Stability of the temporary excavation between Soldier Piles in Unsaturated Residual Granites in Sandton CBD

F.H. van der Merwe
GaGE Consulting, Johannesburg, South Africa

J.H. Engelbrecht
Franki Geotechnical, Johannesburg, South Africa

ABSTRACT: The design of multi-anchored tieback soldier pile walls in Sandton is typically based on either empirical methods or two-dimensional finite element modelling, ignoring the effects of matric suction in unsaturated residual granites. The designer, however, often relies on previous experience in a similar subsurface profile rather than established theory, to assess the ability of the soil to arch between soldier piles. Using experience rather than a sound theoretical basis for design could potentially be problematic. Excavations typically proceed in intervals between 1.5m and 2.5m before a reinforced shotcrete liner is applied. This differs from the way in which timber lagging is typically installed in smaller excavation lifts in the USA. In this study a typical Sandton residual granite profile is considered, and a matric suction profile estimated. This profile and its apparent derived cohesion are then analysed in three-dimensional geotechnical finite element software, and the temporary stability of the soils between the piles established. This paper will therefore serve as a guideline to eliminate the reliance on "experience" to establish the appropriateness of using a soldier pile wall system in unsaturated soils.

1 INTRODUCTION

Most of the northern suburbs of Johannesburg are underlain by basement granites of Johannesburg Dome. These granites are variably weathered giving rise to an undulating bedrock profile which in Sandton results in deeply weathered soils (typically 10 – 15m). These residual granites typically have a high shearing resistance with effective friction angles in the range of 33-38° and negligible effective cohesion. Despite the insignificant cohesion values the main choice of lateral support system in the Sandton area typically consist either a soil nailed system or a multi-anchored soldier pile wall with shotcrete lagging spanning between the concrete piles. These walls are normally strutted by the building's slabs for the permanent solution. Figure 1 shows the soldier pile wall constructed at The Marc. The construction procedure of these soldier pile walls typically occurs in the following manner: auger piles are first installed at a horizontal spacing of 2.0-3.0m c/c, the excavation then proceeds in lifts ranging between 1.5m-2.5m with tieback elements installed vertically down the pile between 2m-4m c/c. Reinforced shotcrete lagging is constructed between the piles before stressing the tiebacks.

This excavation procedure should be problematic if one considers the zero effective cohesion associated with the residual halfway house granites, unless matric suction and the associated apparent/capillary cohesion in the residual soils are taken into account.

Figure 1. Multi-anchored soldier pile wall at The Marc

The effect of capillary cohesion on such a soldier pile system can be illustrated by an experiment undertaken on clean beach sand placed in a wooden box with a sliding trapdoor and cantilever piles. For the case where oven-dried sands are placed in the box and the sliding trapdoor removed the sand collapses on the angle of repose through the piles (Figure 3), whilst with some water added, the sands remain stable between the piles shown in Figure 2. However, if left to dry the sand will flake and loose its capillary cohesion, once again collapsing though the soldier piles. In a more clayey sand, drying out is more difficult due to the affinity for water by the clay.

Figure 2. Experiment undertaken on moist beach sand illustrating the effect of capillary cohesion.

Figure 3. Experiment undertaken on dry beach sand showing failure will occur through the soldier piles at the angle of repose

2 SOIL PROPERTIES AND CAPILLARY COHESION

2.1 *Johannesburg Dome Residual Granite Properties*

The residual granite is characterised by high effective friction angle ranging from 33-38° with negligible effective cohesion (Day and Schwartz, 1994). The residual sandy material generally classifies as a SC or SM according to the Unified Soil Classification method. The Plasticity Index (PI) of 25 samples on various sites in Sandton were considered and the average was found to be 10% with a standard deviation of 5%. These residual soils are normally in a very loose to medium dense state. The swell/shrink potential from the results are normally low and therefore no significant volume change is expected when drying or wetting the soils in their natural state.

2.2 *The Soil-Water Retention Curve (SWRC)*

The soil-water retention properties of a residual granite were determined using tensiometers for suction values below 1700kPa and a Dew Point Potentiometer for suction values exceeding 1700kPa (Jacobsz, 2018). Table 1 and Figure 4 summarises the measured properties of the residual Sandton granites.

Table 1. Summary of measured soil properties

Soil Properties	Adopted values
Specific Gravity, Gs	2.65
Air Entry value	3kPa
Void Ratio, e	0.516
Dry Density	1750kg/m^3
Effective shear strength: c' and ϕ'	0kPa and 38°
Permeability coefficient, K_{sat}	5.3x10^{-8} to 1.5 x10^{-10}m/s

Fredlund and Xing's (1994) equation was used to fit a SWRC through the measured values as shown in Figure 4.

