Collapse of Residual Archaen Granitic Soils in South Africa

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By

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ABSTRACT

The collapse of residual Archaen granitic soils in South Africa is geotechnical hazard that was identified in the 1950's. Collapse has led to hazardous building conditions in the rapidly expanding northern parts of Johannesburg, and in areas stretching across the eastern parts of South Africa, encompassing KwaZulu Natal, Mpumalanga and Limpopo Provinces. Since the development of the oedometer test in the 1960's and the research of Jennings and Knight in the 1970's very little further research has been done on the topic of collapse, despite the development of a number of new testing procedures that can give insight into the properties of these soils. Due to the large surface area of South Africa prone to collapse, and the lack of knowledge and testing methods to identify and quantify this hazard the author felt, that further insight into the collapse of residual Archaen granitic soils was required.

This research serves to evaluate the properties of Archaen residual granitic soils that may be indicative of collapsible soils. The research also compares various test methods and apparatus used to identify and quantify collapse potential, namely the oedometer collapse potential test and the triaxial collapse potential test, and evaluates the effect of soil properties on these methods.

ACKNOWLEDGEMENTS

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1. INTRODUCTION

The first published example of collapse settlement in South Africa was identified during 1955 near Witbank when high amounts of settlement were observed beneath parts of a large steel framed structure underlain by residual Archaen granitic soils (Jennings and Knight, 1957). Further investigation by Brink and Kantey (1961) found that residual granitic soils throughout South Africa are potentially collapsible. Extensive infrastructure development throughout South Africa takes place in regions underlain by residual granitic material. The collapse and settlement of these soils can be problematic, and with new test methods for measuring settlement behaviour and various soil parameters now available it is essential that this topic is revisited. Soil parameters such as void ratio, dry density and degree of saturation are often valuable tools used by geotechnical professionals to identify potentially collapsible soils. This research serves to quantify collapse that may be anticipated in residual Archaen granitic soils in South Africa and shows the effect of various soil parameters on volume change behaviour, and their use in identifying this hazard.

The identification and quantification of collapse in South Africa was explored by Knight (1961) resulting in a description of the mechanism responsible for collapse settlement, and an array of testing procedures to quantify collapse settlement. One of the most widely used laboratory tests to identify collapse is the oedometer collapse potential test, however it is not advised to use results obtained from this index test as design parameters for construction (Jennings and Knight, 1975). Various flaws such as bedding errors and lateral strains are inherent with the oedometer apparatus, which lead to inaccuracy in collapse and settlement measurements. In an attempt to mitigate these errors Rust et al. (2005) developed the triaxial collapse potential test which is theorized to imitate field conditions far better than the oedometer apparatus. It is anticipated that measurements of collapse in triaxial collapse potential test may be used as design parameters for construction. The volume change behaviour of the residual Archaen granitic samples was measured using both the oedometer and triaxial collapse potential tests and the difference between the results and the test methods themselves were evaluated (Heymann, 2000; Rust et al., 2005; Jennings and Knight, 1975; Knight 1961).

2. THE COLLAPSE MECHANISM

Collapsing soils can be defined as soils that decrease in bulk volume under load when water is added to them. Collapse differs from conventional consolidation (settlement), as no water is being pressurized out of the sample, and in many cases a collapsing soil will continue to absorb water whilst collapsing (Dudley, 1970). This occurrence of collapse seemingly does not correspond to Terzaghi's theory of effective stress, as wetting increases pore water pressure, and should cause swell, not collapse (Terzaghi, 1943).

Schwartz (1985) identified 4 criteria required for collapse to occur, these being:

1. The soil must have a collapsible fabric.
2. The soil must be at least partially saturated.
3. There must be a rise in the soil moisture content.
4. There must be an increase in the overburden pressure of the soil.

2.1. Collapsible Fabric

It is generally agreed that collapsible soils comprise a mixture of coarser soil grains held together by finer material which permit intermolecular, electrostatic, capillary and chemical bonds to develop. Although not all of these may be present, the relative strength of the bond type will depend on the structure and makeup of the soil. The soil fabric must be leached and voided to produce the voids/space (the air component of the soil) into which the grains can compress (or rearrange) when shear strength is lost between the soil particles causing the soil to decrease in volume (Schwartz, 1985).

2.2. Partial Saturation

A degree of partial saturation must exist in the fines component of the soil to create the shear strength required to support the coarser soil fraction in an open, leached structure. Studies have proven that in a collapsible soil percentages of fines (particle size smaller than 0.075 mm) of the material are inversely related to the degree of saturation required for collapse to occur. It has been shown that soil above the critical degree of saturation will collapse further under an increased applied load, and conversely a collapsible soil below the critical degree of saturation will not collapse under an increased applied load. This shows that the critical degree of saturation is an important factor in triggering collapse (Schwartz, 1985; Errera, 1977; Jennings and Knight, 1975).

2.3. Rise in Moisture Content

The bond strengths (intermolecular, electrostatic, capillary and chemical bonds) that exist between the fines portion (particle size smaller than 0.075 mm) of the soil are typically sensitive to moisture content (degree of saturation) and the bond strength tends to decrease with increase in moisture content. A soil at a constant partial degree of saturation and under a constant applied load will not suddenly collapse without the addition of further moisture (Osipov & Sokolov, 1994; Lefebvre, 1994).

2.4. Increase in Overburden Pressure

It has been shown that the bulk of in-situ soils in South Africa will not collapse without an increase in overburden pressure. This was termed as the ‘stable surface condition’ by Jennings and Knight (1975) and is the assumption that the natural surface level of the ground will not collapse (will remain stable) under any moisture condition. Therefore, for a natural in-situ material with a collapsible fabric to collapse an increase in applied pressure is required. This presumption is logical, as over a long time period a soil would have been saturated and come to equilibrium with the overburden pressure at all moisture contents.

It can therefore be seen that for a soil to collapse both a collapsible soil structure and a degree of partial saturation are necessary. The applied pressure (load) and the moisture content are at equilibrium in a stable soil, however an increase in either can lead to unstable soil conditions and subsequent collapse (Schwartz, 1985).

3. YIELDING MODEL

Bishop and Blight (1963) showed that collapse can be explained within the theory of effective stress, and that effective stress in partially saturated soils can best be described by 2 separate forces, namely applied stress (total stress) and pore water suction. Applied stress can be visualized as the stress causing shear stresses and consequently instability, whilst pore water suction is a normal stress acting at the grain contacts that can either strengthen or weaken the stability of the soil structure (Bishop and Blight, 1963).

Using the yield model for collapsible soils as set out by Leroueil and Vaughan (1990) it is understood that electrostatic, intermolecular and capillary forces may exist in the finer soil fraction which bonds the coarser soil grain contacts of the soil (it should be remembered that not all of these forces may be present). The strength of the forces present between grain contacts will be related to the moisture content of the soil, and applying a pore fluid suction or total stress of equal magnitude to the soil will have a matching effect on the effective stress. This however is not true for collapsible soils of very high pore fluid suction (low degree of saturation), as a change in pore fluid suction and an identical change in total stress may not have the same effect on the behaviour of the soil (Leroueil and Vaughan, 1990). This can be seen in the three distinct stress paths that have been identified in the volume change behaviour of collapsible, compacted, gneissic soil (Jose et al., 2000):

- Phase 1; at high pore water suction only small deformation is observed.
- Phase 2; at intermediate pore water suction high deformation (collapse) is observed.
- Phase 3; at low pore water suction deformation is negligible, with some secondary soil skeleton compression.

