacity is increasing at a decreasing rate and is tending towards constant value. This is reasonable as the soil must have an ultimate bearing capacity that should be reached eventually at high densities of the soil.

A stronger correlation is shown by the nonlinear relationship given by the quadratic trendline as it has a higher R^2 value of 0.96. The relationship between the total work done per cubic metre of soil bored into and bearing capacity according to the quadratic trendline is that the bearing capacity is increasing at a decreasing rate and is tending towards constant value. This is reasonable as the soil must have an ultimate bearing capacity that should be reached eventually at high densities of the soil.

Given that the data points do not lie far from the fitted linear trendline and because the R^2 value (R^2 = 0.94) is not that low in comparison to that of the quadratic trendline that fits the data best. It might be sufficient to use this relationship when correlating the work required to bore into the soil and the expected bearing capacity of the body of soil or strata. Since it is a much simpler relationship and will not lead to overestimations of the bearing capacity if it is used with the quadratic relationship which will give an estimate of the ultimate bearing capacity.
3.4 Load Test Results

Results from the load tests for all the piles that were successfully installed are depicted in Figure 7. The steepness of the graphs decreases with increasing blows per layer. This shows that the bearing capacity of the soil is increasing with increasing density as the compacted soil can take on more load with less settlement resulting from the increasing load.

Figure 7. Load test results

3.5 Relationship between the work done per m$^3$ and bearing capacity based on the expected and actual penetration

The requirement that the screw pile must advance into the soil at a rate of 1 pitch per revolution is called, in this report, the expected rate of penetration and the work that would result from it, the expected work. This represents ideal conditions were the screw pile advances into the soil with minimal disturbance. What now follows is a comparison between expected and what actually took place when the experiment was conducted.
The expected penetration assumes that the pile advances at a constant rate of 1 pitch per revolution. The actual penetration considers the penetration rate that actually took place. It was observed that the actual rate of penetration was less than the expected rate of penetration. The expected rate of penetration did not take into account the amount of resistance provided by the soil at that particular depth and therefore does not state how much torque and crow force are needed to maintain the constant rate of 1 revolution per pitch. It can be observed from Figure 8 below that the expected rate of penetration underestimates the total work required to rotate the pile through the soil by taking into account resistance of the soil. The expected work shows a stronger linear correlation between bearing capacity and total work done per cubic metre of soil bored into as compared to the actual work. This further proves that a linear relationship exists between the two parameters, i.e. between bearing capacity and total work done per cubic metre of soil bored into.

4 Conclusion

A total of 7 screw pile installations were made and for each installation a quick static load test was carried out. This study has shown a linear relationship between the bearing capacity of a screw pile and the total work done per cubic metre of soil bored into exists and this confirms what was found by Silva et al. (2012). However, a relationship that best fits the data was observed to be nonlinear and was approximated by a quadratic function that shows that bearing capacity of a screw pile increases at a decreasing rate with increasing total work done and tends to a constant value. It was found that the more compact a soil sample was, the greater the total work required to rotate the helical pile through the soil. In addition to the main investigation other relationships were also investigated. Relationship between the shear strength achieved after compaction and the total work done per cubic metre of soil bored into; relationship between the density of soil achieved after compaction and bearing capacity of the screw pile; and the relationship between density of soil achieved after compaction and the total work done per cubic metre of soil bored into. It was found that the total work done per cubic meter of soil bored into increased with increasing shear strength and density of the sample.
References
Ruberti, M. 2015. Investigation of Installation Torque and Torque to Capacity Relationship of Screw Piles and Helical Anchors, Amherst: Department of Civil and Environmental Engineering University of Massachusetts.
Proceedings of the 9th South African Young Geotechnical Engineers Conference, 13, 14 & 15 September 2017 – Salt Rock Hotel, Dolphin Coast, Durban, KwaZulu-Natal

Tzaneen Dam Raising – Geotechnical Investigation

C. van Dyk

1ARQ Consulting Engineers (Pty) Ltd, Pretoria, Gauteng, coert@arq.co.za

Abstract

A field investigation was conducted for the proposed raising of Tzaneen Dam, a composite earth fill embankment dam. This paper describes the part of the investigation that focussed on the inclined clay core of the dam. The sensitive nature of the clay core did not permit conventional rotary core drilling to be employed as this could potentially cause hydraulic fracturing of the core resulting in leaking, internal erosion and ultimately failure of the dam. The geometry of the embankment and inclined configuration of the clay core also added to the difficulty in obtaining info relating to the impervious material. A methodology comprising continuous SPTs and shelby sampling was developed. The majority of the shelby samples had to be hammered in, which raised concerns on the quality of samples obtained. However, good material recovery was achieved and the strength parameters obtained from 10 triaxial tests correlated well with typical strength parameters from literature.

Keywords: Embankment dam, undisturbed sampling, triaxial test, standard penetration test

1 Introduction

The proposed raising of Tzaneen Dam was defined at preliminary design level as part of the Groot Letaba River Water Development Project (GLeWaP) by the Department of Water and Sanitation in 2010 (DWS, 2010).

Tzaneen dam is a composite earth fill embankment dam with a central concrete ogee spillway. The dam wall is approximately 1150 m in length with a maximum height of 52 m and was built in 1977. The proposed raising will add an additional 3m to the spillway and approximately 2.4m to the embankment non-overspill crests. The latter is likely to be undertaken by means of engineered fill and thus compatibility with the existing clay core will need to be assured.

ARQ (Pty) Ltd was appointed to do the geotechnical investigation as well as the design of the proposed raising. The scope of the geotechnical investigation included identifying material sources to be used for the proposed raising and also to assess the foundation conditions. The primary aim, however, was to assess the existing embankment material for subsequent inputs in the stability analyses during the design phase. It is the assessment of the existing embankment material, and specifically the clay core, which is the focus of this paper.
2 Geotechnical investigation of existing embankment material

During the planning and execution phase of the geotechnical investigation, close collaboration was required between ARQ’s geotechnical engineers, geologists and dam engineers. This collaboration was critical to ensure that the correct information required for design was obtained at a high level of quality and within the allocated budget and time constraints.

The embankment geometry had a significant impact on the investigation and is discussed in the following sub-section.

2.1 Embankment geometry

According to as-built information, the existing embankment comprises semi-pervious, impervious (clay core), and filter material, with the core being inclined. The upstream face has a 1V:3H (~18°) slope and the downstream face a 1V:2H (~27°) slope. Both faces are protected with a 150mm thick paving block surface. The existing crest is some 8.7m wide and was specifically designed as such to allow for future raising of the dam wall. A cross-section of the embankment is shown in Figure 1 and a photo in Figure 2.

![Figure 1. Cross section of the embankment](image1)

![Figure 2. Crest and upstream slope of the dam](image2)
2.2 Investigation methodology
Characterization and sample collection of the semi-pervious material was achieved by conventional rotary core drilling conducted from the crest. Six boreholes were drilled from the crest into the semi-pervious material and advanced into the foundation material. Various Shelby samples were taken for laboratory testing including foundation indicator, dispersivity, and tri-axial tests (which included a permeability stage).

The sensitive nature of the clay core (impervious material and inclined configuration), however, did not permit rotary core drilling to be employed as an investigation method as this could potentially cause hydraulic fracturing of the clay core. Hydraulic fracturing (or hydrofracturing) occurs when the vertical effective stress in the clay core is reduced to small enough levels that tensile fracture of the clay core occurs (Djarwadi et al., 2014). In the case of rotary core drilling, the pressures associated with the drilling fluids utilized can cause a reduction in the vertical effective stress (effective stress = total stress minus pore water pressure) to such an extent that tensile fracture of the clay core occurs, resulting in leaking and internal erosion of the clay core and ultimately possible failure of the dam.

The geometry of the embankment (height, slope angles, smooth paved face and inclined clay core) also added to the difficulty of obtaining info for the impervious material, which is integral to the performance of the dam. Other methods were thus considered for the sampling of this material. The investigation of the clay core is discussed in the following section.

3 Sampling of the clay core
As mentioned above, an alternative to rotary core drilling was required to sample the clay core. Required material properties pertaining to the clay core included the grading envelope, Atterberg limits, permeability, and shear strength parameters. Various methods were considered and are discussed in the following sub-sections.

3.1 Methods considered but not used
The following methods were considered for the sampling of the clay core, however they were not used due to various flaws.

3.1.1 Sonic Drilling
Initially sonic drilling was the preferred method for sampling the clay core because no drilling fluid is required. The drilling head is advanced by thrusting, rotating and vibrating it, with the vibration “liquefying” the surrounding particles for effortless penetration. Very good quality core can be recovered using this method which was another advantage of using this method.

The problem however was the size of the rig (Crawler Rig Model SDC550-18, 312C Rigid, 16ton, 6.2m long and 2.5m wide) and the permitted inclination at which it can drill. The maximum permitted inclination is 10°, which rendered drilling an inclined hole from the crest and into the inclined core unfeasible. Furthermore, drilling on the upstream slope of the dam wall (18° slope) would have required the construction of temporary drilling platforms to provide at least an 8° slope - an option which proved prohibitively time consuming and costly. The risk of an anchorage failure could also have resulted in the rig slipping into the water below. Due to these limitations and risk the option of Sonic Drilling was abandoned.

3.1.2 Window sampling
Window sampling was considered using a Dando Terrier rig, however there were concerns relating to whether the required depths (around 7.5m) could be reached in the compacted embankment fill. Working on the slope of the embankment was also a concern. Due to these uncertainties this method was abandoned.
3.1.3 Augering
Augering using “hand held” equipment was considered, however, as with the window sampling option, there were concerns relating to whether the required depths (around 7.5m) could be reached. Furthermore it would not be possible to take undisturbed samples at depth (around 6-7.5m) with this method.

3.2 Chosen method – Continuous SPT and Shelby sampling
After all the above mentioned methods were abandoned, the methodology detailed below was developed for the sampling of the clay core. This method was deemed to satisfy all of the requirements of the investigation. The method was termed: Continuous SPT and shelby sampling.

1. On the upstream slope of the dam, continuous standard penetration tests (SPTs) with a Raymond spoon sampler would be undertaken, with material recovered every 450mm. Samples were wrapped in plastic and stored in core boxes. The continuous SPTs were advanced up to midway into the semi-pervious layer on the upstream side of the dam. SPT blow counts are recorded as per the normal SPT procedure.

2. Once penetrated about midway into the semi-pervious material, the hole (which at that time has a diameter of about 50mm) should be increased to 65mm to enable an undisturbed shelby sample to be taken at the bottom of the hole. It is proposed to do this by pushing a 60° 65mm diameter steel cone welded to a steel rod down the hole. Alternatively, the contractor can propose a different method and submit it to the engineer for approval.

3. After the shelby sample is obtained the SPTs should continue until it has penetrated approximately 1m into the clay core. A competent person (geologist or geotechnical engineer) should be on site to verify that the clay core has indeed been penetrated.

4. The hole is again widened to enable undisturbed shelby samples to be taken at the bottom of the hole. After widening of the hole, 3 undisturbed shelby samples (63mm diameter) are taken from the bottom of the hole directly after each other (head to tail). The first shelby is intended to remove all the waste material that could have been transferred to the bottom of the hole during the widening process. The second and third shelys are for tri-axial testing. The shelby’s must be pushed (not knocked) in. Only when the material stiffness is such that the shelys cannot be pushed in, may gentle knocking be employed by using the SPT hammer. It is emphasized that shelys should be pushed as far as possible, and they may only be knocked if a competent person (geologist or geotechnical engineer) gives approval and acknowledges it is not possible to push them in. A competent person must supervise all the shelby sampling and also verify that the correct material (clay core) is sampled. The shelys should be properly sealed and be placed in the core boxes.

5. After the shelby samples are taken, the entire hole should be filled from the bottom (tremmied) with a water-cement-bentonite grout. The holes must be filled shortly after the last shelby is taken, and holes are not to be left open overnight.

The above methodology was chosen for the following reasons:
• No pressurized water is used during the process which mitigates the risk of hydro-fracturing the clay core.
• It is a simple and quick process.
• The size of rigs required is comparatively small and therefore working on the slope is easier and safer.
• The required depths of some 7.5m can easily be reached.
• A continuous evaluation of the strength and stiffness of the semi-pervious and impervious layers are obtained through the SPT N values. These N values are valuable for deriving inputs for the stability analyses.
• Continuous soil samples are obtained via the Raymond spoon sampler.
• Undisturbed samples are obtained for laboratory testing. Firm to stiff clayey soils are expected and hydraulically jacked thin walled open-drive tube samples (commonly referred
to as Shelby samples) are the recommended sampling method for this type of soil (Clayton et al., 1982).

- Cost efficient.

3.2.1 Execution of the chosen method

The above mentioned methodology was sent out for tender together with quantities and borehole positions. Three tenders were received of which two were deemed to be technically sufficient and reasonably priced. One contractor proposed using a small DPSH rig (the rig weighing as little as 135kg) and using a 60°, 65mm diameter steel cone welded to a DPSH rod to widen the hole in order for the Shelby samples to be taken. The small rig was ideal for working on the slope and did not require a platform to be constructed. However, there were some concerns as to whether the light rig would provide enough downward force for the Shelby samples to be pushed into the clay core which was expected to be firm to stiff.

The successful contractor used a conventional skid mounted rotary core rig (YWE D90 model) and used an N-size split tube barrel to widen the hole before Shelby sampling. The skid mounted rig weighed much more than the small DPSH rig (some 950kg) and was therefore deemed more suitable for pushing Shelby samples. The rig was lowered down the embankment slope on a steel cable connected to a winch mounted to an SUV. The cable passed through a pulley system connected to the back of an 8ton truck which was situated on the crest. Once at the drilling position, steel rods were knocked into the embankment (between the paving blocks) against which a steel beam was placed underneath one of the rig’s skid tracks to level the rig. During the entire operation the cable remained fixed to the rig as a safety line should the platform gave way. Thereafter the methodology given in Section 3.2 was followed. Photos of the drilling rig setup on the upstream slope of the dam are shown in Figure 3 and Figure 4.

![Figure 3. Rig setup in progress on upstream slope of dam](image-url)
The operation was completed within 13 days (continuous SPT and Shelby sampling conducted from 25 May to 6 June 2016). In total 7 holes were investigated with depths ranging from 5.35m to 7.45m. A total of 43 SPT tests were conducted and 32 Shelby samples were taken, 7 in the semi-pervious material and 25 in the clay core. An ARQ engineering geologist was present on site to verify that the clay core was indeed penetrated. He also logged the material recovered from the SPTs. The drilling foreman communicated throughout the operation with the author on aspects such as sampling and the use of the SPT hammer. During the Shelby sampling, especially in the clay core, the rig struggled to push the tubes deep enough to allow for sufficient sample recovery for triaxial testing. Ultimately, almost two thirds of the total Shelby length recovered was knocked in using the SPT hammer. The author was aware that this might overstrain/disturb the samples but it was decided that potentially disturbed samples trumped no samples at all! The results obtained from this operation are discussed in the following section.

4 Results from the clay core investigation

The information gathered included SPT N-values, logging of the associated core, as well as lab testing results, including foundation indicators and triaxial testing. These are discussed in the following sub-sections.

4.1 Foundation indicator results and SPT sample logging

From the samples recovered from the SPTs the semi-pervious material was generally described as a clayey sand and the impervious material as a gravelly clay. From the foundation indicator results the semi-pervious material was classified according to AASHTO as A-6 or A-7-6 (clayey soils) and according to the Unified Classification System (UCS) as CL or SC (clay of low plasticity and clayey sand respectively). It had an average liquid limit of 35, an average grading modulus (GM) of 0.75, and an average plasticity index (PI) of 13. The impervious material was classified as A-6 or A-7-6 according to AASHTO and CL according to UCS. The impervious material had an average liquid limit of 39, an average grading modulus (GM) of 0.59 and an average plasticity index (PI) of 16. A summary of the foundation indicator results of the impervious material is given in Table 1.
Table 1. Summary of foundation indicator results

<table>
<thead>
<tr>
<th>Borehole</th>
<th>Depth</th>
<th>GM</th>
<th>LS</th>
<th>LL</th>
<th>PL</th>
<th>PI</th>
<th>AASHTO</th>
<th>UCS</th>
</tr>
</thead>
<tbody>
<tr>
<td>SD10</td>
<td>4.14-4.69</td>
<td>0.56</td>
<td>7.5</td>
<td>38</td>
<td>21</td>
<td>17</td>
<td>A-6</td>
<td>CL</td>
</tr>
<tr>
<td>SD10</td>
<td>4.69-5.24</td>
<td>0.74</td>
<td>9.5</td>
<td>44</td>
<td>25</td>
<td>19</td>
<td>A-7-6</td>
<td>CL</td>
</tr>
<tr>
<td>SD10</td>
<td>5.24-5.79</td>
<td>0.55</td>
<td>6.5</td>
<td>35</td>
<td>23</td>
<td>12</td>
<td>A-6</td>
<td>CL</td>
</tr>
<tr>
<td>SD11</td>
<td>5.95-6.50</td>
<td>0.37</td>
<td>9</td>
<td>42</td>
<td>24</td>
<td>18</td>
<td>A-7-6</td>
<td>CL</td>
</tr>
<tr>
<td>SD11</td>
<td>6.50-7.05</td>
<td>0.62</td>
<td>9.5</td>
<td>40</td>
<td>23</td>
<td>17</td>
<td>A-6</td>
<td>CL</td>
</tr>
<tr>
<td>SD12</td>
<td>6.49-7.04</td>
<td>0.42</td>
<td>7.5</td>
<td>40</td>
<td>26</td>
<td>14</td>
<td>A-6</td>
<td>CL</td>
</tr>
<tr>
<td>SD13</td>
<td>4.40-4.95</td>
<td>0.58</td>
<td>10.5</td>
<td>46</td>
<td>26</td>
<td>20</td>
<td>A-7-6</td>
<td>CL</td>
</tr>
<tr>
<td>SD13</td>
<td>5.25-5.80</td>
<td>0.67</td>
<td>7</td>
<td>34</td>
<td>24</td>
<td>10</td>
<td>A-6</td>
<td>CL</td>
</tr>
<tr>
<td>SD14</td>
<td>4.28-4.83</td>
<td>0.61</td>
<td>6.5</td>
<td>34</td>
<td>22</td>
<td>12</td>
<td>A-6</td>
<td>CL</td>
</tr>
<tr>
<td>SD15</td>
<td>6.35-6.90</td>
<td>0.75</td>
<td>9.5</td>
<td>40</td>
<td>19</td>
<td>21</td>
<td>A-6</td>
<td>CL</td>
</tr>
</tbody>
</table>

4.2 SPT N-values
In general the SPT N-values ranged from 4 to 39, with an average of 17 and a standard deviation of 7. SPT N-values in the upper zone of the semi-pervious zone (before the first undisturbed sample was taken) ranged from 4 to 18, with an average of 11 and a standard deviation of 4. SPT N-values in the impervious zone (immediately above the first undisturbed samples in the clay core) ranged from 15 to 39, with an average of 24 and a standard deviation of 8. Therefore, on average, the consistency of the upper portion of the semi-pervious zone is soft to firm, and that of the impervious zone as firm to stiff.

4.3 Shelby sampling
Hammering a sample tube into the soil with repeated blows of a drop hammer provides a lower quality sample than pushing the sample in with a steady force (Hvorslev, 1949). Therefore each sample was pushed in as far as possible and only then hammered in if required. As mentioned in Section 3.2.1 almost two thirds of the total Shelby length recovered was knocked in using the SPT hammer, with only a third being pushed. The reason for this is deemed to be a combination of the relatively good consistency of the soil (compacted embankment), the relatively light weight of the drilling rig, and in some instances the gravel particles in the soil which increased the soil resistance to penetration. The recovery of each sample was immediately reported to the engineer and if the recovery was less than 360mm (approximately the amount of sample required for a triaxial test i.e. 3 specimens) an additional Shelby was added. In general the material recovery was good with an average of 83% (450mm) recovery per Shelby. The effect of the hammering on the test results were unknown and it remained to be seen whether the samples recovered would provide realistic results.

4.4 Tri-axial testing
Eleven samples were sent to the laboratory for consolidated undrained triaxial tests (with the exception of SD10 4.69-5.24m which was a consolidated drained triaxial test). One sample was found to be disturbed upon extraction and, as such, was not tested. A summary of the shear strength parameters determined from the triaxial tests are given in Table 2 and a statistical analysis of these parameters are given in Table 3.
Table 2. Strength parameters from triaxial tests

<table>
<thead>
<tr>
<th>Borehole</th>
<th>Depth (m)</th>
<th>c' (kPa)</th>
<th>φ' (°)</th>
</tr>
</thead>
<tbody>
<tr>
<td>SD10</td>
<td>4.14-4.69</td>
<td>6.5</td>
<td>33.4</td>
</tr>
<tr>
<td>SD10</td>
<td>4.69-5.24</td>
<td>8.2</td>
<td>32.5</td>
</tr>
<tr>
<td>SD10</td>
<td>5.24-5.79</td>
<td>5</td>
<td>28.3</td>
</tr>
<tr>
<td>SD11</td>
<td>5.95-6.50</td>
<td>1.4</td>
<td>25</td>
</tr>
<tr>
<td>SD11</td>
<td>6.50-7.05</td>
<td>20</td>
<td>30</td>
</tr>
<tr>
<td>SD12</td>
<td>6.49-7.04</td>
<td>8.4</td>
<td>21.1</td>
</tr>
<tr>
<td>SD13</td>
<td>3.55-4.10</td>
<td>sample disturbed</td>
<td></td>
</tr>
<tr>
<td>SD13</td>
<td>4.40-4.95</td>
<td>6.2</td>
<td>31.1</td>
</tr>
<tr>
<td>SD13</td>
<td>5.25-5.80</td>
<td>26.9</td>
<td>12</td>
</tr>
<tr>
<td>SD14</td>
<td>4.28-4.83</td>
<td>15</td>
<td>32.5</td>
</tr>
<tr>
<td>SD15</td>
<td>6.35-6.90</td>
<td>6.5</td>
<td>18.9</td>
</tr>
</tbody>
</table>

Table 3. Statistical analysis of strength parameters

<table>
<thead>
<tr>
<th>Parameter</th>
<th>c' (kPa)</th>
<th>φ' (°)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Average</td>
<td>8.6</td>
<td>28.1</td>
</tr>
<tr>
<td>Standard Deviation</td>
<td>5.6</td>
<td>5.3</td>
</tr>
<tr>
<td>Coefficient of Variation (%)</td>
<td>65.2</td>
<td>18.8</td>
</tr>
<tr>
<td>Cautious Estimate</td>
<td>5.8</td>
<td>25.5</td>
</tr>
</tbody>
</table>

All of the samples tested had dilatant behavior. The results from SD13 (depth 5.25-5.80m) are clearly not realistic (especially the very low friction angle of 12°) and were therefore discarded. The strength parameters of the remaining 9 samples that were tested, producing an average c' and φ' of 9kPa and 28° respectively, are deemed to be realistic and are within the expected range of values for the specific material type. As an example, NAVFAC DM7 (1971) quotes typical strength characteristics of c'=12 kPa and φ'=28° for a compacted soil classified as CL which is very close to the values obtained from the investigation. For design purposes, however, cautious estimate strength parameters (cautious estimate being defined as the average minus a half of the standard deviation) were utilised and are provided in Table.

Strength and stiffness of soil is affected by sampling (Clayton et al., 1982). The strength of bonded or heavily consolidated soils is especially affected by sampling. For this investigation it seems that the strength of the clay core (a compacted imported soil) was not too greatly affected by the sampling method used as the strength parameters from the triaxial tests were within the expected range of values. The effect of the sampling on the stiffness is difficult to assess especially because local strain measurement techniques were not used during the triaxial testing. Therefore it was not attempted to investigate the affect that the sampling had on the soil stiffness. For design purposes, the stiffness of the semi-pervious and impervious material was derived from the SPT N-values.

As the main aim of the triaxial testing of the investigation was to obtain strength parameters of the clay core, the method used for sampling is deemed to have been successful as parameters correlating well with literature were obtained.
4.5 Permeability tests
Time constraints on the programme, as well as the clayey nature of the material, necessitated the use of side drains during the triaxial testing. Due to these side drains, permeability specific triaxial tests could not be conducted and therefore it was decided to conduct falling head permeability tests on reconstituted samples (as per the densities obtained from the undisturbed samples) to determine the permeability of the clay core material. The results of the permeability tests are given in Table 4.

<table>
<thead>
<tr>
<th>Sample No.</th>
<th>SD11</th>
<th>SD12</th>
<th>SD14</th>
<th>SD15</th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth (m)</td>
<td>5.40-5.95</td>
<td>5.94-6.49</td>
<td>4.83-5.38</td>
<td>6.9-7.45</td>
</tr>
<tr>
<td>Permeability (m/s)</td>
<td>7.50E-08</td>
<td>4.67E-07</td>
<td>1.01E-09</td>
<td>6.45E-08</td>
</tr>
</tbody>
</table>

The permeability values obtained show that the material is practically impervious (BS 8004, 1986) and is therefore fit for use as a clay core in an embankment dam.

5 Conclusion
The inclined clay core of the Tzaneen dam was investigated to establish input parameters for the design of the proposed raising of the dam. The embankment geometry as well as the sensitive nature of the clay core (risk of hydraulic fracturing) posed various challenges during the investigation. Several investigation methods were considered and ultimately SPTs and Shelby sampling was conducted using a skid mounted rotary core rig.

Undisturbed samples were sent to the laboratory for foundation indicator, permeability and triaxial testing. There were concerns regarding sample disturbance during the shely sampling as the SPT hammer was required to obtain sufficient sample in several instances. However, the strength parameters obtained from the triaxial testing correlated well with typical values for the given soil type and therefore sample disturbance is deemed not to have been significant. The investigation, although it had its challenges, is deemed to have been successful in providing input parameters for design purposes.

Acknowledgements
I would like to thank the following people for their involvement in this project:

- Mr Steven Arumugam – Acting Deputy Director General at the Department of Water and Sanitation.
- Mr David Cameron-Ellis – ARQ Dams and Hydro Director and dam engineer on the project.
- Mr Deon Blignaut – Engineering Geologist.
- Ms Madaleen Booysen – ARQ Geologist.
- Mr Steve Mandumba – ARQ Engineering Geologist.
- Ms Katy O’Brien – ARQ Geotechnical Engineer.
- Mr Richard Weppelam and Mr Christo Bekker – RWBE Drilling.
- Mr Hennie Barnard – Geostrada.
References

Department of Civil Engineering. University of Surrey.
Comparison between Various Correlations to Determine Soil Parameters for Platinum Tailings Using Piezocone Testing

P. F. Oelofse

1SRK Consulting, Johannesburg, Gauteng, POelofse@srk.co.za

Abstract

Piezocone testing has become one of the go-to in situ test method in geotechnical practice due to its repeatability, relatively low cost along with a large sampling rate when compared to undisturbed sampling for laboratory tests. This has placed a large emphasis on obtaining the most reliable soil parameters from this test for use in modelling due to the risks associated with tailings dams. Piezocone testing was carried out at a platinum tailings dam and specific soil parameters, including the angle of internal friction ($\phi'$), unit weight ($\gamma$), permeability ($k$), and the state parameter ($\psi$) were inferred using various proposed methods and compared to associated laboratory tests to determine the most appropriate of these methods in terms of calculation effort and reliability for use in practice. Methods were identified that produced reasonable results that, in conjunction with laboratory testing, could be used in geotechnical practice with reasonable and justifiable confidence.

Keywords: piezocone, CPTu, strength parameters, state parameter

1 Introduction

Piezocone (CPTu) testing is becoming increasingly popular and widespread in its use as a cost effective and reliable in situ test method to determine the underlying soil conditions on a wide range of sites, including tailings dams. The main advantage of CPTu testing is the high repeatability of the test as well as the high sampling rate during testing, providing a nearly continuous dataset for a test location.

Although the CPTu test method yields reliable data, the analysis of the data is still discussed to determine the ideal approach to determine soil properties with a reasonable level of confidence for use in detailed designs and assessments. Soil parameters are usually obtained by means of laboratory testing – which, in the case of platinum tailings dams, may not provide a complete understanding of the material properties due to the highly layered nature of tailings dams in general. Most test samples are also usually extracted from a shallow depth (< 5 m below the surface) due to plant limitations. The test samples are also only representative of the soil at that specific point and gives little to no information about the soil in the rest of the profile.
A set of 16 CPTu tests in platinum tailings were analysed using various analysis methods proposed for various material properties, including the friction angle ($\phi'$), unit weight ($\gamma$), permeability ($k_h$) and the state parameter ($\psi$) of the soil. These results were compared (where possible) to laboratory tests conducted on samples from nearby locations on the site to determine the method(s) that would provide the most reasonable estimates of the soil properties that may be used with a reasonable level of confidence with future work, including generating numerical models of a facility for various analyses.

Due to the scope of the study being limited to a comparative analysis of the various methods, only the most significant expressions and equations are shown. Detailed descriptions of the basic calculation procedures using CPTu data and the accompanying parameter calculations can be found in the reference texts.

2 Soil Parameters and Calculation Procedures

A brief overview of the various soil parameters, calculation procedures and the associated intricacies are given below. These are some of the soil parameters most commonly used in geotechnical practice. They include internal angle of friction ($\phi'$), bulk unit weight ($\gamma$) and permeability ($k_h$). These parameters enable one to create numerical models for seepage, slope stability and other geotechnical problems. The state parameter ($\psi$) is mostly used to assess the state of the soil with regard to its critical state and this is used in liquefaction analyses.

Various published methods proposed were used to analyse each of the datasets to compare the results to each other as well as the laboratory tests conducted at similar locations to determine the fitness of each method to relate the soil parameters inferred from the CPTu to the parameters measured in the laboratory with reasonable confidence.

The Robertson, (2010) SBT soil type charts were used to identify the soil type for every data point in a soil profile.

2.1 Angle of Internal Friction, $\phi'$

The angle of internal friction has been calculated using two methods namely:

Mayne (2014) using a logarithmic function for sandy soils:

$$\phi_{sand}' = 17.6^\circ + 11 \log(q_{tt})$$  \hspace{1cm} (1)

With:

$$q_{tt} = \left(\frac{q_c}{P_a}\right) \sqrt{\frac{\sigma'_{vo}}{P_a}}$$  \hspace{1cm} (2)

Where:

$q_{tt}$ = the stress cone resistance normalised for atmospheric pressure.
$q_c$ = the measured cone resistance.
$P_a$ = atmospheric pressure, usually 101.3 kPa.
$\sigma'_{vo}$ = the vertical effective stress.
For clayey soils, the following equation was used from Mayne, (2014):

\[ \phi'_{clay} = 29.5B_q^{0.121}[0.256 + 0.336B_q + \log(Q_t)] \]  

(3)

With:

\[ Q_t = \frac{q_t - \sigma_{vo}}{\sigma_{vo}'} \]  

(4)

Where:

\( Q_t \) = the normalised cone resistance.
\( \sigma_{vo} \) = the vertical total stress.

Another approach was proposed by Kulhawy ad Mayne, (1990), which calculated the peak friction angle:

\[ \phi'_{p} = 17.6 + 11\log(Q_t) \]  

(5)

As the peak friction angle is the sum of the internal angle of friction, \( \phi' \) and the angle of dilatancy, \( \phi_d \):

\[ \phi' = \phi'_{p} - \phi_d \]  

(6)

With the angle of dilatancy described by Lee et al., (2008) as:

\[ \phi_d = \frac{1}{a} \log \left( \frac{q_c/\sigma_{ho}}{b} \right) \]  

(7)

And:

\[ a = 0.13K_0^{-0.115} \]  

(8)

\[ b = 64.09K_0^{-0.17} \]  

(9)

Where:

\( \sigma_{ho} \) = the in situ horizontal total stress.
\( K_0 \) = the coefficient of earth pressure at rest.

2.2 In situ Bulk Unit Weight, \( \gamma \)

Two methods of determining the in situ unit weight of the soil at a point were considered. Robertson and Robertson, (2015), proposed the following method:

\[ \gamma = \gamma_w \left[ 0.271\log(R_f ) + 0.36 \log\left( \frac{q_t}{P_o} \right) + 1.236 \right] \]  

(10)

Where:

\( \gamma_w \) = the unit weight of water.
\( R_f \) = the friction sleeve ratio (different from \( F_r \)).
Mayne, (2010), proposed the next method following a multiple regression analysis on various soils:

\[
\gamma = 1.81 \gamma_w \left( \frac{\sigma'_{vo}}{P_a} \right)^{0.05} \times \left( \frac{q_c}{P_a} \right)^{0.017} \times \left( \frac{f_s}{P_a} \right)^{0.073} \times \left( B_q + 1 \right)^{0.16}
\]

(11)

Where:

\( q_e \) = the effective or excess cone resistance.
\( f_s \) = the sleeve friction.

As can be seen, Robertson’s method is independent of soil stress based parameters, like \( B_q \) and \( \sigma'_{vo} \), which, in the case of Mayne, (2010), initiates an iterative or circular calculation process to more accurately determine the bulk unit weight.

2.3 Soil Permeability, \( k \)

The default method of calculating the soil permeability stems from consolidation theory and is highly dependent on the soil being saturated enough for dissipation tests to be carried out to obtain \( t_{50} \) values. As the presence of water throughout a tailings dam is unwanted and the water table is drawn down by drains, this approach cannot be employed above the phreatic surface and the permeability must be calculated using other methods.

Robertson and Robertson, (2015) gave a set of equations for soil permeability that is dependent on its \( I_c \) value. As the soil type will dictate the calculated permeability, this may not yield the most accurate results, but the expressions should produce values for permeability within the right magnitude of permeability:

If \( I_c \) from Robertson and Robertson, (2015) is between 1.0 and 3.27 (sands and silts), the following expression should be used:

\[
k = 1 \times 10^{(0.952 - 3.04I_c)}
\]

(12)

If \( I_c \) from Robertson and Robertson, (2015) is between 3.27 and 4.0 (clays), the following expression should be used:

\[
k = 1 \times 10^{(-4.52 - 1.37I_c)}
\]

(13)

2.4 State Parameter, \( \psi \)

The state parameter is an indication of the soil state – whether it is denser or looser than the soil’s critical state. This is an important value when considering the liquefaction potential of a soil mass.

Jefferies and Been, (2016) has proposed the following expression to estimate the state parameter of a soil using CPTu data using invariant values:

\[
\psi = -\frac{\log(Q_p/k)}{m}
\]

(14)
With:

\[
Q_p = \frac{3Q_t}{1 + 2K_0}
\]  

(15)

Where:

\(k, m\) are critical state parameters.

Robertson, (2010) has proposed an alternative expression using normalised values:

\[
\psi = 0.485 - 0.314 \log(Q_{\text{m,cs}})
\]  

(16)

Where:

\(Q_{\text{m,cs}}\) = the corrected normalised cone resistance for a clean sand equivalent value.

3 Results

3.1 Angle of Internal Friction, \(\phi'\)

The results of the two methods described by Mayne, (2014) as well as well as Kulhawy and Mayne, (1990) in conjunction with Lee et al., (2008) for sands are shown in Table 1 below and for clays are shown in Table 2 below. The soil types are determined using Robertson, (2010)’s SBT charts. The reference values used to compare the inferred values were obtained from triaxial tests, which indicate a mean \(\psi\) of 32.9° and a standard deviation of 1.1°.

Table 1. Internal friction angle, \(\phi'\) for each method for sands and silty sands.

<table>
<thead>
<tr>
<th>Sample ID</th>
<th>Average (\phi'), Eq. 1 (°)</th>
<th>Standard Deviation, Eq. 1 (°)</th>
<th>Average (\phi'), Eq. 6 (°)</th>
<th>Standard Deviation, Eq. 6 (°)</th>
</tr>
</thead>
<tbody>
<tr>
<td>PZ 1*</td>
<td>36.2</td>
<td>3.1</td>
<td>34.1</td>
<td>1.7</td>
</tr>
<tr>
<td>PZ 2</td>
<td>38.1</td>
<td>2.6</td>
<td>33.9</td>
<td>1.8</td>
</tr>
<tr>
<td>PZ 2-1*</td>
<td>37.1</td>
<td>1.8</td>
<td>34.0</td>
<td>1.6</td>
</tr>
<tr>
<td>PZ 2-2*</td>
<td>37.4</td>
<td>2.3</td>
<td>34.0</td>
<td>1.7</td>
</tr>
<tr>
<td>PZ 3A-1*</td>
<td>34.8</td>
<td>1.6</td>
<td>34.5</td>
<td>1.2</td>
</tr>
<tr>
<td>PZ 3A-2*</td>
<td>34.2</td>
<td>3.6</td>
<td>34.5</td>
<td>2.0</td>
</tr>
<tr>
<td>PZ 3B-1</td>
<td>37.9</td>
<td>4.7</td>
<td>32.6</td>
<td>2.4</td>
</tr>
<tr>
<td>PZ 3-X*</td>
<td>37.2</td>
<td>1.6</td>
<td>34.2</td>
<td>1.3</td>
</tr>
<tr>
<td>PZ 8</td>
<td>40.3</td>
<td>2.4</td>
<td>32.4</td>
<td>1.4</td>
</tr>
<tr>
<td>PZ 12*</td>
<td>40.4</td>
<td>2.3</td>
<td>32.3</td>
<td>1.4</td>
</tr>
<tr>
<td>PZ 20*</td>
<td>35.5</td>
<td>2.4</td>
<td>34.2</td>
<td>1.6</td>
</tr>
<tr>
<td>PZ 21*</td>
<td>36.3</td>
<td>2.5</td>
<td>34.1</td>
<td>1.8</td>
</tr>
<tr>
<td>PZ 22/3</td>
<td>39.4</td>
<td>1.9</td>
<td>32.2</td>
<td>1.2</td>
</tr>
<tr>
<td>PZ 24</td>
<td>37.8</td>
<td>3.2</td>
<td>34.0</td>
<td>2.1</td>
</tr>
<tr>
<td>PZ 39</td>
<td>38.8</td>
<td>2.1</td>
<td>33.2</td>
<td>1.7</td>
</tr>
<tr>
<td>PZ 40</td>
<td>38.8</td>
<td>2.0</td>
<td>33.3</td>
<td>1.6</td>
</tr>
<tr>
<td>Combined</td>
<td>37.6</td>
<td>2.8</td>
<td>33.8</td>
<td>1.8</td>
</tr>
</tbody>
</table>

* Test positions located > 20 m inside the beach area of the tailings dam.
Table 2. Internal friction angle, $\phi'$ for each method for clays and silty clays.