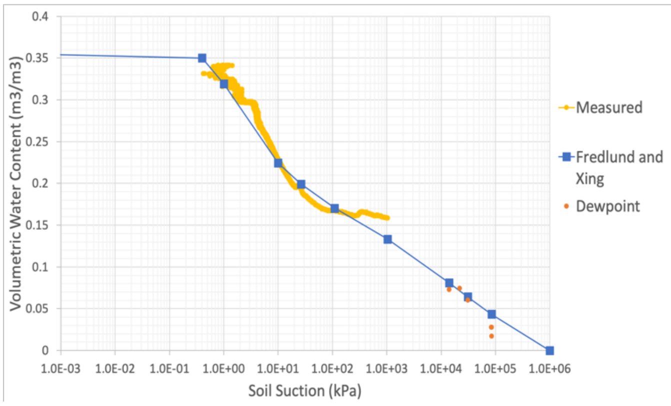

Figure 4. SWRC measured on residual Halfway House Granite

2.3 *Unsaturated Shear Strength*

All unsaturated soils will exhibit increased shear strength due to suction, unless completely dry. This increase in shear strength can be expressed by the following equation (Fredlund and Rahardjo, 2012):

$$c_{app} = (u_a - u_w)\tan\phi^b \quad (1)$$

where c_{app} is the apparent or capillary cohesion component, u_a-u_w matric suction and ϕ^b the non-linear angle indicating the rate of shear strength change with respect to suction.

Due to the excessive cost associated with undertaking unsaturated triaxial tests, it is common practice internationally to derive the shear strength from the SWRC using empirical methods by Fredlund et al. (1996) using the following formulation:

$$\tan\phi^b = \left(\frac{\theta}{\theta_s}\right)^{\kappa}\tan\phi' \quad (2)$$

where

$$\kappa = -0.0016(PI)^2 + 0.0975(PI) + 1 \quad (3)$$

The capillary cohesion at certain points of soil suctions values were derived from equations (2) and (3) and are provided in Figure 5.

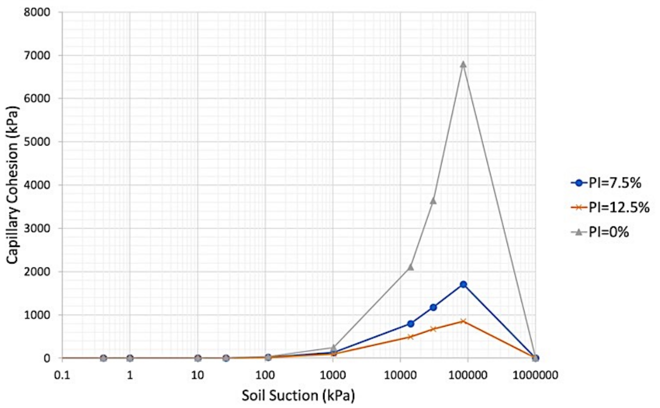

Figure 5. Capillary Cohesion profile for different values of PI(%)

From the above it can be seen that it would be the most conservative to use the cautious estimate value [mean + 0.5 STD Deviation (Schneider, 1997)] using the superior range of PI (of 12.5%).

2.4 *Unsaturated Stiffness*

The modulus of elasticity is normally assumed to be the same above and below the water table in homogenous soils, this is however not correct as the stiffness will increase with suction up to residual suction values and then start to decrease, but never below the saturated values (see Figure 6). This increase in stiffness can clearly be seen by comparing double oedometer results (E_{oed}) at the Natural Moisture Content and at saturation. Whilst the stiffness has no bearing on the ability of the soils to stand up vertically, it does affect deflections and consequent redistributed stresses as a result of soil-structure-anchor interaction.

3 EMPIRICAL APPROACH AND BASIC PRINCIPALS

3.1 *Steel soldiers and timber lagging*

The first soldier piled walls were installed as part of the underground train systems in Berlin and New York using H-beams and timber lagging.

Since then, the designer's experience and empirical methods have been used to establish the suitability of a soldier pile lateral support system with recommendations on lagging thickness, competency of soils and allowable spacing between soldiers. Soldier piles are normally used in competent soils with sufficient stand-up time and not in soft clays or materials below the ground water table.