It has thus been shown that when the finer particles found between larger grains in a collapsible soil are wet the pore water suctions in the finer soil portion are reduced, resulting in a lower soil strength. When the soil strength is reduced to less than the applied stress acting on the soil, the soil is no longer stable, and deformation (collapse) will take place. This generally occurs at intermediate pore water suction (partial saturation) before the soil becomes completely saturated. The soil moisture content at which the soil strength reaches equilibrium with the applied load is commonly known as the “Critical Degree of Saturation” (Schwartz, 1985).

4. PREVIOUS IDENTIFICATION OF COLLAPSE

Collapsible soils may generally be identified by simply attained properties such as particle size distribution, dry density, void ratio or degree of saturation. Many of the authors who suggest values for these criteria warn against using these as an indication of collapsible material. There are instances that soils which fall within these criteria are not collapsible and conversely soils that fall out of these criteria have been found to be collapsible. Despite this soil parameters are widely used by geotechnical professional to initially identify potentially collapsible soils and weather further collapse specific laboratory testing is required.

4.1. By South African Standards

Many methods have been devised to identify collapsible soils, such as the sausage test and the oedometer collapse potential test (Jennings and Knight, 1975). The encumbrance of these tests has been the fact that they are only indicator tests and the qualitative data they produce is not relevant to design on collapsible material. Similarly soil parameters only give an indication of potential collapse and give little insight into the quantity of collapse that can be anticipated.

Brink, Patridge and Williams (1982), and Yates (1980) showed that generally soils with a dry density of between 900 and 1600 kg/m³ are more prone to collapse, but acknowledged that higher dry densities do not necessarily prohibit collapse and vice versa.

Brink (1985) shows two equations derived to estimate the collapse potential of an aeolian and mixed origin soil based purely on dry density. According to the equations Aeolian soils will only exhibit collapse with dry densities of less than 1672 kg/m³ (Schwartz in Brink, 1985), whilst mixed origin soils will only exhibit collapse with dry densities of less than 1590 kg/m³ (Pavlakis, in Brink 1985).

Void ratio is linked to dry density and is also an important factor in collapsibility. In South Africa void ratios of 0.8 or above are seen as indicative of collapse on the collapsible Kalahari Sands (Rust et al., 2005). The research done by Brink and Kantey (1961) found collapsible residual granitic soils with void ratios generally ranging between 0.8 and 1.1, but with some as high as 1.5.

Clay content is also very important in the formation of a collapsible fabric, as this creates the bridges between larger grains which loose shear strength when wet and result in consolidation. Collapse has been known to occur in soils with clay contents varying from 5% to 30%, but collapsible material have generally been found to have a clay content closer to the lower end of this range (Webb and Hall.1967). Furthermore it was hypothesized by Jennings, (1967) that collapse could even occur in a predominantly clay soil where large clay peds substitute as the coarse fraction of the collapsible soil fabric. The type of clay which is found in a soil can also affect the collapsibility, as a soil with a high percentage of (expansive) 2:1 clay may swell, whilst a soil with the same content of (non-swelling) 1:1 clays may collapse. It can thus be seen that the optimum clay content for collapse to occur differs depending on the physical properties and the type of clay/s found in the soil.

The critical degree of saturation is important in the collapse mechanism, this represents the moisture content of the collapsible soil required to result in a breakdown of the chemical bonds between the clay bridges and collapse. This value is affected by the grading of the soil, especially the fines content of the soil. This parameter varies greatly in different soils and is based on the grading, the type of materials making up the soil and the formation of the soil (origin). Literature has identified the critical degree of saturation required for residual granitic soils (with 15% finer than 0.075 mm) to collapse is 52% according to Errera (1977), and the degree of saturation required for collapse of fine silty sands was found to be between 50% and 60% (Jennings and Knight, 1975).

The degree of weathering is thought to be crucial to the development of a collapsible grain structure, specifically in residual soils. It has been found that collapsible residual soils generally fall within areas of a water surplus (Schulze, 1958 in Schwartz, 1985). Schwartz (1985) went further to stress the importance of chemical decomposition and leaching in the formation of collapsible grain structure of residual soils. Other than this very little research has been done on the link between weathering and collapsibility of residual soils.

Schwartz (1985) alludes to the fact that there are still few case histories and studies done on collapsible soils, and that predictions made from current geotechnical engineering practice are still highly unreliable.

4.2. By International Standards

Dry densities have been used internationally to identify collapsible soils. Collapsible loess in china has been shown to have dry densities of between 1100 kg/m^3 and 1400 kg/m^3, with void ratios of approximately 0.8 (Lin, 1994). Dudley (1970) gave a very wide range of dry densities of soils that could indicate a potentially collapsible fabric, from 1100 kg/m^3 to 1700 kg/m^3.

The collapsible fluvial deposits of the San Joaquin Valley (California, USA) have showed maximum collapse with a clay content of 12%, negligible collapse with clay content below 5% and swell with clay content above 30%, these clays were 70% to 80% montmorillonite. This research also concluded that soils with larger pore spaces collapsed more than soils with the same void ratio but smaller pore spaces and that drier samples collapsed more (Dudley, 1970).

Research by Jose et al., (2000) has shown that in collapsible compacted gneissic soils the majority of volume change generally tends to occur at a pore water suction of 30 kPa to 10 kPa. This material showed gradual collapse from suctions above 90 kPa to 10 kPa and no further collapse settlement at pore water suctions of between 10 kPa and 0 kPa.

5. COLLAPSE FABRIC OF RESIDUAL GRANITIC SOILS

Brink and Kantey (1961) were the first in South Africa to identify extensive collapse settlement occurring on residual granitic soils. They identified this phenomenon on the residual soils from the three types of granite found in South Africa, the Cape Granite Suite, the Bushveld Igneous Complex Granites and the Archaen Granites. Collapse of residual Archaen granitic soils has been a major cause of road failure in South Africa (Knight and Dehlen, 1963). It has also been shown that compacted gneissic soil (similar in grading and petrology to Archaen residual granitic soils) undergoes collapse. This collapse was found to relate to the fines percentage, moisture content, stress imposed and initial dry density of the material (Jose et al., 2000).

Criteria for the formation of collapsible fabric forming in residual granitic soils have been identified as:

1. Areas underlain by granitic bedrock with a composition consisting mainly of quartz, feldspar and mica (Brink and Kantey, 1961).
2. There must be a water surplus (precipitation exceeds evaporation) to produce the amount of water needed to leach the profile. Collapse in residual granitic soils has been seen to only occur in areas of a water surplus (Schulze, 1958; Brink and Kantey, 1961).
3. Slopes where drainage down gradient within the soil profile causes the condition of internal drainage required to remove colloidal material and leach the profile (Brink and Kantey, 1961).