<table>
<thead>
<tr>
<th>Sample ID</th>
<th>Average $\phi'$, Eq. 3 ($\degree$)</th>
<th>Standard Deviation, Eq. 3 ($\degree$)</th>
<th>Average $\phi'$, Eq. 6 ($\degree$)</th>
<th>Standard Deviation, Eq. 6 ($\degree$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>PZ 1*</td>
<td>20.0</td>
<td>3.7</td>
<td>26.1</td>
<td>2.0</td>
</tr>
<tr>
<td>PZ 2</td>
<td>21.9</td>
<td>3.5</td>
<td>27.6</td>
<td>1.6</td>
</tr>
<tr>
<td>PZ 2-1*</td>
<td>18.2</td>
<td>3.9</td>
<td>27.8</td>
<td>1.6</td>
</tr>
<tr>
<td>PZ 2-2*</td>
<td>21.5</td>
<td>3.8</td>
<td>28.0</td>
<td>1.7</td>
</tr>
<tr>
<td>PZ 3A-1*</td>
<td>23.9</td>
<td>3.1</td>
<td>25.6</td>
<td>2.4</td>
</tr>
<tr>
<td>PZ 3A-2*</td>
<td>23.6</td>
<td>3.3</td>
<td>24.9</td>
<td>2.6</td>
</tr>
<tr>
<td>PZ 3B-1</td>
<td>22.6</td>
<td>7.2</td>
<td>28.8</td>
<td>2.3</td>
</tr>
<tr>
<td>PZ 3-X*</td>
<td>16.7</td>
<td>2.7</td>
<td>25.2</td>
<td>2.0</td>
</tr>
<tr>
<td>PZ 8</td>
<td>19.9</td>
<td>5.4</td>
<td>24.7</td>
<td>2.7</td>
</tr>
<tr>
<td>PZ 12*</td>
<td>18.2</td>
<td>5.2</td>
<td>24.0</td>
<td>2.6</td>
</tr>
<tr>
<td>PZ 20*</td>
<td>23.6</td>
<td>4.0</td>
<td>25.3</td>
<td>2.4</td>
</tr>
<tr>
<td>PZ 21*</td>
<td>20.7</td>
<td>5.7</td>
<td>25.0</td>
<td>2.3</td>
</tr>
<tr>
<td>PZ 22/3</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>PZ 24</td>
<td>19.3</td>
<td>3.1</td>
<td>26.7</td>
<td>2.0</td>
</tr>
<tr>
<td>PZ 39</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>PZ 40</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Combined</td>
<td>21.8</td>
<td>4.3</td>
<td>25.6</td>
<td>2.4</td>
</tr>
</tbody>
</table>

* Test positions located > 20 m inside beach area of the tailings dam.

Figures 1 a) and b) show representative plots for the calculation methods applied. It shows a comparison between the methods across different soil types ranging from sandy, to silty and clayey.

It can be seen that the corrected values (the dotted line) reduce the magnitudes for data points or soil layers that yield very high ($>40\degree$) or very low ($<20\degree$) friction angles. Negative corrections are usually applied to sandy soils and positive corrections are calculated to clayey soils by using the expression as shown in Equation 7. A $K_0$-value of 0.5 was assumed for the calculation purposes. The calculated $\phi'$-values skew proportionately to $K_0$ being chosen higher or lower with this method.

For sandy soils, the method proposed by Mayne, (2014) produced an average $\phi'$ of 37.5$\degree$ and a standard deviation of 2.8$\degree$, whereas Kulhawy and Mayne, (1990) with Lee et al., (2008), produced an average $\phi'$ of 33.8$\degree$ and a standard deviation of 1.8$\degree$.

For clays, Mayne, (2014) produced an average $\phi'$ of 21.8$\degree$ and a standard deviation of 4.3$\degree$, whereas Kulhawy and Mayne, (1990) with Lee et al., (2008), produced an average $\phi'$ of 25.6$\degree$ and a standard deviation of 2.4$\degree$. 

308
Figure 1. Comparison of the two methods for calculating $\phi'$ from CPTu data. a) is an extract from PZ 3-X and b) is an extract from PZ 21, indicating corrections for both sands and clays.

3.2 In situ Bulk Unit Weight, $\gamma$

The soil unit weight was calculated using the two methods in section 2.2. Representative outputs are shown in Figure 2. The output from Mayne, (2010) is only after one iteration has been carried out.

At the initial portion of a test profile (0–2m), Mayne’s, (2010) method significantly underestimates the unit weight when compared to Robertson and Robertson, (2015), but consistently exceeds the Robertson’s values at greater depths. The average bulk unit weights measured for materials from similar locations to PZ3-X and PZ21 were 20 and 17 kN/m$^3$ respectively. Both methods seem to reflect reasonable unit weights in sandy and denser materials, whereas both methods also overestimate the unit weights in clayey materials with lower unit weights measured.
Figure 2. Comparison of the two methods for calculating $\gamma$ from CPTu data. a) is an extract from PZ 3-X – a more sandy to silty material and b) is an extract from PZ 21 – a more silty to clayey material.

### 3.3 Soil Permeability, $k$

The most common USCS classifications for platinum tailings range from low plasticity silts and silty sands, (ML and SM), to low plasticity clays and high plasticity silts, (CL and MH). The expected permeability for SM to ML ranges from approximately $5 \times 10^{-8}$ to $5 \times 10^{-6}$, where the expected permeability for CL to MH ranges from approximately $5 \times 10^{-10}$ to $5 \times 10^{-8}$. Table 3 presents a summary of the calculated permeability of the sandy and clayey soils respectively.
Table 3. Permeability, k for each method for sands and silty sands.

<table>
<thead>
<tr>
<th>Sample ID</th>
<th>Average k, Sands (m/s)</th>
<th>Average k, Clays (m/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>PZ 1*</td>
<td>3.2×10^{-5}</td>
<td>3.1×10^{-9}</td>
</tr>
<tr>
<td>PZ 2</td>
<td>8.3×10^{-5}</td>
<td>3.1×10^{-9}</td>
</tr>
<tr>
<td>PZ 2-1*</td>
<td>1.5×10^{-5}</td>
<td>2.2×10^{-9}</td>
</tr>
<tr>
<td>PZ 2-2*</td>
<td>1.9×10^{-5}</td>
<td>4.7×10^{-9}</td>
</tr>
<tr>
<td>PZ 3A-1*</td>
<td>2.9×10^{-6}</td>
<td>3.2×10^{-9}</td>
</tr>
<tr>
<td>PZ 3A-2*</td>
<td>1.2×10^{-4}</td>
<td>2.5×10^{-9}</td>
</tr>
<tr>
<td>PZ 3B-1</td>
<td>9.2×10^{-5}</td>
<td>8.2×10^{-10}</td>
</tr>
<tr>
<td>PZ 3-X*</td>
<td>4.8×10^{-6}</td>
<td>2.0×10^{-7}</td>
</tr>
<tr>
<td>PZ 8</td>
<td>1.8×10^{-4}</td>
<td>-</td>
</tr>
<tr>
<td>PZ 12*</td>
<td>1.1×10^{-4}</td>
<td>-</td>
</tr>
<tr>
<td>PZ 20*</td>
<td>3.6×10^{-5}</td>
<td>1.8×10^{-9}</td>
</tr>
<tr>
<td>PZ 21*</td>
<td>4.6×10^{-5}</td>
<td>2.9×10^{-9}</td>
</tr>
<tr>
<td>PZ 22/3</td>
<td>1.1×10^{-4}</td>
<td>-</td>
</tr>
<tr>
<td>PZ 24</td>
<td>2.3×10^{-4}</td>
<td>5.0×10^{-9}</td>
</tr>
<tr>
<td>PZ 39</td>
<td>9.2×10^{-5}</td>
<td>-</td>
</tr>
<tr>
<td>PZ 40</td>
<td>5.5×10^{-5}</td>
<td>-</td>
</tr>
</tbody>
</table>

* Test positions located > 20 m inside the beach area of the tailings dam.

The sandy soils tend to show a permeability of a single magnitude higher than the upper limit approximation that was expected, whereas the more clayey soils seem to fall within expected ranges.

3.4 State Parameter, $\psi$

The state parameter is an indication of the soil state relative to its critical state, which is in turn, an indication of the material’s potential for liquefaction. For the purposes of conservativism, it was assumed that the clayey soil type is also susceptible to liquefaction as the material may only exhibit clay-like behaviour due to its extremely fine grind and its close packing causing very low permeability, rather than a chemical component and a plate-like structure in the material attracting water, as is the case in natural clays.

Figure 3 presents a comparison between the methods proposed by Jefferies and Been, (2016) and Robertson, (2010) to estimate $\psi$.

Jefferies and Been, (2016)’s method is highly sensitive to the material type in question, as clayey materials tend to yield higher values for $\psi$, indicating a very loose and liquefiable material. Robertson, (2010) tends to yield more conservative values for $\psi$ in sandier materials, closer to the critical state value where $\psi = 0$. 
Figure 3. Comparison of the two methods for calculating $\psi$ from CPTu data.

a) is an extract from PZ 3-X – a more sandy to silty material and
b) is an extract from PZ 21 – a more silty to clayey material.

4 Conclusions

4.1 Angle of Internal Friction, $\phi'$
The results from the laboratory testing were only compared to that of the sandy soil type’s calculations. This is due to the shallow depth of the sample’s extraction, the relatively close proximity to the deposition wall where the coarser fraction of tailings tend to settle out, coupled with the effect of the remoulding process mixing in any small clayey layers within the samples.

The remoulding and possible inclusion of clay layers in a test sample could possibly decrease the mean friction angle in a triaxial test. Given this, the most reasonable method considered above for calculating $\phi'$ from CPTu data was Kulhawy and Mayne, (1990) with the dilatancy correction from Lee et al., (2008). The average and standard deviations calculated with this method were within an error of 1° and within reasonable ranges for the material types in question. This method is both applicable to sandy and clayey soils and provide reasonable estimates for $\phi'$ from CPTu data.

4.2 In situ Unit Weight, $\gamma$
While both the methods from Robertson and Robertson, (2015) and Mayne, (2010) yield similar results for estimating the soil’s unit weight, Mayne, (2010) requires significantly more calculation effort due to the iterative nature of the expression including parameters that rely implicitly on the soil’s unit weight. This leads to increased calculation time, especially of iterations must be carried out for thousands of data points per dataset.
Robertson and Robertson, (2015) can be calculated with minimal calculations preceding it, and therefore lends itself to a higher efficiency when calculating $\gamma$ and $\sigma_{v0}$ for large datasets.

When comparing the unit weights calculated to the unit weights measured in the laboratory, both methods’ calculated values seem to compare reasonably well to the measured values for sandy materials – like PZ3-X, whereas a significant overestimation is seen when applied to more clayey profiles as seen in the data from Pz21.

### 4.3 Soil Permeability, $k_h$

When comparing the calculated values to the expected values for soils with a similar USCS classification, the sandy soils’ permeability tend to be overestimated by roughly one order of magnitude. For the clayey soils, the permeability estimated falls well within expected ranges.

Although this method is no substitute for permeabilities derived from dissipation tests, it provides a reasonable alternative to estimate the in situ permeability if dissipation tests or the presence of a phreatic surface is unavailable.

### 4.4 State Parameter, $\psi$

The state parameter calculated using both methods from Jefferies and Been, (2016) and Robertson, (2010) tend to show reasonable agreement when considering sandy and silty soils. Robertson, (2010) tends to show better grouping of the data as well as a relatively low material type sensitivity with depth progression, whereas Jefferies and Been, (2016) shows significant sensitivity with material type. This parameter is very useful in identifying zones of similar state inside the soil when modelling tailings dams.

The state parameter is a heavily inferred value from a limited number of measurements per data point. Although Robertson, (2010) seems to produce a less conservative state parameter value than Jefferies and Been, (2016) in this particular case, one must consider both methods in all cases to make a reasonable assessment of the material state.

### References


Typical Parameter Ranges for Classification Laboratory Tests

H. F. T. Barnard¹, V. Venter²

¹Geostrada, Aurecon, Pretoria, Gauteng, Hennieb@geostrada.co.za
²Geostrada, Aurecon, Pretoria, Gauteng, Vian.Venter@geostrada.co.za

Abstract

It is important for any Geo-professional to be able to evaluate the quality of laboratory test results, and therefore the main aim for this paper is to provide typical parameter ranges derived from laboratory tests. The outcome of this paper is based on laboratory test data collected by Geostrada Engineering Laboratory in Pretoria on various projects and soil types across South Africa over the past three years. The parameters that will be covered by this study will include grading modulus (GM), Atterberg limits, Maximum Dry Density (MDD), Optimum Moisture Content (OMC) and California Bearing Ratio (CBR). The typical ranges from this paper will provide guidance for Geo-professionals to evaluate laboratory test results and to identify any possible outliers in test data. The authors derived a correlation between GM and MDD for different soil types, which can be used to estimate the optimum compaction behavior of the soil based on the data presented in this paper.

Keywords: Laboratory tests, typical ranges, grading modulus, maximum dry density, optimum moisture content.

1 Introduction

The laboratory tests for any geotechnical project may range from simple classification tests to complex geotechnical testing (Mayne et al., 2002). Geo-professionals require a basic understanding of different laboratory tests and the factors affecting the results, in order to fully grasp the benefits that laboratory tests may have on any project. In this paper, the authors will provide typical parameter ranges for the most common laboratory tests such as particle size distribution and compaction tests. The test data used in this paper were measured by Geostrada, an SANAS Accredited Engineering Laboratory in Pretoria, on various projects and soil types across South Africa over the past three years (2014 to 2016). In the last three (3 No.) years, Geostrada have tested particle size distribution, Modified AASTHO compaction as well as California Bearing Ratio on more than 2000 samples. All the samples were classified using the Unified Soil Classification System (USCS, 1953), based on the criteria as summarized in Table 1. For the purpose of this paper, only the three (3 No.) material types that were tested the
most will be presented and discussed. Table 2 shows the number of tests done on each material type and indicates that the GC, SC and CL materials were tested the most.

Table 1. Unified Classification System (USCS, 1953).

<table>
<thead>
<tr>
<th>Major divisions</th>
<th>Group symbol</th>
<th>Group name</th>
</tr>
</thead>
<tbody>
<tr>
<td>Coarse grained soils more than 50% retained on or above No.200 (0.075mm) sieve</td>
<td>Clean gravel &lt;5% smaller than No.200 Sieve</td>
<td>GW well-graded gravel, fine to coarse gravel</td>
</tr>
<tr>
<td></td>
<td>Gravel with &gt;12% fines</td>
<td>GP poorly graded gravel</td>
</tr>
<tr>
<td></td>
<td>Clean sand &lt;5% smaller than No.200 Sieve</td>
<td>SW well-graded sand, fine to coarse sand</td>
</tr>
<tr>
<td></td>
<td>Sand with &gt;12% fines</td>
<td>SP poorly graded sand</td>
</tr>
<tr>
<td>Fine grained soils 50% or more passing the No.200 (0.075mm) sieve</td>
<td>Inorganic</td>
<td>ML silt</td>
</tr>
<tr>
<td></td>
<td>Organic</td>
<td>CL clay of low plasticity, lean clay</td>
</tr>
<tr>
<td></td>
<td>Inorganic</td>
<td>MH silt of high plasticity, elastic silt</td>
</tr>
<tr>
<td></td>
<td>Organic</td>
<td>CH clay of high plasticity, fat clay</td>
</tr>
<tr>
<td>Highly organic soils</td>
<td>Pt</td>
<td>Peat</td>
</tr>
</tbody>
</table>

Table 2. Number of tests performed on each material type.

<table>
<thead>
<tr>
<th>USCS Material Type</th>
<th>Number of tests</th>
</tr>
</thead>
<tbody>
<tr>
<td>GC</td>
<td>267¹</td>
</tr>
<tr>
<td>GM</td>
<td>45</td>
</tr>
<tr>
<td>GM-GC</td>
<td>82</td>
</tr>
<tr>
<td>SC</td>
<td>773¹</td>
</tr>
<tr>
<td>SM</td>
<td>207</td>
</tr>
<tr>
<td>SM-SC</td>
<td>199</td>
</tr>
<tr>
<td>CL</td>
<td>312¹</td>
</tr>
<tr>
<td>ML</td>
<td>10</td>
</tr>
<tr>
<td>ML-CL</td>
<td>10</td>
</tr>
<tr>
<td>CH</td>
<td>65</td>
</tr>
</tbody>
</table>

Note: ¹ Material presented in this paper.
2 Evaluation of Test Data

A database was developed with the test results from the previous three years as tested by Geostrada Laboratory. The criteria used to populate this database was that a sample was tested for all the following tests:

- Particle size Distribution (Road and foundation indicators);
- Maximum Dry Density and Optimum Moisture Content (MOD AASHTO); and
- California Bearing Ratio.

The parameters evaluated for this study include Grading modulus (GM), Liquid Limit (LL), Plastic Index (PI), Linear Shrinkage (LS), Maximum Dry Density (MDD), Optimum Moisture Content (OMC) and the 95% California Bearing Ratio (CBR) values. For each parameter the 20th and 80th percentile values were determined in order to form the typical range that includes 60% of the test data for each soil type. The median value for each parameter was also calculated for each soil type. The data is presented as modified box and whisker plots showing the minimum and maximum values as well as the 20th and 80th percentile values.

A correlation between GM and MDD was derived for the different material types in order to estimate the optimum compaction behavior of the soil based on particle size distribution. The correlation between GM, MDD and OMC can further be used to provide dry density and moisture content values for geotechnical testing on reconstituted samples.

3 Discussion of Test Results

3.1 Grading Modulus

The Grading modulus (GM) is an empirical figure obtained by adding the percentage of soil passing the 2.0 mm, 0.425 mm and 0.075 mm sieves respectively, and subtracting the total percentage from 300. The sum is then divided by 100 to provide the GM value for each test. Figure 1 shows the GM values calculated for the different material. The median GM value for the CL material was 0.58 and of the 312 CL samples the GM values was below 0.25 for only 23 tests (approximately 7%). The median GM values for the SC and GC material was calculated as 1.42 and 2.02, respectively. Although the GM values for the SC material span over a wide range, 90% of the data showed a narrower range between 0.75 and 2.00 as indicated on Figure 1.

3.2 Atterberg Limits

At Geostrada laboratory, the Atterberg limits are presented on our test reports by three parameters namely Liquid limit (LL), Plastic Index (PI) and Linear Shrinkage (LS). The liquid limit of a soil is defined as the moisture content in percent at which a soil transfer from a plastic state to a liquid state of consistency. The LL can be determined by means of a Casagrande liquid limit device (TMH1: A2, 1986) or Falling cone apparatus (Das, 2010). The test data presented in this paper was determined by the Casagrande method.

The next process in the Atterberg limits is to determine the moisture content in percent at which the soil transfer from a plastic state to a semi-solid state of consistency (TMH1: A3, 1986). The plastic index (PI) is calculated as the difference between the LL and PL values. The PI is the moisture content range in which the soil shows plastic behavior (Das, 2010).

The linear shrinkage (LS) is determined by filling a 150 mm long steel pan with soil and is allowed to dry in an oven at 110 ºC overnight. The LS is then measured the following day by means of a caliper device, as the amount of linear shrinkage observed. The linear shrinkage in mm units is then divided by 150 mm in order to present the LS as a percentage. The LS gives an indication of the shrinkage potential of the clay minerals within the soil (TMH1: A4, 1986).
3.2.1 Liquid Limit
Figure 2 shows the typical parameter ranges for the LL measured in the laboratory over the past 3 years. The median LL value for the CL material was measured as 36% and all the CL material results showed a fairly narrow range between 20% and 50%. The median LL values for the SC and GC material was measured as 29% and 33%, respectively. Although the results showed higher LL values for both GC and SC materials, it seems that by excluding the upper 5% of data, the 95 percentile LL values for the GC and SC material will be 55% and 45%, respectively as indicated on Figure 2.

3.2.2 Plastic index
The typical parameter ranges for the Plastic index (PI) calculated from the test data are presented in Figure 3. The median PI value observed for the CL material was 16, with the 20th percentile 12% and 80th percentile 21%. From the 772 tests conducted on SC material, 60% of the values ranged between 9% and 15% with a median PI value of 11%. The median PI values for GC material was calculated as 13%, which is higher than the median PI value for the SC material. Although some results in figure 3 show PI values greater than 30%, it is interesting to notice that only about 2.5% of the total 1352 test results measure PI values greater than 27%.
Figure 2. Typical Liquid Limit ranges.

Figure 3. Typical Plastic index ranges.
3.2.3 Linear shrinkage
The typical parameter ranges for the Linear Shrinkage (LS) values are presented in Figure 4. The median LS value measured for the CL material was 8%, with the 20th percentile 6% and 80th percentile 10.5%. The median LS values for the SC and GC material was calculated as 5.5% and 6.5%, respectively. Again the median LS values for GC material was measured higher than the median LS value for the SC material. The same trend was observed in the LL and PI test results and the only explanation may be that in reality there are more clay particles that stick to the clayey gravel (GC material) oppose to the clayey sand (SC material) and therefore the portion of GC material passing the 0.425 mm sieve behave slightly more plastic compared to the SC material. Although some results showed higher LS values for all material types, the results show that by excluding the upper 5% of data, the 95 percentile LS values for the GC, SC and CL material will be 12%, 10% and 12%, respectively as indicated on Figure 4.

3.3 Maximum Dry Density
Figure 5 shows the MDD values tested for the different material types. The graph also indicate the 5th percentile and 95th percentile MDD values for all three material types. The median MDD value for the CL material was measured as 1820 kg/m$^3$, with the 20th percentile 1700 kg/m$^3$ and 80th percentile 1910 kg/m$^3$. The median MDD values for the SC and GC material was measured as 2000 kg/m$^3$ and 2042 kg/m$^3$, respectively. For all three material types the 20th percentile and 80th percentile MDD values only differ with about 200 kg/m$^3$ which give a fairly narrow band for 60% of the test data.

3.4 Optimum moisture content
The moisture content at which any soil can be compacted to the maximum dry density is defined as the optimum moisture content (OMC). Figure 6 shows the OMC values measured for the different material types. Although some results showed higher OMC values for all material types, the results show that by excluding the upper 5% of data, the 95 percentile OMC values for the GC, SC and CL material will be 15%, 15% and 20%, respectively as indicated on Figure 6. The median OMC value for the CL material was measured as 13.3%. The median OMC values for the SC and GC material was both measure at around 9.5%.

3.5 Ninety-Five percent California Bearing Ratio value
Figure 7 shows the typical 95% California Bearing Ratio (CBR) ranges that can be expected for different material types. CBR tests were performed on a portion of the total grading and compaction tests. From the 262 CBR tests conducted on GC material, 60% of the values ranged between 11 and 38 with a median 95% CBR value of 26. For the SC material, less than 5% of the 744 results revealed CBR values less than 2.0 as indicated. The median 95% CBR value observed for the SC material was 19, with the 20th percentile 7.6 and 80th percentile 32. For the CL material, less than 20% of the results showed values greater than 10 and less than 5% of the CL results showed values greater than 20.
Figure 4. Typical Linear Shrinkage ranges.

Figure 5. Typical Maximum Dry Density ranges.
Figure 6. Typical Optimum Moisture Content ranges.

Figure 7. Typical 95% CBR ranges.
4 Correlation between Grading Modulus and Maximum Dry Density

Often geo-professionals only managed to retrieve disturbed samples from site and would like to perform geotechnical tests on reconstituted samples. Therefore, the maximum dry density and optimum moisture content, measured in compaction tests are required. The normalized MDD (N\text{3000}) vs GM plots as shown in Figure 8 for the SC material, were developed for all material types. These plots were normalised on 3000 kg/m\textsuperscript{3} and provide equations to estimate the relationship between the grading modulus and maximum dry density.

In order to estimate the OMC, the MDD vs OMC values for each material type were plotted in order to derive a relationship between these parameters. Figure 9 shows the plot for the SC material. Therefore, by using a combination of the two equations presented in Table 3, geo-professionals can estimate (1) the MDD by multiplying the normalised value by 3000 kg/m\textsuperscript{3}, as well as (2) the OMC value by using the calculated MDD value derived from the grading modulus. The significance of these plots are that geo-professionals can save the costs to determine the MDD and OMC by means of additional compaction tests in the laboratory and rather request more geotechnical testing for design purposes. Table 3 provide the equations derived from the plots for the different material types.

Table 3. Correlations between GM, MDD and OMC

<table>
<thead>
<tr>
<th>USCS Material Type</th>
<th>MDD (N\text{3000}) vs GM</th>
<th>MDD vs OMC</th>
</tr>
</thead>
<tbody>
<tr>
<td>GC</td>
<td>y = 0.0638x + 0.5498</td>
<td>y = -32.62x + 2364.78</td>
</tr>
<tr>
<td>SC</td>
<td>y = 0.0359x + 0.6145</td>
<td>y = -34.46x + 2343.10</td>
</tr>
<tr>
<td>CL</td>
<td>y = 0.0499x + 0.5774</td>
<td>y = -27.97x + 2203.11</td>
</tr>
</tbody>
</table>

Figure 8. Normalised MDD (N\text{3000}) vs GM plot for SC material.
5 Conclusions and Recommendations

The graphs presented in Figure 1 to 7 provide guidance for geo-professionals to evaluate laboratory test results in terms of particle size distribution and compaction testing. This paper also provided a correlation between grading modulus and optimum compaction behavior of different material types. The equations presented in Table 3 can be used to estimate MDD and OMC from the grading modulus values obtained from normal grading analyses. The significance of these plots are that geo-professionals can save the costs to determine the MDD and OMC by means of additional compaction tests in the laboratory and rather request more geotechnical testing for design purposes. The authors hope that the typical parameter ranges presented in this paper will assist geo-professionals to gain a better understanding of typical results that can be expected from laboratory tests.

References

An investigation on the Relationships between the Petrographic, Physical and Mechanical Characteristics of Sandstones from the Newspaper Member of the Natal Group Sandstones

D. Naidoo¹, M. Ferentinou²

¹University of KwaZulu-Natal, Durban, KwaZulu-Natal, dayini.naidoo@yahoo.com
²University of Johannesburg, Johannesburg, Gauteng, mferentinou@uj.ac.za

Abstract

The Natal Group sandstones, which constitute approximately 90% of the Natal Basin, are frequently used in the construction industry. It is therefore important to understand the materials’ physical and mechanical properties, as well as the physical and petrographic factors that influence the mechanical strength of sandstones. Representative block samples of the Newspaper Member of the Marianhill Formation were collected from four different locations, namely Cliffdale, Scottburgh, Coedmore Quarry and the Palmiet Nature Reserve. These samples were used to investigate the relationship between the petrographic, physical and mechanical properties of Natal Group sandstones using simple linear regression. Petrographic and geochemical analysis was conducted to determine the samples’ mineral compositions and texture. The composition of the sampled sandstones were predominantly quartz, K-feldspar and plagioclase. The grain size ranged from fine to coarse grained and the shape of the grains ranged from sub-angular to sub-rounded. The porosity of the sandstones were generally low while the density was moderate to high. The mechanical properties of the sampled sandstones were determined using standardized geotechnical tests namely, the Schmidt Hammer Rebound (SHR) test, Unconfined Compressive Strength (UCS) test, Point Load Index (PLI) test and Brazilian Disc strength test. The UCS of the sandstones ranged from low to high strength with most having a high intact strength. The indirect tensile strength was determined using the PLI and the Brazilian Disc strength test. The strength values achieved from the PLI test indicated that the sampled sandstones were of high strength. The quartz percentage, as well as the mean grain size had no substantial correlation to the UCS values. However, linear correlations were detected between PLI and Brazilian Disk strength. Mechanical and physical properties correlated linearly.

Keywords: Natal Sandstones, Uniaxial Compressive Strength, Linear Regression
1 Introduction

Sandstones are described to be the most important rock type in the Natal Group as they make up 90% of the Marianhill Formation and most of the Natal Group Basin (Marshall, 2002). Due to the widespread occurrence of sandstone within the Natal Basin, it is frequently used as a construction material, therefore highlighting the need to fully understand their engineering properties as well as the physical and petrographic factors that influence the mechanical strength of sandstones.

The most conventional method of determining intact strength is through the Uniaxial Compressive Strength (UCS) test. Adequate core samples required for UCS testing are often not easily accessible and sample preparation for determining strength parameters in laboratories are time consuming and expensive (Diamantis et al., 2009). In light of the difficulty in obtaining and preparing core samples needed for UCS testing, empirical equations have been derived by several researchers, which predict the UCS of sandstones from indirect tests. Measurements from indirect tests such as the Point Load Index (PLI) and Schmidt Hammer Rebound (SHR) test, can be conducted in the field and require very little sample preparation.

The mechanical strength of sandstones are also affected by inherent characteristics such as petrographic and physical properties (Acikalin et al., 2008). The petrographic characteristics known to affect the mechanical behavior of sandstones, such as the UCS, include grain size distribution, type of grain contact, mineral composition and mean grain size (Bell and Lindsay, 1999). Physical properties that affect the mechanical strength of sandstones include porosity and density.

The aim of this study is to derive empirical equations using simple linear regression in order to determine the UCS of the Newspaper Member sandstones, taken from various locations, via indirect testing methods. Furthermore, this study will reveal the relationships existing between the physical, petrographic and mechanical properties of sandstones.

2 Literature Review

Bell and Lindsay (1999) conducted research based on the Newspaper Member of the Natal Group sandstones where they related the petrographic characteristics to the geo-mechanical properties of a sandstone sample obtained from a drill hole sunk just over 300m in Kloof, KwaZulu-Natal. They found that the percentage of quartz in the sandstone had a positive correlation with the UCS where an increase in quartz percentage led to increase in the strength of the sandstones. The relationship between the median grain size and the indirect tensile strength was also investigated and it was found that a reduction in grain size led to an increase in Brazilian Disk strength. It was also revealed that dry density has an effect on the strength of the sandstones where an increased dry density is associated with an increased UCS and indirect tensile strength. Bell and Lindsay (1999) also investigated the relationship between the sandstone SHR and the dry density and porosity. It was found that, as density values increased, there was also a notable increase in the Brazilian Disk Strength and UCS of the sandstones. Furthermore, the relationships between the mechanical properties themselves were also investigated by Bell and Lindsay (1999). They discovered that an increase in one strength parameter lead to an increase in the other parameter. This relationship was also evident between the Brazilian Disk strength and PLI of the Newspaper Member sandstones.
3 Research Methodology

3.1 Sampling Process
Sample locations, namely Cliffdale, Scottburgh, Coedmore Quarry and the Palmiet Nature Reserve, were chosen based on the outcrop occurrence of the Newspaper Member sandstones as well as the accessibility of obtaining fresh samples. The sampling process was non-selective in order to achieve a wide spatial distribution. Twelve sandstone samples were collected from various outcrops at the four different localities as presented in Table 1, in order to conduct the necessary petrographic and geochemical analyses as well as the laboratory and geotechnical test necessary to achieve the aim of the current study.

Table 1. Details of each sample location.

<table>
<thead>
<tr>
<th>Sample Location</th>
<th>Geographic Coordinates</th>
<th>Number of Samples Collected</th>
</tr>
</thead>
<tbody>
<tr>
<td>Palmiet Nature Reserve</td>
<td>S 29° 49’ 24.6”, E 030° 55’ 34.4”</td>
<td>2</td>
</tr>
<tr>
<td>Coedmore Quarry</td>
<td>S 29° 54’ 1.22”, E 030° 57’ 5.12”</td>
<td>5</td>
</tr>
<tr>
<td>Scottburgh</td>
<td>S 30° 16’ 49.5”, E 030° 45” 33.8”</td>
<td>2</td>
</tr>
<tr>
<td>Cliffdale</td>
<td>S 29° 45’ 32.7”, E 030° 40’ 31.2”</td>
<td>3</td>
</tr>
</tbody>
</table>

3.2 Petrographic and Mineralogical Analysis
Four thin sections were prepared from the selected locations and were analyzed under the Leica Olympus BX 41 microscope. Petrographic analysis was performed under crossed nicols to determine the mineralogical composition, mean grain size, shape of grain contacts, shape and sphericity of the grains making up the sandstones of the Newspaper Member. The sandstone samples taken from Cliffdale and Scottburgh were examined under a magnification of 2X whereas those obtained from Coedmore Quarry and the Palmiet Nature Reserve were examined under a magnification of 10X. The grain size of 100 grains from each thin section was determined using the microscale on the photomicrograph. The mean grain size was subsequently calculated for each thin section. The shape of the grains was determined using the Powers (1982) chart. Grain shape contact was determine using the contact shape types outlined by Taylor (1950).

3.3 Geochemical Analyses
The mineralogical composition of the sandstones was determined by performing X-ray diffraction (XRD) using the Pan-analytical Empyrean X-ray diffraction machine. The results obtained from the test were analyzed using HighScore Plus software.

3.4 Laboratory and Geotechnical Tests
3.4.1 Physical Properties
For the purpose of this study the bulk density and porosity of twelve representative core samples was determined, the latter using the standard saturation method which was carried out in accordance with the standard procedures outlined by ISRM (1979).

3.4.2 Geotechnical Test
The twelve representative core samples were subjected to axial and diametrical N-type SHR tests that was conducted in accordance with the ISRM (2007) recommended guidelines. The tests were performed forty times on each sample; twenty times axially and twenty times diametrically, in order to determine the rebound height which provides an indication of materials strength. A total of ten representative samples were prepared for the UCS tests in accordance with the ISRM (1978) standards. The UCS test was performed using a SANS UCS.
machine, which applies a load of 0.5-1.0Mpa/s. The PLI was determined for selected sandstone samples in accordance with the standard procedures outlined in the ISRM (1985) standards. The PLI test was performed on previously prepared NX-size core samples, which had a diameter of 54mm. The test was performed three to six times, diametrically and axially, based on availability of samples from each location. The Brazilian Disc strength test was used to indirectly measure the tensile strength of the studied sandstones.

3.5 Statistical analysis
The raw data obtained from the physical parameters, mechanical properties and petrological characteristic test results were subjected to linear regression in order to define the correlation coefficient between the two variables. The correlation coefficient derived provides an indication of the relationship existing between the input variables.

3.6 Petrographic and Geochemical Analyses
The grain size of the sandstone samples obtained from Cliffdale has a grain size range of 0.33 mm-1.47 mm and a mean grain size of 0.75 mm. Long and sutured grain shape contacts were observed however, the long grain shape contact prevailed. The Cliffdale sandstones are composed of 76% quartz, 11.3% plagioclase and 12.8% K-feldspar.

The grains of the Scottburgh sandstones are moderately sorted sub-angular to sub-rounded and exhibit low sphericity. The Scottburgh sandstones have a grain size range of 0.17 mm-1.20 mm and a mean grain size of 0.62 mm. The floating contact is the prevailing grain shape contact observed among the grains of the Scottburgh sandstones. Quartz is the predominant mineral in the Scottburgh sandstone and makes up 88.7% of the mineralogical composition followed by K-feldspar, which makes up 11.3%. The majority of the grains that compose the Scottsburg sandstones are sub-angular to angular in shape, have a low sphericity with a grain size range of 53.33-206.67 μm and a mean grain size of 126.27 μm.

The grains which make up the sandstones of Coedmore Quarry are moderately sorted and the floating grain shape contact is dominant. Quartz, K-feldspar and plagioclase make up 75.4%, 15% and 9.6% respectively of the Coedmore Quarry sandstones.

The grains comprising the sandstones taken from the Palmiet Nature Reserve are sub-angular to sub-rounded in shape, have a low sphericity and are moderately to well sorted. The grain size range of these sandstones is 33.33-233.33 μm with a mean grain size of 128.40 μm. The majority of the grains that make up the Palmiet Nature Reserve sandstones are in long contact with each other. The geochemical analyses conducted on the sandstone samples obtained from the Palmiet Nature Reserve showed that the sandstones are composed of 72% quartz, 12.8% K-feldspar and 14.2% plagioclase.

4 Results

4.1 Physical Properties Analyses
Figure 1 shows the average bulk density (g/cm3), porosity (%) and dry density (g/cm³) of the studied sandstones of the four localities. The studied sandstones have a mean bulk density of 2.67 g/cm³, a mean porosity of 1.5% and a mean dry density of 2.65 g/cm³.
4.2 Mechanical Properties Analyses

The mechanical properties of the Natal Group sandstones were determined by conducting geotechnical tests which include the SHR test, UCS test, PLI test and Brazilian Disk Strength test. A summary of the geotechnical test results is presented in Figure 2.

![Bar graph showing the average physical properties of the studied sandstones](image)

**Figure 1.** Bar graph displaying the physical properties of the Natal Group sandstones.

<table>
<thead>
<tr>
<th>Area</th>
<th>UCS (MPa)</th>
<th>Point Load Index (MPa)</th>
<th>Brazilian Disc strength (MPa)</th>
<th>Schmidt Hardness</th>
<th>Porosity (%)</th>
<th>Dry Density (g/cm³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Palmiet Nature Reserve</td>
<td>46.72</td>
<td>5.79</td>
<td>10.4</td>
<td>42.8</td>
<td>1.87</td>
<td>2.55</td>
</tr>
<tr>
<td>Clifdale</td>
<td>4.21-11.51</td>
<td>7.89-13.73</td>
<td>22.50</td>
<td>1.56-2.34</td>
<td>2.40-2.5</td>
<td></td>
</tr>
<tr>
<td>Scotburgh</td>
<td>24.59-70.89</td>
<td>3.52-11.18</td>
<td>8.40-12.32</td>
<td>20.54</td>
<td>0.81-2.02</td>
<td>2.14-2.42</td>
</tr>
<tr>
<td>Coedmore Quarry</td>
<td>25.27-40.70</td>
<td>3.64-7.37</td>
<td>6.61-11.25</td>
<td>22.39</td>
<td>1.78-2.27</td>
<td>2.64-2.80</td>
</tr>
</tbody>
</table>

**Figure 2.** Table summarizing the physical and mechanical properties of the Natal Group sandstones from the four different study locations.
4.3. Relationship between the Petrographic characteristics and mechanical strength of the Natal Group Sandstones

Figure 3 graphically displays the relationship between the quartz content and the UCS of the investigated sandstones of the Natal Group.

Figure 3. Bar graph showing the relationship between the quartz content and UCS.

4.4 Relationship between the physical and mechanical properties

The linear relationships between the laboratory derived porosity and mechanical strength of the studied sandstones are presented in Table 2.

Table 2. Correlations between studied sandstones porosity and mechanical strength.

<table>
<thead>
<tr>
<th>Mechanical Strength</th>
<th>Linear Equation</th>
<th>Correlation Coefficient</th>
</tr>
</thead>
<tbody>
<tr>
<td>UCS</td>
<td>y = -17.12x + 71.76</td>
<td>0.85</td>
</tr>
<tr>
<td>Axial SHR</td>
<td>y = -10.61x + 64.19</td>
<td>0.81</td>
</tr>
<tr>
<td>Diametrical SHR</td>
<td>y = -13.63x + 65.63</td>
<td>0.81</td>
</tr>
<tr>
<td>Brazilian Disk Strength (BDS)</td>
<td>y = -2.87x + 16.03</td>
<td>0.73</td>
</tr>
<tr>
<td>PLI</td>
<td>y = -2.62x + 10.86</td>
<td>0.98</td>
</tr>
</tbody>
</table>

4.5 Relationship Between the mechanical properties:

Linear regression was used to derive equations relating the test results obtained from indirect testing methods to the laboratory derived UCS as well as related relationships between the indirect tensile strength. The proposed equations are presented in Table 3.
Table 3. Derived equations.