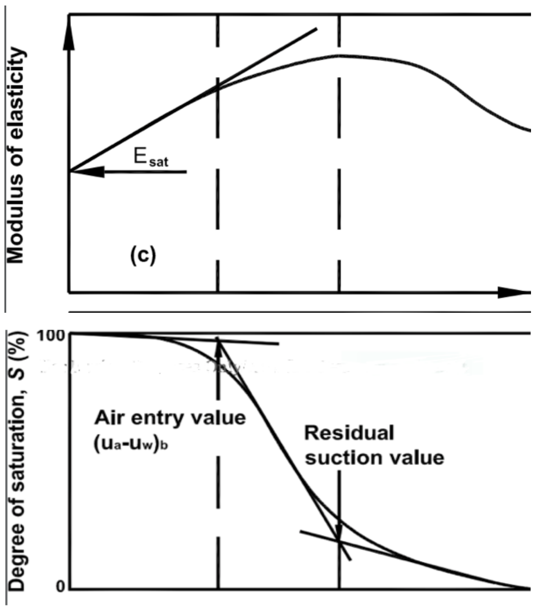

Figure 6. Increased stiffness over the range of suction values (Oh & Vanapalli, 2009)

The Goldberg Zoino chart provided in the FHWA's (Federal Highway Administration)1970 table adjusted in the CalTrans Trenching and Shoring Manual (2011) summarises SC and SM materials competency as follows; Medium Dense to Dense SM is a competent soil to construct a soldier piled wall in, when above the water table; whilst SC soil can potentially be a difficult soil in a medium dense to dense state below the water table and potentially dangerous in a loose state below the water table.

The timber lagging between soldiers are then further detailed in the Goldberg Ziono chart for varying clear spans between soldier's over a specific depth, whilst a maximum excavation depth of 18m governs.

3.2 *Bending Capacity*

The bending capacity required for timber lagging has in the past been based on empirical models, with various mathematical models trying to describe the reduced soil pressure on the timber lagging as a result of arching between piles and redistributed stresses. Shotcrete is however much stiffer than timber, leading to smaller deflections and a much higher pressure. An increase in stiffness of the soils behind such a wall as discussed in 2.4 would reduce the moment in the lagging.

3.3 *Failure Mechanism*

Perko and Bouden (2008), postulated that a localised arching failure mechanism will occur due to caving at the base of a soil silo through the temporary unsupported face of the excavation between soldier piles, as shown in Figure 7. This yielding 'silo' is opposed by the shear resistance between the yielding and stationary mass, the friction resistance against the piles and shotcrete lagging, and the cohesion from the exposed soil at a specific lift before placing lagging. This phenomenon should also induce a reduced pressure on the lagging at depth as the friction between the wall and the soil along the sides of the silo would increase with depth.

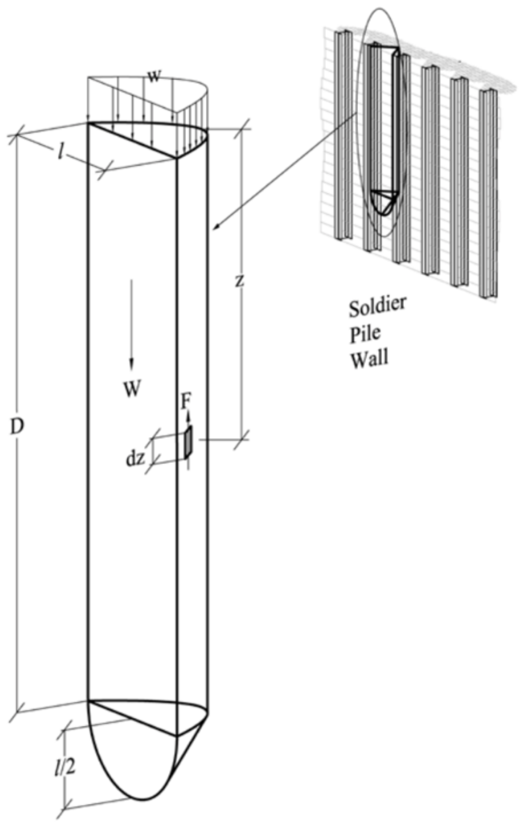

Figure 7. Soil Silo Geometry (Perko and Bouden, 2008)

3.4 *Unsupported Vertical Cut*

Conventionally it is assumed that a cohesionless soil cannot stand unsupported vertically. This is evident from the upper-bound maximum cut height (H_c) equation provided below for drained conditions, with c' set as zero:

$$H_c = \frac{4c\prime - 2q\sqrt{K_a}}{\gamma\sqrt{K_a}} \tag{4}$$

The capillary cohesion derived from the SWRC can however be used in equation (4) to replace the zero-effective cohesion. The surcharge would generally be equal to γH at the depth under consideration; some strain should occur which will reduce the pressure due to friction acting on the sides of the silo/soldier piles system, as described in Section 3.3.