It must be noted that the paper by Brink and Kantey (1961) assumes that soil formation and all weathering has taken place under topographic conditions similar to those at present. Many of the collapsible residual granites in South Africa are not necessarily in areas of internal drainage and a water surplus, but have developed collapsible grain structure through weathering in the Createaceous to Early Cenzoic age, during cyclic erosion associated with the African erosion surface (Partridge and Maud, 1987).

A map of Southern Africa showing areas of water surplus and regions underlain by granitic bedrock can be seen in Figure 5.1. All of the samples recovered for this research were taken from areas underlain by Archaen granitic bedrock, with a water surplus on slopes perceived to have a condition of internal drainage.

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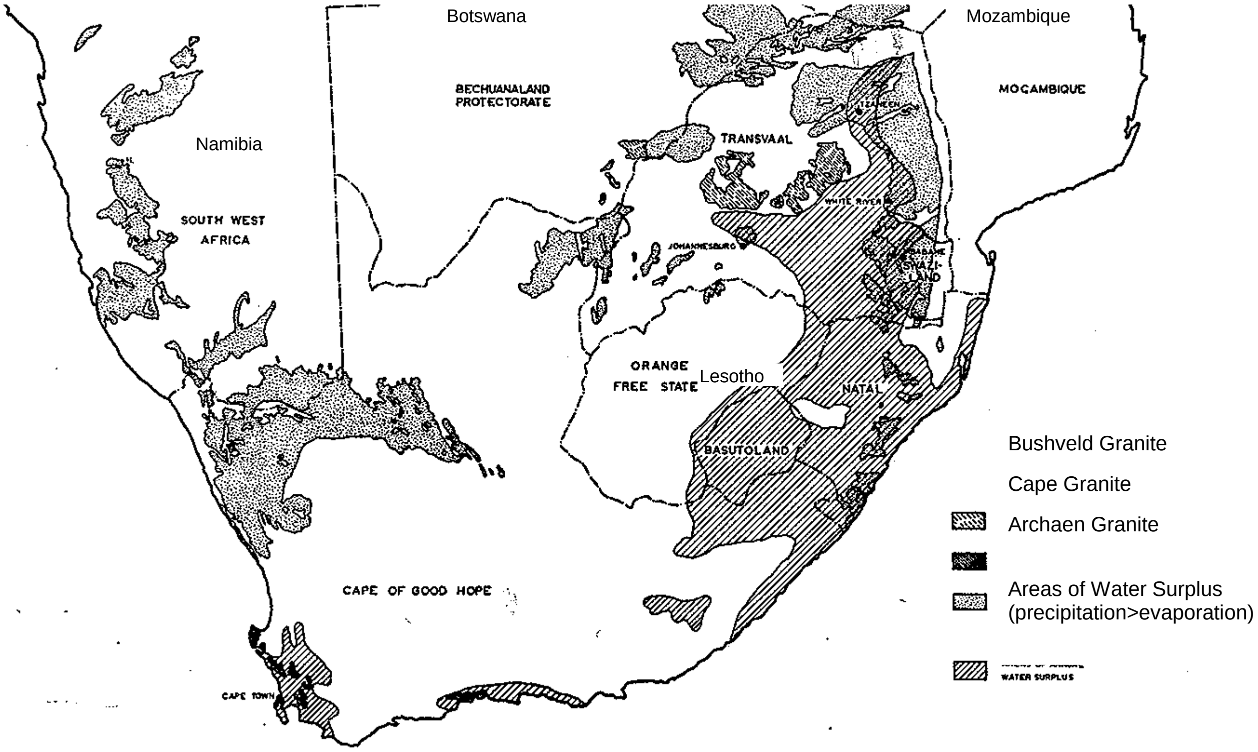


Figure 5.1: Showing areas of granitic bedrock and areas with a water surplus (Brink and Kantey, 1961).

6. THE DEVELOPMENT OF TRIAXIAL COLLAPSE TESTING

The majority of the work done on quantifying collapsibility has revolved around the one-dimensional oedometer apparatus. However Errera (1977) theorized a method of using triaxial equipment to model collapse behaviour more accurately. The use of the collapse potential as measured in the oedometer apparatus is seen as an index test and Jennings and Knight (1975) stress that these results are not to be used as design parameters. Despite some geotechnical professionals using these results as design parameters the majority of the geotechnical fraternity believe that the inherent errors in the oedometer test are too great to accurately quantify collapse settlement (Schwartz, 1985; Rust et al., 2005).

The development of a triaxial collapse potential test was seen as a way of possibly mitigating many of the errors characteristic of the oedometer apparatus. Schwartz (1985) expressed problems with using a triaxial apparatus in the quantification of collapse. He predicted that this method would not catch on with commercial laboratories and not be adopted by geotechnical professionals, due to the cost, complicated sampling and sample preparation required (Schwartz, 1985; Errera, 1977).

Studies by Jose *et al.* (2000) on collapsible compacted gneissic soil were carried out using a triaxial permeameter. The material was loaded to 25 kPa, 50 kPa, 100 kPa and 200 kPa respectively and then saturated. Saturation took place in increments from the initial pore water suction of 370 kPa to 90, 60, 30, 10, 5 and finally 0 kPa (fully saturated).

Taking advantage of advances in technology, Heymann (2000) designed and tested the triaxial collapse potential test. He used a triaxial cell to better simulate the radial stresses imposed upon an in-situ material, and used isotropic stress to collapse the sample as opposed to a load being transferred vertically through a plate which can lead to bedding errors. This test differs from previous collapse tests, as it uses local instrumentation to measure axial strains, which further mitigates inaccuracies due to bedding errors and its longer gauge length (±55 mm as opposed to the ±20 mm of the oedometer) decreases the effects of local anisotropy. The radial stress applied on the sample also reduces the inaccuracies caused by lateral strain (squashing) experienced in oedometer testing (Maswoswe, 1985; Rust et al., 2005).

7. AIMS OF INVESTIGATION

This research is aimed at investigating the collapsibility of residual granitic soils, specifically those derived from the weathering of Archaen granitic rocks. It includes the findings and assumption of previous authors coupled with state of the art test methods to gain a better look at the properties of these soils and how this affects the collapsibility of these media.

Since the development of the oedometer test in the 1960's and the research of Jennings and Knight in the 1970's very little further research has been done on the topic of collapse. This research serves to use updated testing methods such as the triaxial collapse test, x-ray fluorescence and diffraction, and microscopy to re-evaluate collapse.

This research is carried out specifically on residual Archaen granitic soils, as collapse has been reported to occur widely in this medium and there has been a lack of research into this material's collapsibility (with majority being carried out on the windblown Kalahari Aeolian sands). The properties of the sampled residual granitic soils are assessed to ascertain any soil properties that may be indicative of collapsible soils. The research also compares various test methods and apparatus used to identify and quantify collapse potential, namely the oedometer collapse potential test and the triaxial collapse potential test, and evaluates the effect of soil properties on these methods.

8. METHODOLOGY

The selection of the location of the sampled, potentially collapsible, residual granitic soils was carried out using the criteria set out by Brink and Kantey (1961). The required bedrock geology (Archaen granitic bedrock) was identified using geological maps and areas of surplus water were found using climate data (Weinert, 1964; Brink and Kantey, 1961). With this information and study of previous cases of collapse on residual granitic soils four areas were selected. The four areas selected were Tzaneen, Bushbuckridge, Midrand and Paulpietersburg.