<table>
<thead>
<tr>
<th>Equation Number</th>
<th>Derived Parameter</th>
<th>Proposed Equation</th>
<th>Correlation Coefficient</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>UCS</td>
<td>UCS = 1.86 (R*) -40.39 : axial SHR</td>
<td>0.73</td>
</tr>
<tr>
<td>2</td>
<td>UCS</td>
<td>UCS = 1.50 (R) -19.03 : diametrical SHR</td>
<td>0.74</td>
</tr>
<tr>
<td>3</td>
<td>UCS</td>
<td>UCS = 8.15 (PLI) -16.99</td>
<td>0.96</td>
</tr>
<tr>
<td>4</td>
<td>UCS</td>
<td>UCS = 5.11 (BDS) -5.10</td>
<td>0.78</td>
</tr>
<tr>
<td>5</td>
<td>BDS</td>
<td>BDS = 1.60 (PLI) +0.90</td>
<td>0.72</td>
</tr>
</tbody>
</table>

*Schmidt Hammer Rebound Value

5 Discussion

There are significant relationships that exist between the petrographic, physical and mechanical properties of selected sandstones of the Newspaper Member of the Natal Group sandstones for which linear equations have been derived.

According to Folk’s (1974) classification of sandstones, the studied sandstones of Cliffdale, Scottburgh and Coedmore Quarry plot in the arkosic category and are classified as arkosic sandstones whereas those obtained from the Palmiet Nature Reserve are classified as sub-arkosic.

The mechanical properties of the studied sandstones were determined by the geotechnical tests performed. The intact strength of the studied sandstones was determined by subjecting the ten core samples to UCS testing. Sandstones of the Coedmore Quarry exhibited the greatest mean UCS value of 63.21 MPa. This was followed by sandstones of Cliffdale which have a mean UCS value of 53.06 MPa. The Palmiet Nature Reserve sandstones had the second lowest UCS value of 46.72 MPa. The Scottburgh sandstones displayed the lowest intact strength with a mean value of 32.99 MPa. According to Bieniawski’s (1979) strength classification of rock materials, the sandstones of Coedmore Quarry and Cliffdale are classified as medium to high strength sandstones whereas those of the Palmiet Nature Reserve and Scottburgh are classified as low strength sandstones.

The SHR followed the same trend as the UCS with the studied sandstones of Coedmore Quarry having the highest mean SHR and the studied sandstones from Scottburgh displaying the lowest SHR. The mean axial SHR values exceeded that of the mean diametrical SHR values.

The tensile strength of the studied sandstones was determined indirectly by conducting the PLI and Brazilian Disc strength tests. The studied sandstones from Coedmore Quarry had the highest mean PLI and Brazilian Disc strength, with values of 9.56 MPa and 14.28 MPa respectively. Cliffdale had the second highest mean PLI and Brazilian Disc strength, with values of 6.62 MPa and 12.12 MPa respectively. The studied sandstones from the Palmiet Nature Reserve had the second lowest mean PLI and Brazilian Disc strength, with values of 5.79 MPa and 10.40 MPa. Scottburgh’s sandstones displayed the lowest mean PLI of 5.53 MPa as well as the lowest mean Brazilian Disc strength with a mean value of 7.13 MPa. The PLI of the investigated sandstones from the four different localities classified as high strength based on Bieniawski’s (1979) strength classification of rock materials. A trivial difference was noticed between the axial and diametrical PLI which are indicative that the sandstones are not anisotropic. Although all the studied sandstones are from the Newspaper Member of the Natal
Group, Brady and Brown (2006) mentioned that the strength can vary widely due to factors such as the degree of weathering and porosity.

It is expected that numerous petrographic factors have an influence on the engineering properties of rocks. This study’s focus was on the mineral composition, grain shape and mean grain size. There were no significant relationships existing between the percentage of quartz comprising the sandstones and the UCS of the studied sandstones. Similar results were found by Acikalin et al. (2008). In contrast, positive correlations between the two parameters were established by Bell and Lindsay (1999) as well as Ulusay et al. (1994). The strength of the sandstones did not increase with decreasing mean grain size nor did the sphericity of the grains significantly influence the strength of the studied sandstones. Plachik (1999) also found that the mean grain size and sphericity of grains did not have any meaningful influence on the strength of sandstones whereas Fahy and Guccione (1979) reported that sandstones with smaller mean grain sizes are expected to pose higher strength.

The relationship between porosity and the mechanical strength of the selected sandstones were investigated and it was found that porosity has a major influence on the UCS, SHR, Brazilian Disc strength and PLI. Negative linear relationships exist between porosity and mechanical strength of the sandstones, with correlation coefficients of 0.85, 0.81 0.73 and 0.98 existing between porosity and UCS, axial and diametrical SHR, Brazilian Disc strength and PLI respectively. As the porosity increases, the strength of the rock decreases. Similar results were reported by Bell and Culshaw (1998). The highest strength was exhibited by the sandstones of Coedmore Quarry and this is attributed to these sandstones having the lowest porosity value of 0.49%.

Significant positive linear relationships occur between UCS, SHR, PLI and Brazilian Disc strength. The highest correlation coefficient of 0.96 was produced by the linear relationship between the PLI and UCS. The relationships between the axial SHR, diametrical SHR, Brazilian Disc strength and UCS yielded correlation coefficients of 0.73, 0.74, 0.78 and 0.83 respectively. Similar investigations conducted by Gupta et al. (2015) and Ajalloeian et al. (2014) also found positive relationships between the indirect testing measures and UCS.

The linear equations produced in this study relating the indirect tests to UCS can be used when sandstones have similar petrographic and physical characteristics, as shown in the sandstones under investigation in the current study.

The relationship between the PLI and Brazilian Disc strength were also considered in the current study of selected sandstones of the Newspaper Member of the Natal Group. A positive linear relationship exists between the two tensile strength parameters, with a correlation coefficient of 0.72. An increase in one tensile strength parameter resulted in an increase in the other. Shrestha et al. (2006) and Alemdag (2012) also investigated the relationship between the tensile strength parameters of sandstones and found significant positive relationships existing between the two parameters.

6 Summary and Conclusion

Sandstone samples of the Newspaper Member (Marianhill Formation, Natal Group) were collected from four localities namely Cliffdale, Scottburgh, Coedmore Quarry and the Palmiet Nature Reserve. They were subjected to petrographic and geochemical analyses as well as laboratory and mechanical tests to determine the relationships that exist between the petrographic, physical and mechanical properties. The results showed that the sandstones are composed mainly of quartz, K-feldspar and plagioclase and are classified as arkosic with others being sub-arkosic. As far as the grains of the sandstones of the Newspaper Member are concerned, grain size ranged from fine to coarse grained and the shape of the grains were sub-
angular to sub-rounded. The porosity of the Newspaper Member sandstones was generally low with moderate to high dry densities. When the sandstones were tested in unconfined compression, they ranged from low to high strength with most having a high intact strength. The indirect tensile strength were determined by the PLI and Brazilian Disc strength test. The PLI test resulted in strength values that indicated that the studied Newspaper Member sandstones possessed high strength.

The study identified the undermentioned relationships:

- The percentage of quartz as well as the mean grain size had no substantial influence on the strength of the studied sandstones.
- Porosity had a significant negative relationship with the strength of the sandstones. The sandstone samples of Coedmore Quarry displayed the highest strength which can be attributed to its low porosity.
- The relationships between the mechanical properties were significant and the linear equations relating to the indirect test to UCS produced high correlation coefficients that can be used when sandstones have similar properties on the sandstones of the current study.
- A significant positive relationship exists between the point load strength and Brazilian Disc strength.

In general, the relationships between the mechanical properties and physical properties as well as the relationships amongst the mechanical properties themselves proved to be more significant than the relationship between the petrographic and mechanical properties.

References


Bell, F.G. and Lindsay, P. 1999. The petrographic and geomechanical properties of some sandstones from the Newspaper Member of the Natal Group near Durban, South Africa. *Engineering Geology*, 53, pp 57–81.


Investigating Dolomitic Land

L. R. Richer¹, A. G. A’Bear²

¹ LR Geotech, Johannesburg, Gauteng, lindi@lrgeotech.co.za
² Bear GeoConsultants, Johannesburg, Gauteng, tony@bgconsult.co.za

Abstract

Hazard investigations of dolomitic areas are one of the few fields within geotechnical investigations that are regulated and allow for peer review. As such, geotechnical investigations of dolomitic areas should be good examples of efficient geotechnical investigative and reporting techniques.

An area of concern with dolomitic investigations relates to the nature of the investigation undertaken. In most cases investigations are carried out by applying the minimum requirements set out in the standards. This has resulted in terrain evaluation processes, which have been in use since the 1970s, being disregarded.

This paper argues that exploration of dolomitic areas requires different techniques depending on the nature of the overburden and the depth to bedrock. By applying iterative land evaluation processes properly and by using appropriate techniques, a far more precise evaluation of the inherent hazard classification, with respect to the potential for sinkholes and subsidences to develop, may be arrived at.

Keywords: sinkhole hazard, dolomite investigation, terrain evaluation, geological model

1 Introduction

Areas underlain by dolomite, a soluble rock, are subject to the development of karst features, such as sinkholes. These pose a significant hazard to property and may even be life threatening. The following factors are used to evaluate the degree of hazard associated with sinkhole and subsidence development (Buttrick, 1992):

- Mobilising agencies, most importantly ingress water from leaking services or ponding of water on surface;
- Bedrock morphology, significantly the bedrock pattern, involving the wavelength and amplitude of pinnacle and gryke development.
- The presence of cavities and fissures, and the depth to them;
- Nature of the blanketing layer, including its potential to erode into underlying cavities and it’s potential to absorb or reduce the velocity of water flowing vertically through it; and
• The depth to the present groundwater level and its position relative to bedrock and overburden.

The standards set in SANS 1936-1-4:2012 (SABS, 2012a-d) and SANS 634 (SABS, 2012e) detail the requirements for geotechnical investigations of dolomitic land and specifically provide for peer review in situations where the regulatory authority disagrees with the findings of the consultant, or vice versa.

The major influence on the type of investigation carried out is the depth to dolomite. When shallow dolomite bedrock is present it becomes imperative to define bedrock morphology and when deep bedrock is present the characterisation of the overburden becomes of greater importance.

2 Geotechnical Model and Dolomitic Terrain Evaluation Process

SANS 1936-2:2012 (SABS, 2012b) states that “Rational assessment shall be based on a geotechnical model” with regards to inherent hazard classification of dolomite land. The compilation of the geotechnical model should include three components (SABS, 2012b):
• The geological model,
• The overburden model, and
• The geohydrological model.

The geological model should include the following (SABS, 2012b):
• geological setting, dynamics and site geology;
• geomorphology;
• bedrock lithology and karst features;
• presence and distribution of intrusions;
• presence and distribution of historic karst features (sinkholes, subsidences, swallow holes, etc.);
• presence and extent of voids in overburden and bedrock;
• overburden thickness and composition;
• weathering and weathering products; and
• drainage.

The nature of the overburden determines whether arching over a void with catastrophic collapse or gradual settlement with collapse will occur. Therefore, determining the shear strength and permeability of that overburden is important.

The critical parameters forming the geohydrological model are (SABS, 2012b):
• depth to groundwater;
• position of perched and regional groundwater with respect to bedrock;
• variation of groundwater level over time, taking the likelihood of future de-watering into account; and
• groundwater abstraction and recharge conditions.

Terrain evaluation is an iterative process which may require more than one level of investigation before a definitive geological and geotechnical model can be produced. Buttrick (1992) developed the “Method of Scenario Supposition” to provide a set of factors for dolomite stability characterisation. Table 1 overleaf shows the steps involved in this method.
Table 1. Method of Scenario Supposition (Buttrick and van Schalkwyk, 1995)

<table>
<thead>
<tr>
<th>Step</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Step 1</td>
<td>Field reconnaissance and desk study of site</td>
</tr>
<tr>
<td>Step 2</td>
<td>Preliminary zoning utilising tools such as air photo interpretation and geophysics</td>
</tr>
<tr>
<td>Step 3</td>
<td>Preliminary boreholes to characterise “preliminary” zonation</td>
</tr>
<tr>
<td></td>
<td>Characterisation process (scenario supposition). Individual borehole profiles are reviewed within context of the selected scenarios</td>
</tr>
<tr>
<td>Step 4</td>
<td>Evaluation factors</td>
</tr>
<tr>
<td></td>
<td><strong>Sinkhole formation</strong></td>
</tr>
<tr>
<td></td>
<td>Mobilisation agency/agencies</td>
</tr>
<tr>
<td></td>
<td>Receptacle development</td>
</tr>
<tr>
<td></td>
<td>Potential development space (i.e. potential sinkhole size)</td>
</tr>
<tr>
<td></td>
<td>Nature of blanketing layer/s</td>
</tr>
<tr>
<td></td>
<td>Mobilisation potential of blanketing layer/s</td>
</tr>
<tr>
<td></td>
<td><strong>Subsidence (doline) formation</strong></td>
</tr>
<tr>
<td></td>
<td>Mobilising agency</td>
</tr>
<tr>
<td></td>
<td>Nature of blanketing layer/s</td>
</tr>
<tr>
<td></td>
<td>Mobilisation potential</td>
</tr>
<tr>
<td></td>
<td>Lateral extent</td>
</tr>
<tr>
<td>Step 5</td>
<td>Pooling of individual borehole characterisations and amending of preliminary zoning, taking historical information into account.</td>
</tr>
<tr>
<td>Step 6</td>
<td>Finalised risk zonation characterised in terms of a certain risk of certain sized features forming</td>
</tr>
<tr>
<td>Step 7</td>
<td>Selection of appropriate development type and precautionary measures</td>
</tr>
<tr>
<td>Step 8</td>
<td>Implementation of appropriate development design and precautionary measures</td>
</tr>
<tr>
<td>Step 9</td>
<td>Vigilance and maintenance</td>
</tr>
</tbody>
</table>

This method of scenario supposition should be used to create the geotechnical model required for SANS1936. The authors recommend an addition to the method at Step 5 of the process. It is recommended that the validity of zones and data should be evaluated and it should be established whether further or different techniques are required and either go back to Step 2 or proceed to Step 6.

3 Definition of Shallow Dolomite

The draft version of SANS 1936-2 (SABS, 2009) had a definition of shallow dolomite as being “where the average bedrock head is less than 8 m”. There is no definition in the current SANS 1936-2 (SABS, 2012b). In the authors experience it possible that areas in which dolomite pinnacles actually outcrop can still have an average depth to bedrock well in excess of 8 m if deep grykes, solution features usually associated with weathering along joints, are present. The average depth to bedrock in boreholes may not truly reflect the average depth to bedrock across a site as boreholes are often sited to avoid outcrop or pinnacle heads.

Shallow dolomite areas could perhaps be better defined as being those areas in which bedrock, or pinnacle bedrock is commonly found within reach of a large track mounted excavator. This would indicate that a significant portion of the site is underlain by rock at depths of 4 m or less.
4 Investigations of Shallow Dolomite Areas

Traditionally, shallow dolomite is considered to have a high hazard rating in terms of the potential for sinkholes to develop, usually resulting in an automatic classification of the site with an Inherent Hazard Class rating of 5 (Buttrick et. al., 2001) indicating that it has a high potential for small sinkholes to develop. There is clear evidence that the current approach defining all shallow dolomite sites as being highly hazardous is not appropriate (A’Bear and Richer, 2011).

When shallow bedrock is present, it becomes vitally important to determine the bedrock morphology as it is the size of opening leading to cavern which determines sinkhole size. Gravity surveys are the most frequently applied geophysical method for investigating dolomitic terrain in South Africa. It has become the norm to apply a station spacing of 30 metres when carrying out gravity surveys even though it is well understood, and even recommended, that the station spacing be related to the depth to bedrock. Conventional gravity surveys on shallow dolomite terrain tend to show little variation in depth to bedrock and are unable to pick up the narrow grykes which are characteristic of such terrain. This is largely due to the grid spacing being too wide.

Previous work (A’Bear and Richer, 2011 and A’Bear et al., 2015) has shown the value of a closer spacing when rock head is shallow. Narrowing the grid spacing to 5 m or even 10 m tends to improve the detail recovered dramatically (See Figure 1 and 2) (A’Bear et al., 2016) but this is usually only cost effective on small sites.

On larger sites, more than one or two hectares in area, the increase in cost is usually too large for the project to bear. It is nevertheless recommended that all sites be covered by a gravity survey, unless too small, and that this should be done on as small a grid as possible. Should this not be feasible then it is recommended that a smaller grid be applied within the footprint of specific structures or several traverses with closely spaced stations be added into the conventional survey. These should consist of at least four traverses and should be done in two orthogonal directions. This should allow for smaller features to be picked up and some estimate made of direction and spacing.

Trenching using large excavators to expose the bedrock over long distances has been used successfully on a number of sites by the authors (See Figure 3). This allows for pinnacle spacing and gryke widths to be determined as well as the nature of the gryke infill to be examined. The geotechnical parameters of the gryke infill can sometimes be determined by carrying out laboratory tests on undisturbed samples or at least be estimated from test carried out on disturbed samples combined with conventional profiling.

Drilling to establish the depth of the grykes and the nature of them is still required. However, it is strongly recommended that clusters of boreholes be used to characterise the area with spacing between boreholes in each cluster being set at between 5 and 10 m. As shallow dolomite sites usually result in a substantial reduction in the total depth drilled it becomes cost effective to drill more holes but in a different pattern from the norm. In this sense, it is recommended that the minimum number of boreholes recommended per hectare by the authorities be increased by at least 15% in shallow dolomite sites.

There appears to be some validity in looking at a parameter using the variance in depth to bedrock within a shallow dolomite site (A’Bear and Richer, 2011). This appears to give a reasonable method of differentiating high from low hazard sites as it reflects the extent of development of solution features.
Figure 1. Gravity survey comparison on a site in Tembisa, Gauteng. Solid lines represent the 5 m grid and dashed lines the 30 m grid.

Figure 2. Comparison of 5 m and 30 m gravity survey grids on a site in Centurion, Gauteng. Solid lines represent the 5 m grid and dashed lines the 30 m grid.
5 Investigation of Deep Dolomite

Where deep dolomite bedrock is present, overburden properties become of overriding importance. If the overburden is composed of highly permeable material ingress water is able to reach velocities enabling erosion of the overburden provided it is capable of being eroded. If the blanketing layer comprises low permeability materials, percolating water is unlikely to be able to erode the material as it cannot achieve the critical flow velocity required to entrain particles.

The nature of the overburden also determines whether arching over a void ultimately resulting in catastrophic collapse will occur or whether gradual settlement with collapse will occur. Therefore, determining the shear strength and permeability of that overburden is important. Techniques to obtain samples to ascertain these properties could include, trenching and auger holes to get to the lower lying horizons. In-situ testing, such as plate jacking tests, can be carried out in these excavations and borings. Undisturbed samples can sometimes be retrieved and taken to the laboratory for relevant testing.
Well documented work (Buttrick, 1986 and Day, 1981) has shown that residual dolomite (wad) which is free of chert bands can be successfully tested and characterised in the field and laboratory. Testing along these lines has recently resulted in areas previously considered to be highly hazardous being released for development on the basis that the wad was proven to be competent overburden.

6 Conclusions

When starting off any geotechnical investigation on dolomite it is important to determine whether it is a shallow dolomite or deep dolomite site. This will help to decide whether bedrock morphology or the nature of the overburden is of overriding importance. As a start the following steps should be used:

- Desk study of available information – should information be available this may indicate which approach to make when investigating the site
- Geophysics - particularly a gravity survey with a grid spacing of no less than 30 m, except where the context indicates otherwise
- Limited drilling

Using this information, preliminary zones can be drawn up and it can be determined whether shallow or deep dolomite is present.

If the site is a shallow dolomite site where bedrock morphology is of importance, then the following should be part of the investigation:

- An evaluation of the bedrock morphology and this may involve:
  - More detailed gravity surveys,
  - Trenching,
  - Revised drilling approach, and
  - A statistical analysis (A’Bear and Richer, 2011).
- An evaluation of the gryke infill and this may involve in-situ and laboratory testing.

If the nature of overburden is of importance then the following should be done if possible:

- Alternative geophysical investigations,
- Additional drilling,
- Auger drilling,
- In-situ tests and laboratory tests to characterise the strength, permeability and erodibility of the overburden or its components.

This is reiterative process which may require more than one level of investigation before a definitive geotechnical model can be produced.

References


SABS. 2012e. SANS 634 Geotechnical Investigations for township development. Pretoria, SABS Standards Division.

Measurement of Clay Fracture Strain with a Bending Beam Test

D. H. Marx

1 Jones & Wagener (formerly University of Pretoria), Rivonia, Gauteng, marxdavidh@gmail.com

Abstract

Clay liners are used as impermeable barriers in the mining and landfill sectors. However, these liners are susceptible to cracking due to differential and local settlement of the foundation material. The cracking can result in the liner leaking, compromising its ability to perform as an effective barrier. To determine the fracture strain of the clay, tensile stress-deflection curves can be measured with a four-point bending beam test. This article discusses such a test setup, as well as the steps followed for analysis. From the tensile stress-deflection curves micro- and macro-crack growth could be identified. Subsequently, the corresponding tensile strains for fracture could be identified. Once these strains are known the liner can be designed to be stiff enough, or the foundation can be treated, to prevent cracking from rendering the barrier unusable.

Keywords: clay, fracture mechanics, four-point bending test, tensile strength

1 Introduction

In recent decades the field of civil engineering has seen a greater drive towards environmental responsibility. In the discipline of geotechnical engineering this drive is giving rise to the importance of clay liners. These liners, constructed from several layers of compacted clay, are, amongst others, used to isolate pollutants from the environment. In landfills waste is deposited on multi-layer liner systems, comprising of compacted clay liners and geosynthetics, to prevent leachate generated in the waste from contaminating underlying groundwater.

Another important application for these clay liners is in the field of opencast mining. Several South African coal deposits occur below, or near, sensitive wetland areas. In some cases the wetlands are destroyed as part of the mining process. Subsequently, during mine closure these areas are rehabilitated and the wetlands are rebuilt. Clay liners are used as basis for the new wetland with fauna and flora established on top.

In both these examples, landfills and wetland rehabilitation, the clay liners are used as a barrier to the flow of liquids. In landfills the clay liners prevent leachate above the liner from contaminating the underlying ground water. Similarly, in wetland rehabilitation clean water in
the rehabilitated wetland is prevented from filtering down into the old mines. However, in both landfills and wetland rehabilitation the foundation for the clay liners may comprise of soft, heterogeneous materials. Backfilled opencast mines, and old waste in the case of piggyback landfills, are both prone to differential settlement. Differential settlement results in the liners distorting and cracking. The cracks increases the permeability of the liner, and thus compromises its ability to perform as an effective barrier.

To mitigate the effect of the differential settlement several mitigation measures, such as geogrid reinforcement, can be implemented. However, before these mitigation measures can be designed, the behaviour of the clay in tension has to be understood. Specifically, the maximum strain that can be imposed on the clay before fracture occurs should be known. This paper describes a test to measure the tensile behaviour of clay, and to identify crack growth and the corresponding fracture strain.

2 Fundamentals of fracture mechanics

Before the strain required for fracture of clay can be measured, fracture itself has to be defined. In Figure 1 three photos are shown of a clay beam in various stages of fracture. To identify which of these three stages: a) internal cracking, b) onset of observable cracking and c) significant observable cracking, qualifies as the onset of fracture the theory of fracture mechanics will briefly be explored.

Most materials, including clay liners, are riddled with defects or micro-cracks (Vallejo, 1994). As a clay liner distorts due to differential or local settlement, tensile stress will be generated in the clay. This stress will drive the micro-cracks to grow and merge into so-called macro-cracks. The macro-cracks will eventually grow large enough to be visible.

Crack growth is driven by so-called energy sources, e.g. elastic strain energy stored inside the material. However, in ductile materials, such as clays, a number of energy sinks are present that absorbs the energy released during fracture, retarding crack growth (Hallett & Newson, 2001). Some energy sinks includes plastic deformation, micro-crack growth, and friction between the two crack surfaces. Consequently, due to these energy sinks energy has to be continuously added to the material for crack growth to continue, i.e. the work done on the material has to increase. Thus, clay liners do not suddenly fail catastrophically.

The emergence of the micro- and macro-crack growth can be identified from the tensile load-deflection curve of the material (Turner & Kolednik, 1994; Karihaloo, 1995). In Figure 2 a typical tensile load-deflection curve is shown. From points O to A the behaviour is linear. Once micro-cracking damages the material (A) it supports less and less load for a given displacement increment (i.e. it becomes non-linear). When the curve reaches its peak (B) the macro-cracks
opens. If the material is elastic-plastic the load supported will remain constant while the deflection increases (B-C). However, for a quasi-brittle material the load will drop and finally plateau (B-D).

![Figure 2](image.png)

**Figure 2.** Typical tensile load-deflection curve of a material that cracks. B-C is an elastic-plastic material while B-D is a quasi-brittle material.

To measure a tensile load-deflection curve similar to that provided in Figure 2 one of several tensile tests can be used. The direct tensile test is fundamentally the most representative test of the tensile behaviour of a soil (Ajaz & Parry, 1975). However, this test is temperamental - with samples prone to failing at the clamps due to stress concentrations. Consequently, several authors used four point bending beam tests to study the fracture behaviour of clays (Ajaz & Parry, 1975; Thusyanthan, et al. 2007; Plé et al., 2012). In these tests two loads are applied to the centre of a clay beam resting on two supports. As these loads generate a moment in the beam, tensile stress is induced at the base of the beam, allowing for the fracture behaviour to be studied. The bending beam test has the benefit that it replicates the fracture of clay liners which are rarely in direct tension

### 3 Methodology

Four-point bending beam tests were done on kaolin beams in a Lloyds 5 kN press fitted with a Lloyds LC 5kN (15643) load cell. The clay beams rested on two support bars of 16.1 mm diameter that were 98 mm apart, as shown in Figure 3. Two top bars are lowered at thirds of the span to load the beam. These were connected to the load cell. One of the support bars, and both the loading bars, could swivel out of in plane. The accuracy of the load cell was ±5% and the resolution 0.25 N.

The top bars were lowered at a rate of 3 mm/min while photos of the beams were captured at 6 s intervals with a Canon 100D camera equipped with a fixed 40 mm lens. These photos were subsequently used for digital image correlation analysis.

The clay beams were consolidated to 95% of its Proctor density, equivalent to 1415.5 kg/m³. This density was achieved at a consolidation stress of 608 kPa. Consequently the beams were one-dimensionally consolidated to 610 kPa.
To prepare the beams the clay was mixed with a motorised mixer at a gravimetric moisture content of 100% for a minimum of 10 min to ensure a homogeneous mixture. Subsequently, a vacuum was applied for the final mixing stage to remove air bubbles in the mixture. After mixing, the slurry was poured into an oiled rectangular steel mould (710 mm x 155 mm in plan). All clay that was in contact with the oil was removed after consolidation. Double drainage of the clay was ensured by placing thick, non-woven, separation geotextiles above and below the slurry.

The filled mould was placed in a consolidation press fitted with a hydraulic piston that pressed onto a 10 ton load cell. From the load cell the force was transferred to a 40 mm thick solid steel section that distributed the applied load evenly over a channel beam directly on top of the separation geotextile. The load applied to the clay was incrementally increased to 12.5 kPa. Hereafter it was doubled until a final consolidation stress of 610 kPa was reached.

The time for 95% consolidation for a 50 mm high clay slurry at a 103 kPa was 140 min ($\sigma' = 100$ kPa) and for a 30 mm clay layer at 626 kPa was 15.2 min. After the load remained constant for more than 30 minutes, the clay was judged to have consolidated sufficiently for the next loading stage. The final load increment of 610 kPa was sustained for 8-10 hours.

Finally, after the clay was fully consolidated at 610 kPa, it was swelled in steps of approximately 100 kPa per hour down to 50 kPa. The clay remained at this final swell pressure of 50 kPa for a minimum of two hours to allow for the pore pressures to equilibrate. Afterwards the clay was demoulded, the surface levelled and the beams cut. Beams between 27.9 mm and
29.4 mm thick, 50 mm wide and 150 mm long were cut. Washed sand grains were added to the surface of the beams to provide texture to improve the digital image correlation analysis.

4 Digital image correlation

Soil movement in some geotechnical problems can be measured by capturing a series of photos of the problem over time and comparing the relative movement between the images (frames). The technique is known as Digital Image Correlation (DIC) or in some geotechnical publications as Particle Image Velocimetry (PIV) (Stanier et al., 2015). Two DIC software packages, GeoPIV-8 (White et al., 2003) and GeoPIV-RG (Stanier et al., 2015) were used for this study.

Firstly, the DIC technique isolates a patch in a reference image. Secondly, subsequent images are searched for the location of this patch to determine its relative displacement. The process is repeated for a number of patches spread over the image to generate a displacement field.

The reference image can either be kept constant through the analysis (compare image 1→2, 1→3, 1→4, ..., 1→i), or it can be updated (compare image 1→2, 2→3, 3→4, ..., i→i+1). The first method, a leapfrog analysis, might suffer from loss of correlation as the soil deforms excessively. For the second type, a sequential analysis, "random walking errors" might occur. A "random walking error" occurs when movement is measured while none has taken place (Stanier et al., 2015). Consequently, care has to be taken when choosing an updating scheme.

The GeoPIV-8 and GeoPIV-RG software packages have sophisticated calibration algorithms to compensate for lens distortion, refraction through a glass pane, irregular shaped pixels on the camera’s sensor and alignment errors (White et al. 2003). However, considering that: 1) camera sensor technology has come a long way since the software was initially released, 2) there was no glass between the camera and the beam and 3) the camera and lens combination took photos of very low distortion; only a single calibration factor was used for the bending beams. To calculate the calibration factors, the movement of the two loading bars were tracked with GeoPIV-8 and compared to the movement recorded by the loading press.

5 Results

The tensile stress at the base of the beams in the four point bending beam test can be calculated as:

\[
\sigma = \frac{PL}{bd^2}
\]  

Where \( P \) is the applied load, \( L \) the length of beam, \( b \) the width and \( d \) the depth.

A typical tensile stress –deflection curve is shown in Figure 4a. In this work tensile stress is shown as positive (and compressive negative), contrary to popular geotechnical convention. The initial segment of the tensile stress-deflection behaviour of the beams was unexpectedly nonlinear. As linear behaviour is expected until micro-cracks had formed, further investigation of the stress-deflection data was conducted.

Firstly, the displacement used in the stress-displacement plots was reconsidered. Consequently, the displacement of a single patch of 3.4 x 3.4 mm at the effective centre of the beam was tracked using GeoPIV-8. The analysis was repeated for both a patch at the base and a patch at the top of each beam.
There was little difference in movement at the top and bottom of the beam. Furthermore, once the stress was plotted against the displacement measured with DIC, the initial behaviour of the curve was much closer to the expected linear behaviour (Figure 4b). The non-linear behaviour recorded by the loading press could have been due to the loading bars punching into the clay, resulting in the skewed displacement measurements. Once the contact area between the clay and the loading bars had increased sufficiently, the expected linear behaviour was recorded by the loading press.

A further consideration for interpreting the stress-deflection curves was that the clay beams were soft enough to deflect under own weight. Consequently, the measured (DIC) displacement was offset by this initial mid-span deflection (see Figure 4c).

Finally, the load (stress) applied had to be adjusted for the self-weight of the beam. The equilibrium of the system was investigated as depicted in Figure 5. The reactions due to the self-weight of the beam are $R_C$ and $R_D$. To convert the distributed load of the self-weight to an equivalent point load offset, the reactions at the supports due the load offset ($R_A$ and $R_B$) should equal $R_C$ and $R_D$. Given an average moisture content for the five beams of 31%, and a $G_s$ of 2.661 for kaolin, the unit weight of the beams at full saturation was calculated as 18.75 kN/m$^3$. For the beam in Figure 4 the reactions are $R_C = R_D = 3.01$ N = $R_A = R_B$, thus the offset is $P = 6.04$ N. From here onwards all stress-displacement plots are those of the corrected data as shown in Figure 4d.
In Figure 6 tensile stress-deflection curves are shown for five of the kaolin beams tested. The corrected tensile stress-deflection curves in Figure 6 are initially linear up to approximately 6 mm of displacement whereafter the behaviour became non-linear. The implication is that micro-cracks have formed inside the clay. After the non-linear behaviour, the tensile stress-deflection curve reached a peak and then plateaued. Consequently, macro-cracks had formed inside the clay, as the beam had reached the maximum load it could support. The stress plateau, rather than stress drop, observed in Figure 6 represents the ductile behaviour expected of a) a clay and b) a small fracture specimen. The tests were terminated before total collapse of the sample could occur, as shear stress commenced at the bottom supports.

In Figure 6 the occurrence of micro-cracks, identified as a change in slope of the stress-deflection curve, and the opening of macro-cracks, at the peak load, are indicated with black dots. Incidentally, the two outliers visible were for a beam that was significantly thinner than the others, reinforcing the principle that fracture behaviour is specimen size dependent (Karihaloo, 1995). The five stress-deflection curves, as well as the displacements for crack growth, were averaged to obtain the stress-deflection curve in Figure 7.

After the tensile stress-deflection curves were measured the linear horizontal strain along the base of the beams was calculated from the displacements of the DIC analysis. Due to its own weight the beams settled before any load was applied (see Figure 6d). The strain due to this deflection was added to that calculated from the DIC analyses. In Figure 7 the tensile strain at the base and compressive strain at the top of the beam is shown. A 15.23 mm wide gauge (approximately half of the loading width) was used to calculate the strain.
Figure 7. Strain-deflection results for the bending beam tests

At the same displacement where micro-crack growth occurred (identified where the stress-deflection curve deviated from linear), the tensile strain-deflection curve also deviated from linear. Consequently, micro-crack formation can also be defined from the tensile strain-
deflection curve. This correlation implies that micro-crack growth can be identified in physical modelling of clay liners where only the strain, and not the stress, in the clay liners (or beams) is available.

In contrast to the tensile strain the change in compressive strain aligned with the macro-crack growth, thus a different central displacement. However, both the change in compressive and tensile strain occurred at the same magnitude of strain. This difference between the compressive and tensile strain behaviour could be related to difference in radius of curvature differs at the top and bottom of the beams. Furthermore, once macro cracks formed in the clay a greater load was applied to a smaller cross-sectional area of the beam. Consequently, the beam yielded in compression.

The tensile strain where micro-cracks occurred was -8.71% at a stress of -66.5 kPa for the average curve shown in Figure 7. Macro-cracks were judged to have opened at a stress of -75.3 kPa and a compressive strain of 8.73%. Even though these values are representative of a fully saturated kaolin beam consolidated to 610 kPa it is not applicable to all clay liners or beams. Fracture strain in literature varied from 0.08% to 2.5% for several clay types, measured with full scale bursting, direct tension and bending beam tests (Ajaz & Parry, 1975; Edelman et al, 1996; Thusyanthan et al, 2007; Gourc et al, 2010 and Plé et al, 2012). The significant difference in magnitude of tensile strain at fracture is due: 1) the consolidation pressure of the sample and the initial effective stress in the sample (Thusyanthan et al, 2007), 2) the size of the virtual strain gauge used for the analysis and 3) the sample size (see Clayton et al (2016) for the effect of sample size on the undrained shear strength of clay). Finally, none of the literature reported whether the strain is for micro- or macro crack growth.

5.1 Designing for fracture
Deciding to design for micro- or macro-cracks depends on the design scenario. A small increase in the void ratio of a saturated clay due to cracking can result in a significant increase in permeability (Fredlund et al. 2010):

$$k = \frac{2}{fA^2} \cdot \frac{e^3}{1+e}$$  \hspace{1cm} (2)

where $k$ is the permeability (m/s) at a reference temperature of 20°C, $f$ the angularity of the particles, $A$ the specific surface area of the grains (mm²/mm³) and $e$ the void ratio.

For a clay liner where the primary function is a liquid barrier, leaking through the liner, i.e. the forming of cracks, will be the ultimate limit state. Consequently, the liner should be designed to prevent micro-cracks from forming. In a different scenario the liner could have a structural function and the liquid barrier might be less critical. In this case the serviceability limit state of the liner will be leakage, while collapse of the liner will be the ultimate limit state. As the tensile stress-deflection curve reaches a peak when macro-cracks opens, at ultimate limit state the design should prevent macro-cracks from opening. Arguably, this second case can be applicable to wetland rehabilitation where some loss of clean water through the liner will still be acceptable.

6 Conclusion

Firstly, a methodology for a four-point flexural test on clay beams to measure the tensile-stress deflection curves was discussed. Thereafter, the fundamental principles of fracture mechanics were successfully applied these curves. It was found that in tension clay behaves in three distinct stages: 1) initial linear elastic load-displacement behaviour, 2) forming of micro-cracks and 3)
opening of macro-cracks. Once micro- and macro-cracks were identified it was possible to measure the tensile stress and strain in the clay when these cracks formed. For liners that are primarily liquid barriers preventing micro-crack growth is critical, while for liners with a structural function preventing macro-crack growth is critical.

Acknowledgements

The financial assistance of the Deutscher Akademischer Austausch Dienst (DAAD), the National Research Foundation of South Africa (NRF) and the Geosynthetics Interest Group of South Africa (GIGSA) is acknowledged by the first author. Opinions expressed and conclusions presented are those of the authors and are not necessarily to be attributed to the NRF or GIGSA.

References

Towards the Development of an Indirect Method of Predicting the Shear Strength Properties of Shales

P. Naidoo1

1SRK Consulting, Durban, KwaZulu-Natal, pnaidoo@srk.co.za

Abstract

Urban sprawl in the Durban metropolitan area has necessitated for the development of areas underlain by shale which were previously deemed unsuitable. Shale is a highly degradable material which weathers easily when exposed to the physical environment. Representative samples of both fresh and weathered shale material of the Pietermaritzburg Formation were sampled from four localities within the Durban-Pietermaritzburg area and subjected laboratory testing. The results from the laboratory tests were used to develop a modified method of indirectly predicting the shear strength properties of shales. A shale rating system was subsequently modified and developed which uses simple index tests to rate shales and indirectly predict the shear strength properties of shales. The rating system divides shales into three main categories based on a range of index properties and shear strength parameters. The rating system will be useful to geotechnical engineers as they are bound by financial limitations and time constraints.

Keywords: Degradable materials; Shale rating system.

1 Introduction

Population growth has necessitated for the relocation of people to land that was once deemed unsuitable. These unsuitable areas often consists of highly degradable, flawed, discontinuous and inhomogeneous geological materials of low strength, low lying areas which are prone to flooding or even environmentally unfavourable areas (Walkinshaw and Santi, 1996). Shale is a type of argillaceous material which consists of silt and clay sized particles (Tucker, 2001) and is one of the most problematic type of degradable materials (Walkinshaw and Santi, 1996). When exposed to the physical environment, shales degrade rapidly which includes physical and chemical changes (Pidwirny, 2006). Taylor (1948) also stated that shales degrade easily on exposure to air and water. Additional problems caused by shales include settlement, landslides and borehole instabilities (Strohm et al., 1981; Bell & Maud, 1996, 1997; Hopkins & Beckham, 1998).