4 ANALYSIS

4.1 *Ground modelling*

The importance of Finite Element Analysis in the geotechnical engineering field, among others, remains resolute due to the 3D complexities not only within the various soil strata but in their interaction on structural elements as well. With that being said, any FEM analysis can only be as accurate as the quality of its input data.

Accurate modelling of the ground profile remains critical for any geotechnical analysis. The unfortunate truth is that the geotechnical information provided for design purposes rarely comply with national standards and code of practices and is therefore inadequate for an accurate analysis. Emphasis is placed on sound engineering knowledge, judgement and experience for a successful and reliable analysis. The selection of the correct constitutive soil model is also critical to best simulate the behaviour of the material as realistically as possible.

4.2 *Suction profile*

Determining the suction profile, either by prediction or measuring, is the most complex part of undertaking an unsaturated analysis. This also proved to be the most difficult to derive from the information provided for a few of the sites in Sandton as the density and gravimetric moisture content are normally only provided for the upper surficial layers in a profile if considering spread foundations at shallow depths. In Sandton, water is typically encountered at the rock-soil interface or deeper in the rock profile. The bedrock level varies significantly due to the nature of the granitic profile. Sandton, on average, gets 600 - 800mm of rainfall a year whilst 400 - 600mm of soil evaporation can be expected. This implies that the precipitation is approximately equal to the actual evaporation over each year. Therefore, due to the lack of better information, the matric suction profile was assumed to be controlled by the hydrostatic pore-water pressure line for this study. This hydrostatic pore-water pressure was assumed to act over a residual profile with a depth of 15m to bedrock. The capillary cohesion was estimated to range from 19kPa at surface to 0kPa at bedrock for a hydrostatic pore-water pressure profile using the superior PI profile.

4.3 *Other Considerations and construction issues*

Geological intrusions, historic constructions works and the condition of pre-existing services remain one of the few challenges that considerably influences the design and safe construction of a retaining structure.

The Archaean Granite typical to the Sandton area are known to be intruded by Diabase dykes and quartzite pegmatite sills providing specific design and construction difficulties. A detailed geotechnical investigation remains critical and should be correctly specified according to the scope and risk profile category of the project as set out in SANS 10160-5:2011 supplemented by the SAICE, Geotechnical Division, *Site Investigation Code of Practice*, (2010).

A detailed desktop and on-site investigation must be undertaken to establish all subsurface services and structures that may influence the design and construction phases and must be accounted for as accurately as possible. However, the significance of a well-developed and actioned observational methodology procedure is invaluable as it the only means to accurately evaluate the as-built in-situ conditions, allowing the designer to react and amend their design as deemed necessary. A simple back analysis of a recently excavated lift, for example, can provide early confirmation of the apparent cohesion of the material.

4.4 *Three-dimensional finite-element geotechnical analysis- Assessment of required capillary cohesion*

Rocscience's RS3 (2018) was used to undertake the three-dimensional geotechnical finite element modelling. To simplify the output derived from the 3D model, an embedded soldier piled wall was considered with a maximum height of 9m (typically at these heights a soil nailed system can be considered). Non-convergence in an excavation stage indicates that the model is unstable. The excavation stages for this example were modelled in 3m lifts with anchors, installed at 0.5m above these platforms. Circular concrete piles were placed at a 3m c/c spacing with shotcrete lagging placed vertically and without an arched geometry in front of the piles. One such model is shown in Figure 8.

The excavation procedure was modelled firstly with an initial phase to establish the in-situ stresses, secondly a pile installation stage, thirdly an excavation stage, fourthly a lagging installation phase and fifthly an anchor stressing stage with steps 3 to 5 being repeated up to the final installation depth.