8.1. Sampling

Once on site the bedrock geology was evaluated in the area and the soil was profiled to identify the residual granitic horizons. The topography/gradient of the site was assessed to verify that a condition of 'through profile drainage' existed. The depth, structure and mineralogy of the residual soils at these sites were assessed and the residual soils observed to have the most collapsible fabric were then sampled. These samples were generally retrieved at road or railway cuttings (as a Tractor-Loader-Backhoe (TLB) was not available).

The profiles from which the samples were retrieved were described according to SANS 633 (2009). Photographs and co-ordinates of the sampling sites were also taken. Undisturbed block samples were carved out of the residual soil horizon (with minimum dimensions of 250 mm x 250 mm x 250 mm) using shovels, picks, geological picks and carving knifes. These block samples, whilst still in-situ, were covered in molten wax (Figure 8.1) to prevent moisture loss and to give the sample added strength when being removed from the horizon. The samples were then removed and completely covered in wax before being wrapped in layers of cling film and finally covered in aluminium foil as prescribed by Heymann and Clayton (1999). This method of packaging the samples was shown to decrease moisture loss to a minimum whilst maintaining the soil structure of the sample (Heymann and Clayton, 1999). The samples were transported to the laboratory with great care to minimize the damage to the structure of the sample prior to testing (Figure 8.2).

Figure 8.1: Sample being covered in wax

Figure 8.2: Sample being transported

8.2. Standard Laboratory Tests

Disturbed samples were taken in conjunction with the undisturbed samples at the same locality. The grading, hydrometer analysis, Atterberg limits and particle density of the undisturbed samples was tested by the Soillab (Pty) Ltd laboratory in Pretoria.

The laboratory results of the samples are presented in Appendix B.

8.3. X-Ray Diffraction and X-Ray Fluorescence Testing

X-Ray Diffraction (XRD) and X-Ray Fluorescence (XRF) testing was conducted to ascertain the exact mineralogical and chemical composition of the samples and give an indication of the degree of weathering, and gauge the effects this has on the structure and subsequent collapsibility. Both the XRD and XRF analyses were undertaken by the University of Pretoria, Faculty of Natural and Agricultural Sciences Analytical Laboratory.

The XRD and XRF analysis of the samples are shown in Appendix C.

8.3.1. XRD Analysis

The XRD analysis was carried out using a back loading preparation method in an X'Pert Pro powder diffractometer with X'Celerator detector and variable divergence and receiving slits with Fe filtered Co-Kα radiation. The different phases were identified using X'Pert Highscore plus software. The weight percentage of the phases was estimated using the Rietveld method (Autoquan Program), with errors on the thrr sigma level. Amorphous phases, if present in the analysis, were not taken into consideration.

8.3.2. XRF Analysis

The XRF analysis was carried out using an ARL 9400XP+ Wavelength dispersive XRF Spectrometer with Rhodium tube, LiF200, LiF220, GER, AXO6 and PET analysing crystals, flow-proportional and Scintillation detectors. The phases were identified using WinXRF software for major and trace element analyses, with Quantas and Uniquant software as additional semi-quantitative programs.

8.4. Degree of Weathering

Literature has identified the role played by weathering and leaching in the formation of collapsible soil structure particularly in residual granitic soils (Schwartz, 1985; Dudley, 1970; Brink and Kantey, 1961). It is anticipated that the degree of weathering may give good insight into the volume change behaviour of residual granitic soils. Little research has been conducted into the relationship between weathering and collapse, as it is difficult to quantify the amount of weathering that has taken place in a soil. Two methods were used to quantify weathering in the samples and investigate the possible relationship with collapse.

The mineralogical composition of the soils, determined using X-ray diffraction (XRD) analysis, was used to evaluate the amount of primary granitic minerals (granite rock forming minerals, are typically quartz, feldspar, hornblende, mica, etc.) against the amount of secondary minerals (minerals altered by weathering and not present in original granite rock, such as kaolinite, chlorite, etc.). This relationship is thought to give a good indication of the amount of weathering a soil has undergone.

The degree of weathering of the samples was also assessed using the Chemical Index of Alteration (CIA;Nesbitt and Young, 1982). This was calculated from the molecular make up of the samples obtained from X-Ray Fluorescence (XRF) analysis using Equation 8.1.

$$CIA=[Al_2O_3/[Al_2O_3+Na_2O+K_2O+CaO]].100 \qquad \text{Equation 8.1}$$

A problem identified with assessing the weathering of the soils is that much of the soil may have been eroded or leached out of the profile, and with no way of knowing the components, type or quantity of material that has been lost it is difficult to gauge the true degree of weathering.

8.5. Microscopic Examination

Due to the importance of soil structure in collapse settlement, undisturbed pieces of the samples were evaluated by means of microscopy. A trial sample was viewed using a scanning electron microscope. However the preparation method for this machine was too damaging to the soil structure of the undisturbed samples and as the required magnification to view the soil structure could be achieved using an optical microscope, only the optical microscopy was used. The samples were analyzed with a Weiss optical microscope at magnifications of X1 to X100, as these magnifications were found to best illustrate the structure of the sampled soils. The micrographs of the undisturbed samples were then analyzed, alongside the consolidated samples, to view the amount of voids and the average size of voids in each sample as well as the change in the structure of the soil from the in-situ state (un-consolidated) to the collapsed state (consolidated).

The micrographs taken of the samples are shown in Appendix D.

8.6. Pore Fluid Suction Pressure Measurements

The pore water suctions of the undisturbed samples at Natural Moisture Content (NMC) were measured using the filter-paper method as proposed by Chandler and Gutierrez (1986) (Figures 8.3 and 8.4). This gave the pore water suctions of the sampled soils in their in-situ state (when sampled) and was used to identify the degree of saturation and moisture content of the samples.