Shales are distributed extensively within the KwaZulu-Natal province and constitute one half of the volume of sedimentary rocks in the shallow earth’s crust (Picard, 1971; Blatt, 1982; Walkinshaw and Santi, 1996; Merriman et al., 2003). The Pietermaritzburg Formation outcrops frequently in the Durban area and because of its association with landslides, it has been regarded as unstable (Bell and Maud, 1996; 1997). Shales are very problematic rocks in terms
of sampling and laboratory testing. It is therefore very difficult to obtain representative samples of shales for geotechnical testing as it exhibits varying degrees of fissility and disintegrates into thin layers. As a result of the ongoing development of areas that contain shales which were previously considered as being unsuitable and due to the problems that are associated with shales, it is necessary to develop a modified and simple indirect method of predicting the shear strength properties of shales. This method of rating and predicting the shear strength properties of shales should use simple index tests to avoid the time spent and cost implications of performing laborious tests such as the large scale shear box test.

2 Field Work and Laboratory Testing

2.1 Sample Collection and Laboratory Tests
Four areas of shale outcrops were sampled within KwaZulu-Natal; three of the four localities are found within the Pietermaritzburg metropolitan area whilst the fourth locality is in the Cornubia Housing and Industrial Development area situated in Verulam, KwaZulu-Natal. The sites were chosen for sampling because of the very good exposures and accessibility of the shales of the Pietermaritzburg Formation and; these sites were not affected by major structural discontinuities such as faults or intrusions. These shale exposures allowed for the sampling of fresh and weathered shale material as well as the sampling of the residual material.

X-ray diffraction (XRD) tests were performed on the shale material from each locality in order to determine the chemical and the mineralogical composition of the shale samples. A series of geotechnical laboratory tests were performed at the University of KwaZulu-Natal Engineering Laboratory to determine the index properties of the residual soil material and the physical properties of the shale rock samples from the four localities. Large shear box tests were performed at the University of Witwatersrand on the shale rock material during the winter vacation in 2014. The shear apparatus used is a modified and larger version of the conventional shear apparatus and the basic testing procedure was done in accordance with Head (1984) and the ISRM (2000).

2.2 Settlement Analysis
Shales, siltstones and dolerite, were used as fill material to construct platforms for the housing and industrial development at the Cornubia housing and industrial development. Two areas within the Cornubia housing and industrial development comprising three sites (Gralio site, Fountains Site and the Vumani Site) were selected for the monitoring of settlement. At the Fountains site, shale was the major constituent fill material. Shales, siltstones and dolerite were used to construct platforms at the housing sector (Vumani and Gralio site) of the Cornubia development. Within these areas, individual platforms were chosen for settlement monitoring since they represent areas of deepest fill (between 5-8 m) and it is expected that the greatest amount of settlement would occur within these areas as these areas contain a large volume of shale material. The settlement monitoring was performed using the precise leveling technique as described by Schofield and Breach (2007).

3 Laboratory Results and Discussion

3.1 Geochemical Results
The results obtained from the XRD analyses using the Richveld analysis tool in Highscore Plus revealed that the common minerals are quartz and muscovite mica. Quartz was identified as being the most abundant non-clay mineral which was present in the fresh and weathered shale materials from each locality. The minor minerals present are nacrite, siderite and dickite (see Naidoo, 2015 for a detailed discussion of the analyses).
3.2 Geotechnical Results

The results from the slake durability tests revealed that the most significant trend in these slaking cycles is seen by the weathered shale material from the Cornubia development as the shale material continuously degrades at a higher rate as compared to the shales from the other localities. This may be due to the lower percentage of quartz and a higher percentage of illite as evident from the XRD results. Based on the classification system by Hopkins and Beckham (1998) and Broch and Franklin (1972), the shales from each locality could be regarded as having high to extremely high slake durability indices and as hard shales respectively.

The Point Load Strength test results showed that the Point Load Strength is much higher for the darker, fresher shale material as compared to the weathered shale materials. The axial strength of the shale samples tested from each locality are higher than the diametrical strength. When failure is initiated parallel to the bedding planes (anisotropy) of sedimentary rocks, the resulting diametrical strength is lower than the axial strength (Broch, 1983; ISRM, 2000). In addition, the observed diametrical strength is lower than the axial strength since the force applied by the individual platen are parallel to the lamination planes and to the degree of fissility which reduces the force which is required to break shale (Bell and Maud, 1996; Bell et al., 1997). Based on the Point Load Strength index after Brock and Franklin (1972), the shale materials have a high Point Load Strength.

The shear strength properties of Underwood’s (1967) classification system were used to describe the shale samples from each locality since the information regarding shales of the Pietermaritzburg Formation is very limited. Using this classification, all samples tested are characterised as being favourable according to their φ’. However, the shale samples from Localities 1 to 4 are characterised as being unfavourable according to their c’ despite that these samples have a high frictional component.

3.3 Settlement Analyses

Using the precise levelling technique to obtain the monthly observed settlement from each site, a prediction was made to determine the total expected settlement which equates to 1% for each platform. Each prediction was calculated using the average settlement at a particular site (i.e. Fountains, Gralio, and Vumani) to determine the time taken for the fill to settle approximately 1% of its fill height. The rate of settlement from each site which ranges from 1-2%, is higher than the usual expected amount of settlement as stated by NAVFAC (1982). A straight line assessment (using a projected/estimated amount of settlement) has not accounted for the initial rapid settlement on completion of the platforms since the monitoring of settlement commenced approximately 2 months after. In addition, the settlement analyses presented only represents 80-90 % of the settlement which excludes future long-term settlement or differential settlement beneath the building footprints which will be exerted by the structure to be built on these platforms.

4 Development of a Modified Rating System

4.1 Introduction

Several researchers (e.g. Strohm et al., 1978; Franklin, 1981; Santi and Rice, 1991; Hopkins and Beckham, 1998; Hajdarwish and Shakoor, 2006) have proposed procedures for the design and construction of embankments by performing laboratory tests on highly degradable materials (Walkinshaw and Santi, 1996). These procedures were initially created and designed for shales especially when shales and mudrocks were used in the construction of embankments and pavements (Walkinshaw and Santi, 1996). Thus, the use of shales for the stability design of an engineered fill embankment and for road layerworks requires knowledge of the anticipated long term shear strength properties of the fill.
These shale rating and classification systems focused on the geotechnical properties of shales and the overlying residual soil material. Hajdarwish and Shakoor (2006) showed in their laboratory investigations, that it is not possible to use a single geological property such as lamination or engineering property such as the void ratio, specific gravity and dry density to predict the shear strength of shales. For example, shales which have a high slake durability index does not necessary imply that it has a high shear strength. They suggested that a variety of engineering properties such as the slake durability index (SDI), Point Load Strength and Atterberg limits should be used to predict the shear strength parameters of these materials.

Franklin (1981) developed a shale rating chart to rate shales by assigning shale ratings from 1 – 9 which are based on geotechnical tests such as Atterberg limit, the Point Load Strength test and the slake durability test for the construction of embankments. In Franklin’s (1981) shale rating chart, the Point Load Strength test and slake durability test were used to obtain a shale rating if the shale sample has an SDI that is greater than 80 % whilst the Atterberg limit and the slake durability test were used if the shale sample has an SDI that is less than 80 %. In this study, Franklin’s (1981) original shale rating chart (using shale ratings from 1-9) was modified to incorporate the geochemical analyses and the geotechnical properties. Additionally, Franklin’s (1981) shale rating chart was modified to incorporate the jar slake test and to use three shale ratings (i.e. R, 1-3) instead of using nine shale ratings. The modifications will enable for the shear strength parameters (c’ & φ’) of shales to be predicted within reasonable limits and to provide an indication of settlement and clay content when shales are used as fills.

4.2 The Development of a Simple Shale Rating System to be used by Geotechnical Engineers.

Three shale ratings were used to substitute for the nine shale ratings as proposed by Franklin (1981) (as shown by Figure 1) and includes the jar slake test. The shale ratings from 1-3 were based on the laboratory results that are presented in Section 3 and from the results from various researchers (Naidoo, 2015). The shale rating of 1 was given to highly weathered shale materials which would represent the residual clays and gravel material which have formed from the progressive weathering of the shale rocklike material. On the other hand, the shale rating of 3 is given to shale materials and would represent the in-situ fresh shale materials. The shale rating of 2 refers to shales that often behave between R1 and R3 which is in response to their reaction to the physical environment and exhibit a variation in their geotechnical properties. This means that depending on their exposure to the environment, the prevailing environmental conditions such as high rainfall and the amount of vegetation present, shales will be subjected to varying degrees of weathering and by an expected change in their geotechnical properties. The broader shale rating of 2 also caters for shales which have high slake durability indices but are difficult to predict their shear strength parameters when used in a fill embankment (see Table 1). The shale rating of 2 is a suitable representation when shales are used in the construction environment as shale materials that are used comprise a mixture of soil and rocklike (granular) material thus having a variation in their engineering properties.

After a geotechnical engineer has conducted the various index tests (such as the Atterberg limits, slake durability test, jar slake test) on shales to obtain the index values, the geotechnical engineer should then compare their results with the values in Table 1 to determine the rating (e.g. R1/ R2/ R3). For example, shales which display an SDI of 80 %, an IJ of 4 and a Point Load Strength of 2 MPa will be awarded a shale rating of 2. Thereafter, Table 2 provides an estimation of the effective shear strength parameters of a shale material which is based on the shale rating (R1/ R2/ R3) from Table 1. The range of c’ and φ’ have been grouped according to the laboratory results and from a literature search by authors such as Underwood (1967); Cripps and Taylor (1981); Wilson (1983); Hopkins (1988); Bell and Maud (1996; 1997); Hopkins and Beckham (1998); Hajdarwish and Shakoor (2006); Drennan, Maud and Partners (2010); Aghamelu et al. (2010) and Toniolo (2012).
Figure 1. Broader groups for rating shales based on the Plasticity Index, the slake durability index, the jar slake test and the Point Load Strength test (modified after Franklin, 1981).

Table 1. Using simple index test results from the literature to characterise the shale material according to shale ratings.

<table>
<thead>
<tr>
<th>Index test</th>
<th>R1</th>
<th>R2</th>
<th>R3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Atterberg limit</td>
<td>Liquid limit</td>
<td>&gt;55</td>
<td>55-34</td>
</tr>
<tr>
<td>Plastic limit</td>
<td>&gt;26</td>
<td>26-16</td>
<td>&lt;16</td>
</tr>
<tr>
<td>Plasticity index</td>
<td>&gt;29</td>
<td>29-18</td>
<td>&lt;18</td>
</tr>
<tr>
<td>Jar slake (Ij)</td>
<td>&lt;3</td>
<td>3-4</td>
<td>5-6</td>
</tr>
<tr>
<td>Slake durability (SDI) (%)</td>
<td>&lt; 60</td>
<td>60-88</td>
<td>&gt; 88</td>
</tr>
<tr>
<td>Point Load Strength (MPa)</td>
<td>&lt; 1</td>
<td>1-2.5</td>
<td>&gt; 2.5</td>
</tr>
</tbody>
</table>

Table 2. Using shale ratings (R) to predict the shear strength of shales and associated settlement.

<table>
<thead>
<tr>
<th>Shale Rating (R)</th>
<th>1</th>
<th>2</th>
<th>3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Description</td>
<td>Highly weathered</td>
<td>Moderately weathered</td>
<td>Fresh to slightly weathered</td>
</tr>
<tr>
<td>$\phi'$</td>
<td>&lt; 19°</td>
<td>19 - 33°</td>
<td>&gt; 33°</td>
</tr>
<tr>
<td>$c'$ (kPa)</td>
<td>0-4</td>
<td>5-19</td>
<td>&gt; 19</td>
</tr>
<tr>
<td>Settlement (%)</td>
<td>&gt; 3 (&lt; 5 years)</td>
<td>1.1-3 (5 years)</td>
<td>&lt; 1.1 (5 years)</td>
</tr>
<tr>
<td>Mineralogy (%)</td>
<td>Quartz content: &lt; 30</td>
<td>Quartz content: 30-60, Clay minerals: &lt; 30, Quartz content: &gt; 40</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Clay minerals: &gt; 40</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

When using shales as fill material, a prediction of the estimated settlement is also provided in Table 2. The estimated range of settlement values that are presented in Table 2 have been determined from the monitoring of settlement of the shale platforms from the Cornubia development and from the available literature (e.g. NAVFAC, 1982; Hopkins and Beckham,
1998). All values presented below are reasonable estimates particularly when using shales of the Pietermaritzburg Formation.

Hopkins and Beckham (1998) stated that a knowledge of the effective stress parameters (c' & φ') is essential to determine the stability of slopes during the design of slopes for highway shale embankments. In addition, the long term durability of shales is one of the basic requirements for the design of shale embankments (Hopkins and Beckham, 1998). For example, shales that are classified as hard and durable (SDI > 90; I_j = 6) can be used as rockfill (Hopkins and Beckham, 1998). However, intermediate shales which are classified as hard and non-durable (50 < SDI < 90; I_j = 3-5) are difficult to compact and require heavy machinery (Hopkins and Beckham, 1998). Thus, a variation of shear strength parameters are used during slope stability studies to determine the stability of a slope. Using the shear strength parameters of R2 shale rating as an example, a best case scenario (c' = 19 kPa & φ' = 33°) should be used in conjunction with the worst case scenario (c' = 5 kPa & φ' = 19°) to perform the slope stability analyses. Furthermore, the average shear strength values should be used (c' = 12 kPa & φ' = 26) as this would account for the variability in material properties.

The largest range of cohesion values for the shale rating of 2 as compared to the shale rating of 1 or 3 were chosen because the shale rating of 2 is representative of the behaviour of shales which are used in the field environment. Engineers rarely encounter very fresh and competent shale material and if fresh shale samples are used, these materials is expected to weather easily when exposed to the physical environment hence the narrow range in the shear strength parameters of R3. Also, they will not use highly weathered shale materials as shales sometimes weather to a fine grained mass (R1). An estimation on the clay content and quartz content is provided in Table 2. It is expected that highly weathered shales would have a high percentage of clay minerals and a low percentage of quartz whilst fresh rocklike shales is expected to have a high quartz content and low clay content.

4.3 Testing the Validity of the Modified Rating System

The results obtained from the index tests which were performed on the shale samples from the different localities in this study were used to test the validity of the modified rating system. This was done in order to validate whether the shear strength parameters obtained from the laboratory testing are within the range of values of the predicted shear strength parameters based on the rating system. Firstly, using Figure 2, the index properties were used to determine the rating of the shale samples from the different localities. For example, the SDI was used in conjunction with the I_j values and Point Load Strength results of the shale materials from Localities 1 and 3 whilst the SDI and the I_j values were used from Localities 2 and 4 to determine the shale rating R using Figure 2. The Atterberg limit results from Localities 1 and 3 were not used as the SDI of the shale samples are greater than 80% as suggested by Franklin (1981). Thereafter, the predicted shear strength values of a particular shale rating that is presented in Table 2 was compared to the shear box results (See Naidoo, 2015).

Using Figure 2 and Table 1, the results show that the shale samples tested from the different localities are categorised as having a rating (R) of 3. The predicted values of c' and φ' using the modified shale rating chart were found to be similar or within the range of values that were obtained from the shear box tests with the exception of the c' value of the fresh shale samples from Locality 4, and the φ' value of the weathered material from Locality 1. Thus, there is a slight difference between the predicted shear strength results using the modified shale rating system to the shear box results. For example, there is a slight difference in φ' between the predicted value of > 33° using the rating system to the shear box test result of 32.2° of the weathered shale sample from the Cornubia development. However, there is a large difference in the c' value between the predicted value using the modified rating system and the shear box value of the fresh shale sample from Locality 4.
Since index tests were not done on the mixture of the shale samples from Localities 1 and 3, the results obtained using Table 2 showed that the predicted shear strength parameters are within the values that were obtained from the shear box tests for a shale rating of 3. It is expected that the mixture of the fresh and weathered shale samples from Localities 1 and 3 should represent the shale materials that are used in the construction environment and should have a shale rating of two. However, the results revealed that the “mixture” of the shale samples fall within a rating of 3 and may be due to the mixture of the shale sample containing more fresh shale materials than weathered shale materials. Thus, it is important to consider the current state of weathering of a shale sample as Cripps and Taylor (1981) stated that the greatest variation found in the engineering properties of mudrocks can be attributed to the effects of weathering. Although Hajdarwish and Shakoor (2006) revealed that there are strong correlations between the friction angle and mineralogy and between the cohesion and SDI of mudrocks respectively, this relationship was not observed by the shale samples in this study. These observations further emphasised that a single lithological characteristic or engineering property cannot be used as an independent value to predict the shear strength property of shales or mudrocks and that there are a variety of factors to be considered.

In addition to rating the shale materials from each of the four localities, the geotechnical properties of the Kentucky shales from the Kentucky area in the U.S.A were subjected to the modified shale rating system. The index test results which include the Atterberg limits, the I values and SDI values were used to obtain a shale rating (R) for the Kentucky shales. Using the modified shale rating system as shown in Figure 3, a shale rating (R) of 2 and 3 were obtained for the Nancy and Drakes shales and, Hance shales from the Kentucky area respectively from the index test results. Thereafter, the estimated range of φ for a shale rating of 2 which are presented in Table 2 were similar to the φ’ values which were determined by Hopkins and Beckham (1998). However, the maximum values of c’ which were determined by Hopkins and Beckham (1998) were higher than the predicted values of c’ for a shale rating of 2 for the Nancy shales and Drakes shales which are shown in Table 2. Thus, the shale rating chart was successful in predicting the φ’ values for the Nancy and Drakes shale samples using the results from the index tests but the values for c’ were higher than the range of values that are presented in Table 2. Although the range of c’ for a rating of 2 as suggested by Table 2 was not sufficient for c’ for the Nancy and Drakes shales, a rating of 2 should be used to rate these shales, as it will cater for the variability in the material properties of the shales in general.

The results from the monitoring of settlement over 3 years which was undertaken at two shale fill test stations in the Kentucky area are presented in Table 3. Table 2 further provides an
estimated percentage of settlement for a 5 year period of a particular shale rating when used for the construction of shale embankments. For example, the test fill station 498+50 in the Kentucky area settled approximately 0.9 % of its fill height over a 3 year period as shown in Table 3. Thus, if the monitoring peg was not destroyed at the test fill station in the Kentucky area and the settlement monitoring continued for a five year period, the amount of settlement could have increased therefore falling into the shale rating of 2 as proposed in Table 2. Furthermore, there is a small difference between the predicted amount of settlement for a shale rating of 2 (1-1.3 %) as shown in Table 2 to the measured amount of settlement from the test fill stations (0.5-0.9 %) as shown in Table 3.

![Testing the proposed field rating system for engineers on shales and mudrocks.](image)

**Table 3.** The geotechnical properties that were used to obtain a shale rating for the Kentucky shales (modified after Hopkins and Beckham (1998)).

<table>
<thead>
<tr>
<th>Plasticity index</th>
<th>Jar slake index</th>
<th>SDI</th>
<th>Measured shear strength parameters</th>
<th>Shale rating</th>
<th>Predicted shear strength parameters</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>c′</td>
<td>φ′</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Unweathered Kentucky shales</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Drakes</td>
<td>7</td>
<td>4</td>
<td>65</td>
<td>9.62-53.3</td>
<td>26.8-32.6°</td>
</tr>
<tr>
<td>Hance</td>
<td>5</td>
<td>5</td>
<td>93</td>
<td>23.9-87.8</td>
<td>25.9-31.7°</td>
</tr>
<tr>
<td>Nancy</td>
<td>9</td>
<td>3</td>
<td>62</td>
<td>10-46.2</td>
<td>25.5-29.5°</td>
</tr>
</tbody>
</table>

**Settlement (using Kentucky shales)**

<table>
<thead>
<tr>
<th>Test station no.</th>
<th>Fill height (mm)</th>
<th>Measured Settlement (3 years)</th>
<th>%</th>
<th>Settlement prediction from Table 9.5 for R2</th>
</tr>
</thead>
<tbody>
<tr>
<td>498+50</td>
<td>19180</td>
<td>120-170</td>
<td>0.9</td>
<td>1.1-3 (5 years)</td>
</tr>
<tr>
<td>317+50</td>
<td>15240</td>
<td>70-80</td>
<td>0.5</td>
<td>1.1-3 (5 years)</td>
</tr>
</tbody>
</table>

**4.4 Limitations of the Proposed Shale Rating System**

The modified shale rating chart presented in Figure 3 showed that occasionally it is not possible to get a single point of intersection of three geotechnical tests. For example, a shale rating of 2 was given to the Drakes shale sample from Kentucky as presented in Figure 3 despite that the 3 geotechnical tests which were used did not intersect at a single point as a result of three tests.
being used. To avoid the problem of obtaining two shale ratings for a particular sample, the Atterberg limits test should only be used if the slake durability index of a shale material is less than 80% as suggested by Franklin (1981) whilst the Point Load Strength test should be used to rate shales if the slake durability index is greater than 80%.

5 Conclusion

The objective of this study was to undertake a multidisciplinary method of indirectly determining the shear strength properties of shales. During the course of this research, laboratory tests were undertaken to determine the geochemical and geotechnical properties of shales of the Pietermaritzburg Formation. The outcome of these tests has led to the development of a modified method of predicting the shear strength of shales using shales of the Pietermaritzburg Formation as a case study which involves using simple index test and associated settlement. Furthermore, the validity of the proposed rating system was verified with values obtained previously from the literature.

References


Toniolo, F. 2012. A geological and geotechnical investigation of the Pietermaritzburg Formation shales in KwaZulu-Natal, BSc (Hons.) Unpublished honours project, University of KwaZulu-Natal.


Wilson, C. 1983. *Symposium on Geotechnical Engineering in areas in Natal underlain by Ecca Shale*. Durban Branch in conjunction with the Geotechnical Division of the S.A. Institution of Civil Engineers.
Guidelines for Problematic Soils in Southern Africa – a Laboratory Perspective

T. Grobler\textsuperscript{1}

\textsuperscript{1}Soillab, Pretoria, Gauteng, groblert@soillab.co.za

Abstract

The document Identification of Problematic Soils in Southern Africa was published by the Department of Public Works in 2007. The document provides an overview of test methods to quantify soil strength and to assess failure mechanisms. Although the document provides some guidelines on specific test methods, it does not provide comprehensive detail of the testing requirements and testing procedures.

A combination of in-situ investigations and laboratory testing remains paramount for geotechnical investigations. A lack of either of these aspects in an investigation can necessitate the engineer or geologist to revert to historical data or empirical correlations, or both. Empirical correlations unquestionably have a role to play in geotechnical engineering, due to time and budget constraints. Being in the information age, several published papers on empirical correlations have become available to geotechnical engineers around the world. Although useful information can often be gained from these sources, caution needs to be exercised.

Keywords: Problematic Soils, Geotechnical Laboratory Testing, Sampling, Empirical Correlations

1 Introduction

Karl Terzaghi who is regarded as the founder of modern day soil mechanics, who also conducted the first geotechnical laboratory testing in Constantinople (now Istanbul) in 1924, wrote:

\begin{quote}
I came to the United States and hoped to discover the philosopher’s stone by accumulating and coordinating geological information... It took me two years of strenuous work to discover that geological information must be supplemented by numerical data which can only be obtained by physical tests carried out in a laboratory. (Terzaghi, 1936).
\end{quote}

According to Day (2013), one of the hallmarks of the South African geotechnical industry is the spirit of cooperation and sharing of knowledge between geotechnical engineers. According to him this is caused by two factors. Firstly the great pioneers of geotechnical engineering in
South Africa, such as Jennings & Brink, left a legacy of a willingness to share information and to advance knowledge throughout the industry. Secondly, it may be due to the scarcity of geotechnical engineering skills in the country fostering a spirit of cooperation. The reasons mentioned in combination with the good collaboration between academics, consultants and contractors in the geotechnical industry have led to close collaboration between the affected parties.

The recognition of the role of geotechnical laboratory testing and the culture of close collaboration between affected parties in the geotechnical industry enable the geotechnical laboratory to provide its input and perspective of the geotechnical conditions. This also provides the opportunity to provide guidance on matters such as sample requirements and best practices for sampling techniques.

The scope of this paper only includes specialist geotechnical laboratory testing. It does not include routine indicator and Atterberg limit testing.

2 Confidence in Laboratory Testing

In the paper “Are we getting what we pay for from geotechnical laboratories?” (Jacobsz and Day 2008) and the thesis “A Contribution to the Advancement of Geotechnical Engineering in South Africa” (Day 2013), particular concern was expressed about the quality and credibility of geotechnical laboratory results. Particular focus was placed on Foundation Indicator testing (particle size distribution, including hydrometer and Atterberg limits); shear strength determination was also mentioned in Day’s thesis, referring to effective stress triaxial tests.

Day (Day 2013) reported that blind samples of the same material were distributed to four laboratories for foundation indicator testing. Results were poor and scattered. Grading envelopes didn’t compare well and major variations in Atterberg limit results were observed. The large variations in results were attributed to poor laboratory performance. Equipment-, procedural- and personnel problems were identified as the major contributors to poor results. The final sentence of the article “Are we getting what we pay for from geotechnical laboratories?” stated “What is needed is a commitment from the commercial laboratories to respond to the challenge” (Day et al 2008).

Since this challenge was posed, laboratories have responded and invested in new equipment and invested in personnel by appointing persons with engineering degrees and post-graduate engineering degrees in key positions.

It is mentioned in Day’s thesis that “The situation regarding strength testing of soils is even worse (than routine indicator and Atterberg limit testing)”. Day mentioned that at that time samples for a multinational project were sent to various commercial and academic laboratories. However, owing to poor and unreliable results from laboratories, all results excluding the results from one laboratory were discarded. All further testing for this specific project was carried out by laboratories overseas.

For a significant period specialists in the geotechnical industry only relied on academic laboratories for specialist geotechnical testing.

South African specialist commercial geotechnical laboratories have since invested in new state of the art equipment. Details on the implementation of the new generation of geotechnical equipment are provided by Grobler (2014) in a paper titled “Advances in and automation of commercial geotechnical laboratory testing and testing equipment”.
The investment in equipment and personnel was mentioned by Heymann (2016) and the role of the commercial laboratories has subsequently increased. There are currently several individuals and companies in the laboratory fraternity driving proficiency programs and the outcomes have been positive.


The introduction of the document titled Identification of Problematic Soils in Southern Africa states “This document serves as a manual on aspects of problem soils in South Africa, which may influence the design and economic appraisal of civil engineering projects performed by consultants on behalf of the Department of Public works” (Department of Public Works 2007). The document focuses on the consultant’s view. This article aims to provide the laboratory’s input relating to problem soils.

3.1 Sampling

A test result can only be as good as the sample that is received by the laboratory. Although this is a bit of a cliché in the laboratory industry, the importance of suitable sampling techniques and quantities of samples are imperative. Poor sampling techniques and providing the wrong type of sample often cause problems or delays with regard to testing.

The laboratory is rarely involved with sampling itself but it is recommended that consultants or contractors (as applicable) consult with laboratory managers or laboratory specialists on the size and type of sample required before sampling is conducted. Advice on sampling techniques in difficult soil types or soil conditions can also be discussed with laboratory specialists. After sampling of undisturbed samples has been conducted, it is important that the sample’s integrity and in-situ moisture content be preserved. For block samples it is recommended that block samples be handled as described by Heymann (1999).

3.2 Types of Testing Depending Potential Problematic Soil

Although it is the consultant’s or contractor’s responsibility to ensure that the appropriate tests be selected, it is advisable for younger geotechnical engineers or engineering geologists to consult with an experienced person or a laboratory specialist on the type of testing. Where further details are applicable such as confining stresses and normal loads need to be considered it is recommended that an experienced geo-specialist be consulted.

3.3 Soils with a Potentially Collapsible Grain Structure

Several transported and residual soil types may exhibit a potentially collapsible grain structure. The document Identification of Problematic Soils in Southern Africa describes collapsible soils as:

soils which can withstand relatively large imposed stresses with small settlements at a low in situ moisture content, but will exhibit a decrease in volume and associated additional settlement with no increase in the applied stress if wetting up occurs. The change in volume is associated with a change in soil structure. (Department of Public Works 2007).

Two possible geotechnical tests can be conducted to confirm or quantify the collapse potential of soil.

The collapse potential test is the quicker, cheaper and cruder test to confirm that a soil has a collapsible grain structure. A soil specimen, cut into an oedometer ring, is loaded at its natural moisture content at similar load increments to an oedometer test to 200kPa. At 200kPa the sample is inundated with water and the change in void ratio at 200kPa is used to calculate the
collapse potential of the soil. Figure 1 illustrates and Equation 1 indicates how the collapse potential is calculated.

\[ CP(\%) = \frac{\Delta e_c}{1 + e_0} \times 100 \]  

Figure 1. Main elements of a collapse potential graph

<table>
<thead>
<tr>
<th>CP</th>
<th>Severity of Problem</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 - 1%</td>
<td>No problem</td>
</tr>
<tr>
<td>1% - 5%</td>
<td>Moderate trouble</td>
</tr>
<tr>
<td>5% - 10%</td>
<td>Trouble</td>
</tr>
<tr>
<td>10% - 20%</td>
<td>Severe trouble</td>
</tr>
<tr>
<td>&gt; 20%</td>
<td>Very severe trouble</td>
</tr>
</tbody>
</table>

With the collapse potential result it is not possible to calculate settlement other than the settlement at 200kPa.

The double oedometer test is used to quantify collapse settlement. Two undisturbed samples are cut into oedometer rings. On the one specimen an oedometer test is conducted with the material at its natural moisture content. An oedometer test is performed on the second specimen with the specimen in a saturated condition. Changes in void ratios can then be used to calculate
normal settlement (settlement of the soil at its natural moisture content) and collapse settlement (settlement of the soil in a saturated condition).

Sample preservation for both these types of testing is critical, since the material cannot be remoulded to conduct these types of testing. Special care must be taken with samples and delivery at an inherent moisture content is critical. If the in-situ material in its current state is above its optimum moisture content, it may be necessary to let the soil dry slightly, to a moisture content just below its optimum moisture content. If this procedure is not followed, collapse settlement of the sample may take place prior to the sample being inundated with water and results may indicate normal settlement and not give an accurate results with regards to collapse settlement. Care must be taken in this drying process and samples must not be dried in an oven, as this might induce high suction pressures on the soil and might change the soil structure.

3.4 Expansive Soils

The document Identification if Problematic Soils in Southern Africa defines expansive soils as: “Soils in which variations in moisture content result in volumetric change, i.e. swell or shrinkage of the soil skeleton are defined as expansive soils” (Department of Public Works 2007). Expansive soils are noted to be the most commonly occurring problematic soils in Southern Africa. Expansive soils can either be residual or transported soils and the degree of potential expansiveness is generally dictated by the clay mineral which constitutes the soil.

Although indicator testing and Atterberg limit determination can provide an indication of the potential expansiveness, swell tests performed in an oedometer ring can quantify swell parameters, such as the % swell and the swell pressure. The way in which the % swell is calculated and the swell pressure is determined are indicated graphically in Figure 2 below. This test is often referred to as a free swell test.

![Typical % swell and swell pressure](image)

Figure 2. Typical % swell and swell pressure

Similar to the collapse potential and double oedometer tests, the initial moisture content is critical. If the initial moisture content is too high the sample has probably already expanded and when the sample is saturated, additional swell will be minimal. It is important to allow the sample to air dry to a moisture content well below the optimum moisture content. The sample should be dried out before the test specimen is cut into the oedometer ring. If the sample is cut out and then allowed to dry, the specimen shrinks, leaving voids in the testing specimen. However caution must be exercised to not dry out the sample too much before testing, as this would cause significant excessive suction pressures in the soil, which could cause problems
with sample preparation. Thus the sample preparation process, even before the sample is cut into the ring, involves monitoring and conditioning.
An example of a clay sample that shrunk in an oedometer cell is illustrated in Figure 3.

![Shrunken clay sample in an oedometer ring](image)

Figure 3. Shrunken clay sample in an oedometer ring

### 3.5 Other Problem Soils

Other problem soils are also encountered in Southern Africa, but will only be mentioned briefly in this document.

Highly compressible soils are often encountered, but the level of compressibility can fairly easily be quantified with simple oedometer testing.

Dispersive soils can be classified by using the suite of four tests to determine dispersivity. The suite of four tests comprise chemical testing (exchangeable sodium and cation exchange capacity), double hydrometer testing, pinhole testing and the crumb test.

Pedogenic materials, although not classified as problem soils, are extremely difficult to test in a geotechnical laboratory. It is often simply not possible to cut undisturbed samples due to the degree of cementation or large and regular nodules in the sample (or both). Remoulding of pedogenic material is not recommended as all cementation is broken during the remoulding process.

### 4 The Use of Empirical Correlations in Geotechnics for Shear Strength Parameter Determination

Although the use of empirical correlations does not form part of the scope of the problem soils document, the determination of shear strength parameters is often required for various types of geotechnical engineering projects.

According to Ameratunga, Sivakugan & Das (2016, pg i):

> Empirical correlations play a key role in geotechnical engineering designs and analysis. Laboratory and in situ testing of soils can add significant cost to a civil engineering project. By using appropriate empirical correlations, it is possible to derive many design parameters, thus limiting our reliance on these soil tests.

In geotechnical engineering empirical correlations are typically used to determine the shear strength parameters of soils. This is probably due to the fact that laboratory testing to determine shear strength parameters are time-consuming and expensive.

Although empirical correlations may be useful in certain cases, caution must be exercised and empirical correlations should not be misused. With the dawn of the information age and the
availability of information on the internet not all empirical correlations are relevant for different soil types.

Figure 4 indicates the results of a study conducted on a variety of materials, where the secant friction angle is plotted against the liquid limit.

Trend lines for various CF (clay fraction) values are plotted on the graph. Although definite trends and correlations between the liquid limit and the friction angle are evident, it will not be suitable to simply read off a friction angle value from the graph. Ranges are fairly broad and various outliers are observed.

![Graph showing secant friction angle vs. liquid limit](image)

Figure 4. Friction angle as a function of liquid limit. (Stark & Hussain 2012)

5 Case Study

A case study is used to illustrate the importance of conducting thorough laboratory testing. An evaluation, analyses and comparison of two similar sandy silt samples from two different terraces, where slope failure occurred, are used. On visual inspection the two samples were similar and no discernable differences were observed. The samples are named Terrace A and Terrace B. The primary objective of the laboratory testing was to determine effective stress parameters, from which slope remedial measures could be designed. The material was a residual tillite.

The testing comprised sieve analysis and Atterberg limit determination, consolidated undrained triaxial testing and XRD (X-ray diffraction) testing. Electron microscope photos of specimens from both terraces were taken. Testing was conducted in a SANAS accredited laboratory and was performed in accordance with the relevant TMH and BS test methods.

5.1 Sieve Analyses, Hydrometer and Atterberg Limit Determination

The indicator testing on the two samples revealed that the samples were very similar. Figure 5 indicates the sieve analysis and hydrometer results. Table 2 provides a summary of the indicator results.
5.2 Triaxial Testing

Consolidated undrained triaxial testing was performed on both samples. During the consolidation stage the volumetric strain on both samples was between 2.7% and 4.5%. The shear stage indicated typical behaviour expected from silty material; initial contraction, followed by dilation. During the testing the pore pressures initially increased, but was followed by a steady decrease.

The stress paths of the two samples are indicated in Figure 6 and Figure 7. Shear strength parameters are indicated in Table 3.
Table 3. Shear strength parameters

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Terrace A</th>
<th>Terrace B</th>
</tr>
</thead>
<tbody>
<tr>
<td>Friction angle (°)</td>
<td>31.6</td>
<td>29.3</td>
</tr>
<tr>
<td>Cohesion (kPa)</td>
<td>0.4</td>
<td>4.1</td>
</tr>
</tbody>
</table>

Soil strength parameters generally fall within typical parameters for a silt, as described in literature. When comparing the results to results found in literature the friction angles obtained from testing be slightly high for silt, it might be due to a fairly high sand content.

The stress paths for the two samples had similar shapes and the two samples had similar strength parameters. However, there was a discernible difference between the two samples. In the case of silt strength parameters are generally governed by the mineralogy of the material.

5.3 XRD Testing

XRD testing was performed on both samples. XRD testing is a crystallographical test method used to quantify the specific mineral contents of samples. Table 4 provides a summary of the XRD results.
Table 4. Summary of XRD results

<table>
<thead>
<tr>
<th>Mineral Composition</th>
<th>Terrace A</th>
<th>Terrace B</th>
</tr>
</thead>
<tbody>
<tr>
<td>% Quartz</td>
<td>64.12</td>
<td>55.75</td>
</tr>
<tr>
<td>% Muscovite</td>
<td>24.20</td>
<td>35.64</td>
</tr>
<tr>
<td>% Kaolinite</td>
<td>11.68</td>
<td>8.61</td>
</tr>
</tbody>
</table>

Based on the XRD results it was evident that Terrace A had a much higher quartz content compared to Terrace B. Terrace B has a higher muscovite content.

5.4 Electron Microscope Photos

Electron microscope photos were taken with magnifications of x95 to x9500 to view particle shapes and to view the agglomerations of particles. Individual particles were subsequently analysed for element content.

Figures 8 and 9 present electron microscope photos taken of Terrace A and Figure 10 presents a photo taken of Terrace B.

Figure 8. Agglomeration of particles and particle shapes of the sample taken at Terrace A.

Figure 9. Agglomeration of particles and particle shapes of the sample taken at Terrace A.
From the electron microscope photos it is evident that the material taken from Terrace A has more rounded particles and the agglomeration of particles were common. Particles from Terrace B were more plate like. This confirms the presence of more quartz particles in Terrace A and more muscovite particles in Terrace B. The element analysis indicated that silica and oxygen elements were more common in particles from Terrace A, confirming the higher presence of quartz.

5.5 Case Study Conclusion
Foundation indicator testing and XRD testing indicated the samples from Terrace A and Terrace B were similar. It was, however, evident from electron microscope photos and triaxial results that there were minor differences between the samples taken from the two terraces. Small differences in mineralogy between the samples gave rise to small differences in the shear strength parameters. The strength parameters of silt were highly influenced by the mineralogical composition of the soil. The sample taken from Terrace A had slightly more quartz particles and this had an influence on the strength parameters obtained.

6 Conclusion
The geotechnical laboratory industry has historically been an integral part in the development of the geotechnical industry as a whole. Specialist laboratory testing is required to confirm and quantify problem soils. Sample preparation and correct testing techniques are critical to provide accurate test results. It is a key requirement that laboratories realize the major role they play in providing results which are analysed by engineering geologists and geotechnical engineers. Although the use of empirical correlations has a role to play in geotechnical engineering, specialist tests are still required to allow for safe and economical designs. Doing further advanced testing such as XRD testing and taking electron microscope photos of samples on a more regular basis along with triaxial testing, may allow for a better understanding of soil mechanics.
References


Grobler, T. 2014. Advances in and automation of commercial geotechnical laboratory testing and testing equipment. *Proceedings of the 8th South African Young Geotechnical Engineers Conference*.


A Review of the Geotechnical Properties of Wad from the Oaktree Formation in Centurion, South Africa

D. Bester¹, A. G. A’Bear², P. W. Day³

¹Bear GeoConsultants (Pty) Ltd, Johannesburg, Gauteng, deon@bgconsult.co.za
²Bear GeoConsultants (Pty) Ltd, Johannesburg, Gauteng, tony@bgconsult.co.za
³Jones and Wagener Consulting (Pty) Ltd, Johannesburg, Gauteng, day@jaws.co.za

Abstract

Dolomite residuum in the form of wad is frequently regarded as an impediment to the development of dolomitic areas within South Africa. Currently, an overburden profile containing wad is viewed as a higher inherent hazard with respect to sinkhole and/or subsidence development in comparison to a profile where wad is absent. However, it is widely recognized that relatively few sinkholes and subsidences occur on the Oaktree Formation, despite the abundance of wad.