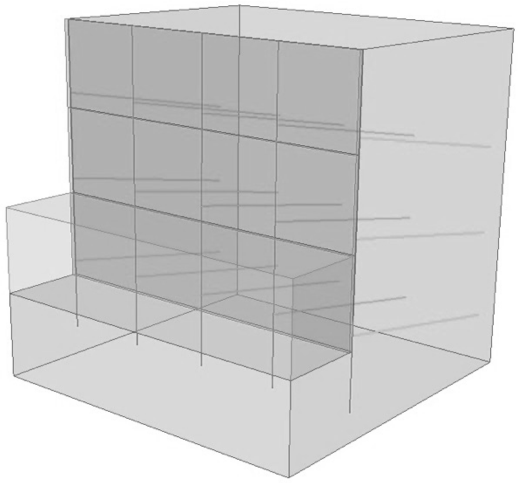

Figure 8. 3D Modelling of soldier pile wall

The Mohr-Coulomb constitutive model (CM), with lower-bound residual strength parameters, was used in the analyses as the main purpose of this study was to establish if the soils between the soldier piles will collapse at a certain capillary cohesion value. Conventionally for deep excavations a non-linear elasto-plastic constitutive model is required to establish the expected displacement (SLS). The linear-elastic perfectly plastic Mohr-Coulomb CM does not accurately model displacements but is suitable to establish if failure will occur. A strain plots is shown in Figure 9. The first model assumes a capillary cohesion value at Base Excavation Line (BEL) of 8kPa

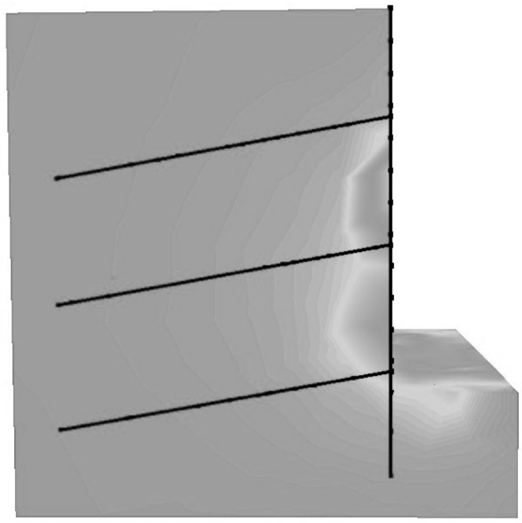

Figure 9. Shear strain plots for the 8kPa

which corresponds to the suction profile at superior PI, i.e. PI=12.5%, with capillary cohesion of 19kPa at surface. This model did not converge at the final excavation before application of the final shotcrete liner and the shear strain, γ, was larger than 2%. A second model to back analyse the required capillary cohesion at the BEL, determined this cohesion to be 20kPa which would correspond to a suction profile ranging from 0 at bedrock to 37kPa at surface. This capillary cohesion value, even with these lower-bound residual strength parameters, is lower than that required with equation 4.

However, in order to justify the postulated suction profile, the groundwater level must either be substantially deeper than that assumed, be significantly affected by evaporation, have a much lower PI value, very different water retention properties or have higher residual capillary cohesion values.

This highlights the importance of establishing the ground water profile for a specific site as well as the importance of adequate testing in particular determining the volumetric water content, grading and PI even at the most basic level.

4.5 *Other checks*

Certain other checks were also undertaken that are important to note for soils in an unsaturated state. It was established by increasing the stiffness of the soils, as can be expected for an unsaturated soil, that the moment in both the piles and shotcrete lagging decreased. This could imply major cost savings to the client.

4.6 *Future research*

To study the long-term serviceability performance of these walls the suction profile needs to be understood fairly well. Horizontal collapse could possibly also occur close to the surface if infiltration occurs as larger lateral stress will be experienced than in the in-situ state, inducing larger moments at the top of the wall. It will be useful to determine the increased stiffness of the soils and establish the true difference in stiffnesses to ascertain what benefit can be attained in terms of a reduced pile diameter or rebar reduction or mesh and shotcrete thickness reduction due to reduced moments in the elements. Additionally, the effect of increasing the conventional 3m maximum spacing can be assessed to establish what the effect would be on displacement and lagging requirements of these soldier piled walls.

5 CONCLUSIONS

This study highlights the importance of understanding the soil-water retention properties and the suction profile with depth accompanying the use of a soldier pile wall in unsaturated soils with shotcrete lagging. Additionally, it emphasises the importance of adequate laboratory testing and three-dimensional analysis to assess the problem holistically. Three-dimensional assessment of these walls would also result in better displacement predictions over the length and breadth of the excavation.

6 RECOMMENDATIONS & OTHER CONSIDERATIONS

The following investigation and design considerations/procedures are recommended:

1) Auger holes must be excavated whilst sampling intact samples at a vertical interval of 3m.
2) From these samples the natural moisture content, in-situ density, grading and Atterberg limits must be established.
3) Materials representing the materials retrieved must be send for characterisation of the soil-water retention properties (SRWC) at similar in-situ densities.
4) Ground water measurements over time must occur.
5) The capillary cohesion profile can be established with depth from steps (1) – (3) and based on formulation provided in this paper.
6) The ability of the soils to stand up at various depths can be modelled in three-dimensional geotechnical software with the capillary cohesion profile derived.

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