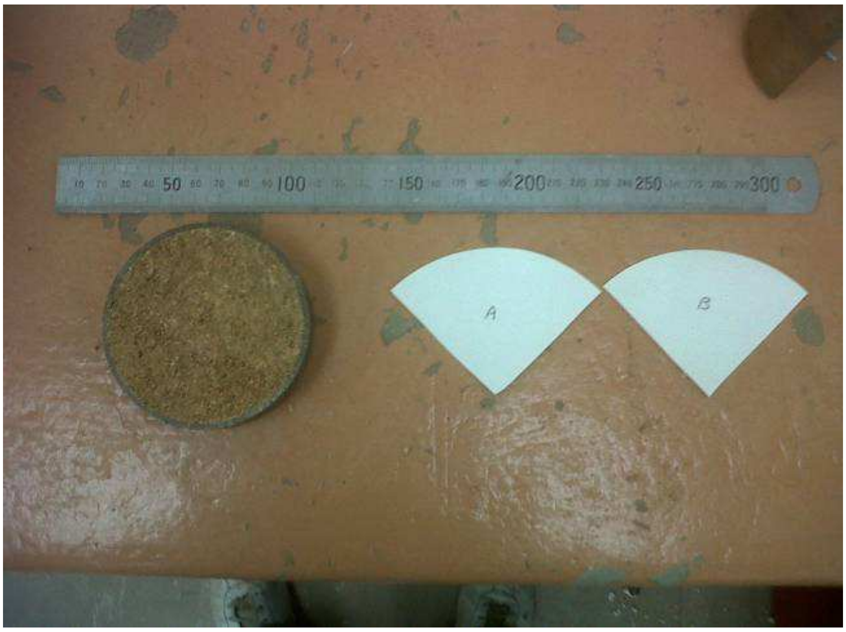


Figure 8.3: Pore fluid suction measurement

Figure 8.4: Pore fluid suction measurement

8.7. Oedometer Collapse Potential Testing

Consolidometer testing was carried out on the undisturbed samples in the procedure prescribed by Knight (1961). The soil was cut from the undisturbed samples using a knife and an oedometer ring, approximately 20 mm high with a diameter of ±75 mm (Figure 8.5). Once cut from the block sample (Figure 8.6) the top and bottom of the oedometer sample were levelled with the top and bottom of the ring. Any gaps or voids, created during the levelling of the oedometer sample, were then filled in by compacting disturbed soil into the gaps using a knife. In this way the exposed surface of the soil was made as continuous and level as possible to minimize bedding errors (Figure 8.7).

Figure 8.5: Cutting of oedometer sample from block sample

Figure 8.6: Un-smoothed oedometer sample

Figure 8.7: Smoothed oedometer sample

The oedometer sample was then placed between upper and lower sintered brass porous disks to allow free drainage of the sample during testing (Figure 8.8). The sample was positioned in an oedometer pressure cell (Figure 8.9) and put under consolidation stress (bedding load) of 12.5 kPa for approximately 24 hours. The stresses were then increased incrementally to 12.5, 25, 50, 100 and 200 kPa. The sample was then inundated with water and again left to strain over a 24 hour period (at 200 kPa). Then the stress increment increases continued to 400 and finally to 800 kPa with the sample under saturated conditions. With each incremental increase the sample was left until the strain rate at that pressure had become minimal, less than 0.25% of the sample height per hour (calculated as 0.05 mm/hour). The strain of the sample was measured during the incremental loading by a mechanical strain gauge attached to the oedometer pressure cell.

Figure 8.8: Sample positioned between porous disks to allow free drainage of the sample

Figure 8.9: Oedometer apparatus used in testing

8.8. Triaxial Collapse Potential Test

The collapsibility of the undisturbed samples was also tested using the triaxial collapse potential test, developed by Heymann (2000) to quantify the collapse of deep collapsible beach/dune sands of Mozal, Mozambique. These sands are similar to the aeolian sea sand found in the KwaZulu-Natal coastal area, known as Berea sands. This testing required the use of a conventional triaxial cell and high pressure submersible LVDT’s (linear variable differential transducers).

The pressure cell used is a standard cell used for the triaxial testing of 75 mm diameter samples. The cell has a unique machined base pedestal which reduced the pedestal (on which the sample sits) from 75 mm to 50 mm in diameter in order to create more space in the cell for the placement of the LVDT’s on the sides of the sample. A schematic showing the set up of a triaxial collapse potential test is shown in Figure 8.10.

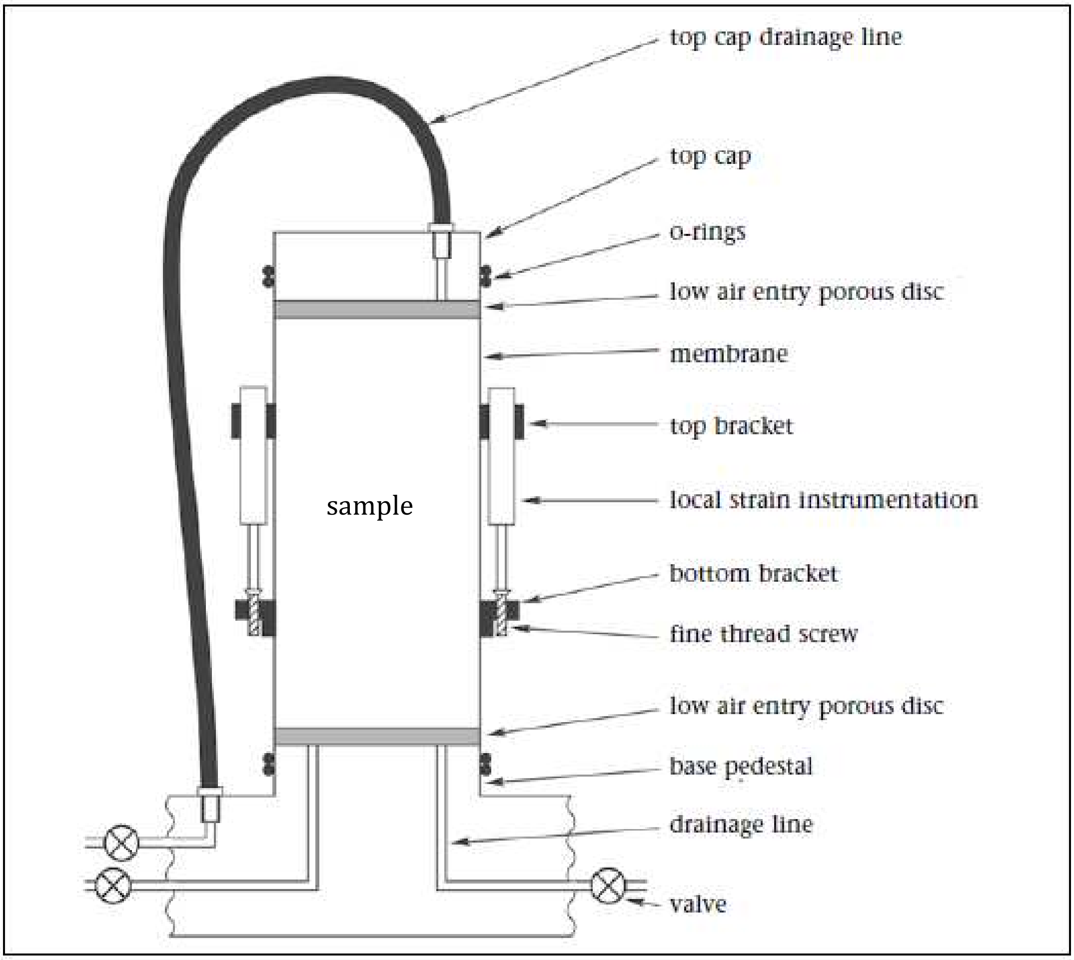


Figure 8.10: The triaxial collapse potential test (Rust et al., 2005).

8.8.1. LVDT(Linear Variable Differential Transducers)

Two LVDT's were used for the displacement measurements during the triaxial collapse testing (Figure 8.11). The LVDT's used, were designed to work at high pressures and whilst submerged.