In this paper, the properties of wad as determined by conventional field and laboratory test procedures is assessed. Undisturbed wad samples from sites underlain by dolomites of the Oaktree formation have been tested by soils laboratories. The geotechnical properties measured in earlier work are presented and compared to new data, with aims to aid understanding the behavior of wad. A case study is presented where the evaluation of the geotechnical properties of wad has resulted in a reduced hazard evaluation.

Keywords: Geotechnical, wad, dolomite, hazard.

1 Introduction

Dolomite residuum in the form of wad is frequently regarded as problematic with respect to the development of dolomitic areas within South Africa. Many consultants and regulators view an overburden profile containing wad as representing a higher inherent hazard than a profile where wad is absent. This view stems from either the difficulty in retrieving undisturbed samples or a habit within industry by which sampling or submission of wad soils for materials testing rarely occurs. The main reason for this most likely relates to wad usually occurring in profiles containing chert gravels, making sampling and conventional geotechnical testing difficult. Significant research conducted in the early 1980’s unearthed some of the geotechnical properties of wad, predominantly from chert poor dolomite formations. This research provided insight into the behaviour of wad dating back 30-35 years. Wad soils can be classified as a clayey silt or silty clay with a low density and a high void ratio. This combination of
geotechnical properties makes it difficult to derive economically viable engineering solutions for smaller projects, such as residential developments on single stands.

This paper presents a case study of the work completed for a residential development in the Monavoni area in Centurion, South Africa. Various geotechnical consultants investigated the area at preliminary and supplementary levels. The classification system during these prior investigations allocated an inherent risk class (IRC) on a scale of 1 to 8 per borehole profile, where IRC 1 presents low risk of small sinkholes and IRC 8 a high risk of very large sinkholes developing. The inherent risk class allocated to the site previously varied between an IRC3b and an IRC6/7, which suggests a high potential for small sinkholes and a high potential for medium to large sinkholes to develop. Current standards have moved towards the zonation of the site in accordance with the hazard of sinkhole development as opposed to risk, also on a similar scale of (1 to 8), known as the Inherent Hazard Class (IHC). To accurately assess the results obtained from the design level investigation, a review of the available information is needed.

2 Review on the Geotechnical Properties of Wad

The available literature on wad predominantly stems from South Africa and dates back to the 1980’s. Literature from other countries, mainly refer to residual limestone soils, which are chemically not too far removed from wad although the comparison of grading results shows significant differences to that experienced in a South African context. This is evident in the manner with which wad is described in the field. The limited, yet relevant literature dealing with wad soils has established a basis for comparison of further work.

The most recent descriptions of wad in literature follow from Day (1981). He terms wad as an insoluble residue, derived from the weathering of manganese rich dolomite. Further stating that the material can be divided into two separate categories based on the structure thereof. “Structured” and “non-structured” varieties of wad and ferroan soil are described, where “structured” wad refers to material with some degree of relict structure, inherent to the parent material. “Non-structured” wad describes material where no relict structure is apparent. Wagener (1982) defines wad as a residue comprising manganese or iron, which is dark brown or purplish in colour and develops in areas where dolomite weathers, typically exhibiting low densities and high void ratios. When discernable the structural categories are useful indicators for the competence of the material (Buttrick, 1986). Dated between 30 to 35 years ago, the available research on wad is possibly limited by the geotechnical laboratory testing of the time. Some insight into how the modern investigation of dolomites came to fruition is first presented in Day and Wagener (1981).

2.1 Day and Wagener, 1981

The paper by Day and Wagener (1981) discusses the investigative techniques used on dolomite terrain in Orkney on the West Rand. Techniques are discussed and compared in order to set out objectives for investigations on dolomite. These objectives are to determine the properties and thickness of the overburden, condition and depth of bedrock as well as the level and seasonal variation in groundwater. The bases of investigative techniques and the information required to assess the hazards related dolomite land remain similar to this day. Techniques identified are divided into “Quantitative” and “Qualitative” methods to differentiate those which give numerical value for properties from methods from which ground conditions may be inferred. Investigations for the development of Rooihuiskraal in Centurion resulted in the publication by Day (1981) on the properties of wad, coincidently also situated on the Oaktree formation.
2.2 Day, 1981
Day reiterates the unique composition of the Oaktree Formation, which contains dolomite with a manganese content in excess of 1%. This concentration of manganese is markedly higher than the overlying formations and only eclipsed by the underlying Black Reef which may contain concentrations as high as 3% (Brink, 1979). The suburb of Rooihuiskraal, similarly to the Monavoni area, is situated immediately north of the contact between the dolomite and the Black Reef quartzite and is extensively intruded by syenite. Thick wad profiles were located by mapping of service trenches. Both “intact wad” and “reworked wad” are identified and considered representative of homogenous and completely reworked materials respectively. The in-situ field testing and laboratory testing of undisturbed samples were carried out at natural moisture content and after saturation (Day, 2013).

The conclusion from Day (1981) indicates highly variable results for wad with dry densities ranging between 225 kg/m³ and 1327 kg/m³ and averaging 670 kg/m³. The grading of the material is generally a clayey silt or silty clay with high liquid limits and moderate PI. Similarly noted by Brink (1979), the reworked wad is found to be highly compressible with an average elastic modulus of 5 MPa and exhibits a reduction in stiffness when saturated and loaded, not too differently to the windblown sands on the Highveld (Day, 2013). The intact wad shows higher strength with an average elastic modulus of 24 MPa. Examination of the texture of wad under an electron microscope revealed a texture described by Wagener (1982) as resembling “rice crispies”.

2.3 Wagener, 1982
Elaboration on the geotechnical investigation techniques by Wagener (1982) in his doctoral thesis, discussed methods such as rotary core drilling, in situ testing and large diameter auger drilling. Significant work on the geotechnical properties of wad are outlined and discussed in the thesis. The conclusion of this research developed a classification for dolomite sites based on depth to rockhead, namely Class A (0-3m), Class B (3-15m) and Class C (>15m). Wagener also urges investigators to gather geological and hydrogeological data from as many methods as possible and to evaluate the data open mindedly.

2.4 Buttrick, 1986
The most detailed and recent review on the properties of wad are presented by Buttrick (1986) in his master’s dissertation. This work covers the geotechnical properties and structural importance of the residual textures of wad soils in detail. Wad and ferroan soils are distinguished and a comparison of the micro and macro structures is presented. The influence of the secondary features and discontinuities imposed on the soils are discussed and found to result in both poor and very positive behavioral characteristics of these soils.

In the summary of the geotechnical data available shown in Table 1 below, wad is seen to be highly variable in most parameters. The most pertinent values are the bulk density, void ratio, compression index, cohesion, friction angle and elastic modulus. Buttrick further notes the importance of detailed descriptions of these soils during investigation and notes additional topics of research in wad.
Table 1. The Geotechnical properties of wad from Buttrick (1986).

<table>
<thead>
<tr>
<th>Geotechnical Test/Parameter</th>
<th>Laminated Wad</th>
<th>Massive Wad</th>
<th>No of samples</th>
</tr>
</thead>
<tbody>
<tr>
<td>Typical Grading</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Geotechnical Test/Parameter</td>
<td>Minimum</td>
<td>Maximum</td>
<td>Minimum</td>
</tr>
<tr>
<td>Typical Grading</td>
<td>Clayey Silt to Sandy Silt</td>
<td>Sandy Clayey Silt</td>
<td></td>
</tr>
<tr>
<td>Atterberg limits</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>LL</td>
<td>28</td>
<td>113</td>
<td>27</td>
</tr>
<tr>
<td>PL</td>
<td>22</td>
<td>154</td>
<td>21</td>
</tr>
<tr>
<td>PI</td>
<td>3</td>
<td>26</td>
<td>11</td>
</tr>
<tr>
<td>Linear Shrinkage</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>LS</td>
<td>1</td>
<td>11</td>
<td>2</td>
</tr>
<tr>
<td>Specific gravity</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>SG</td>
<td>2.2</td>
<td>3.10</td>
<td>1.94</td>
</tr>
<tr>
<td>Density kg/m³</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Bulk</td>
<td>373</td>
<td>1686</td>
<td>664</td>
</tr>
<tr>
<td>Dry</td>
<td>220</td>
<td>1221</td>
<td>406</td>
</tr>
<tr>
<td>Natural moisture content</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Degree of Saturation</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Sr</td>
<td>21</td>
<td>98</td>
<td>22</td>
</tr>
<tr>
<td>1/Mv</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>MPa</td>
<td>0.6</td>
<td>58</td>
<td>1.9</td>
</tr>
<tr>
<td>Compression index</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.1</td>
<td>6.7</td>
<td>0.19</td>
<td>5.1</td>
</tr>
<tr>
<td>Over consolidation ratio</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>OCR</td>
<td>1.1</td>
<td>7.8</td>
<td>0.8</td>
</tr>
<tr>
<td>Void Ratio</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>e₀</td>
<td>1.3</td>
<td>16.6</td>
<td>0.97</td>
</tr>
<tr>
<td>c’ kPa</td>
<td>23</td>
<td>63</td>
<td>4</td>
</tr>
<tr>
<td>Triaxial Tests (Consolidated drained)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Ø’ Degrees</td>
<td>21</td>
<td>25</td>
<td>23</td>
</tr>
<tr>
<td>c kPa</td>
<td>26</td>
<td>74</td>
<td>1.0</td>
</tr>
<tr>
<td>Ø Degrees</td>
<td>15</td>
<td>19.3</td>
<td>21.5</td>
</tr>
<tr>
<td>Shearbox</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Ø Degrees</td>
<td>18.5</td>
<td>53</td>
<td>24</td>
</tr>
<tr>
<td>Shear vane</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>c kPa</td>
<td>38</td>
<td>74</td>
<td>4</td>
</tr>
<tr>
<td>Vertical Plate</td>
<td>E_initial MPa</td>
<td>60</td>
<td>589</td>
</tr>
<tr>
<td>Load Tests</td>
<td>E_final MPa</td>
<td>24</td>
<td>100</td>
</tr>
<tr>
<td>Horizontal Plate</td>
<td>E_initial MPa</td>
<td>0.1</td>
<td>27</td>
</tr>
<tr>
<td>Load Tests</td>
<td>E_final MPa</td>
<td>0.5</td>
<td>7.2</td>
</tr>
<tr>
<td>Permeability</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Very Low</td>
<td>Intermediate</td>
<td>Low</td>
<td>Very Low</td>
</tr>
<tr>
<td>Dispersiveness</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Crumb Test</td>
<td>Class 1</td>
<td>Class 3</td>
<td>Class 1</td>
</tr>
<tr>
<td>SCS Test</td>
<td>Non-Dispersive</td>
<td>Slightly-Dispersive</td>
<td>Non-Dispersive</td>
</tr>
<tr>
<td>ESP Test</td>
<td>Non-Dispersive</td>
<td>Intermediate</td>
<td>-</td>
</tr>
<tr>
<td>Pinhole Test</td>
<td>-</td>
<td>-</td>
<td>Clear</td>
</tr>
</tbody>
</table>

The array of different geotechnical parameters provides a broad overview of the behavior of wad and provides a basis for comparison of data from other sites in the area. The work by (Wagener, 1982) on wad, yielded very similar results to those above and have not been included in Table 1.

The work on wad by Buttrick, eludes to the differences in geotechnical properties between “intact” versus “reworked” wad as well as the “laminated” versus “massive” varieties thereof. Further avenues for investigating wad include detailed petrogrographic and chemical analyses.
correlated with the inherently anisotropic macrostructures in the case of laminated wad. The differences in geotechnical properties of intact, reworked and remolded samples as well as the causes of liquefaction must be investigated in greater detail (Buttrick, 1986).

3 Case Study of Monavoni Area in Centurion

Initial investigations of the case study area incorporated a gravity survey on a 30 m grid and percussion drilling. The analysis of borehole data presented a moderate to high associated risk of sinkhole development. This reportedly due to the presence of thick wad profiles across the site, which is typical of the Oaktree formation in the area. The case study area is shown along with the areas investigated by Buttrick (1986), notably covering more than one of the dolomite formations.

![Map showing the investigated areas from Buttrick (1986) as well as the study area.](image)

Figure 1. Map showing the investigated areas from Buttrick (1986) as well as the study area.

The properties of wad from the study area are essential for a thorough assessment of the competence of the overburden. Three sites from the Monavoni area, namely extensions 3, 11 and 23 form part of initial investigations, aimed at determining the viability of a detailed study of one site in particular. Undisturbed Shelby tube samples of wad were retrieved for testing from each site. Grading, Atterberg limits, dispersity and consolidated drained triaxial tests, including falling head permeability, were carried out on each sample from the three sites.

Penetrometer testing, using a Dynamic Probe Super Heavy (DPSH), revealed a tendency of saturated wad to liquefy with the dynamic of testing in this manner. The percussive nature of the testing spikes the porewater pressure at the tip of the cone at each impact, destroying the soil structure ahead of the cone (Day, 2015). This is noted from a marked reduction in the penetration resistance at depths which notably coincide with the average water table depth.
(Bear Geoconsultants, 2016). In such instances, alternative investigative techniques may be better suited to determining the consistency of the materials on site. For example, profiling auger holes to a depth beyond the noted drop in penetration resistance, at the same location. This may rule out the possibility of a cavity/void.

3.1 Field Investigation
Previous work on Extension 11 concentrated on the stability of the site whereas the current investigations are design-level investigations. The work undertaken during these latter investigations includes:

- Rotary percussion boreholes, the bulk of which were drilled using reverse circulation drilling methods,
- Test pitting,
- Large diameter auger holes,
- Cross-hole plate load tests,
- Dynamic probe super heavy (DPSH) tests, and
- Window sampling with SPT tests.

Additional laboratory testing included further grading and indicator tests, consolidated drained triaxial tests, permeability determinations, oedometer tests, collapse potential tests and soil dispersiveness testing. Field testing included hand penetrometer and vane shear tests carried out in the sidewalls of the large diameter auger holes as well as the retrieval of numerous undisturbed samples from slightly above, near and below the water table.

3.2 Testing Results
Grading results for the wad samples correlate well with available data presented in earlier work. However, it is questionable as to whether Van der Merwe’s method of classifying clays is applicable to the classification of wad. The high liquid limits may result from the adsorption of water onto metal (Fe and Mn) oxide surfaces, which make up much of the substance of wad, rather than the adsorption of water into a clay structure (Bear Geoconsultants, 2016). In addition to the grading of the materials, the compressible nature of the soils can be assessed from the available data, as summarized in Table 3 overleaf.

<table>
<thead>
<tr>
<th>Property</th>
<th>Monavoni Data (13 samples)</th>
<th>Previous Data (68 samples)</th>
<th>Combined (81 samples)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Average</td>
<td>Range</td>
<td>Average</td>
</tr>
<tr>
<td>Clay content</td>
<td>41%</td>
<td>14–60%</td>
<td>19%</td>
</tr>
<tr>
<td>Grading Modulus</td>
<td>0.20</td>
<td>0.09–0.52</td>
<td>0.59</td>
</tr>
<tr>
<td>Plasticity Index</td>
<td>26</td>
<td>10–40</td>
<td>17</td>
</tr>
</tbody>
</table>

Table 3. Comparison of Drained Elastic Moduli (E’ or 1/Mv), (Jones & Wagener, 2016).

<table>
<thead>
<tr>
<th>Test type</th>
<th>Monavoni Data (n=8)</th>
<th>Previous Data (n=12)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Triaxial compression (E’) saturated</td>
<td>Average</td>
<td>Range</td>
</tr>
<tr>
<td>(n=5)</td>
<td>6.5MPa</td>
<td>3–15MPa</td>
</tr>
<tr>
<td>Oedometer Tests (1/Mv) saturated</td>
<td>(n=4)</td>
<td>(n=4)</td>
</tr>
<tr>
<td>(n=4)</td>
<td>14MPa</td>
<td>7–26MPa</td>
</tr>
<tr>
<td>Plate load Tests (E’) at natural moisture content</td>
<td>(n=8)</td>
<td>(n=8)</td>
</tr>
<tr>
<td>(n=8)</td>
<td>27MPa</td>
<td>12–54MPa</td>
</tr>
</tbody>
</table>
The differences in the values obtained from the different test types may be attributed to disturbance of the samples, prior to testing, both during sampling and preparation of the test specimens. Plate load testing gives the compressibility of the wad at natural moisture content in contrast to triaxial and oedometer testing which is done under saturated conditions.

![Figures 2 and 3, showing the distribution of Compression Index (left) and the Elastic Modulus (right) test results.](image)

![Figures 4 and 5, showing the Over Consolidation Ratio (OCR) against Depth (m) (left) and the Collapse Potential (%) versus the Dry Density (kg/m$^3$) (right).](image)

Samples retrieved near surface tend towards over consolidation (OCR>1), which may relate the varying moisture content near surface and seasonal desiccation (Jones & Wagener, 2016). The OCR reduces with depth as degree of saturation increases. Although there is no clear correlation between dry density and collapse potential or compressibility, samples from Monavoni lie within the same ranges as the previous data. All four samples subject to collapse potential tests show negligible collapse near 1%.

The summary of the result, presented in Table 4 below, shows remarkably similar friction angles and cohesion values to earlier data. The strength of the wad soils correlate positively, despite the inadequacies of testing methodology during the 1980’s. The friction angles obtained are similar to those expected of a firm clayey silt or silty clay but the cohesion is higher (Jones & Wagener, 2016).
Table 4. Comparison of strength test results, Day (2016).

<table>
<thead>
<tr>
<th>Property</th>
<th>Monavoni Data</th>
<th>Previous Data</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Average</td>
<td>Range</td>
</tr>
<tr>
<td><strong>Triaxial Tests</strong></td>
<td>(n=7)</td>
<td>(n=7)</td>
</tr>
<tr>
<td>Total stress:</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Friction angle (ø)</td>
<td>18.7°</td>
<td>17.1 – 22.4°</td>
</tr>
<tr>
<td>Cohesion (c)</td>
<td>49kPa</td>
<td>30 – 74kPa</td>
</tr>
<tr>
<td><strong>Effective stress:</strong></td>
<td>(n=7)</td>
<td>(n=7)</td>
</tr>
<tr>
<td>Friction angle (ø’)</td>
<td>22.0°</td>
<td>16.9 - 28.0°</td>
</tr>
<tr>
<td>Cohesion (c’)</td>
<td>39kPa</td>
<td>20 - 61kPa</td>
</tr>
</tbody>
</table>

| **Drained Shear Box Tests** |               |
|                            | (n=5)         | (n=5)         |
| Friction angle (ø’)        | 30.1°         | 24.6 - 34.4°  |
| Cohesion (c’)              | 8kPa          | 0 – 24kPa     |

Comparison of the above data has largely been done for materials from the same geological formation. Incorporating data from different formations in Buttrick (1986) for comparison, yields surprising correlations in the results. Permeability of the wad samples from Monavoni range from $1.5 \times 10^{-6}$ to $1.2 \times 10^{-8}$ m.s$^{-1}$. This is similar to the results obtained by Buttrick ranging from intermediate to very low permeability. Dispersivity results of both the crumb and pinhole methods yielded similar results as well, suggesting that wad is not a highly dispersive soil, but rather ranges from non-dispersive in the crumb test intermediate in the pinhole test. Recent work suggests a combination of many different types of testing be carried out to accurately deduce whether soils are dispersive or not. However, even 30 years ago, researchers had their doubts as to the efficacy of such testing on materials as unique as wad (Maharaj et al., 2015).

4 Discussion

Analyzing the range of data available from the investigation is best carried out in terms addressing the factors commonly assessed during stability investigations, namely:
- Bedrock morphology,
- Caverns and fissures,
- Overburden, and
- Groundwater drawdown.

As previously mentioned, there are extensive syenite intrusions in the area. As a result, the 30m grid gravity survey conducted correlates poorly with the actual occurrences of bedrock. From a stability perspective, the presence of such intrusions reduced the likelihood that any cavities which may exist at depth in the profile will develop into sinkholes. Although previously reported to occur on the Monavoni X11 site, no cavities were encountered in the large diameter auger holes, some of which were drilled immediately adjacent to areas where previous investigations identified potential cavities. It is most likely the areas showing poor sample recovery and air loss in the earlier boreholes, are a reflection of the drilling techniques used rather than an indication of cavities (Bear Geoconsultants, 2016).

The formation of sinkholes, although possible, is regarded as improbable given the properties of wad previously mentioned. Low dispersivity and permeability, negligible collapse potential and the reasonable shear strength and compressibility, suggests a low risk of the materials mobilizing into a receptacle should it be present. No sinkholes have occurred in both undeveloped and developed areas within a 3.5 km radius of the site, roughly 60% of which are...
underlain by dolomite. The potential for sinkhole formation is regarded as limited but subsidence is likely to occur in the event of dewatering. If dewatering takes place, gradual subsidence is likely to occur due to the increase in effective stress any wad which is below the level of the current water table. Maximum settlements in the range of 75 mm are calculated from the elastic modulus at 1% strain, derived from triaxial tests. The thickness of wad in profile ranges from being absent to 8.8 m at various depths below surface. Dewatering considered highly unlikely as no mining related activity is likely close to the site. Constant monitoring of the groundwater level is a mandatory part of the Dolomite Risk Management Strategy (DRMS) for the development.

Taking into consideration all of the above research, the Monavoni X11 site is re-evaluated to present the a low to high hazard of small sinkhole development in the non-dewatering scenario (IHC2/5). The development is to be designed for the less likely dewatering scenario, which anticipates a medium hazard of medium sized sinkholes and a high hazard of small sinkholes developing (IHC3/5). For this classification, current standards require concrete raft foundations, capable of spanning a 5m loss of support (SANS, 1936-3). The peer review by Jones & Wagener (2016) of the work detailed in the report by Bear Geoconsultants (2016) is in broad agreement with the overall re-evaluation of the hazard conditions on the site. This is largely motivated by a 30-year track record of no problems being reported in surrounding areas. After thorough deliberation, regulatory authorities subsequently approved the application for development within this area.

5 Conclusion

A review of the relevant literature on the properties of dolomitic residuum in the form of wad is presented. Not all wad is regarded as problematic with respect to the development of dolomitic areas within South Africa. Wad from the Oaktree formation, being chert poor, allows the wad to be assessed using conventional soil mechanics tests which show the properties of the wad to be similar to those of a firm clayey silt or silty clay, despite its low density. The results of these tests and the good correlation with data gathered over the past 30 – 35 years allows the performance of the wad to be rationally assessed in the same was as for other soils.

A case study from the Monavoni area is presented and discussed. This study aims not only to promote appropriate investigations on dolomite land but in particular appropriate investigations into the properties of wad soils. Consultants should view an overburden profile containing wad as an opportunity for further investigation and incorporate appropriate investigative techniques when assessing the competence of the overburden. Investigations and analyses must be conducted with an open mind and based on sound scientific methodology.

Further work into the properties of wad soils is imperative to better understanding the processes of sinkhole formation and propagation. Assessment of the geochemical properties, soil water characteristics as well as undrained triaxial tests with pore water pressure measurements are likely to provide valuable data for assessing the role of wad in the profile. Additionally, the modelling of ground water flow, at incremented hydraulic gradients, through undisturbed samples may reveal the dominant processes for the erosion of saturated low density materials into a void.
References


Hydrometer Under the Microscope

P. K. Monye¹, P. R. Stott², E. Theron³

¹Central University of Technology, Bloemfontein, Free State, priscillakmonye@gmail.com
²Central University of Technology, Bloemfontein, Free State, stott.philip@gmail.com
³Central University of Technology, Bloemfontein, Free State, etheron@cut.ac.za

Abstract

For many years the hydrometer was the only technique that was internationally standardised for geotechnical particle size distribution analysis of fine soils. Recently Britain has added the pipette method as a preferred alternative. The pipette method is considered to give slightly more accurate and consistent results. Both procedures stand on the theoretical foundation of Stokes’ law. The law assumes constant density of the particles being tested, that larger particles settle faster than small particles when in a liquid suspension, and that there is no interference between particles and other particles or obstructions in the suspension. It also assumes all particles to be spherical. This paper studies the hydrometer theory by analyzing settled sediments and compares the effectiveness of two dispersion procedures, one formerly specified by TMH1, with that specified by SANS 3001 which replaced TMH1.

Keywords: Clay fraction, Dispersing agents, Hydrometer analysis, Methylene Blue

1 Introduction

One of the most problematic aspects of South African soils is the behavior of expansive clay, which continues to contribute to structures and roads infrastructure failure. Hydrometer analysis is used to determine clay fractions. Clay fraction and plasticity index are used to determine soil heave potential using Van der Merwe’s diagram (1964). The most important characteristics of soil behavior depend on clay mineral content rather than particle size as given by the hydrometer. Hydrometer analysis assumes that clay particles range from 2 microns (µm) downwards, and silt to be particles larger than 2µm and smaller than 65 µm. However, clay particles like kaolinite, illite and halloysite can be considerably larger than 2µm, also silt particles size may range down to 1µm.

Hydrometer analysis monitors change in density of a settling suspension. It relies on Stokes’ law which assumes all particles to be spherical; it assumes complete dispersion of clay particles at the time of testing and assumes that fine particles are not carried down by coarse particles. The aim of the investigation is to assess some of the questionable aspects of the hydrometer by isolating and testing sand and silt fractions after settlement. A Microscope is used to examine the samples and compare with what is expected by the hydrometer theory. Methylene blue
(MB) is added since it is known to be effective in labelling clay minerals by swapping places with exchangeable cations.

2 Research Background

Most methods for predicting heave severity rely on clay fraction estimation. Van der Merwe’s diagram is used to predict heave potential by using clay fraction and plasticity index. This method can be flawed if clay fraction determination is unreliable. Some critical aspects of soil behavior depend on clay mineral content rather than particle size. The following possible shortcomings were noted by Savage (2007): (i) Stokes’ law assumes all particles to be spherical and clays are flaky, (ii) De-flocculation of many clays is seldom fully completed at the time of testing, (iii) Clay particles are partially carried down by the larger particles and (iv) A relative density of 2.65 is assumed for all particles. Rodrigues et al. (2011) noted that soil mineralogy should be taken into account and probably different treatments should be considered for different clay types.

The Hydrometer monitors the change in density of settling suspension. Rolfe et al. (1960) pointed out that the analysis averages the specific gravity over the submerged part of the instrument and therefore depends on the shape and depth of hydrometer submergence which might bring errors to the test results.

Nettleship et al. (1997) compared automated centrifuge sedimentation method and hydrometer analysis using particles of kaolinite clay. The results obtained from their investigation indicated that the hydrometer underestimated the fraction of submicron clay particles, which suggested that the difference in result might be due to the time dependent aggregation of fine clay particles.

3 Research Methodology

Soil samples from Heidedal, a suburb of Bloemfontein, Free State were tested and treated as per SANS 3001 GR3 and TMH1(1986), with sodium hexametaphosphate plus sodium carbonate (referred to as Calgon) and sodium silicate plus sodium oxalate solution before examination by microscope. While examining the samples with the microscope methylene blue was added. Results were then compared with the hydrometer theory.

3.1 Methylene Blue Adsorption Test

Methylene blue adsorption (MBA) is a measure of the clay particle surface area, which is a function of clay type and an indicator of water adsorption potential. (Çokça, 2001). MBA is a simple and reliable method to obtain clay mineral properties in soils. There are two methods named “spot” and “turbidimetric” method used in practice. The procedure used in the spot method was used for this investigation it is the most commonly used method in the engineering field. In determining the methylene blue value (MBV) the amount of MB solution adsorbed is used. The Association Française de Normalization (AFNOR) and American Society for Testing and materials (ASTM) standards have adopted the methylene blue stain test. The complete methylene blue index (MBI) method is accessible as ASTM designation C837-09. In this investigation MB is used only to label clay minerals. The MBV and MBI were not determined.

3.2 Microscopic examination

Soil samples were prepared and treated as per SANS 3001 Gr3, 2012 and TMH 1, 1986. After mechanical stirring they were transferred to settle in the settlement containers designed for this study. After a week of settlement, water was syphoned off and the sediments were dried at 45°C. Dry sediments were separated into sand and silt particle layers before microscope examination.
An optical microscope was used with a 9 megapixel camera attached to it. A small sample of each separated layer was suspended in a small amount of distilled water and a drop was extracted to be placed on the microscope slide. Photographs were taken before and after MB addition.

4 Findings and Discussion

4.1 Heidedal Sample TP1LR1

4.1.1 Coarse Sand Layer

A sample from Heidedal Test pit 1 layers was used for this investigation. Methylene blue was added to the sample, dispersion for this soil was not complete but reasonable. Figure 1 shows a grain from the sand layer after dispersion with Calgon. The grain appears to have a thin, deeply blue stained coating of very small particles, almost certainly a smectite, and probably montmorillonite. The lower right side of the large grain appears to consist of clay particles surrounding a silt core and appears to form a bridge attaching to another sand grain. Figure 2 shows a sample of the same soil dispersed with sodium silicate and sodium oxalate solution. The sand grain has deeply blue stained patches, it appears that almost the entire sand grain is covered by a thick layer of very small particles, probably smectite. The dispersion of samples treated by Calgon appears to be more successful than that treated with sodium silicate and sodium oxalate solution.

Figure 1. Coarse Sand layer from Heidedal soil, Test Pit 1 Layer 1 (TP1, LR1). Dispersant sodium hexametaphosphate plus sodium carbonate.
4.1.2 Fine Sand Layer

Figure 3 shows a sand particle. The lower part is covered by a thick layer of clay while the upper part has only a thin coating of small deeply blue stained particles. Clay and silt sized particles are visible in Figure 4 which is from the corresponding layer in the sample dispersed with Sodium Oxalate plus Sodium Silicate. Some of the sand grains and silt-sized particles show deep stain (probably indicating montmorollinite) while others show little methylene blue staining (probably indicating kaolinite). Again the Calgon appears to be more successful in dispersion, but still far from satisfactory.
4.1.3 Coarse Silt Layer
The coarse silt particles in figure 5 appear to have blue stained edges. Most of the smaller silt and clay particles are completely blue stained. Diffused nebular structures are visible and appear to be clouds of extremely small clay particles. Figure 6 is the corresponding layer from the sample dispersed with Sodium Oxalate and Sodium Silicate. The large particles are all completely covered by deeply stained clay particles. Many of the clay particles are extremely small, probably montmorillonite, packed more tightly together than in the sample shown in Figure 5.
4.1.4 Fine Silt Layer
Figure 7 shows a sample from the fine silt layer dispersed with Calgon. Clouds of extremely small clay particles form agglomerations with silt and large clay particles. Figure 8 shows the corresponding layer from the sample treated with sodium silicate/sodium oxalate. There is a significant difference between the sample that was treated with Calgon and that with sodium silicate solution. The fine clay particles appear to stick far more tightly to the large particles and do not form extended dispersed clouds.

Figure 7. Fine Silt layer from Heidedal soil, Test Pit 1 Layer 1 (TP1, LR1). Dispersant Calgon solution.
Figures 9 to 12 show samples from the fine silt layer from a different clay layer from the same test pit. The same pattern can be seen in each case. The Calgon-dispersed samples have tenuous nebulae of very fine dispersed clay particles, while the Sodium Silicate and Sodium Oxalate-dispersed samples show fine clay particles clinging more closely to silt particles.
Figure 10. Fine Silt layer from Hededal soil, Test Pit 1 Layer 2 (TP1, LR2). Dispersant Sodium silicate and sodium oxalate solution.

Figure 11. Fine Silt layer from Hededal soil, Test Pit 1 Layer 3 (TP1, LR3). Dispersant Calgon.
5 Conclusion

Hydrometer theory assumes that the bigger particles settle first. The hypothesis to be examined is that clay and silt particles may be carried down by larger particles earlier than expected in the settlement process. The dispersion of soil samples was significantly different between the mixture of sodium hexametaphosphate plus sodium carbonate and the mixture of sodium silicate plus sodium oxalate solution. The soil samples treated with sodium hexametaphosphate showed slightly better dispersion while that of sodium silicate appeared to be less effective.

The expectation of smaller particles settling with bigger particles as suspected by Savage appears to be confirmed. Sand layers for all treatments show a far smaller tendency to draw down smaller particles compared to silt layers. More clay-sized particles content is evident in the silt layers. Calgon seems to disperse the fine clay particles into clouds which form agglomerations with silt particles, whereas the Sodium Oxalate and Sodium Silicate leaves these fine clay particles more closely attached to larger particles.

The study seems to confirm the view that for high-clay content soil, the hydrometer may be unreliable for determining the clay fraction. This could be the reason why a number of projects have failed due to heaving clay while the geotechnical results predicted little chance of heaving. For soils with high-clay content a more cautious approach appears to be required.

References


Rolfe, B.N., Miller, R.F. and McQueen, I.S. 1960. Dispersion characteristics of montmorillonite, kaolinite, and illite clays in water of varying quality, and their control


Pipe Leakage Detection by Measuring Change in Soil Temperature

D. L. Maphanga

1University of Pretoria, Pretoria, Gauteng, u13152417@tuks.co.za

Abstract

Worldwide, pipes are used to transport different types of liquids and gases including oil. Different types of pipe materials are used, with PVC being the most frequent material used to transport water (Pal, 2008). However, there is a shortage of water supplied to many communities in arid countries around the world. The loss of water from distribution systems is due to different factors, but the most important contributing factor is pipe leakage. This paper describes experiments investigating the detection of pipe leakages, in buried distribution pipes, by measuring the change in temperature of the soil surrounding the pipeline.

Keywords: Soil temperature, Thermistor, Pipe leakage

1 Introduction

Based on work done by Pal (2008) there is a shortage of water worldwide due to water loss, referred to as unaccounted for water (UFW), caused by numerous factors. One of these factors is leakage which is mostly associated with the aging of pipelines, deterioration or extreme pressure forced by rapid valve variation (Golmohamadi, 2015). This is a concern especially in developing countries (Rabe et al., 2012). Small leaks lead to a negative impact on the economy and risk to public health (Golmohamadi, 2015). According to the BBC News (2015), Prof. Joby Boxall of the Sheffield University and his team researched the health risks posed by leaking pipes. Their research results revealed that not only treated water is discharged through pipe-leaks, but dirty water is drawn in. The problem occurs when there is a sudden decrease in water pressure that causes dirty water to be sucked in. This leads to risks of pathogens possibly causing disease.

In 2012, the Water Research Commission (WRC) published a report presenting the results of the data obtained from 132 municipalities that were surveyed out of 237 municipalities in South Africa. The results showed that over 75% of the total volume of Municipal water being supplied throughout South Africa, 36.8% is on average estimated to be the non-revenue water - water that has been treated and is “lost” before it reaches the customer (Rabe et al., 2012). Of the non-revenue water, it is further estimated that 25.4% is lost as a result of physical leaks.
The study was conducted over a 4-year period, from 2007 to 2011. The following values presents the average estimated amounts of water losses that were estimated in percentages in nine different South African provinces:

- Northern Cape 52.0%
- Free State 45.2%
- Mpumalanga 44.7%
- KwaZulu-Natal 43.5%
- Limpopo 36.3% and Gauteng 35.9%
- Eastern Cape 29% and North West 29.7%
- Western Cape 23.9%

A report from the Water Research Commission (2012) stated that of the total volume of water being supplied throughout the globe, 36.6% is estimated to be world’s average non-revenue water. This shows that South Africa’s water losses are in line with the world’s average estimated non-revenue water. However, South Africa has to save water since it is a water-scarce country and the 30th driest country in the world, with an average annual countrywide rainfall of about 40% less than the world’s average rainfall (Rabe et al., 2012). According to Kings (2013) R8 billion was lost in the year 2012 due to leaks. The National Planning Commission said that, “fixing water leaks is one of the cheapest and quickest ways of ensuring a water crunch does not happen” (Kings, 2013).

2 Temperature measurement by thermistor

A thermistor is a specialized resistor, sensitive to temperature, so that its electrical resistance changes with change in temperature. Thermistors are made of semiconductors (Omega Engineering, 2000) and have an accuracy of about ±1°C (Electronics Hub, 2015). Negative Temperature Coefficient (NTC) and Positive Temperature Coefficient (PTC) thermistors are available (Moore, 2003). NTC thermistors are most common. They are commonly used because of their high sensitivity and low cost (AVX, 2006).

The relationship between the temperature and NTC thermistor resistance is given in the equation below (Electronic Hub, 2015).

\[
\frac{R(T)}{R(T_0)} = e^{-\beta \left(\frac{1}{T} - \frac{1}{T_0}\right)}
\]

(1)

Where:

- \( R(T) \) = Resistance at some temperature (in K)
- \( R(T_0) \) = Resistance at an initial measurement (reference) temperature T (in K)
- \( \beta \) = Dissipation or thermistor material constant (usually expressed in mW/°C or the amount of power required to induce a temperature rise of 1°C)

From the Electronics Hub (2015) paper, PTC thermistors operate by increasing in resistance with increase in temperature. Moreover, there is a linear relation between resistance of the PTC thermistor and temperature. The following equation shows the linear relation:

\[
\Delta R = k(\Delta T)
\]

(2)
Where:

\[ \Delta R = \text{Change in resistance (in } \Omega \text{)} \]

\[ \Delta T = \text{Change in temperature (in } K \text{)} \]

\[ k = \text{Temperature coefficient} \]

Figure 1 represents the relationship between resistance and temperature of respectively NTC and PTC thermistors.

3 Pipeline Leak Detection using Thermistors

A simple concept was investigated to detect pipeline leaks using NTC thermistors. NTC thermistors were placed below a model pipeline. When a leak occurs at a specific location on the pipeline, there will be a change in temperature of the surrounding soil around the pipeline due to the leaking liquid from the pipeline. The change in temperature of the surrounding soil depends on the following effects:

The leaking water is generally colder than the soil temperature for buried pipelines and the soil temperature is changed due to leaking water.

Figure 2 represents a numerical simulation that shows the temperature change on the surrounding soil around gas pipe leak.
Figure 2. Leak induced a temperature drop which cools down the surrounding soil for a gas (Omnisens, 2001).

The black circle in the middle of the diagram represents the pipeline surrounded by the compacted soil. The contours represent the cooling of the soil in relation to the distance from the pipeline and its water leakage. From the above illustration, soil remains cooler the closer it lies to the pipe (denoted by the dark grey colour). The further from the pipeline the warmer the surrounding soil is (denoted by the light grey colour). The rest of the contours represents the distribution of the temperature change on the soil.

To verify the shape of the flow of water as a result of proving the simulation in Figure 3, a leaking pipe experiment was conducted. The following figures show the formation of the water flow shape. The soil surrounding the leak pipe is moistened in a half-circular formation, moving further away from the leakage. Therefore, the shape formed is similar to that of a ripple.

![Figure 3. The formation of water flow shape.](image)

From the above figures, it was concluded that it would be ideal to place NTC thermistors below a leaking pipeline in order to optimize the detection of leakage, as the leaking water will flow downwards as a result of gravitational force.