Figure 8.11: LVDT used for testing

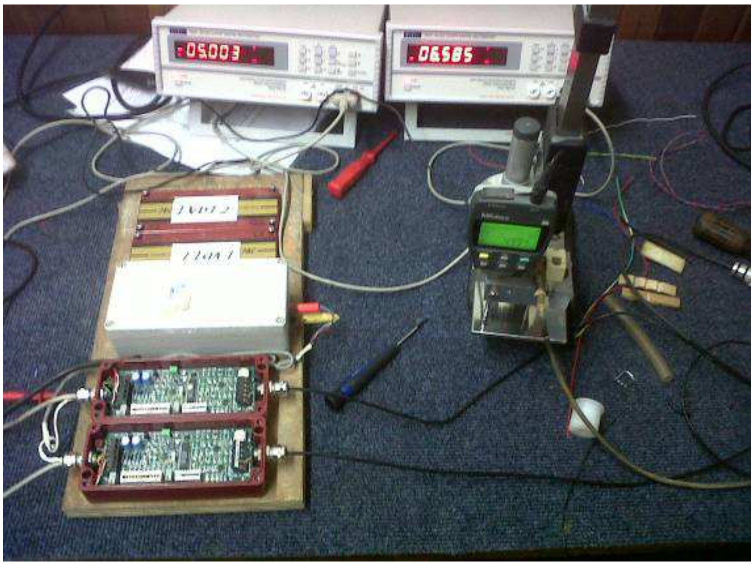


Figure 8.12: Calibration of LVDT's using Miltutoyo strain gauge

The LVDT's were calibrated to ensure accurate and fine measurement of any displacement of the sample, using a Miltutoyo strain gauge, accurate to 1 micron. The LVDT's were calibrated to have a range of 10 mm, with a maximum extension at -5.000 volts (0 mm) and a maximum contraction at +5.000 volts (10 mm; Figure 8.12). To help simplify the conversion of voltage readings into displacement measurements the LVDT's were calibrated so that a change of 1 volt was equivalent to 1 mm in displacement. The LVDT's were calibrated to be accurate to within 10 microns throughout their range. Hysteresis of the LVDT's was not measured, as this was thought to be minimal, due to the frictionless design of these devices, and superfluous to the findings of the study.

8.8.2. Sample Setup

Firstly an undisturbed piece of the desired soil was cut from the block sample, and using a knife and other carving equipment was carved meticulously to a cylindrical sample of diameter ±50 mm and height of ±100 mm (Figures 8.13 and 8.14). During the carving process a 50 mm diameter porous disk was placed above the sample being carved to give an indication of the required diameter. Height measurements were also taken frequently to ensure the sample was being carved to the correct height. It cannot be stressed enough how imperative it is, to the triaxial testing process, that great time and care is taken when carving these samples, especially when working with non-cohesive materials.

Figure 8.13: Piece of block sample (Midrand) being carved to triaxial sample dimensions

Figure 8.14: Completed carved triaxial sample (Midrand)

Two dry sintered brass porous disks were then placed on either end of the sample and it was quickly covered in a latex triaxial membrane (Figure 8.15), whilst in the membrane the sample was weighed and the exact height and diameter of the sample was measured (Figure 8.16). The weight and thickness of the membrane and the porous disks must be taken into account when calculating the weight and dimensions of the sample.

Figure 8.15: Membrane being placed around sample (Paulpietersburg)

Figure 8.16: Sample (Paulpietersburg) being weighed and measured once covered in membrane with porous disks

Figure 8.17: Sample (Paulpietersburg) placed on base pedestal with top cap and o-rings

The sample is then placed on the special pedestal within the triaxial pressure cell and the top cap is placed on the top of the sample. O-rings are placed around the membrane which over lapped the top cap and the base pedestal respectively to create a water tight seal between the sample and the pressurized water of the triaxial cell (Figure 8.17). The latex membrane is pervious to air but not to water.

Once the sample is in position on the pedestal and the top cap is in place, markings (using a permanent marker) are made on the membrane a vertical distance of 55 mm apart (the gauge distance of the test) and this is repeated on the opposite side of the circular sample (180° apart; Figure 8.18).

Two pairs of specially machined brackets (shown in Figure 8.19) are used to attach the LVDT's to the sample. The brackets are carefully adhered to the sample membrane (using super glue) with the top face of the top bracket and top face of the bottom bracket positioned parallel to the markings, and with each pair vertically aligned (Figure 8.20). A screw is attached to the lower bracket of each pair on which the LVDT pin will sit, and with which fine adjustments can be made to the extension of the LVDT (gauge length). Great care must be taken to ensure that the brackets are vertically aligned and are the correct distance apart (55 mm). If this is not the case, the LVDT pin will not rest on the intended screw, the sample will be ruined and sample preparation will have to commence from the beginning.

Figure 8.18: Markings made to indicate the section of the sample to be tested and assist in placing the LVDT’s

Figure 8.19: Specially machined brackets used to attach the LVDT’s to the sample

Figure 8.20: Brackets correctly adhered to the sample (vertically aligned and the correct distance apart)

Elastic bands are then placed around the two upper brackets and similarly around the two lower brackets (Figure 8.21) to further secure the brackets against the sample. The LVDT’s are then placed in the brackets and tightened into position, by tightening the upper open bracket (Figure 8.22).

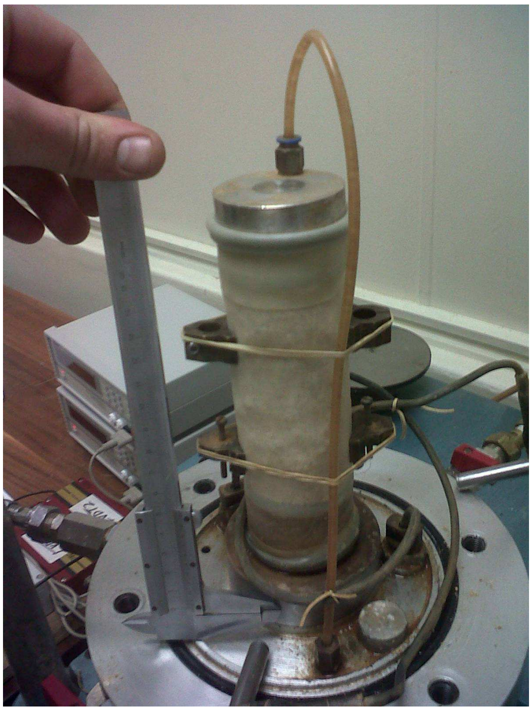


Figure 8.21: Brackets attached and supported by elastic bands

Figure 8.22: LVDT’s in place and secured to sample using brackets

The LVDT pins are then placed in the LVDT tubes, and made to rest on the screws attached to the lower brackets. These screws are then used to delicately position the LVDT pins as close as possible to maximum extension (-5.000v or 0 mm) within the LVDT tube.

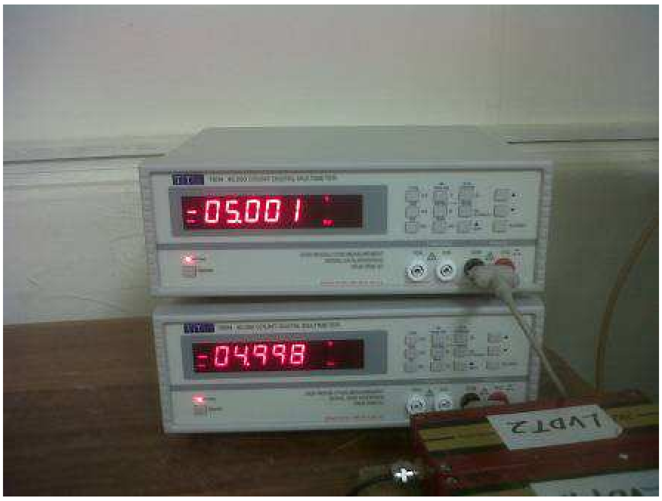


Figure 8.23: Setting of the LVDT's to within 0.01v of maximum extension (-5.000v)

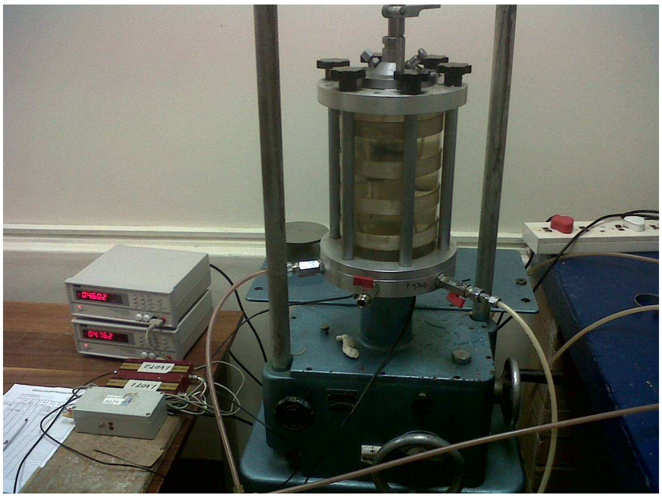


Figure 8.24: Pressure cap placed over sample, submersion of the cell causing slight strain of the sample

Vaseline is placed on the o-ring at the contact between the triaxial cell cap and the base of the cell upon which it sits, to further seal the cell. The triaxial cell cap is then very carefully placed over the sample (Figure 8.24). Great care must be taken to ensure that there is no contact or movement of the LVDT's or LVDT wiring during this process. If contact is made, the LVDT's will have to be readjusted before another attempt at positioning the pressure cell cap.

8.8.3. Evaluation of Collapse

Once the sample with attached LVDT's is in the pressure cell, the cell is filled with water under the pressure head from the de-aired water source through the cell pressure duct (keeping the top valve of the pressure cell open to allow the escape of air from the cell). The last remnants of air in the cell are pushed out using the application of minimal pressure, the top valve of the cell is then closed. The bedding load of 25 kPa is applied for at least 24 hours. A bedding load of 25 kPa is used in the triaxial test (unlike the 12.5 kPa bedding load used in the oedometer tests), as the readings from the triaxial pressure application instrumentation only start at 20 kPa. During the de-airing of the pressure cell slight strain of the sample will take place (Figure 8.23), but this strain should be disregarded and the strain reading at the completion of the bedding load cycle should be used as the zero point.

After the initial bedding load of 25 kPa for 24 hours, the stresses on the sample are increased incrementally, each time allowing for the reduction of the creep rate to less than 0.25% of the gauge length per hour or less than 0.1375 mm/hour. The stress on the samples is raised incrementally as in the oedometer collapse potential tests, to 50 kPa, 100 kPa, 200 kPa, whilst the (settlement) reaction of the LVDT's (local strain instrumentation) is noted at various time intervals. At 200 kPa the creep rate is allowed to drop to acceptable levels (less than 0.1375 mm/hour) and the sample is then saturated applying water at a pressure of 15 kPa through the back pressure duct. Water is allowed to pass through the sample from the back pressure and out the top pressure valve, whilst the (collapse) response of the LVDT's is noted. The incremental increases of stress are then continued to 400 kPa and finally to 600 kPa, with the soil saturated. The pressure application instrumentation of the triaxial apparatus could only apply a maximum pressure of slightly over 600 kPa and therefore the sample could not be loaded to 800 kPa as in the oedometer apparatus. After the sample had been subjected to the highest stress the unloading cycle commenced, and the stress on the sample is reduced in increments to cell pressures equivalent to those used in the loading cycle, allowing for the reduction of the creep rate to acceptable values at each increment. The (rebound) reaction of the LVDT's is again noted during the unloading cycle.

Throughout the test the top pressure valve is left open to release and prevent the build up of pore air pressure, whilst the pore water pressure valve is left closed, as these measurements are not required in this test. The back pressure valve is also closed until the saturation of the sample at 200 kPa, after which it is left open. The cell pressure valve is open throughout the test to control and measure the cell pressure.

8.9. Calculation of Volume Change Behaviour

The void ratio is plotted against applied load to assess the volume change behaviour of the samples. To calculate the initial void ratio at the bedding load, firstly the bulk density is calculated using Equation 8.2, from this the dry density is calculated using Equation 8.3 and the initial void ratio is calculated using Equation 8.4. The change in void ratio of the odometer collapse potential test is calculated from the displacement measurement of the strain gauge using Equation 8.5. The condition of 3 dimensional strain that occurs in a triaxial cell meant that the change in void ratio for this test is calculated from the measured axial strain (from the LVDT's) multiplied by 3 assuming strain in all dimension is equal, as shown in Equation 8.6. The collapse potential and settlement are then calculated from the change in void ratio using Equations 8.7 and 8.8 respectively (Jennings and Knight, 1975).

$$\rho(bulk)=W(soil)/Area(sample) \qquad \text{Equation 8.2}$$

$$\rho(dry)=\rho(bulk)/(1+w) \qquad \text{Equation 8.3}$$

$$eo=[\rho(particle).1000/\rho(dry)]-1 \qquad \text{Equation 8.4}$$

$$\Delta e(oed)=[\Delta H/Ho].[1+eo] \qquad \text{Equation 8.5}$$

$$\Delta e(tx)=[[3.\Delta L]/Lo].[1+eo] \qquad \text{Equation 8.6}$$

$$CP(\%)=[\Delta e/[1+eo]].100 \qquad \text{Equation 8.7}$$

$$Set(\%)=[\Delta e/[1+eo]].100 \qquad \text{Equation 8.8}$$

The properties of the soil determined from the different tests carried out were analyzed along with the results of the collapse potential tests. The variances between the results of the triaxial and oedometer collapse potential tests were assessed to ascertain differences in the way volume change is measured in these apparatus'.

9. PROBLEMS AND MEASUREMENT DIFFICULTIES EXPERIENCED DURING TESTING

9.1. Material properties of residual granitic soils affecting testing

9.1.1. Non-Homogeneous Samples

The non-homogeneous nature of granitic rock and consequently residual granitic soils is problematic in collapse and settlement testing procedures. The smaller the sample size the more exaggerated this problem becomes. A plate load test will cover a larger area and range of variation, and therefore give more representative results, whilst laboratory tests such as the triaxial and oedometer collapse potential tests which require a small sample size will only encompass a small amount of variation and be affected greatly by non-homogeneous samples.

Intrusive igneous rocks such as granite are known to vary petrologically over short distances and often contain inclusions such as quartz veins and hydrothermally altered zones. This can lead to differing weathering products in adjacent areas in the residual granitic soil, with differing collapse and settlement characteristics (Brandl et al., 2006b).