### 4 Soil Temperature Analysis and Leak Tests

The following section presents experimental tests that were performed at the laboratory and a soil temperature analysis that was conducted on site, at the Experimental Farm of the University of Pretoria. NTC thermistors were used to measure the change in temperature of the soil due to change in the controlled ambient room temperature where the experiment was carried out, as
well as to detect a leak whenever there is leakage. The formula below was used to calculate temperature from the thermistors resistance:

\[
T = \left[ \frac{1}{\left(0.00025374 \ln(R) + 0.0011958\right)} \right] - 273 \tag{3}
\]

Where,

T= Temperature read by the thermistor for that resistance (in °C)
R= Resistance of the thermistor (in Ω)

Figure 4, below, represents the curve relating temperature to thermistor resistance.

![Temperature vs Resistance Graph](image)

**Figure 4.** Calibration curve for the approximated formula in relation with actual temperature.

### 4.1 Measuring Temperature Changes in a Natural Soil Profile

The NTC thermistors were used to measure the change in temperature of the soil due to natural changes in atmospheric temperature and solar radiation. The data that was retrieved from NTC thermistors (using a data logger) was used to understand the mechanism of natural soil temperature change under daily and seasonal fluctuation. Furthermore, the data was also used to find a depth below which daily and seasonal temperatures do not drastically affect the soil temperature. This was necessary to isolate soil temperature changes resulting from leaking water only.

An experiment was conducted at the Experimental Farm of the University of Pretoria, where a 3 meter hole was excavated using a light tractor loader backhoe (TLB). Thermistors were placed in the hole to measure the temperature distribution of the soil with increase in distance below ground level. Figure 4 below shows the NTC thermistors placement in the 3m hole. At the time of writing, temperatures have been recorded for a period of 4 months. In the figure below, T1 to T10 refer to the number of the thermistor. Dots represent the thermistors and how they are positioned below and above ground. The black line that joins the dots represents the
wire cable that joins thermistors. Furthermore, the cable is joined to a data logger where data was retrieved. Data logger (Data taker model DT615) was used to record the data.

Figure 5. Placement of thermistors in soil at different depths.

Figure 6 and Figure 7 show temperature data recorded at difference depths during a warm week and a week during which heavy rains occurred respectively.

Figure 6. Ground temperature data obtained during a warm week.
Figure 6 shows data that was obtained during a week of no rain with an average daily air temperature of about 30°C (not shown). The thermistors that were placed at 0.5m and deeper below ground showed little change in temperature (a maximum change of about 0.4°C at 0.5m). On the contrary, Figure 7 represents data that was obtained during a week of heavy rain. The maximum daily rainfall (80.3mm) of the week took place on 22 February 2017. As a result, the change in temperature at 0.5m below ground level was 2°C, i.e. from 25°C to 23°C. The change in temperature at 0.75m was found to be 1.5°C, i.e. from 24.1°C to 22.5°C. The change in temperature decreased with depth.

4.2 Leak Detection Test
A water pipe leak test was simulated in a transparent plastic container that measured 590mm x 790mm x 400mm (h x l x w). This was done to understand temperature change mechanisms during leak detection. The container was placed in temperature controlled enclosure. The following figures show the layout of the experimental set up. The circles represent the thermistors and their positions. Labels T1 to T6 refer to the numbering of each thermistor. The soil surface is denoted by the horizontal line. The point where leakage was introduced to the box through a pipe is represented by the triangle. The temperature of the leak water, introduced at a flow rate of 140 millilitres per minute, was measured at 21°C.
Figure 8. Layout of the experimental container.

The first experiment was conducted over a period of 2 hours and 37 minutes, which would have resulted in 22 litres being lost due to leaking water. Figure 9, shows the results that were obtained after the experiment, plotting temperature (in °C) against time.
Results from thermistor 4 and 5 are not presented because the leakage rate was too low to reach them in the allotted time into the leakage test. The leak was introduced at 10:25:40pm, after the introduction of the leak, T1 was the first thermistor to detect the leak. The dotted line represents the point in time where leaking water was first detected. The initial detected temperature of leak water was 25.3°C at 10:29:40pm. This means that 560 millilitres of water was lost before any detection. T2 was the second thermistor to detect the leak water, represented by the dotted line as well, at a temperature of 25.3°C at 11:19:24pm. This resulted in 7.56 litres loss of water before the water leak was detected. It was followed by the third detection being T3, where point of detection is marked by the dotted line, at a temperature of 25.4°C at 12:16:56pm, and resulted in 15.54 litres loss of water before water leak could be detected. Lastly, T6 shows a slow decrease in temperature as a result of the cooling soil due to thermal flow interaction before leaking water reached the thermistor.

Interestingly, whenever the leaking water came into contact with the thermistor, a small increase in temperature was noticed before the expected decrease. This was noticed with all the three graphs from thermistors T1, T2 and T3. The temperature increase for T1 is calculated to be 0.3°C, for T2 is 0.2°C and T3, 0.1°C. The average initial increase in temperature when leak water came in contact with the soil was 0.2°C.

According to Mohsenin (1980), a wetting of the surface area of any particle as a result of coming into contact with a liquid or gas releases some energy. As such, dry soil materials are expected to release a small amount of energy when first coming in contact with liquid, gas or vapour because a wet surface has less surface free energy compared to a dry surface. As an example, the free surface energy of quartz is reduced by 72 mJ·m⁻² through the emersion in liquid water (Parks, 1984).

A third experiment was conducted with two different soil samples being the light brown washed quartz sand (washed quartz sand) from a commercial source and the red brown silt fine sand of transported origin (transported soil). This was done to understand the release of surface free energy concept further. The following figures represent the two samples that were used. In this experiment thermistors were placed in a 40mm diameter beaker, 70mm below soil surface in

Figure 9. Leak water test results.
both experiments. The height of the soil for both beakers was 125mm. Water was introduced in both beakers and data was recorded with the data logger that was connected to the thermistors to observe and analyse data showing the increase in temperature when water comes contact with into soil for different soil samples.

Figure 10 shows the positioning of the thermistor, labelled T1 for thermistor 1, inside the beaker. The circle in the beaker represents the thermistor, whereas the horizontal line represents the soil surface. The curve from the circle (thermistor) represents the wire connecting the thermistor with data logger. Figure 11 shows the two soil samples that were used in this experiment placed inside beakers.

![Figure 10. Visual position of thermistor inside beaker.](image)

![Figure 11. Soil samples used for experiment: transported soil (left) and washed quartz sand (Right).](image)

Figure 11 represents the experiment layout that was taken into account for both soil samples. The horizontal line represents the soil level in both soil samples in a beaker.

- Water flow rate 140 millilitres per minute,
- Water temperature 23°C and
- Soil mass for both samples, 1.1kg.

The following figure, Figure 12, shows the results that were obtained after the test, plotting temperature (in °C) against time (in hh:mm).
From the above figure, the washed quartz sand sample experienced a higher increase in temperature when it came into contact with water in comparison with the transported soil. This phenomenon is due to the fact that washed quartz sand has a higher amount of free surface energy than the transported soil and hence it releases more energy compared to transported soil.

5 Discussion

In most countries, PVC pipelines are installed between 0.5 to 0.7 m below the ground to transport water. The depth range of installation proves to be beneficial for the detection of leakages by means of temperature measurement due to small natural temperature variation at this depth. From the results obtained in Figure 6, the change in temperature at 0.5 m below ground level was 2°C after heavy rains. The laboratory data, however, shows a change in temperature of 4°C for a thermistor that was placed 50mm below the leaking pipeline at an initial temperature of 25.5°C. As a result, it can be assumed that a leak can be detected through this method even following a rainy day. Overall, there is an expected change in temperature during the wetter days. However, should a measured temperature change below a water pipeline be greater than expected, then it can be assumed that there is a leakage.

6 Conclusions

The use of thermistors allows the monitoring of leakages and hence better management of pipelines, by allowing pipeline operators to make fast decisions, whenever there is a leak, on the maintenance and operation of pipelines. The experimental results presented in this paper shows that it is possible for a leak to be detected and localised. The leak detection is done by measuring the change in temperature of the two mediums, soil and water. While, the location of the leak is done by observing the change in temperature between thermistors along a pipeline.
References


An Experimental Correlation Study on Soil Properties of Lime Stabilised Soil Samples

P. H. Bhengu¹, D. Allopi²

¹Dept. of Civil Engineering, Durban University of Technology, KwaZulu-Natal, bhengupk@gmail.com
²Dept. of Civil Engineering, Durban University of Technology, KwaZulu-Natal, allopid@dut.ac.za

Abstract

The effective utilisation of lime can substantially enhance the stability, permeability, and load-bearing capacity and other of the road underneath soil layers. The study presented here is a correlational quantitative and qualitative study which seeks to relate more than one variable from the same group of subject being analysed (i.e. the use of lime). The purpose of using correlations in this study is to figure out which variables are connected. The variable referred to above relate to both the engineering and microstructural properties of soil samples treated with lime (engineering/construction purposes). This paper analyses the strength development in lime-stabilized soil samples based on physical properties and microstructural properties considerations. Qualitative and quantitative study on engineering properties tests such as consistency limits, Soil compaction, Unconfined Compressive Strength (UCS), California Bearing Ratio (CBR) were conducted on soil samples treated with lime variations (i.e. 2%, 4%, 6%, 8% and 10% by weight of soil). Further qualitative and quantitative properties consideration through XRD, Scanning Electron Microscope (SEM) and Energy Dispersive Spectroscopy (EDS) were conducted with the intention of determining the types and relative amounts of the minerals or mineral elements present in the soil (soil mineralogy) because of its influence on the soil behaviour, its use in soil classification, and its relevance to soil genetic process. The test results indicated that the inclusion of lime to the two soil samples changes the soil properties. Many of the important engineering properties (distribution and connectivity of pores, particle size, shape and distribution, along with the arrangement of grains and grain contacts of soils can be enhanced by the addition of lime, with XRD, SEM and EDS/EDX graphics showing that lime treatment changes significantly the soil fabric depending on curing time and water content, thus justifying the correlation of soil properties.

Keywords: Physical properties, Microstructural analysis, UCS, X-RD, SEM, EDS/EDX.
1 Introduction

In South Africa, one of the prime priorities of highway administrators (i.e. SANRAL, Department of Transport, Municipalities and other) is to increase productivity and decrease the rate of road degradation, most importantly the national highways which acts as the linking mode in-between the provinces and other. The Improvement of the nation's highways performance is normally focused on the quality of the roadway surface pavements. Pavement performance largely and primarily rely on the behaviour of pavement and subgrade soil layers which can be evaluated through an analysis of soil properties. In general, these soil properties are engineering and microstructural properties.

The aforementioned engineering and microstructural properties studies on lime-treated soils are increasingly used to upgrade the understanding of the engineering properties and macroscopic properties of compacted lime treated natural soils (Goegr and Markgraf, 2006; Millogo, Hajjaji and Ouedraogo, 2007; Romero and Simms, 2008; Deneele et al., 2010; Horpibulsuk et al., 2010; Liu, Pemberton and Indraratna, 2010; Ajayi, 2012; Khattab and Hussein, 2012; Makusa, 2012; Mukhtar, Khattab and Alcover, 2012; Solanki and Zaman, 2012; Insight, 2013; Muhmed and Wanatowski, 2013; Onunkwo, A.P and Onyekuru, 2014). These studies (i.e. engineering vs microstructural properties of lime stabilised soil) involve the use of techniques or methods to analyse the arrangement and distribution of particles, particle assemblies and pores and their contacts and connectivity in different soils as has been justified in many experimental studies. Mukhtar, Khattab and Alcover (2012) journal paper focuses on providing an explanation of improvements of geotechnical properties that arise in the expansive soil behaviour due to the lime-clay reactions, mainly a pozzolanic reaction, using microscopic analysis. The changes in the main geotechnical properties (plasticity, unconfined compressive strength, swelling pressure and permeability) associated with the micro level texture and structure of untreated and lime-treated compacted FoCa clay samples were studied using x-ray diffraction, thermogravimetric analysis, scanning electron Microscopy (SEM) and transmission electron microscopy (TEM) methods. Based on the author’s results, it was identified that the treated FoCa clay has a greater number of packed layers in comparison to untreated or natural FoCa clay with the above mentioned techniques proving to be useful in observing the appearance of a cementitious phase or pozzolanic phase that is constituted of a hydrate of calcium silicate aluminate (CSAH). Another qualitative and quantitative study on the microstructure was carried out using a scanning electron microscope, mercury intrusion pore size distribution measurements, and thermal gravity analysis of the soil.

Three influential factors in this investigation were water content, curing time, and cement content. The results revealed that cementitious stabilization (cement, lime and other related) improves the soil structure by increasing inter-cluster cementation bonding and reducing the pore space as was observed by the Horpibulsuk et al. (2010) study. To soils made of materials such as bricks, lime proves to be an appropriate manufacturing component as well. Microstructural changes of adobe bricks made of lime-clayey raw material mixes were investigated by x-ray diffraction, infrared spectrometry, differential thermal analyses, scanning electron microscopy and energy dispersive spectrometry. The impact of these changes on the mechanical resistance and water absorption of adobe bricks was evaluated. It was found that lime additions resulted in the development of calcite and poorly crystallized calcium silicate hydrate (CSH) (Millogo, Hajjaji and Ouedraogo, 2007).

2 Materials and methods of testing

2.1 Sample preparations, tests conducted on soils and testing procedures

Bags of naturally occurring soil samples were collected from two pre identified locations in the parts of the province of KwaZulu Natal in South Africa (i.e. Umlazi and Mkuze South and

408
North part of the province respectively). Both tests relating to engineering properties and microstructural properties identification for soil samples were conducted in studying the correlating variables on the treated soil samples treated with lime. For engineering properties of the soil identification, tests such as consistency limits (Atterberg tests), Maximum Dry Density and Optimum Moisture Content determination, UCS, Curing Soaking- for CBR penetration, CBR Penetration were prime tests conducted. These tests were conducted in relation to South African Technical Methods of Highway 1 (TMH1) under subsections, methods A1, A2, A3, A7, A9, A14 and other. For microstructural properties identification of the soil, fine soil grains treated with lime variations and passing 0.425mm sieve size were used for RXD analysis. The soil sample were x rayed from 2° to 70° theta using Cu Kα radiation, with 2° to 70° degrees range 2θ chosen with the intention to provide enough x-ray diffraction peaks to identify most common soil minerals on the soil samples that were investigated, with tiny samples for SEM taken from the samples that were used for UCS.

3 Presentation and results

Table 1. Consistency limits, MDD & OMC properties of the soil

<table>
<thead>
<tr>
<th>Property</th>
<th>Sample 1</th>
<th>Sample 2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Liquid limit (%)</td>
<td>23.43%</td>
<td>31%</td>
</tr>
<tr>
<td>Plastic limit (%)</td>
<td>10.61%</td>
<td>21.94%</td>
</tr>
<tr>
<td>Plasticity index (%) (before treatment)</td>
<td>12.82%</td>
<td>9.05%</td>
</tr>
<tr>
<td>Plasticity index (%) (After lime treatment)</td>
<td>9.21%</td>
<td>8.42%</td>
</tr>
<tr>
<td>Linear shrinkage (mm)</td>
<td>4.24mm</td>
<td>5.57mm</td>
</tr>
<tr>
<td>Maximum Dry Density (MDD), (Average)</td>
<td>1660kg/m³</td>
<td>1682kg/m³</td>
</tr>
<tr>
<td>Optimum Moisture Content (OMC), (Average)</td>
<td>20.4%</td>
<td>20.3%</td>
</tr>
</tbody>
</table>

3.1 Consistency limits, MDD & OMC properties results discussion

Table 1 shows consistency limits (atterberg limits) results for the two soil samples tested. As per the results, it can be seen that upon lime application, the plasticity of the soil decreases. One of the attributes to the above relates to a decrease in the liquid limit and the increase in the plastic limit of the soil. An analysis on the maximum dry density and the optimum moisture content of the lime treated two soil samples at different contents of lime performed to determine the relationship between the moisture content and the dry density can also be seen in table 1. The result are series of tests conducted to examine the impact of different lime contents at 2%, 4%, 6%, 8% and 10% on the ASSTHO Maximum Dry Density (MDD) and the Optimum Moisture Content (OMC) of soil at specified comp active efforts.

The results of the compaction tests conducted on the soil samples showed that the addition of lime resulted in the improvement in the characteristics of the natural two soil samples. The first soil sample (i.e. one) recorded an average of 1660kg/m³ dry density from actual densities of (1643 kg/m³ at 20 MC for 2% lime; 1667 kg/m³ at 22 MC for 4% lime; 1677 kg/m³ at 22.5 MC for 6% lime; 1670 kg/m³ at 23 MC for 8% lime and 1645 kg/m³ at 23.1 MC for 10% lime. The second soil sample (i.e. two) recorded an average of 1682kg/m³ dry density from actual densities of (1663 kg/m³ at 17.5 MC for 2% lime; 1731 kg/m³ at 18.2 MC for 4% lime; 1752 kg/m³ at 18.5 MC for 6% lime; 1674 kg/m³ at 19 MC for 8% lime and 1591 kg/m³ at 19.1 MC for 10% lime. Lime content at range of 4% - 8% indicates density increase of the stabilized soil samples. For both samples (i.e. samples 1 and 2), results showed that further addition of lime decreases the density and increasing moisture content. The above reported situation (i.e. the
increase in the moisture content vs decrease in dry density in relation to the addition of lime) results from less comp active effort due to connectivity of soil pores, the arrangement of the soil grains.

3.2 Unconfined Compressive Strength curves (UCS)

Figures 1 and 2 shows the unconfined compressive strength due to the application of lime, with immediate and substantial improvement in the strength of the soil samples stabilized with 2% 4% 6% 8% and 10% lime contents, compacted at the respective MDD and OMC, and cured in accordance with TMH1. The result indicate that the UCS (one engineering property of the soil) of the soil sample can be significantly improved by lime stabilization.

Constant increase in strength was indicated by the lime stabilized material for sample 1 at each lime content with highest strength record of 810Kpa at lime content of 8% compared to 382Kpa after curing of sample 2 at the same lime content. Sample 2 indicated a slow grow in strength compare to sample 1. This may be from a number of contributory factors ranging minor testing methodology problems of soil samples to problems relating to the loss of cementitious content in stabilised soils (carbonation) taking place while soil samples were mixed with lime and moist soil in the open air. The above increase in strength of USC soil samples tested has relations to properties as can be justified by both the (microstructural and engineering properties) ranging from the distribution and connectivity of soil pores, soil particle size, shape and distribution of the soil particles, the arrangement of the soil grains and the grain contacts etc. A classic further justification to the above relates to the variation in swelling/expansion of the soil sample 1 at different compaction efforts contents. Soil sample 1 compacted at a slightly higher optimum content will exhibited lower swelling than soil sample 2 compacted to the same porosity at slighter lower optimum content.
3.4 California Bearing Ratio (CBR) results discussion
The improvement in the strength of almost all CBR penetration soil samples can be justified by the formation of agglomerates due to the application of lime. The resistance of the lime treated soil sample during CBR Penetration test results illustrated the importance of the size, shape and arrangement of soil aggregations, as well as the distribution and connectivity of pores as can be justified by the micrographs present on the microstructural properties results (following subsection), on soil samples treated with lime, and how such aggregations and pores can change during CBR penetration. Lime treated soil samples at either 55 comp active efforts or 25 comp active efforts both mechanically and manually showed constant increases in load required to penetrate the surface of the lime stabilised for almost soil samples.

The constant increase in the CBR values for almost all the tested soil samples is a direct indication of the principal chemical reactions taking place during lime-soil stabilisation (Aldaoood, Bouasker and Mukhtar, 2014).

3.5 X-Ray Diffraction results discussion
Figures 3-8 shows the XRD graphs for both soil samples treated with lime different lime variations by X-ray diffraction. The X-ray diffraction technique provided detailed information about the atomic structure of crystalline of the two soil samples that were lime treated and tested.

The Y-axis shows the counts (number of X-rays received and processed by the detector) and the X-axis shows the energy level of those counts. From XRD, soil tested consisted of quartz, albite, microcline, montmorillonite, biotite, rutile and chrysotile, as identified from the peaks for minerals that were present on the soil.

3.5.1 X-Ray Diffraction graphs

Untreated soil sample 1

Sample 1 – Randomly selected

Figure 3. Sample 1 – Untreated

Figure 4. XRD Graph, sample 1 randomly selected
3.6 SEM & EDS/EDX results discussion
SEM (Scanning Electron Microscopy) and EDS (Energy Dispersive Spectroscopy) analysis was carried out on the tiny soil samples after the unconfined compression tests. Lime treatment changed significantly the soil fabric depending on curing time and water content. Figure 9 to figure 18 shows the SEM micrographs of treated soil samples. Images in the SEM display compositional contrast that results from different atomic number elements and their distribution. As can be seen from figure 9 to figure 18, soil particles treated with different content of lime were observed and displayed flaky texture.

Flaky texture confirms the formation of needle-like crystalline formations in the soil sample such as ettringite (The mineral name for calcium substance (Portland cement association, 2001). Further to the above mentioned, the figures show the non-existence (or less) of the pores within the soil particles indicating the change in microstructure of the soil due to addition of stabilizer (lime) hence promoting the strength of the tested soil samples. For EDS/EDX elements were found in the soil, majority of the soil chemical elements evaluated were significantly influenced by use of lime to the three soil samples. Chemical elements, ordered by their atomic number (number of protons), electron configurations, and recurring chemical properties from the SEM on the soil samples treated with different lime contents were identified in the soil samples tested. These (elements) ranged from Magnesium (Mg), Potassium (K), Calcium (Ca), Titanium (Ti), Iron (Fe), Aluminium (Al), Silicon (Si) Oxygen (O), Carbon (C) to Sodium (Na).
With limestone being the source of Ca and Mg and in the presence of water the carbonates dissolve and the Hydroxide, some of the elements listed above form part of the chemical composition making up the hydrated lime used for this experiment. Among these are Calcium, Magnesium, iron, and silicone as can be seen on the EDS/EDX images. Furthermore from the above listed elements, the element Carbon listed relates not to the element that was found in the soil but the non-porous carbon double sided adhesive tape which was used for EDS/EDX tests. The double sided adhesive permitted quick mounting of samples without using liquid or colloidal adhesives.

Due to chemical reactions occurring leading to the distribution and the connectivity of pores, the soil particle size, the soil shape and the distribution, along with the arrangement of the grain and the contacts of the grains treated with lime at different contents, soil particles exhibited cementitious substances resulting from the use of hydrated lime and enhance by the changes in soil properties (i.e. engineering and microstructural properties).

### 3.6.1 SEM Micrographs

![Figure 9. Sample 1 – 2%Lime](image)

![Figure 10. Sample 1 – 4%Lime](image)

![Figure 11. Sample 1 – 6%Lime](image)

![Figure 12. Sample 1 – 8%Lime](image)
4 Conclusion

The performance of lime to the soil was observed and discussed through the engineering properties and microstructural analysis of the soil samples. The concluding remarks in relation to the entire study are presented as below.

As per the experimental results, it can be seen that many of the important properties of soils can be enhanced by the addition of lime. To develop an understanding of the possible mechanisms involved, a series of experiments through variation of parameters/properties were carried out, and the following sub conclusions drawn from the result of series of tests aimed at studying the influence of lime to the engineering properties of the two soil samples. It was observed that with an increase in lime content while the plasticity of soil reduces with increased lime content.
The results further showed that beneficial effects are obtained by the addition of lime contents to soil samples with dry density of the soil sample decreasing with the increase in lime content. The result for the Unconfined Compression Strength test conducted on the two naturally occurring soil samples stabilized with 2% 4% 6% 8% and 10% lime contents, compacted at the respective MDD and OMC, and cured in accordance with TMH1 indicated that the soil samples can be significantly improved by lime stabilization. Factors such as loss of cementitious content in stabilised materials leading the formation of calcium carbonate when lime treated soils are in the open air might have promoted by carbonation as can be justified by inconsistency of some of the UCS test values.

The CBR test was performed by measuring the pressure required to penetrate surface of the compacted soil samples, at different comp active efforts (i.e. 55 CBR mechanically compacted, 25 CBR mechanically compacted, and 55 CBR manually compacted). For the two tested soil samples treated with different lime contents reported a constant increase in CBR when compacted either at 55 comp active efforts or 25 comp active efforts, mechanically. The constant increase in the CBR values for almost all the tested soil samples in a direct indication of the principal chemical reactions taking place during lime-soil stabilisation.

X-ray diffraction technique tests for providing detailed information about the atomic structure of crystalline of the three soil samples and SEM/EDX for the elemental analysis or chemical characterization of the soil samples tested was discussed. From XRD, soil tested consisted of Quartz, albite, microcline, montmorillonite, biotite, rutil and chrysotile, as identified from the peaks for minerals that were present in the soil.

Further, in studying the soil samples under magnification for microstructural analysis so as to determine how they perform under a given application, SEM-micrographs indicated the formation of flaky texture of cementitious form. This flaky texture which confirms the formation of needle-like crystalline formations in the soil sample were shown in the micrographs of the lime treated soil samples due to reactions which contributed to the increase of the strength of stabilized soil samples.

According to EDS/EDX, certain elements were found in the soil. These (elements) ranged from Magnesium (Mg), Potassium (K), Calcium (Ca), Titanium (Ti), Iron (Fe), Aluminium (Al), Silicon (Si) Oxygen (O), Carbon (C) to Sodium (Na). Majority of the soil elements evaluated were significantly influenced by use of lime and other factors to the two soil samples. Elements such as Ca and Mg relating to limestone being the source of these elements in the presence of water during manufacturing.

Based on the experimental results, it can be prime concluded that the use of hydrated lime for the experiment effectively enhanced the soil properties as can be justified by the analysis of the engineering properties and the analysis of the microstructural properties of the soil through SEM, EDS/EDX micrographs of lime treated soil sample indicating cementitious substances and other of the particles due to chemical reactions occurring to the soil particles treated with lime bringing about a correlation between the above mentioned variables.

References


Insight, E. S. 2013. Microstructural analysis of optical materials.
Makusa, G. P. 2012. Soil stabilization methods and materials,. Sweden Civil, environmental and natural resources engineering: Division of mining and geotechnical engineering.
Potential Aggressive Ground Conditions on Buried Concrete and Copper Earth Mat at a Substation in Orlando, Soweto

S. Nyathi

Abstract

In 2013 Aurecon conducted a geotechnical investigations for the extension of a substation in Orlando, Soweto. During this investigation groundwater seepage was not encountered in any of the excavated test pits. Deposition of salt and ingress of acidic groundwater into excavations during construction indicated potentially aggressive ground conditions.

Further geotechnical investigations were subsequently recommended and groundwater seepage was encountered in all the additional test pits.

Basson (1989) and BRE (2001 and 2003) specifications, indicate that these conditions are very highly aggressive and excessively corrosive to concrete structures. DIN 50929 Part 3 (1985) indicates that the severity of these soil and water conditions indicates medium to high aggressiveness towards copper. pH values are ranging between 2.6 to 4.5 which indicates very acidic conditions.

Since the concrete footings and copper earth mat are already installed, the construction of impermeable upstream (HDPE geosynthetic geomembrane) barrier between the substation and the adjacent tailings dam is recommended.

Keywords: Groundwater seepage, aggressive conditions, concrete, copper, impermeable barrier.

1 Introduction

In 2013 Aurecon was appointed to conduct geotechnical investigations for the extension of a substation, which is located south-west of Johannesburg. During this previous investigation groundwater seepage was not encountered in any of the test pits excavated to a depth of 1.8m. During construction of the substation extension deposition of salt on the platform and ingress of acidic groundwater into excavations indicated that potentially aggressive ground conditions could be present.
This paper presents the findings of the follow up investigation. With focus on the following:

- The laboratory test results, specifically that relating to the potential corrosivity and aggressiveness.
- Geotechnical considerations relating to the expected aggressive ground conditions and the likely (long-term) impacts on the substation extension.
- Recommendations for mitigating the potential effects of seepage of aggressive groundwater on the concrete footings as well as the copper earth mat.

2 Site Description and geology

The Substation is located to the south-west of Johannesburg. It is located on a site bounded on the northern side by the N17 road and to the eastern side by a tailings dam. The investigated area is located between the current construction of substation extension and the tailings dam.

According to the published geological information (1:250 000 geological map, Sheet 2626 West Rand, Council for Geoscience) the site is underlain at depth by strata of the Witwatersrand Supergroup. The bedrock underlying the site belongs to the Turfontein Sub-group, part of the Central Rand Group, and comprises quartzite, conglomerate and shale.

3 Investigation Methodology

The current investigation comprised the excavation of test pits with a tractor-loader-backhoe (TLB), soil profiling, soil- and water sampling. The previous investigation comprised of excavation of five test pits (PTP01 to PTP05) across the then proposed extension. For this additional investigation four test pits (PTP06 to PTP09) were excavated along the problem area. Three (3 No) of the test pits were excavated between the tailings dam and Platform 3, with one test pit located at the substation entrance gate.

A two-person team carried out the test pitting in order to comply with accepted safety requirements as reflected in the South African Code of Practice (SAICE 2007). The fieldwork was carried out on the 17 January 2017 in accordance with the guidelines of SAIEG-AEG-SAICE (1990).

Prior to field work the proposed investigation area was scanned to ensure no buried services were intersected during the investigation.

Soil and water samples were taken from representative horizons and submitted to engineering materials laboratory in order to characterize and classify the in situ materials and their geotechnical properties and testing the potential corrosivity and aggressiveness to buried elements (concrete and the copper of the earth mat). The following testing was conducted:

- Foundation Indicators (No 5.), and
- DIN & BRE corrosivity test (No 5).

Table 1. Test pit summary

<table>
<thead>
<tr>
<th>Test pit No</th>
<th>Refusal (m)</th>
<th>Water Seepage (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Follow up Investigation (2017)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>PTP06</td>
<td>2.1</td>
<td>1.8m</td>
</tr>
<tr>
<td>PTP07</td>
<td>2.1</td>
<td>1.6m</td>
</tr>
<tr>
<td>PTP08</td>
<td>1.6</td>
<td>1.3m</td>
</tr>
<tr>
<td>PTP09</td>
<td>2.5</td>
<td>2.5m</td>
</tr>
</tbody>
</table>
4 Ground- and surface water conditions

During the previous investigation none of the excavated test pits intersected groundwater seepage. Due to acidic groundwater seepage and surface water from the adjacent tailings dam, all four test pits for the current investigation encountered groundwater seepage. This seepage occurred above quartzite bedrock within the pebble marker and residual sand layers and within the pedogenic material in PTP09. The direction of the groundwater seepage is from the adjacent tailings dam to the substation.

4.1 Origin of aggressive water

The gold tailings dam adjacent to the substation is in all likelihood a source of the acidic surface and ground water. According to Rudd (1973) pyrite is the main mineral that causes water pollution in the Witwatersrand gold tailings dams. Rudd (1973) indicates that pyrite in its exposed state is acted on by oxygen and bacteria. It is oxidized to form sulphuric acid and iron hydroxide, and is further converted to iron oxide (the red and yellow surface water), while the sulphuric acid reacts to form sulphates, which are all soluble and contaminate water in which they are dissolved.

Many of the tailings dams in the Johannesburg area were not lined and many were not vegetated, providing a source of extensive surface and groundwater contamination to surrounding areas (Oelofse et al (2008)). This sulphuric acid from the tailings dam appear to leach towards the substation.

4.2 Laboratory Results

Limited chemical tests were conducted in the previous investigation, comprising determination of the pH value as well as the conductivity. These results are summarised in Table 2. Based on significance of soil resistivity on corrosivity Duligal (1996) provides the following table for evaluation of the conductivity of soil (Table 2).

<table>
<thead>
<tr>
<th>Soil conductivity (mS/m)</th>
<th>Soil resistivity (Ohm.cm)</th>
<th>Corrosivity classification</th>
</tr>
</thead>
<tbody>
<tr>
<td>More than 50</td>
<td>0 – 2000</td>
<td>Extremely corrosive</td>
</tr>
<tr>
<td>25 – 50</td>
<td>2000 – 4000</td>
<td>Very corrosive</td>
</tr>
<tr>
<td>20 – 25</td>
<td>4000 – 5000</td>
<td>Corrosive</td>
</tr>
<tr>
<td>10 – 20</td>
<td>5000 – 10000</td>
<td>Mildly corrosive</td>
</tr>
<tr>
<td>Less than 10</td>
<td>&gt;10000</td>
<td>Not generally corrosive</td>
</tr>
</tbody>
</table>

Based on Evans’ guideline (1977) a soil pH less than 6 indicates serious corrosion potential.
Table 3. Summarised chemical test results from the previous investigation

<table>
<thead>
<tr>
<th>Hole No</th>
<th>Depth (m)</th>
<th>Horizon</th>
<th>pH</th>
<th>Conductivity (mS/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>PTP01</td>
<td>0.5-1.1</td>
<td>Colluvium</td>
<td>5.57</td>
<td>6.2</td>
</tr>
<tr>
<td>PTP03</td>
<td>0.1-0.7</td>
<td>Colluvium</td>
<td>5.53</td>
<td>1.0</td>
</tr>
<tr>
<td>PTP05</td>
<td>0.8-1.1</td>
<td>Residual Quartzite</td>
<td>5.29</td>
<td>1.49</td>
</tr>
</tbody>
</table>

From the guidelines in Table 2 and the results in Table 3 it was concluded that:
- The pH values of less than 6 indicate that corrosion may occur.
- The conductivity measurements indicate the materials are not generally corrosive or only mildly corrosive.

Observations of the platforms during construction of the substation extension indicated potentially aggressive ground conditions. A single sample of seepage water indicated the strongly acidic water. Although footings were cast in the ground there were deposits on surface that further suggested high salt concentrations within the soil.

For this additional investigation representative soil and water samples were collected for laboratory testing in order to test the potential corrosivity and aggressiveness to buried elements (concrete and copper of the earth mat).

A full suite of DIN and BRE tests were conducted on three (3 No.) water samples and three (3 No) soil samples to determine the potential aggressiveness to concrete and copper on the ground.

4.3 Concrete
Aggression towards concrete or buried metals is primarily driven by the types of ionic substances present in contact water. Different types of ions cause either an expansive change and/or a loss of properties of the cement; subsequently affecting the strength of the concrete. The summary of chemical test results is presented in Table 4 on the following page.
Table 4. Summary of results from chemical tests for both water and soil samples

<table>
<thead>
<tr>
<th>Test</th>
<th>Water from footings</th>
<th>PTP08 water</th>
<th>Surface water near tailings dam</th>
<th>PTP06 0.7 –</th>
<th>PTP08 1.1 –</th>
<th>PTP08 1.5 –</th>
</tr>
</thead>
<tbody>
<tr>
<td>pH</td>
<td>3.0</td>
<td>2.6</td>
<td>3.6</td>
<td>3.7</td>
<td>4.5</td>
<td>4.4</td>
</tr>
<tr>
<td>Electric conductivity, mS/m</td>
<td>309</td>
<td>-</td>
<td>-</td>
<td>26.9</td>
<td>120</td>
<td>128</td>
</tr>
<tr>
<td>Chlorides, mg/l</td>
<td>40</td>
<td>135</td>
<td>126</td>
<td>13</td>
<td>2.6</td>
<td>7.3</td>
</tr>
<tr>
<td>Calcium (Ca), mg/l</td>
<td>203</td>
<td>458</td>
<td>552</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Sulphates, mg/l (H₂O soluble)</td>
<td>1878</td>
<td>5177</td>
<td>2790</td>
<td>1787</td>
<td>618</td>
<td>623</td>
</tr>
<tr>
<td>Total acidity as CaCO₃, mg/l</td>
<td>&lt;5</td>
<td>2520</td>
<td>600</td>
<td>140</td>
<td>16</td>
<td>16</td>
</tr>
<tr>
<td>Total alkalinity as CaCO₃, mg/l</td>
<td>&lt;5</td>
<td>&lt;5</td>
<td>&lt;5</td>
<td>&lt;5</td>
<td>8</td>
<td>8</td>
</tr>
<tr>
<td>Ammonia nitrogen, mg/l</td>
<td>32</td>
<td>0.8</td>
<td>3.2</td>
<td>3.9</td>
<td>2.8</td>
<td>10.5</td>
</tr>
<tr>
<td>Nitrate nitrogen, mg/l</td>
<td>-</td>
<td>&lt;0.1</td>
<td>&lt;0.1</td>
<td>2.6</td>
<td>&lt;0.1</td>
<td>1.8</td>
</tr>
<tr>
<td>Magnesium, mg/l</td>
<td>162</td>
<td>446</td>
<td>146</td>
<td>29</td>
<td>10</td>
<td>19</td>
</tr>
<tr>
<td>Resistivity (ohm. cm)</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>3717</td>
<td>833</td>
<td>781</td>
</tr>
<tr>
<td>% moisture</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>10.9</td>
<td>2.6</td>
<td>8.3</td>
</tr>
<tr>
<td>Leaching Index (after Basson)</td>
<td>5242</td>
<td>4059</td>
<td>4733</td>
<td>4997</td>
<td>4502</td>
<td>4575</td>
</tr>
<tr>
<td>Spalling Index (after Basson)</td>
<td>327</td>
<td>610</td>
<td>319</td>
<td>242</td>
<td>182</td>
<td>148</td>
</tr>
<tr>
<td>Basson Aggressiveness Index</td>
<td>5569</td>
<td>4668</td>
<td>5052</td>
<td>5239</td>
<td>4889</td>
<td>4722</td>
</tr>
<tr>
<td>Final Basson Index (corrected for flow)</td>
<td>4178</td>
<td>3501</td>
<td>3789</td>
<td>3929</td>
<td>3667</td>
<td>3542</td>
</tr>
</tbody>
</table>

An aggressiveness index was established by Basson (1989) to estimate the potential aggressiveness. Table 5 provides guidance for the interpretation of index of aggressiveness.

Table 5. Aggressiveness index interpretation (Basson, 1989)

<table>
<thead>
<tr>
<th>Final index</th>
<th>Aggressiveness</th>
</tr>
</thead>
<tbody>
<tr>
<td>&lt;350</td>
<td>Non- to mildly aggressive</td>
</tr>
<tr>
<td>350 to 750</td>
<td>Mildly to fairly aggressive</td>
</tr>
<tr>
<td>750 to 1000</td>
<td>Highly aggressive</td>
</tr>
<tr>
<td>&gt;1000</td>
<td>Very highly aggressive</td>
</tr>
</tbody>
</table>

In addition, various chemical properties of the sampled water can be used to augment findings of the index of aggressiveness discussed above. The guideline for the interpretation of these properties are summarised in Table 6 below.
Table 6. Recommended limits for assessing aggressiveness (Basson, 1989)

<table>
<thead>
<tr>
<th>Property of water</th>
<th>Degree of aggressiveness of water</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Moderate</td>
</tr>
<tr>
<td>pH</td>
<td>6 to 8</td>
</tr>
<tr>
<td>pH minus CaCO3 saturated pH</td>
<td>-0.2 to -0.3</td>
</tr>
<tr>
<td>Calcium hardness as mg CaCO3/l</td>
<td>200 to 300</td>
</tr>
<tr>
<td>Total ammonium ion as mg NH4/l</td>
<td>30 to 50</td>
</tr>
<tr>
<td>Magnesium ion as mg Mg/l</td>
<td>100 to 500</td>
</tr>
<tr>
<td>Total sulphate ion as mg SO4/l</td>
<td>150 to 1000</td>
</tr>
<tr>
<td>Chloride ion as mg Cl/l</td>
<td>500 to 1000</td>
</tr>
</tbody>
</table>

Final indices (see Table 4) for aggressiveness were found to exceed 1000. According to Table 5, the aggressiveness is very highly aggressive. Table 6 indicates that the analysed soil and water samples are excessively corrosive to concrete structures.

4.4 Copper

The assessment of soil for corrosivity was done using Table 1 in DIN 50929 Part 3 (1985). Investigated parameters collected during fieldwork and measured parameters from the laboratory results are used for the rating. Table 7 and Table 8 summaries the soil and water assessment results.

Table 7. Corrosion assessment from the soil samples (DIN 50929 Part 3, 1985)

<table>
<thead>
<tr>
<th>Parameter investigated or measured</th>
<th>Rating</th>
</tr>
</thead>
<tbody>
<tr>
<td>Type of soil: heavily contaminated soil, $Z_1$</td>
<td>-12</td>
</tr>
<tr>
<td>Soil resistivity, Ω cm, $Z_2$</td>
<td>-6</td>
</tr>
<tr>
<td>Moisture content, $Z_3$</td>
<td>0</td>
</tr>
<tr>
<td>pH value, $Z_4$</td>
<td>-1</td>
</tr>
<tr>
<td>Buffer capacity, acidity up to pH 4.3, mmol/kg, $Z_5$</td>
<td>-6</td>
</tr>
<tr>
<td>Sulphide content, mg/kg, $Z_6$</td>
<td>-3</td>
</tr>
<tr>
<td>Chloride and sulphate, mmol/kg, $Z_7$</td>
<td>-2</td>
</tr>
<tr>
<td>Sulphates, (H2O soluble), mmol/kg, $Z_8$</td>
<td>-3</td>
</tr>
<tr>
<td>Location of structures with respect to groundwater, $Z_9$</td>
<td>-1</td>
</tr>
<tr>
<td>Soil homogeneity, horizontal, $Z_{10}$</td>
<td>-2</td>
</tr>
<tr>
<td>Soil homogeneity, vertical, $Z_{11}$</td>
<td>0</td>
</tr>
</tbody>
</table>
Table 8. Corrosion assessment from the water samples (DIN 50929 Part 3, 1985)

<table>
<thead>
<tr>
<th>Parameter investigated or measured</th>
<th>Rating</th>
</tr>
</thead>
<tbody>
<tr>
<td>Type of water: Flowing water, N₁</td>
<td>-3</td>
</tr>
<tr>
<td>Location of structure: water/air interface, N₂</td>
<td>1</td>
</tr>
<tr>
<td>Chloride and sulphate, mol/m³, N₃</td>
<td>-7</td>
</tr>
<tr>
<td>Alkalinity, mol/m³, N₄</td>
<td>4</td>
</tr>
<tr>
<td>Calcium, mol/m³, N₅</td>
<td>2</td>
</tr>
<tr>
<td>pH value, N₆</td>
<td>-3</td>
</tr>
</tbody>
</table>

Table 9 analyses the soil and water using the ratings as prescribed in DIN 50 929 Part 3 (1985).

Table 9. Estimating copper corrosion from (DIN 50929 Part 3, 1985)

<table>
<thead>
<tr>
<th>Soil Condition</th>
<th>DIN 50 929</th>
<th>Value</th>
<th>Severity</th>
</tr>
</thead>
<tbody>
<tr>
<td>Acid soil</td>
<td>1a</td>
<td>Z₁ = -12 and Z₅ &lt; -6</td>
<td>High</td>
</tr>
<tr>
<td>Sulphide / Contamination</td>
<td>1b</td>
<td>Z₆ = -3 and Z₁ = -12</td>
<td>Medium/ high</td>
</tr>
<tr>
<td>Ammonia</td>
<td></td>
<td>&lt;21.0mg/kg</td>
<td>High</td>
</tr>
<tr>
<td>Chloride &amp; Sulphate</td>
<td>7</td>
<td>Z₇ &gt; 10 - 30</td>
<td>Medium</td>
</tr>
</tbody>
</table>

The results indicates high severity of copper corrosion for acid soils, sulphate and contamination indicates medium and high severity respectively, chloride and sulphate indicates medium severity with ammonia indicating high severity. The severity of these soil conditions indicates medium to high aggressiveness towards copper.

5 Geotechnical considerations and recommendations

The main geotechnical consideration is the acidic seepage that would logically be considered to originate from the adjacent tailings dam. It is however emphasized that no studies have been conducted to date to confirm the origin of this seepage.

5.1 Groundwater seepage

Groundwater seepage occurs between 1.3m and 2.5m depths. A drainage ditch is present along the toe of the tailings dam embankment; designed to capture the surface run-off from the embankment and re-direct this to the return water dam. This surface water in the ditch was sampled and tested. Results indicate the water is very aggressive. Although the surface run-off from the tailings dam is intended to be intercepted by this drainage ditch, it is assumed that some infiltration of this aggressive water does occur, and that is contributes to the sub-surface flow, which migrates towards the substation.

5.2 Effects of aggressive ground conditions

The conditions encountered on site are very highly aggressive according to Basson (1989). These aggressive ground conditions can cause gradual deterioration or decay on concrete
resulting in weakening of structures with time. This can cause variety of problems in concrete structures, from performance problems, reduced lifespan of structures and structural failures. While the effects of corrosion are understood, the timespan for the effects to manifest themselves are less well documented. There is therefore a great opportunity with the removal of the original concrete pylons, to investigate the time-effects of this corrosion, and to calibrate the understanding of the future behaviour of the substation footings. This is further discussed in section 7 below.

5.3 Protective measures
According to BRE specifications (2001), coating of concrete and copper for these specific aggressive ground conditions would have been preferred. However, for this substation the footings are already in the ground and coating is not a practical protective measure. Alternative protective measures comprising construction of a physical barrier between the substation and the tailings dam and monitoring of the already contaminated ground on the substation footprint are therefore recommended. These measures are discussed below.

The recommended impermeable barrier consisting of a subsoil drain combined with an impervious liner will need to be installed on the upstream side of the substation as indicated in drawing (Figure 1). The barrier will likely need to be keyed into the underlying bedrock (typical bedrock depth ranges between approximately 1.5m and 2.5m depth) in order to intercept the groundwater flow effectively.

Figure 1. The proposed impermeable barrier in red and the adjacent tailings dam in blue

This barrier must be able to withstand the aggressive conditions, according to the BRE specifications. Layout shown in the drawing is subject to final confirmation after additional geohydrological investigations. According to BRE specifications (2001) the main requirements of the physical barrier are:

- To provide an impermeable barrier;
- To be resistant to sulphates and other deleterious chemicals;
- Have a neutral effect on the concrete substrate;
- To be resistant to mechanical damage;
• To be easy to construct;
• Have long term durability; and
• To be cost effective.

Intercepted contaminated surface and groundwater will then be diverted to the existing return water dams adjacent to the tailings dam. This is to avoid further environmental issues, and also to avoid the client being liable for collection and treatment of this water prior to re-release. If the origin can be confirmed as the tailings facility then diversion of the intercepted water to the existing return water dams, by gravity, would be favoured. One these water dams is located to the east, near the N17 and N1 intersection about 0.7km form the substation and the other is located south of site at the corner of the tailings dam approximately 1.2km from the substation.

5.4 Remediation and monitoring of substation
Due to the soils above quartzite bedrock being highly permeable, no remediation of the already contaminated ground within the substation is considered at this stage, i.e. after construction of the physical barrier. The view is taken that after measures are introduced to intercept the seepage, successive rainy seasons will likely dilute the aggressive leachate in the substation footprint, which might further be expected to migrate with groundwater from the substation.

It would be necessary to institute regular monitoring to continue testing the aggressiveness of water and soil within the substation footprint after the barrier has been constructed. The monitoring will be in a form of piezometer installation on all platforms as well as on the existing substation. The piezometers must have a lockable cap for security. Due care shall be required during installation to prevent damage to earth mat. Cathodic protection will be recommended should aggressive conditions persist after this monitoring period.

5.5 Environmental consideration
The proposed subsurface drainage system will intercept the aggressive groundwater upstream of the substation. Formal evaluation of the legal ramifications lie beyond the scope of this geotechnical report. Mention is made however of possible issues that may arise as a result of the contaminated groundwater being extracted from below surface and then being released at surface, beyond the substation. It is assumed that the seepage originates from the tailings dam, and for this reason it is assumed that the best option would be to divert this intercepted water back to the existing return water dams associated with the tailings dam. Two options appear to exist in this regard; but a decision as to the optimal solution would be dependent on a detailed survey. Interaction with the Owner of the tailings dam would also be essential.

6 Conclusions
• While the concept of the physical barrier has been explained this does not constitute a detailed design. It is necessary that a groundwater investigation be conducted to confirm the subsurface flow details, specifically the direction of flow. This investigation is necessary in order to finalise the depth and layout of the geomembrane barrier for optimal performance.
• A further important reason for this hydrological investigation would be to confirm the origin of the seepage water as being the tailings dam. This is a crucial step that would be required prior to consideration of diversion of the intercepted water back to the existing return water dams. Even so, negotiations with the owner of the tailings facility would have to follow. If the intercepted water cannot be diverted back, the client would be liable for collection and treatment of this water prior to re-release, and this is not considered a favoured outcome.
• A detailed topographical survey would be required in order to evaluate the feasibility of gravity feed of the diversion of this intercepted flow to the return water dam/s, and also which of the two options would be favoured.

• It is understood that the original pylons adjacent to the substation will later be redundant and will be removed. While the effects of corrosion are understood, the timespan for the effects to manifest themselves are less well documented. There is therefore a great opportunity with the removal of the original concrete pylons, to investigate the time-effects of this corrosion, and to calibrate the understanding of the future behaviour of the substation footings.

• It is therefore recommended that the old pylon footings are investigated in detail after removal, including aspects of reduction in concrete strength, deterioration of reinforcing bar, as well as the overall concrete condition, including potential honeycombing etc. A detailed microscopic and chemical study, as well as strength testing of concrete cores is recommended. A detailed testing programme can be defined in this regard.

• A further consideration is that, the aggressive groundwater problems investigated here are not unique to this substation. Similar conditions are likely to affect a significant proportion of client’s infrastructure in close proximity to other tailing dams and old mines. It is therefore recommended that the findings of this investigation be rolled out to the other elements of infrastructure that are deemed to be at risk, and that these elements are investigated to confirm the effects of such aggressive soil and water conditions.

References


Investigating the tensile behaviour of unsaturated gold tailings using the Brazilian Disc Test

T. A. V. Gaspar¹

University of Pretoria, Pretoria, Gauteng, tav.gaspar@gmail.com

Abstract

This study investigates the tensile behaviour of unsaturated gold tailings across a range of moisture contents. Unsaturated samples were tested using a modified version of the Brazilian Disc Test used in conjunction with Digital Image Correlation to measure surface displacements in samples as they were loaded. Results of the study indicated that while conventional interpretation of the Brazilian Disc Test is adequate for brittle materials, at higher moisture contents samples tended to illustrate a more complex behaviour. For ductile responses, it was found that blindly adopting the maximum achieved load to calculate tensile strength can often result in a gross overestimation. Furthermore, it was found that the relationship between tensile strength and moisture content was qualitatively similar to the material’s Soil Water Retention Curve (SWRC), with matric suction providing a diminishing contribution to strength at lower moisture contents.

Keywords: unsaturated soils, tensile strength, Brazilian Disc Test, Soil Water Retention Curve, Digital Image Correlation

1 Introduction

In geotechnical engineering practice, it is generally assumed that in the absence of cementation and particle interlock, soils possess little to no tensile strength. However, with the development of unsaturated soil mechanics, it has become increasingly recognised that partially saturated soils can develop a significant amount of tensile strength which can be important to consider in certain applications. Typical engineering problems which are dependent on the tensile strength of unsaturated soils involve collapsible soils, the stability of soil arching over a cavity (an aspect particularly important for the study of sinkhole formation), temporary excavations, bearing capacity and slope stability analyses.

The aim of this study was to investigate the tensile behaviour of an unsaturated silty sand (gold tailings) at varying degrees of saturation. Tensile testing was performed using a modified version of the Brazilian Disc Test (BDT) used in conjunction with Digital Image Correlation (DIC) such that surface displacements could be monitored in the sample throughout loading. Finally, the interdependency of matric suction and tensile strength are reviewed by considering the development of matric suction with a change in moisture content.
2 Background

2.1 Tensile strength in unsaturated soils

In contrast to saturated soils, partially saturated soils are primarily characterised by the simultaneous presence of water and air in the soil matrix (Lu and Likos, 2004). The existence of the third air phase results in the manifestation of capillarity, a term used to describe the combined effect of matric suction, acting across the air-water interface and surface tension which acts along it. A graphical illustration of the above two mechanisms has been provided in Figure 1.

![Figure 1. Dominant mechanisms contributing to inter-particle tensile strength](image)

It is however important to recognise that the amount of inter-particle strength generated is not merely a function of the magnitude of matric suction, but also of the contact area over which matric suction and surface tension act. The dependency of these two phenomena on contact area is most pronounced at lower moisture contents. At low degrees of saturation, the magnitude of matric suction will reach its highest values. However, due to the reduced volume of the liquid bridges in the soil matrix, the inter-particle forces of attraction will tend to diminish. In addition to the reduction in the magnitude of these inter-particle forces, a reduction of moisture content also results in a reduced number of liquid bridges (inter-particle forces) within the soil mass (Gallapoli et al., 2003).

Recognising the dependency of unsaturated soil strength on the degree of saturation, partially saturated soils are generally considered to exist in one of three states. The capillary regime is a term used to describe the portion of soil located above the phreatic surface which is kept saturated due to the presence of a negative pore water pressure (commonly referred to as the capillary fringe) (Lu and Likos, 2004). In the capillary regime, the only point where soil-water comes into contact with the air-phase is at the boundary of the soil mass. This has an important implication on the inter-particle attractive forces since the contribution of surface tension is only present at the boundaries of the soil matrix. In its body, however, only negative pore-water pressure contributes to tensile strength. Within the capillary regime, the water content of the soil will remain relatively constant (close to the saturated water content) with an increase in suction. Suction will continue to increase until the air-entry pressure of the soil is reached. At this point, air begins being sucked into the macropores of the soil structure. The soil then begins to desaturate and transition into the funicular regime. In the funicular regime, the water phase is continuous and exists both as liquid bridges as well as pores entirely filled with water. Finally, if the soil is dried out further to its residual moisture content, a steep increase in matric suction will be observed for very little changes in saturation. In this state, referred to as the pendular regime, the water phase becomes discontinuous and exists primarily as thin films surrounding individual soil particles (Kim and Sture, 2008).
The research conducted in this study was aimed at investigating, as far as possible, the tensile behaviour of a silty sand (gold tailings) across all three saturation regimes.

2.2 Tensile testing of unsaturated soils

While matric suction and surface tension are phenomena which govern many characteristics of partially saturated soils, it is their combined effect that brings about a mechanical property which bears practical significance to geotechnical engineering, namely tensile strength. Recognising its importance, many testing methods have been proposed to measure the tensile strength of unsaturated soils. A method to measure the tensile strength of cohesive materials was proposed by Heibrock et al. (2003). The procedure involved loading a vertically aligned tubular sample until failure occurred at its centre. While this measurement technique was found to be useful for the testing of cohesive soils, it has been highlighted that to measure the tensile strength of a granular material, a certain amount of sample confinement is required (Lu et al., 2005). Another method used to test unsaturated compacted clays was proposed by Stirling et al. (2015). The approach incorporated the use of a modified direct shear apparatus as illustrated in Figure 2. Similar testing methods using ‘bow-tie’ shaped specimens have also been used for the testing of unsaturated sands (Kim and Sture, 2008).

![Tensile testing equipment proposed by Stirling et al. (2005)](image)

Figure 2. Tensile testing equipment proposed by Stirling et al. (2005)

An issue related to these direct testing methods is linked to the volume of material required for each test. Due to the large sample sizes, it becomes difficult to repeatedly prepare multiple specimens at consistent void ratios. Furthermore, the time required to establish equilibrium of moisture content and matric suction increases significantly for larger samples. Due to the abovementioned shortcomings, it was decided that a modified version of the Brazilian Disc Test (BDT) be utilised for this study. Originally used for the testing of concrete, the BDT is an indirect method which involves loading a disc specimen along its vertical axis, resulting in the development of horizontal tensile strain in the perpendicular direction. The tensile strength of the material is then calculated from an elastic solution originally proposed by Timoshenko (1934) as provided in Equation 1:

\[
\sigma_t = \frac{2P}{\pi DL}
\] 

(1)
where $P$ is the measured failure load and $D$ and $L$ are the diameter and length of the specimen respectively. Due to its relative simplicity, the BDT is a test which has gained substantial popularity over a range of disciplines, being used to measure the tensile strength of rock, concrete and weakly cemented soils. However, despite its widespread usage, it is a test which has often been criticised. According to the Griffith failure criterion, cracking of the sample must initiate at its centre for the result to be indicative of a tensile failure (Erarslan et al. 2012; Li and Wong 2013). Despite this crucial condition for the success of a BDT, is has repeatedly been observed that failure of the sample often begins elsewhere. A study by Fairhurst (1964) revealed that for small contact angles, failure would occur away from the centre of the sample. Similarly, Hudson et al. (1972) indicated that cracking would always initiate directly under the loading points when flat loading platens were used. For these reasons, extensive research has been done on investigating the effect of contact conditions on the observed failure mechanisms in BDTs. One such study conducted by Erarslan et al (2012) investigated the effect of loading conditions for BDTs performed on Brisbane tuff. The study revealed that the use of 30° curved loading strips consistently resulted in the finest, most centrally located cracks. Subsequent studies conducted by Gaspar (2017) have found this modification to be equally appropriate for the testing of various unsaturated soils. As a result, this study incorporated the use of 30° curved loading strips as a modification to the conventional BDT. Detailed descriptions of the setup have been provided by Gaspar (2017).

3 Experimental procedure

The experimental work carried out for this study involved the testing of unsaturated gold tailings across a range of moisture contents, using a modified version of the BDT. The tensile testing was performed in conjunction with Digital Image Correlation (DIC) which was utilised to examine surface displacements of the samples as they were loaded. Figure 3 illustrates this test setup.

Figure 3. Test setup as viewed from the a) front and b) side (including positioning of camera)

In addition to the tensile testing performed, the Soil Water Retention Curve (SWRC) of the soil was measured. The purpose of measuring SWRCs was to provide a qualitative indication as to the variation in matric suction with moisture content as well as to approximately highlight the transition between saturation regimes. It should however be noted that the initial density of the samples used to measure the SWRCs was not equivalent to all the disc samples tested in tension. Recognising the significant dependence of a soil’s moisture retention properties on its initial density, quantitative correlations between the results of these tests were not made.
The approach followed for the measurement of SWRCs was based on the MIT technique, proposed by Toker et al. (2004). The procedure involved preparing a sample at $S_r = 100\%$ with a high capacity tensiometer embedded in it. The specimen was then placed on an electronic balance and allowed to dry out under controlled conditions, with matric suction and mass being recorded continuously for the duration of the test (mass readings were used to back calculate the changing gravimetric moisture content of the sample). This technique was implemented up until cavitation of the tensiometer had occurred. To measure suctions exceeding the cavitation pressure, the filter paper technique, originally proposed by Gardener (1937) was used.

3.1 Materials

The material considered for this study classified as a silty sand according the Unified Soil Classification System – USCS (ASTM Standard D2487. 2011) and was sampled from a gold tailings storage facility in South Africa. Figure 4 presents the Particle Size Distribution (PSD) and Soil Water Retention Curve (SWRC) for the soil. It is worth highlighting that the retention properties of an unsaturated soil are dependent on particle size distribution, and thus pore sizes of that soil. This interdependency of material properties is reflected in Figure 4 where it can be seen that for the relatively uniformly graded gold tailings, the development of matric suction in the funicular regime remains relatively constant.

![Figure 4. a) PSD and b) SWRC for gold tailings](image)

3.2 Sample preparation procedure

The parameter varied in this study, was the gravimetric moisture content of the respective samples. Each specimen was prepared from a reconstituted slurry. The slurry was deposited into a mould in three separate layers and vibrated for a period of 20 seconds after the placement of each layer to allow even distribution in the mould. Chang et al. (2011) stated how the use of moist tamping results in a flocculated fabric, in contrast to slurry deposition which was found to produce a relatively uniform fabric (a desirable property for this study).

After placement of the slurry into the moulds, samples were placed in an oven at 65°C and allowed to dry. During the drying stage, samples were periodically weighed such that the amount of moisture loss (and thus gravimetric moisture content) could be monitored. Once a sample had reached the desired moisture content, it was demoulded and trimmed to a final height of 25 mm. The mould and hence sample diameter was 50 mm. Samples were then wrapped with a combination of cling film and aluminium foil (as suggested by Heymann and Clayton (1999)) and left to equilibrate for a period of approximately 16 hours.
4 Results

4.1 Brazilian Disc Tests

Due to the large fraction of sand sized particles in the gold tailings, the magnitude of matric suction that could be developed in the capillary regime was minimal. As a result, samples prepared close to their saturated moisture contents would simply collapse under their own weight. Due to this limitation, testing could only be performed on samples in the pendular regime, and the dry and wet sides of the funicular regime. Figure 5 presents the results of three tests conducted at varying moisture contents.

![Figure 5. Tensile stress vs compression relationship for gold tailings across different saturation regimes](image)

An investigation of the results presented in Figure 5 reveal that for each test, three distinct changes in behaviour are present. Each stress vs compression curve displays an approximately linear relationship up until a point of inflection is reached. Following the point of inflection, each curve tends to flatten, up until it reaches a plateau/initial peak stress. Finally, the results illustrate an increase in stress up until the peak load is reached, after which the load drops rapidly. Conventional interpretation of BDTs dictates that the maximum load should be adopted when calculating tensile strength. However, due to the observed changes in behaviour occurring before this maximum value, a more detailed analysis of sample behaviour was warranted.

Figure 6 presents the stress vs compression results, together with a strain analyses performed using data obtained from DIC measurements. The results in Figure 6 specifically highlight the horizontal tensile strain in the sample, at each of the three changes in behaviour mentioned above. Sub-plot a) in Figure 6 presents the stress vs compression results, with the development of horizontal tensile strain plotted on a secondary vertical axis. Sub-plot b) illustrates the distribution of horizontal tensile strain along the loaded axis of the sample. Finally, sub-plot c) has been included to illustrate the state of the sample at each of the three stages of the test, with contours of horizontal tensile strain superimposed onto each image. From the results in Figure 6 c), it is seen that Point 2 is the first instance where surface cracking, initiating at the sample centre, becomes visible to the naked eye. At this point, the sample had begun to split at its centre. According to the Griffith failure criterion, the exact centre of the disc is the unique point at which conditions for tensile failure equal to the uniaxial tensile strength are met (Li and Wong, 2013).
Furthermore, at this stage, the conditions for elastic strain compatibility are no longer met and thus the elastic solution provided by Equation 1 becomes invalid. However, if the results in Figure 6 are investigated, it is seen that the stress vs compression curve remains approximately linear up until the point of inflection (Point 1). A similar relationship observed in concrete was described by Karihaloo (1995) from the perspective of fracture mechanics. Karihaloo (1995) described how the non-linearity in a load-deformation curve following this point of inflection was due to the formation of microcracking. Another characteristic which can be identified from Figure 6 a) is that Point 1 corresponds with the onset of horizontal tensile strain across the centre of the disc. At this stage of the test the sample has reached a state of first yield, prior to which tensile splitting cannot occur. Following first yield, progressive failure of the sample ensues and micro-cracks begin to grow and coalesce until a surface crack becomes visible. Application of Equation 1 for the calculation of the mobilised tensile stress beyond first yield is therefore likely to become increasingly inaccurate. For this reason, it was deemed more appropriate to adopt the stress at first yield as a conservative estimate of tensile strength for the results presented in Figure 5 and Figure 6.

To emphasise the danger of adopting the maximum load as being indicative of the tensile strength of a weak and ductile material, consider the results for a test conducted at a moisture content of w = 12%. Figure 7 illustrates the stress vs compression results, together with an image of the sample some time before the peak load had been reached. The results in Figure 7 illustrate a case where, at approximately 1.5 mm of vertical displacement, only half the sample remained between the loading strips. Despite this, the measured load continued to increase significantly. It is thus clear that the measured increases in load following this stage of the test were not due to tensile splitting, but to the compression of one half of the sample. This result
strongly emphasises the magnitude of error associated with blindly adopting the maximum load to calculate the tensile strength of a ductile material.

![Figure 7](image1.png)

Figure 7. Tensile stress vs displacement for sample at w = 12%

From the results shown in Figure 5, it is seen that the material behaviour of gold tailings remains relatively constant for the moisture contents considered, with all tests illustrating a ductile response. Only after the material had been dried out almost completely, was a change in behaviour observed. Figure 8 illustrates the result of a sample tested at w = 0.2%.

![Figure 8](image2.png)

Figure 8. Tensile stress vs displacement vs tensile strain for a sample at w = 0.2%

While the wetter samples tested were found to exhibit a relatively ductile response, a distinct transition to brittle behaviour was observed for tests conducted at low moisture contents. From Figure 8 it is seen that the stress vs compression curve rises at an approximately linear gradient to the maximum load. Furthermore, the development of horizontal tensile strain remains unchanged until the peak stress is reached. The above result illustrates that, while sample ductility can complicate test interpretation, for brittle responses, the maximum achieved load is representative of a splitting failure in the disc specimen.
By interpreting test results using the criteria set out above, for ductile and brittle responses, BDT results across a range of moisture contents were analysed. Figure 9 illustrates the measured tensile strengths as a function of gravimetric moisture content. Furthermore, the SWRC measured for gold tailings has been reproduced in Figure 9 such that qualitative correlations can be made between the variation of tensile strength and matric suction with moisture content.

The results presented in Figure 9 highlight how measured tensile strengths remained constant for samples tested in the funicular regime, increasing slightly in the pendular regime, mirroring similar trends in the variation in matric suction in the soil’s SWRC. The amount of strength increase in the pendular regime is however significantly less than the matric suction increase over the same range. This trend emphasises the diminishing contribution of matric suction to overall soil strength.

5 Conclusions

The results presented in this study illustrate that, while conventional interpretation of the Brazilian Disc Test was adequate for brittle materials, at higher moisture contents samples tended to illustrate a more complex behaviour. For ductile responses, it was found that blindly adopting the maximum achieved load to calculate tensile strength can often result in a gross overestimation. It is therefore proposed that the initial inflection point in the stress-compression curve, referred to here as the stress at first yield, be used to provide a conservative estimate of ductile samples’ tensile strength. Additionally, it was found that the relationship between tensile strength and moisture content was qualitatively similar in shape to the SWRC of the soil.

References


Laboratory Measurement of Rock Joint Stiffness by Experimental Techniques

A. W. Bezuidenhout

1SRK Consulting Pty (Ltd), Johannesburg, Gauteng, bezuidenhoutalvino@gmail.com

Abstract

This paper presents an overview of theoretical and experimental techniques used to compute joint stiffness. Currently, there are no accurate, conventional techniques used to measure or compute joint stiffness. The author discusses a micro-mechanical approach and successful laboratory techniques used to measure joint stiffness. An innovative experimental approach designed by the author is proposed to be adopted as a conventional joint stiffness testing technique. The approach makes use of static and dynamic techniques that requires Digital Image Correlation (DIC) and Seismic wave transmission respectively. Both static and dynamic approaches are discussed in detail. The author’s experimental design is presented as a recommendation for future research.

Keywords: joint stiffness; digital image correlation; seismic wave transmission.

1 Introduction

Natural rock masses typically consist of a network of intact rock blocks separated by discontinuities such as joints, fractures, faults, bedding planes and other geological features. Generally, discontinuities reduce the elastic modulus and strength of a rock mass, potentially resulting in instability (Li et al, 2010), thus playing an important role in the way most rock masses deform and fail under loading (Resende, 2010).

This paper will include a literature review of joint stiffness computing and testing techniques, as well as outline a potential joint stiffness testing procedure. Joints are brittle fractures within rock masses and represent planes of weakness in rock masses. The three types of joints are closed-, open- and filled joints, as shown in Figure 1.
2 Review of current joint stiffness measurement techniques

A literature review of two current joint stiffness measurement techniques was completed and is outlined in the following section. The techniques discussed are considered to be potential options in the approximation of rock joint stiffness.

2.1 Joint stiffness estimated from rock mass properties

Normal and shear joint stiffness can be estimated using the rock mass modulus, intact rock modulus and joint spacing. This approach assumes that the deformability of the joints and intact rock bridges influence the deformability of the rock mass. The method requires that the joints are orientated normal to the direction of loading and are spaced at distance, \( L \), (from Rocscience, 2016). The normal and shear joint stiffness are calculated using equations 1 and 2, respectively, as shown below:

\[
K_n = \frac{E_i E_m}{L (E_i - E_m)} \quad \text{(1)}
\]

\[
K_s = \frac{G_i - G_m}{L (G_i - G_m)} \quad \text{(2)}
\]

Where:

- \( K_n \) = normal stiffness
- \( K_s \) = shear stiffness
- \( E_i \) = intact rock mass modulus
- \( E_m \) = rock mass modulus
- \( G_i \) = intact rock shear modulus
- \( G_m \) = rock mass shear modulus
- \( L \) = average joint spacing

The method described above does not directly account for joint roughness, aperture, infill or strength, and the application is limited to the loading conditions described above, and thus are considered to be only rough approximations of the true joint stiffness.

2.2 Joint stiffness estimated from joint infill properties

The following method considers the properties of joint infill to approximate joint stiffness and is only applicable to filled joints. Infill properties of the joint are considered in equations 3 and 4 in order estimate the stiffness of the joint. The method considers the following assumptions (from Rocscience, 2016):
Joint infill deforms elastically; and
Joint stiffness can be directly related to the elastic properties of the joint infill.

\[ K_n = \frac{E_o}{h} \]
\[ K_s = \frac{G_o}{h} \]

Where:
\( E_o \) = young’s modulus of infill material
\( G_o \) = shear modulus of infill material
\( h \) = aperture (joint thickness)

2.3 A Micro-mechanical model computing joint stiffness

Pande (2011) presented a model for the computation of normal and tangential elastic stiffness of rock joints. The model assumed that the asperities on a rock joint are hemi-spherically shaped, as shown in figure 2, and may have normal or subnormal (<90°) orientation relative to the joint plane.

The micro-mechanical model demonstrated that the stiffness of a rock joint is dependent on the size, orientation, density, and mechanical properties of the asperities. Equations 2.5 and 2.6 were considered for the computation of normal and tangential stiffness in the cases where asperities were normal and subnormal to the joint plane.

The following assumptions hold where the asperities are normal to the joint plane (\( \theta = 0 \)):
- The size of asperities also determines their density (number of asperities per unit area of joint)
- Asperities on both faces of a joint mate perfectly
- Asperities transmit normal and tangential forces on the joint; therefore, contribute to the normal and tangential deformations attributed to the rock joint. The contribution of all the asperities are summed to obtain the stiffness of the joint.

Pande (2011) presented the closed form solution for normal and tangential (shear) stiffness for two elastic spheres as shown in equations 5 and 6 respectively.
\[ K_n = \left[ \frac{\partial rG^2}{(1-\nu)^2} F_n \right]^{\frac{1}{3}} \]  
(5)

\[ K_s = \frac{2(1-\nu)}{(2-\nu)} \left[ 1 - \frac{F_s}{F_n \tan \phi} \right]^{\frac{1}{3}} K_n \]  
(6)

Where:
- \( r \) = average radius of asperities
- \( G \) = elastic shear modulus of rock material (reduce \( G \) for weathered rock joints)
- \( \nu \) = poisson’s ratio of rock material
- \( \phi \) = Inter-asperity friction angle
- \( F_n \) = applied normal force to the joint
- \( F_s \) = shear force to the joint, \( F_s \leq F_n \tan \phi \)

Assuming there are \( N \) equal hemispherical asperities and that the asperities act as springs in parallel, the stiffness of the joint calculated using equation 7 shown below:

\[ K_{\text{Joint}} = \Sigma_N K_a \]  
(7)

Pande (2011) extended this formulation to the Global Cartesian plane (n’, s’, t’) where (n’, s’) define the axes of the joint plane shown above in figure 2. By resolving contact forces to contact stresses, joint stiffness computed as shown in equations 8, 9 and 10.

\[ K_n = \left[ \frac{\partial nG^2}{(1-\nu)^2} r \left[ \frac{\sigma_{nr}}{N} \right]^{\frac{1}{3}} \right]^{\frac{1}{3}} \]  
(8)

\[ K_s = \left[ \frac{2(1-\nu)}{(2-\nu)} \left[ 1 - \frac{\tau_{sr}}{\sigma_{nr} \tan \phi} \right]^{\frac{1}{3}} K_n \right]^{\frac{1}{3}} \]  
(9)

\[ K_t = \left[ \frac{2(1-\nu)}{(2-\nu)} \left[ 1 - \frac{\tau_{tr}}{\sigma_{nr} \tan \phi} \right]^{\frac{1}{3}} K_n \right]^{\frac{1}{3}} \]  
(10)

Conclusively, Pande (2011) demonstrated that the normal and tangential (shear) stiffness is non-linearly stress dependent and through a simplified model, stiffness is a function of the radius and density of asperities. The model is useful when analyzing jointed rock and it is not possible to perform physical tests.

2.4 A Joint Stiffness Mechanical Model by Zangerl et al (2008)

In contrast to the model proposed by Pande (2011), Zangerl et al (2008) used a macro-mechanical model to estimate normal stiffness in granitic rock. The procedure does not take into account any joint characteristics, but rather uses the macro-mechanical behaviour of the joint to compute normal joint stiffness. Zangerl et al (2008) applied a normal force across a rock sample and presented the change in mechanical aperture as a function of the change in applied normal stress.

The Semi-Log Normal Closure Law defines the mechanical behaviour of the fracture mathematically. The model assumes that a linear relationship between normal stiffness (\( k_n \)) and effective normal stress (\( \sigma_n' \)) exists. The gradient of this linear relationship is a constant, referred to as the stiffness characteristic. The change in aperture of the joint is a function of the stiffness characteristic, shown by equation 11:
\[-\Delta \alpha_m = \frac{1}{\sigma'_{n} \partial K'_{n} / \partial \sigma'_{n}} \ln \left( \frac{\sigma'_{n}}{\sigma'_{n \text{ref}}} \right) \]  

(11)

Where:
\(\Delta \alpha_m\) = change in mechanical aperture (resulting from change in effective normal stress)
\(K_n\) = normal stiffness
\(\sigma'_{n}\) = effective applied normal stress
\(\partial K'_{n} / \partial \sigma'_{n}\) = stiffness characteristic (constant)

The stiffness characteristic is determined by linear regression using \(\sigma'_{n}\) and \(\Delta \alpha_m\) data. The constant can be calculated for individual rock types and used in the stress sensitive normal stiffness equation 12:

\[K_n = \left( \frac{\partial K'_{n}}{\partial \sigma'_{n}} \right) \sigma'_{n} \]  

(12)

Zangerl et al (2008) concluded that the Semi-Log Normal Closure Law was adequate to describe the closure behaviour of a fracture. The law requires one free parameter, the stiffness characteristic, to estimate the normal stiffness at an arbitrary normal stress value.

### 2.5 Barton-Bandis Joint Model (1985)

The Barton-Bandis model considers a non-linear relationship between joint stiffness and normal stress. Bandis et al (1985) investigated rock deformation characteristics under conditions of normal and shear loading – with joint stiffness calculated using equation 13 and 14:

\[K_n = K_{ni} \left( 1 - \frac{\sigma_n}{V_mK_{ni} + \sigma_n} \right)^{-2} \]  

(13)

And

\[K_{ni} = -7.15 + 1.75JRC + 0.02 \left( \frac{JCS}{a_j} \right) \]  

(14)

Where:
\(K_n\) = normal joint stiffness
\(K_{ni}\) = initial joint stiffness
\(V_m\) = maximum joint closure
\(\sigma_n\) = normal stress acting on the joint plane
\(JRC\) = laboratory Joint Roughness Coefficient
\(JCS\) = laboratory Joint Compressive Strength
\(a_j\) = initial joint aperture under self-weight

Normal joint stiffness was calculated as a function of applied normal stress, joint surface characteristics (aperture, joint compressive strength and joint roughness coefficient) and rock mass properties. The results from the study are shown in Figure 3. Bandis et al (1985) confirmed that joint stiffness increases non-linearly with increasing normal stress.
Shear stiffness equations, from Bandis et al (1985), were simplified by Daehnke (1997), by assuming that shear stiffness was only dependent on the Joint Roughness Coefficient (JRC) and normal stress, as shown in equations 15 and 16:

\[ K_t = k_j (\sigma_n)^{n_j} \]  \hspace{1cm} (15)

And

\[ k_j = -17.19 + 3.86 JRC \]  \hspace{1cm} (16)

Where:
- \( K_t \) = Tangential (shear) stiffness
- \( \sigma_n \) = Normal stress
- \( n_j \) = Stiffness exponent = 0.76
- \( k_j \) = stiffness number

### 2.6 Joint Stiffness estimation using Seismic wave transmission

Joint stiffness can also be measured from dynamic testing that involves the propagation of stress waves in jointed rock. Discontinuities in rock affect the propagation of the energy of a stress wave across the medium. An incident wave moving across discontinuities (i.e. such as joints) seldom strikes normal to the discontinuous boundary. Hence, the incident wave will undergo reflection and refraction (Daehnke, 1997). Per Resende (2010), conversion of some of the energy of the wave to sound and heat occurs due to interaction with the discontinuity.

Two important aspects to be aware of regarding the propagation of waves through elastic, isotropic media are:
- The inertial and elastic properties of the medium govern the velocity of the wave.
- In the bulk of an elastic, isotropic body, the two possible modes of wave propagation are P-waves (Dilatational or Primary waves) and S-waves (Distortional or Secondary waves).

The compression and shear wave speeds in the bulk of an elastic, isotropic body are calculated using equations 17 and 18, respectively:
\[ c_p = \sqrt{\frac{E(1-\nu)}{\rho(1+\nu)(1-2\nu)}} \] (17)

\[ c_s = \sqrt{\frac{E}{2\rho(1+\nu)}} \] (18)

Where:
- \( c_p \) = P-wave velocity
- \( c_s \) = S-wave velocity
- \( E \) = Young’s Modulus
- \( \nu \) = Poisson’s ratio
- \( \rho \) = Material bulk density

Ultrasonic waves (\( f \geq 20000 \) Hz) will behave differently depending on the type of discontinuous interface it interacts with. When two rough surfaces are pressed together, they come into contact at the tips of asperities. If an ultrasonic wave is propagated across the interface of the two rough surfaces, the wave will be reflected at regions of no contact (air gaps) and will undergo mode conversion at regions of contact.

The reflection coefficient, \( R \), is the proportion of the wave amplitude (P- or S-wave) being reflected. The Reflection coefficient is computed as a function of the frequency of the incident wave and stiffness of the interface, \( k \). For an interface/contact between two materials with dissimilar acoustic impedances, the reflection coefficient, \( R \), is calculated using equation 19.

\[ R = \sqrt{\frac{(\omega Z_1 Z_2)^2 + k^2(z_1 - z_2)^2}{(\omega Z_1 Z_2) + k^2(z_1 + z_2)^2}} \] (19)

Equation 19 alters slightly in the case where the ultrasonic wave is transmitted through a contact with the same properties (Kendall & Tabor, 1971; Drinkwater et al, 1996), shown by equation 20.

\[ R = \frac{1}{\sqrt{1 + \left(\frac{\omega}{2Z}\right)^2}} \] (20)

Where:
- \( R \) = Reflection coefficient
- \( \omega = 2\pi f \) = Angular frequency
- \( z = \rho v \) = Acoustic impedance
- \( \rho \) = Material density
- \( v \) = velocity

After investigating the reflection coefficient as a function of the frequency of ultrasonic waves, Drinkwater et al (1996), confirmed that Equation 20 shows adequate correlation in computing the stiffness of rough surfaces.

3 Recommended Conventional Experimental Design

3.1 Fundamentals of Soil Dynamics

Navier’s equations are essential to the theory of elasticity in soil mechanics. Navier’s basic equations satisfy equilibrium of stress, strains, and displacements in a linear-elastic continuum under elastostatic conditions. Further derivation of Navier’s equations for the application
involving the propagation of a compression wave in concrete piles of finite length, as seen in figure 4, has the mathematical expression shown by equation 21.

\[ w = f_1(z - ct) + f_2(z + ct) \]  

(21)

Where:

- \( w \) = displacement
- \( f_1, f_2 \) = arbitrary functions
- \( c \) = wave velocity
- \( t \) = time
- \( z \) = length of pile

![Figure 4. Pile of finite length (Verruijt, 2010).](image)

The same approach is relevant to a problem where a compression wave is produced at one end of a non-homogenous pile, shown in figure 5. The pile consists of two materials where the material on the left-hand side and right-hand side have dissimilar magnitudes of stiffness.

![Figure 5. Non-homogenous pile (Verruijt, 2010).](image)

Velocity and stress functions, shown by equations 22 – 25, can be used to develop expressions for reflection and transmission coefficients:

\[ f_1(h - c_1 t) = F_1(t) \]  

(22)

\[ f_2(h - c_1 t) = F_2(t) \]  

(23)

\[ g_1(h - c_2 t) = G_1(t) \]  

(24)

\[ g_2(h - c_2 t) = G_2(t) \]  

(25)

Where:

- \( F_1(t) \) = incident wave function
- \( F_2(t) \) = reflected wave function (reflected at the interface)
- \( G_1(t) \) = transmitted wave function (transmitted at the interface)
- \( G_2(t) \) = reflected wave function (reflected at the end of material 2)

Examining a case where the incident wave interacts with interface and the wave reflecting from the end of the pile \((G_2)\) has not reached the interface yet, then \(G_2(t) = 0\). Now that there are only three unknown variables, \( F_2 \) and \( G_1 \) can be expressed in terms of \( F_1 \), as shown by equations 26 and 27:
\[ F_2(t) = \frac{\rho_1 c_1 - \rho_2 c_2}{\rho_1 c_1 + \rho_2 c_2} F_1(t) \]  
(26)

\[ G_1(t) = \frac{2\rho_1 c_1}{\rho_1 c_1 + \rho_2 c_2} F_1(t) \]  
(27)

This approach relies heavily on the assumption of normal incidence, where the reflection and transmission coefficients are not a function of the incident angle. The reflection and transmission coefficients are calculated using equations 28 and 29 respectively:

\[ R = \frac{F_2(t)}{F_1(t)} = \frac{\rho_1 c_1 - \rho_2 c_2}{\rho_1 c_1 + \rho_2 c_2} \]  
(28)

\[ T = \frac{G_1(t)}{F_1(t)} = \frac{2\rho_1 c_1}{\rho_1 c_1 + \rho_2 c_2} \]  
(29)

However, in the event that the incident waves strike the discontinuity at an angle, the Reflection coefficient is computed using equation 30.

\[ R(\theta_i) = \left( \frac{\left( \frac{\rho_2 \sin \theta_i - \sqrt{(c_1/c_2)^2 - \cos^2 \theta_i}}{\rho_1} \right)}{\left( \frac{\rho_2 \sin \theta_i + \sqrt{(c_1/c_2)^2 - \cos^2 \theta_i}}{\rho_1} \right)} \right)^2 \]  
(30)

There are velocity and stress reflection and transmission coefficients. A representation of the coefficients is shown in figure 6. As the compression waves interacts the interface between the two materials. The stress reflection coefficient may become negative, thus causing a tensile force in the concrete pile.

Figure 6. Reflection and transmission of a shock wave (Verruijt, 2010).

The example of the non-homogenous concrete pile can be applied as an analogy in determining reflection and transmission coefficients in a jointed rock core sample. Figure 7 illustrates the wave interaction with a frictional, cohesive joint (closed joint).
3.2 Digital Image Correlation (DIC)
Digital Image Correlation (DIC) is an optical method that uses a series of images, at multiple stages of deformation, to track moving pixels, in a region of interest (ROI), and computes strains by using algorithms. A high-speed camera is used to monitor the sample. The images are processed by computer software that can produce strain contour maps in any ROI (Hedayat & Walton, 2017).

3.3 Experimental design
The static approach attempts to determine joint stiffness using DIC under quasi-static loading conditions. The sample is placed in a mechanical load press. The macro-mechanical behaviour of the joint could then be used to calculate the joint stiffness. The dynamic approach calculates joint stiffness from ultrasonic wave transmission. Piezo-electric transducers would be used to transmit and detect seismic waves. Digital oscilloscopes display the electrical signals of the waves. The results could then be used to calculate the dynamic stiffness of the joint.

Using these approaches, joint stiffness can be described as a function of joint properties and applied normal stress. A preliminary design of the static and dynamic testing procedures is shown in figures 8 and 9.

Figure 7. Incident P-wave interaction at a cohesive frictional interface (Daehnke, 1997).

Figure 8. Experimental design using only digital image correlation.
4 Conclusions

Future research will involve carrying out the proposed testing procedures. A full review of DIC, seismic wave transmission and results of the proposed tests will be presented in future by the author.

Acknowledgments

Thank you to William Joughin and Robert Armstrong of SRK Consulting Pty (Ltd) Johannesburg for the opportunity to conduct this research, as well as for the guidance and support they provided during my internship with SRK.

References

Daehnke, A. 1997. Stress wave and fracture propagation in rock, Doctoral of Technical Sciences, Vienna University of Technology, Austria.


The Role of Geomorphology in Geotechnical Engineering

N. Mabilo¹, S. Ndlovu²

¹Bear GeoConsultants (Pty) Ltd, Johannesburg, South Africa, neo@bgconsult.co.za
²African Extractive Industries Consultants (Pty) Ltd, Johannesburg, South Africa, sndlovu@aeiconsultants.net

Abstract

Geomorphology, the science and study of earth’s land formations, plays a vital role in the reconnaissance stage of geotechnical site investigations in both fields of Civil and Mining Geotechnics. The geomorphic processes, being either internal/endogenetic or external/exogenetic together with the fluctuating climates to produce complex landscapes. These processes result in the structural history, varied lithology, occurrence of long planation periods, intricate drainage systems. Understanding these landscape formation processes allows for rapid acquisition of data which can permit planning over large areas e.g. for the location, design, and construction of roads. For geotechnical engineering purposes the terrain in an investigation area may be evaluated using two systems; namely the parametric terrain classification and the landscape approach. The latter system allows the classification and mapping of large areas relatively quickly and inexpensively by the interpretation of remote sensing or satellite imagery. This results in the land pattern having a higher level of generalisation regarding the engineering properties of the terrain. Data from terrain evaluation may be used to identify and delineate mapping units resulting in lower levels of generalisation using for example, stratigraphic units and regional stratigraphic units, which may be augmented and interpreted for engineering purposes.

This paper will analyse the geomorphic processes and the outcome of these processes on the acquired data, such data will be classified in the geotechnical site investigation processes and will lead to the final geotechnical recommendations and design.

Keywords: Geomorphological cycle, Terrain classification, Landscape formation, endogenetic, exogenetic

1 Introduction

The soil profile forms the basic unit of foundation material and often also of potential construction material. Such a unit is of interest to any site. The soil profile owes its development and composition to the character of the stratigraphic unit as well as to the geomorphic event which operated locally in the past. The distribution and properties of soils can only be
comprehended in the context of the landscape in which they develop. For example, in sub-
humid tropical environments, on the well-drained upper slopes or crestal areas which have not
been subjected to erosion, leaching tends to produce granular, well-drained soils with a
proportion of voids. Mid-slope zones may be subject to either erosion or deposition and the
degree of leaching may be expected to be less advanced; seasonal groundwater effects may
produce well developed pedogenic horizons in the upper layers of the soil profile. In poorly
drained areas and valleys, groundwater effects are more prominent; the introduction of clay
minerals and the retardation of leaching processes is likely to produce active clay minerals such
as montmorillonite. Within such a succession each zone presents a specific range of engineering
properties, e.g. deep compressible soils often with a collapsible fabric are likely to be associated
with deeply leached upper slopes, and potentially expansive soils with the bottomland areas.
Hard ferricrete for use as road construction materials would be sought on the middle and lower
slopes (Brink et al., 1982). This variation in the composition of the soils depending on their
position on a slope can then allow for targeted investigation sites for finding specific
construction materials etc.

The type of catenary relationship that dominates any particular landscape depends on the degree
to which it has been dissected, on the extent to which the valleys have widened, the climate,
parental material and age of the local planation surface (Brink et al., 1982). These would be
considered exogenetic processes that have resulted in the formation of the soil and landscape
formation. An illustration of this is shown in Figure 1.1 overleaf.

Figure 1. A Catenary sequence of soils developed on granite in a
sub-humid environment (Brink et al., 1982).
2 Geomorphological Processes And Cycles

The two main geomorphic processes are internal, or endogenetic, which originate beneath the earth’s surface and external, or exogenetic which occur at the earth’s surface (Buckle, 1978). King (1975) recognised six geomorphic cycles which are the result of internal endogenetic processes (Brink, 1979). The processes are listed below.

2.1 Internal or Endogenetic Process
Internal or endogenetic processes are further sub divided into two. These are mainly:
- Earth movement, which consists of faulting, folding and warping.
- Vulcanicity or vulcanism

The geomorphic cycles associated with these processes are listed below:

2.2 Gondwana Planation
On the highest parts of the Lesotho and Windhoek highlands, remnants of the ancient pediplain which was inherited from the parent supercontinent are found.

2.3 Post-Gondwana cycle
Dissection during the early Cretaceous represents the first denudational cycle of the independent African continent. Only small areas of the resulting rather variable surface have survived in the present landscape. These areas appear at intervals along the edge of the Great escarpment, notably near the crest of the Drankensberg escarpment and near Windhoek.

2.4 African cycle
Vertical uplift of atleast 1000 m during the early Cretaceous introduced a new base level of erosion. A cycle of erosion which lasted more than 30 million years produced widespread planation of extraordinary smoothness above the level of which a few residual highlands were preserved. Laterite and bauxite, developed in the deep residual soils due to the extreme degree of weathering below this flat surface. Soil profiles belonging to this geomorphic cycle occupy large areas of the subcontinent. These highly leached soils are not very productive and seldom support indigenous trees. The deepest mantles of residual soils are encountered on igneous rocks where the depth to bedrock exceeds 500 m in places. A collapsible grain structure develops in the deep residual granites beneath remnants of this surface. Extensive cavern systems developed beneath the long-static water table in the Malmani and Ghaapplato dolomites. The African cycle has left its legacy of sinkhole development to the present.

2.5 Post-African I cycle
Widespread uplift of a few hundred meters was accompanied in parts of the subcontinent by tilting of the landsurfaces during the early Miocene. The degree of weathering beneath the rolling surface is not as intense as beneath the African surface. Renewed corrosion of dolomite below the static water table produced a further network of caverns below the level of the level of those developed during the African cycle. An example of Post-African I planation residual soils is the Springbok Flats of the Bushveld. These soils pose significant engineering consequence, such as problems associated with expansive clays.

2.6 Post African II cycle.
Following a further uplift of the subcontinent at the end of the Miocene, scarp retreat led to the widening of valley floors and to the development of interior basins and extensive areas of undulating topography in the coastal hinterlands. Examples of this Planation are the Estcourt and Ladysmith basins of Natal. This cycle produced a wide variety of soil profiles which in places contain deep transported soils. Marine transgression during this period carved the coastal plains, such as that of Zululand.
2.7 Quaternary cycle
The most recent upheaval along the escarpment axis, which took place during the early Pleistocene, brought the mountains to their present altitudes. The uplift was accompanied by strong outward tilting of the coastal margins. This resulted in the carving of deep river valleys into the undulating Post-African I landsurface, such as those of the Tugela, Umgeni, Great Fish and lower Orange, which flowed in deep canyons through the tilted continental margins. Formations deposited during this period are the Durnford Formation and the Bluff Formation of the Natal coast. Decalcification and weathering of the upper parts of these and other related formations have given rise in more recent times to the well-known Berea Red Sands.

2.8 External or Exogenetic Processes
External processes are also further divided into two, mainly:
- Denudation or degradation processes, which consist of the destruction of the landscape by works of weathering, mass wasting and erosion.
- Deposition or aggradation processes, which complement of denudation. The eroded material which had been transported by rivers, waves, tides glaciers, ice sheets and wind is layered resulting in the formation of landscapes.

3 Terrain Classification
Understanding geomorphological cycles makes it possible to classify all the terrain of an area into a manageable number of terrain classes and with all the terrain in one class reasonably similar. With this classification then information from any site could also be applied to all other sites in the same terrain class. Provided that the class is sufficiently uniform to permit meaningful extrapolation of information to other sites in it. For geotechnical engineering purposes the terrain in an investigation area may be evaluated using two systems; namely the parametric terrain classification and the landscape approach. The latter system allows the classification and mapping of large areas relatively quickly and inexpensively by the interpretation of remote sensing or satellite imagery. The basic classes must be recognizable on such classes. Due to the scope of this paper only the landscape approach will be discussed (Brink et al, 1982).

The landscape approach recognised terrain classes on their external features and their interrelationships and then describes their properties from the observations at representative sites in each. For example, if rocks such as quartzite are present in the escarpment, it is inferred, without a field visit, that the fan will contain gravelly colluvium. Classes are defined because they are mappable and then their engineering properties are inferred or measured. Two different methods based on the landscape approach have been developed for engineering purposes the land system/land facet classification and the PUCE (pattern-unit-component-evaluation) classification. These classifications are further explained below (Brink et al, 1982).

The basic unit of the land system/land facet classification is the land facet, which is an area of ground with a simple surface form, a specific succession of soil profile horizons and a characteristic groundwater regime. A land facet may be delineated on aerial photographs at scales between 1:10 000 and 1:50 000 and in arid areas it may be possible to achieve delineation on scales as small as 1:80 000. A land system is a recurrent pattern of genetically linked land facets. Usually each land system is dominated by one major geomorphic process. Land systems may be conveniently mapped at scales of 1:250 000 to 1:1 000 000. The boundary between an established and a new land system is recognised when the inter-relationship between land facets changes or the relative sizes of the land facets change significantly or a new land facet or group of land facets occurs in the landscape. Criteria’s in which land systems are distinguished are illustrated in Figure 3.1 below and a block diagram of a land system is illustrated in Figure 3.2 below (Brink et al, 1982).
Figure 2. Some criteria in which land systems can be distinguished (Brink et al, 1982).

Figure 3. Block diagram for the land system (Brink et al, 1982).
The PUCE classification provides an hierarchical terrain classification for engineering purposes on four levels namely the province, pattern, unit, and component. The province is an area of constant geology at group level and its boundaries are delineated either directly from existing geological maps or by airphoto interpretation and geological reconnaissance. The terrain pattern is defined in terms of airphoto pattern and field sampling and is delineated at scales of the order of 1:100 000. Terrain unit are areas occupied by a characteristic association of earthen material with a characteristic vegetative cover. These characteristics are determined chiefly by field study. Terrain units may be mapped at scales of 1:10 000 to 1:50 000. The terrain component is a unit characterised by a consistent soil profile and vegetation association and a constant rate of change of slope. Terrain components are not usually mapped and are not usually associated with any specific terrain unit (Brink et al, 1982).

4 Conclusions

Geomorphology plays an important role in geotechnical engineering site investigation and it allows for terrain evaluation in an investigation area. The geomorphic processes, being either internal/endogenetic or external/ exogenetic together with the fluctuating climates to produce complex landscapes. These processes result in the structural history, varied lithology, occurrence of long planation periods, intricate drainage systems. Understanding these landscape formation processes allows for rapid acquisition of data which can permit planning over large areas e.g. for the location, design, and construction of roads. It is important to note that such data provides a framework and each site still needs to be filled in with information.

References

Geotechnical Properties of the Ground Profile at the Great Dyke of Zimbabwe

B. M. Mmileng¹, H. F. Booyens², H. Gumindoga³

¹SRK Consulting (Pty) Ltd, Johannesburg, Gauteng, BMmileng@srk.co.za
²SRK Consulting (Pty) Ltd, Johannesburg, Gauteng, HBooyens@srk.co.za
³Zimplats, City, Zimbabwe, Howard.Gumindoga@zimplats.com

Abstract

A geotechnical site investigation was conducted to identify soil and rock conditions underlying the new portal site at an established mine in Zimbabwe, in order to determine their engineering properties with respect to foundations for mining infrastructure. The fieldwork included eighty-one test pits and nine rotary core boreholes. The soil profile of the site is typically composed of a surficial transported topsoil layer, which is underlain by a reworked residual clayey soil that transitions into granular soils composed of completely weathered to highly weathered pyroxenite, granite and dolerite or gabbro norite. An alluvial horizon is present in the central portion of the site. Granite underlies the western portion of the site, Gabbro-norite occurs in the eastern portion of the site while pyroxenite, which is occasionally intruded by dolerite, is present across the majority of the site. Representative samples were collected and submitted to a laboratory for geotechnical soil testing. The tested cohesive materials had intermediate to extremely high clay content and plasticity which results in low to very high potential expansiveness. Foundation recommendation was to place heavy structures at the depth of refusal (i.e. soft rock zone). However the depth to bedrock varied considerably and undulating foundation base need to be accounted for. Especially in the gabbro norite zone, where ventilation shafts are planned, competent bedrock is only present as deep as 30 m below ground level and is overlain by a core stone zone, which will be problematic during raise bore drilling.

Keywords: pyroxenite; gabbro-norite; granite, dyke, foundations.

1 Introduction

A geotechnical investigation was conducted at the new portal site at and established mine in Zimbabwe. The main objectives of the investigation was to identify the soil and rock conditions underlying the site in order to determine their engineering properties with respect to foundations for mining infrastructure. The mining infrastructure to be developed comprise an overhead ore loader, surface crusher station and loadback, ventilation shafts, pollution control dams, waste rock stockpiles, access roads and a number of single storey masonry buildings.
The site investigation at the new portal site commenced in February 2016 and included a desk study of relevant information available on the site. The fieldwork comprised intrusive works (rotary core drilling and test pit excavation) in addition to soil profiling and soil sampling across the area.

A total of eighty-one (81) test pits were excavated and a total of nine (9) rotary core boreholes were drilled. The results of all fieldwork data from the investigation and all laboratory data was analysed and evaluated. This paper describes the geotechnical properties of the ground profile as found during the geotechnical site investigation.

2 Site description and geology

According to the published geology map, The Great Dyke underlies the site. The dyke is a layered mafic-ultramafic intrusion into Archean Granites and greenstone belts of the Zimbabwe craton. The dyke is divided into two major successions: a lower Ultramafic Sequence dominated from the base upwards by cyclic repetitions of dunite, harzburgite, pyroxenite and chromitite layers; and an upper Mafic Sequence consisting mainly of gabbro and gabbro-norite. The present site investigation is in agreement with the published geological data as the ground profile of the site is typically composed of a surficial transported topsoil layer that is underlain by a reworked residual clayey soil, which transitions into granular soils. The granular soils are composed of completely weathered to highly weathered pyroxenite (including harzburgite, websterite and dunite rock types), granite, dolerite or gabbro norite. An alluvial horizon is present in the central portion of the site. Granite underlies the western portion of the site. Gabbro-norite occurs in the eastern portion of the site while pyroxenite, which is occasionally intruded by dolerite, is present across the majority of the site. Typical soil and rock horizons that occur on the site can be generally described as follows:

- **Topsoil:** Moist, dark grey to black, soft to firm, micro-shattered, silty clay with abundant roots; or Slightly moist, dark brown or reddish brown, loose, voided, clayey sand with or without fine, medium, coarse grained quartz gravel and cobbles and abundant roots; or Moist, yellow brown, very loose, pinholed, silty sand with abundant roots.

- **Reworked residual:** Slightly moist to moist, dark grey to black, dark brown or reddish brown with or without light grey mottles, firm becoming stiff with depth, shattered, fissured or slickensided, silty clay with or without fine medium coarse grained quartz gravel and cobbles. This soil type is typically referred to as black cotton soil in literature. This horizon is underlain by a transition zone.

- **Transition zone:** Slightly moist to moist, light yellow grey speckled light and dark grey, loose, pinholed, clayey sand. or Slightly moist, dark brown mottled light grey, firm, shattered, silty clay with abundant calcrete nodules; or Slightly moist, dark green grey mottled light yellow grey, firm becoming stiff with depth, shattered fissured and slickensided, silty clay.

- **Alluvium:** Clayey or granular sediment typically underlain by slightly moist, light yellow brown, loose, pinholed and voided, silty SAND with abundant angular to rounded, fine, medium and coarse grained quartz and gabbro norite gravel, cobbles and boulders.

- **Residual Pyroxenite:** Slightly moist, light yellow grey mottled dark grey, loose to medium dense, pinholed, gravelly sand, completely weathered pyroxenite.

- **Residual Granite:** Slightly moist, dark yellow brown stained olive green speckled light yellow brown, medium dense becoming dense with depth, pinholed, silty coarse sand. completely weathered granite.

- **Residual Dolerite:** Dark yellow grey speckled dark grey stained pink, relict textured and structured, gravelly sand completely weathered dolerite.
• **Residual Gabbro Norite:** Slightly moist, light grey mottled dark olive green, very loose to loose, pinholed and voided silty sand with occasional boulders (corestones up to 1.20 m in diameter).

• **Pyroxenite:** Light yellow grey, blotched and speckled dark grey, light grey and olive green, highly to moderately weathered with decomposed zones, coarse grained, inferred highly fractured, very soft to medium hard rock.

• **Harzburgite:** Dark grey, blotched light yellow grey, speckled orange brown and streaked light grey, highly to moderately weathered, fine grained, very intensely to intensely veined (calcite infilled) with very thinly spaced round pyroxene inclusions, highly fractured, very soft to medium hard rock.

• **Websterite:** Light green grey, speckled dark grey and stained light grey, highly weathered with decomposed zones, coarse grained, massive, highly fractured (calcite infilled fractures), soft rock.

• **Gabbro norite:** Light yellow grey, speckled and mottled dark grey and olive green, highly to moderately weathered, coarse grained, very highly fractured, very soft to medium hard rock.

• **Dolerite:** Dark grey to black, speckled dark yellow grey and stained reddish brown, highly weathered to moderately weathered, fine to medium grained, very highly fractured, very soft to medium hard rock.

The soil profiling and core loggings was conducted according to standard methodology provided by Brink and Bruin (2002).

The climatic N-value (Weinert, 1980) of the region is less than 5 and can be described as sub-humid. This implies that chemical weathering is dominant.

3 Laboratory Analysis

The laboratory testing consisted of moisture content, grain size analysis, hydrometer, specific gravity, Atterberg limits, remoulded permeability, California Bearing Ratio, MOD AASHTO and Standard Proctor Compaction Density.

3.1 Foundation Indicator test results

Fifty-one (51) foundation indicator tests were performed and it was found that the completely weathered pyroxenite typically classifies as well graded clay (GC), silty sand (SM), gravel with silt (GM), poorly graded gravel (GP), poorly graded sand (SP) and clayey sand (SC). The material has a low to very low clay content (range 0 to 17%). Linear shrinkage is typically low (6 to 17.4%) and the grading modulus (GM) ranges between 1.24 for the finer soils and 2.47 for the coarser gravelly soils.

The cohesive reworked residual pyroxenite (black cotton soil) classifies as either a silt with high plasticity (MH) or a clay with high plasticity (CH) except for one sample which returned a granular classification. Linear shrinkage is medium to very high with values ranging from 11.8 to 22.6%. The GM of the cohesive material is low, ranging between 0.15–0.78.

Granular residual pyroxenite classifies as either SM, clayey sand SC, well graded sand (SW) or CH. Linear shrinkage is medium to very high with values ranging from 11.1 to 22%. The GM for the granular material range from 1.08–2.55.

The transition zone typically classifies as MH with the exception of one sample which classified as SM. Linear shrinkage is medium to very high with values ranging from 12.1 to 23.7%. Low GM values, ranging from 0.11 to 0.67 were obtained for the silt while the silty sand returned a GM of 1.36.
Topsoil classifies as either MH or SC with low GM values ranging between 0.26 and 0.74. Linear shrinkage is low to medium with values ranging from 8.2 to 15.6%.

Residual granular gabbro norite classifies as silty sand (SM) and clay with high plasticity (CH). The samples with a higher sand content typically showed higher GM values (ranging between 1.07 and 1.10) than the clay having a GM of 0.15. Linear shrinkage is low to medium with values ranging from 3.5 to 13.1% for the granular and cohesive material respectively.

The alluvium classifies as silt with high plasticity (MH), silty sand (SM) and clay with low to medium plasticity (CL). Linear shrinkage is low to high with values ranging from 6.3 to 18.8%. In terms of the GM, the values obtained for the silts and clays ranged between 0.29 and 0.68 and the sand returned a GM of 1.28.

Residual granular granite classifies as clayey sand (SC) and clay with high plasticity (CH). Linear shrinkage is low to medium with values ranging from 6.6 to 11.6% while GM is 1.51 to 1.62 for the granular material and 0.68 for the cohesive material.

Residual dolerite returned a high GM of 2.32 and the material classifies as gravel with silt (GM). The linear shrinkage is high at 18.5%.

The gabbro norite sampled classifies as a silty sand (SM) and returned a GM of 1.55. Linear shrinkage is low at 7.5%.

The Atterberg limits for the fine-grained fractions of the samples are shown in Figure 1.

Figure 1. Plasticity Chart
From Figure 1, the granular, completely weathered pyroxenite plots above (CI, CH and CV) and below (MI, MH and MV) the A-line with the LL ranging between 41 and 78% and thus the plasticity being intermediate, high to very high.

Cohesive reworked residual material plot in the CH to CV and MH and ME zones. Most of the samples plot in the very high to extremely high plasticity ranges with LL ranging from 61 to 116%.

Granular residual pyroxenite plot in the CH and MH, MV and ME zone with LL ranging between 58 and 112% with plasticity being high, very high to extremely high.

Cohesive transition zone material plot in the MH to ME zone, with LL ranging between 50 and 124% with corresponding high to extremely high plasticity.

Granular residual gabbro norite and residual granite plot in a similar CI, CH to MI zone with LL ranging between 44 and 59% for gabbro norite and slightly lower at 36 and 56% for residual granite.

Alluvium plots in the CI to MH and MV zones with a LL of 46 to 90%. The plasticity varies from intermediate, high to very high.

### 3.2 Potential Expansiveness

Each sample was plotted on the Van der Merwe potential expansiveness graph (Figure 2). From Figure 2, it is evident that the materials tested range from low to very high potential expansiveness. Cohesive residual pyroxenite returned low (<2%) to very high (8%) potential expansiveness with the bulk of the samples returning the latter. The granular residual pyroxenite tend to be of low (<2%) to medium (4%) potential of expansiveness while the potential expansiveness of the transition zone varies considerably from low to very high without any specific trends. Topsoil is typically medium to highly expansiveness. The pebble marker returned a medium potential expansiveness and the dolerite showed a low potential expansiveness. The alluvium, residual gabbro norite and residual granite all returned varying degrees of potential expansiveness, but typically show a low to medium potential expansiveness.

### 3.3 MOD AASHTO and Standard Proctor Compaction Density

The Maximum Dry Density (MDD) and Optimum Moisture Content (OMC) relationship were determined for thirteen (13) samples. Three (3) samples of cohesive material were tested using the Standard Proctor compactive effort while the remainder of mostly non-cohesive granular materials were tested using the Modified AASHTO compactive effort.

The residual pyroxenite has an average optimum moisture content of 15.63%, an average maximum dry density of 1845 kg/m³ and an average specific gravity (SG) of 2.64. The MDD of the clayey reworked residual material is generally low as it lies between 1283 kg/m³ and 1472 kg/m³. The behaviour of the material during compaction is typical of highly expansive clays as the MDD of the material is achieved when the material has approximately 10% air voids. This is likely due to the mineralogical nature of the expansive clays that absorbs the moisture added during testing within the crystal lattice and thus the compaction effort is actually working against the pore pressures created by the soil. The MDD of transition zone is generally high (i.e. 2132 kg/m³) while the MDD of the residual granite and the alluvium material is 2039 kg/m³ and 1894 kg/m³ respectively.
3.4 California Bearing Ratio (CBR) Tests

The California Bearing Ratio (CBR) of the material is determined by measuring the load required to allow a standard piston to penetrate the surface of a material compacted to a certain percentage of the MDD. Only five CBR test results were received at the date of the report. From Table 1, none of the samples comply with minimum specification of either the maximum PI of 12 or 3 x GM +10 and/or the minimum CBR level of 7 at 93 % Mod AASHTO and fail the specification for even G9 class materials according to Colto specifications.

Table 1. California Bearing Ratio Test Results Summary

<table>
<thead>
<tr>
<th>ID</th>
<th>Depth (m)</th>
<th>Material</th>
<th>PI</th>
<th>GM</th>
<th>CBR value at % compaction</th>
<th>G-class</th>
</tr>
</thead>
<tbody>
<tr>
<td>BDTP01</td>
<td>2.40-2.80</td>
<td>Completely weathered pyroxenite</td>
<td>25</td>
<td>1.32</td>
<td>17 18 20 21 23 NA</td>
<td></td>
</tr>
<tr>
<td>BDTP02</td>
<td>1.20-1.40</td>
<td>Granular Residual pyroxenite</td>
<td>50</td>
<td>1.08</td>
<td>4 5 5 6 6 NA</td>
<td></td>
</tr>
<tr>
<td>BDTP05</td>
<td>1.40-1.80</td>
<td>Residual pyroxenite</td>
<td>29</td>
<td>1.54</td>
<td>5 6 6 6 7 NA</td>
<td></td>
</tr>
<tr>
<td>BDTP03</td>
<td>1.10-1.40</td>
<td></td>
<td>54</td>
<td>1.70</td>
<td>9 10 10 10 10 NA</td>
<td></td>
</tr>
<tr>
<td>DTP08</td>
<td>0.80-1.40</td>
<td>Pebble marker</td>
<td>36</td>
<td>1.55</td>
<td>6 7 8 9 10 NA</td>
<td></td>
</tr>
</tbody>
</table>

*COLTO Specifications (March, 1998)
3.5 Soil bulk density
Twenty (20) small, undisturbed samples were taken during test excavation to measure the in situ soil density (bulk density) and porosity. The in situ density of the very soft rock pyroxenite varies considerably and the results are probably anomalous, indicative of laboratory error or sample breakage. The average dry density of the granular residual pyroxenite is 1679 kg/m$^3$ but varies considerably between 1380 to 2255 kg/m$^3$. The average dry density of the cohesive reworked residual pyroxenite is low at 1 164 kg/m$^3$, varying between 1140 and 1250 kg/m$^3$. According to Byrne & Berry (2008), for granular soils, an in situ dry density of less than 1450 kg/m$^3$ is indicative of very loose consistency material, while a density of greater than 1 925 kg/m$^3$ is indicative of very dense soils. The average in situ consistency of the granular soils are typically in the medium dense to dense range, which correspond to test pit descriptions.

3.6 Soil Permeability
Three (3) remoulded, cohesive reworked residual soil samples were submitted for falling head permeability tests. The target remoulded density was specified as 95% of the proctor MDD for the sample tested. The average remoulded soil permeability of the cohesive reworked residual material is low, being 1.50 $\times 10^{-8}$ m/s, which correspond to typical textbook reference values for MH (10$^{-7}$ to 10$^{-9}$ cm/s) and CH (10$^{-8}$ to 10$^{-10}$ m/s) soil types (Look, 2007).

4 Discussion and Recommendations
The following section presents a discussion of the geotechnical conditions as observed during field investigation at the various structures being considered for the site as well as the implication of the these conditions for foundation design.

4.1 Overhead ore loader
Given that the expected foundation loads in the loader area is in the order of 350 kPa for typical 1.5x 2 m rectangular footings, the preliminary foundation recommendation is to place foundations at the depth of refusal (i.e. very soft rock zone) which typically occurs at -3.2 to -4.5 m, but may be as deep as -5.2 m. This will ensure that foundations are cast on material with a bearing capacity in the order of 1 to 3 MPa which is about 2.5 to >3 times the required bearing capacity, resulting in a minimum FoS of 2.5.

The depth to bedrock varies considerably in the loader area and care should be taken that corestones are removed from foundation excavations to prevent differential settlement of constructed footings. The varying depth to bedrock may result in an undulating base of foundation and blasting may be required to remove areas of higher elevation or alternatively, low areas may require infill with cement stabilized G4 grade material, to achieve the high bearing capacity requirements.

4.2 Surface crusher station and loadback
Test pit excavation showed that gradual refusal occurred at -3.7 m on pyroxenite bedrock. Rotary core borehole drilling showed that very soft rock pyroxenite was encountered at -2.1, which grades to soft rock consistency at -7.1 m. Depending on the foundation load and dynamic load at the crusher, a conservative foundation depth of -4 m will imply that a safe bearing pressure of around 350 kPa will be achieved. This is based on the ultimate bearing capacity of very soft rock of 1 and 3 MPa and a conservative FoS of 3.

4.3 Ventilation shafts
Refusal of test pits excavated in the area of the ventilation shaft occurred at -1.3m to -3.4 m mostly on gabbro norite boulders. Appropriate foundation depth for conventional single storey masonry structure with a low (< 150 kPa) foundation load is approximately -1.9 m (i.e. below the depth of clayey reworked residual and / or transition zone soils). Alternatively, soil rafts can be considered once an appropriate depth of black cotton soil has been removed. Underlying
granular residual soils may however contain loose consistency zones that may require in-situ densification prior to construction of foundations.

Raise boring could be specified for installation of the proposed ventilation shafts. The biggest risk to construction of the ventilation shaft is the loss of lateral support between the hard rock layers and the collar during the raise bore operations. Without sufficient lateral strength in this zone, failure and breakout is likely to occur that could adversely affect the collar. The soft rock zone present in the boreholes is poses the potential hazard of loose corestones. The depth of this layer varies between -24.6 and >36.8 m. A dolerite dyke is inferred in the rotary core boreholes drilled in the area which is also taken as a potential zone of deficient material. These horizons may become unstable when raise boring is conducted through it as dolerite and gabbro norite boulders are likely to be dislodged during drilling, leaving a very jagged surface which may be prone to failures. Dislodged core stones are also likely to damage the raise bore equipment and prove dangerous to personnel clearing out drill debris below ground during construction.

4.4 Pollution control dams

From the test pits excavated in the dam area it was found that, the dam basin will be relatively easily excavatable to a depth of -2 to -3 m. The cohesive reworked residual material have a low average MDD (approximately 1400 kg/m$^3$) and it is likely that there will not be sufficient cohesive material for dam wall building purposes. The CBR of the completely weathered granular pyroxenite is poor and it is likely that the soil will not have sufficient strength for dam wall building purposes. It is therefore likely that suitable G4 type material will have to be imported for dam wall construction.

If sufficient cohesive reworked residual soil can be stockpiled during dam basin excavations, the material will have an adequately low remoulded permeability to serve as dam liner material. According to the South African Regulations as highlighted by the Department of Water Affairs, any soil used for a compacted soil liner must have a minimum Plasticity Index (PI) of 10 and a maximum that will not result in excessive desiccation cracking. The maximum particle size must not exceed 25mm, and the soil shall not be gap-graded. Clay liners must typically be compacted to a minimum dry density of 95% of Standard Proctor maximum dry density, at a moisture content of within 2% of Proctor optimum.

From the above it is likely that cohesive residual soils will have to be amended with lime to reduce the PI to inhibit desiccation cracking, given the average PI of this horizon of 46.

4.5 Access roads

Given that the access roads will accommodate road freight train ore hauling, care should be taken with road layer works construction as the haul vehicles are heavier than typical traffic. For the granite areas, excavation for spoil of soils is recommended to the depth of completely weathered granular granite (typically starting at 0.55 to 2.65 m bgl). Similarly, in the pyroxenite areas, excavation for spoil of cohesive material overlying granular residual pyroxenite is recommended (typically -1.4 to -3 m depth).

The exposed residual soil should be ripped and then compacted using an impact roller. This will negate the effect of collapsible sand properties observed in the residual soils during test pitting and serve to establish a suitable base for road layer works construction purposes, provided the in situ CBR can be improved to around 10 by means of impact rolling. In areas where thick instances of cohesive soil will need to be removed, consideration should be given to backfilling with compacted dump rock as mass fill to the base of road layerworks.

Similar to the above, given that cohesive soils have tested highly expansive, excavation to spoil of such materials underneath roads are also recommended for the remainder of site access roads.
Where a thick cohesive horizon is present, it is recommended that at least two thirds of the horizon should be removed. Granular residual pyroxenite and / or gabbro norite that is sourced on site should be suitable for selected subgrade provided a CBR of at least 10 at 98% Mod AASHTO density can be achieved and that the PI can be lowered to 12 by the addition of lime.

4.6 General infrastructure
The general infrastructure area where administration buildings, workshops, change rooms etc. are to be constructed, tends to be areas of relatively shallow bedrock (refusal at 1 to 3 m below ground level).

For single storey buildings, the preferred foundation solution is removal of cohesive soils and founding on granular residual material, which has been subjected to in situ densification to negate instances of collapsible soil.

It should be noted that the granular residual soils do classify as medium to highly expansive in some instances and standard precautions pertaining to site drainage (i.e. to keep water away from foundations) should be applied. Similarly, all instances of cohesive soil in foundation trenches should be removed for spoil prior to construction.

5 Conclusions
The area of investigation lies on the Great Dyke of Zimbabwe as evident in the test pits and boreholes. Granite underlies the western portion of the site. Gabbro-norite occurs in the eastern portion of the site while pyroxenite, which is occasionally intruded by dolerite, is present across the majority of the site. The most significant geotechnical constraints that will influence construction on site is difficult excavation due to the presence of corestones and heaving clays.

References