Over time, leaching of certain zones in residual granite soils can cause piping, with leached channels and pipes resulting in higher porosity and hydraulic conductivity in these areas and a higher potential for further leaching along these preferential drainage paths. Structural joints and discontinuities which existed in the granitic parent rock may result in preferential drainage paths and leached zones, horizons and planes in the residual granitic soils. These leached zones have differing properties from the surrounding soil, and may have higher void ratios, higher porosities, and lower strength. Based on these assumptions leached zones should show higher collapse and settlement. If samples from these zones are tested for collapse or settlement it may cause the overestimation of these values or vice versa if a less leached sample is tested

(Muxart et al.,1994). An example of these preferential drainage zones and resultant anisotropy in a residual granitic soil can be seen in Figure 9.1.

Figure 9.1: Preferential drainage paths in sample (Bushbuckridge) indicated by black arrows

9.1.2. Moisture Content and Degree of Saturation

Results from collapse potential testing, using both the oedometer and the triaxial apparatus, can be difficult to interpret, because if a tested sample has a degree of saturation close or above the critical degree of saturation (specific to that material) for collapse to occur, the results may show minimal collapse and only conventional settlement (Schwartz, 1985). This effect may be especially detrimental in instances where the geotechnical field investigation of a site to be developed is done when a potentially collapsible soil is above the critical degree of saturation (e.g. the wet season) and the collapsibility is not identified due to this effect. The structure may then be constructed when the potentially collapsible soil's degree of saturation is below this critical state (e.g. the dry season), and collapse may occur. The time of year and moisture content of the soil during sampling and construction are important factors in the potential collapse of a medium and it is important that these are noted and analyzed by the geotechnical professional.

9.2. Oedometer Test

Jennings and Knight (1975) stress that collapse potential test results should be used as an index value to the severity of the collapse problem, and not to determine design parameters. Despite this warning Schwartz (1985) describes that in geotechnical practice many oedometer collapse potential values are being considered as more than just index values, and used as design parameters.

Some problems have been identified with the testing of collapse using the oedometer apparatus, such as bedding errors and unconfined lateral strain. For these reasons, values ascertained from oedometer collapse potential testing should only ever be used as an index to collapse settlement and not as input to engineering design.

9.2.1. Bedding Errors

The majority of residual granitic soils are generally comprised of coarse grained sand and gravel particles. Furthermore these soils may contain large quartz grains and weathered vein quartz which can be up to coarse gravel, cobble or even boulder size in some instance. This is problematic in oedometer testing, as 1 mm diameter quartz asperites protruding from each side of an oedometer sample can lead to a bedding error of 2 mm, which is approximately 10% of the height of the sample, therefore causing a bedding error of 10% (Rust et al., 2005). It can be seen that bedding errors can potentially greatly affect values attained from oedometer testing especially with coarser grained materials. This error is theorized to lead to an over estimation of collapse and settlement measurements in the oedometer apparatus, as compared to in-situ conditions.

9.2.2. Lateral Strain

In a one-dimensional oedometer test, great care must be taken to make sure the sample is cut to fit inside the oedometer ring as tightly as possible to impart the sample with the maximum achievable lateral reinforcement. If the sample fits loosely inside the ring no lateral stress will develop (as exists in situ) and therefore horizontal strain of the sample may occur. Whilst vertical strain is measured horizontal strain is not and this horizontal strain will result in an increased vertical strain of the sample, due to Poisson's effect. Lateral strain causes an increase in the vertical strain of a sample as there is space for the soil grains to move both horizontally and vertically. This error will lead to an overestimation of settlement and collapse as only the artificially increased vertical strains are used to determine volume change (Rust et al., 2005; Maswoswe, 1985).

9.3. Triaxial Collapse Potential Test

9.3.1. Principal Stress Ratio in Triaxial tests

In the triaxial collapse potential test the axial stress placed on the sample causes both axial and horizontal strain in the sample. The measured volume change (collapsibility) of the sample is determined from only the axial strains as these are the only strains measured. In a triaxial test total magnitude of volumetric strain is dependent on mean effective stress and is independent of the principal stress ratio (axial stress/radial stress), but the individual components of volumetric strain depend on the principal stress ratio (Lawton *et al.,* 1991). It can therefore be seen that the measured axial strain is dependent on the principal stress ratio of the tested sample and in future this should be taken into account when carrying out triaxial collapse potential tests. The measurement of both axial and horizontal strain would give a better insight into the mechanical deformation properties of the sample.

9.3.2. Sampling and Sample Preparation

Sampling for the triaxial collapse potential test is difficult and time consuming. Samples should be as large as possible, and completely intact to maintain the in-situ soil structure. The most effective form of block sampling as per the method by Heymann and Clayton (1999) requires that the sample, whilst in-situ, is cut from the soil horizon, then covered in wax which entails the melting of wax on site, requiring a number of instruments, namely candles, a heat source (generally gas cylinders are the most effective) and a paint brush (used for application of the wax). Furthermore the sample must be wrapped in cling wrap and aluminium foil, this must be done carefully and slowly to avoid damage to the sample. Transporting these large samples is also hazardous as the sample may be damaged under its own weight (approximately 30-40 kg) and carrying these samples is difficult. It recommended that large block samples are transported and kept on cushions or pillows to avoid damage to the in situ soil structure.

The cutting of retrieved samples into triaxial test samples without damaging the in-situ structure of the soil is difficult and time consuming and it is also impossible to cut perfectly cylindrical samples by hand. Upon cutting the block samples into smaller triaxial samples many fall apart, as the coarse gravel grains and weathered quartz vein material (often found in residual granites) catch on the cutting tool being used, dislodging a much larger pieces of soil. Coarse residual granitic materials also prevent the sides of the sample from being perfectly smooth. As the samples are cut by hand and the dimensions are estimated by eye the samples are not perfectly cylindrical, this differs from re-compacted triaxial samples that are compacted in a perfectly cylindrical mould. It is recommended that the retrieved undisturbed field sample has dimensions of at least 250 mm x 250 mm x 250 mm, and that the method of sampling prescribed by Heymann and Clayton (1999) is used.

9.3.3. High Sensitivity of the LVDT’s

The high sensitivity of the LVDT’s is problematic during testing, as any movement of the LVDT’s or associated wiring would change the position of the LVDT’s enough to change the readings of the LVDT’s substantially. This is most important during the placement of the triaxial cell cap over the soil sample and attached LVDT’s, as the LVDT’s had been calibrated to their zero point or maximum extension (-5.000volts or 0 mm), and any change in the LVDT’s will then affect the range of the LVDT’s. It is advised that at least two people carry out this task. After the cell cap has been positioned the LVDT’s will show slight displacement of the sample during filling of the cell with water, but once the cell is filled and a pressure is applied, the LVDT’s show very little movement other than strain of the sample caused by pressure change and/or saturation.

9.3.4. Cost

The costs associated with the triaxial collapse potential test are far higher than those of oedometer testing. This is one of the main limiting factors when considering the application of the triaxial collapse potential test.

The higher costs of triaxial collapse potential testing are due to higher price of:

- The triaxial cell and LVDT equipment required.
- The sample recovery, packaging, transport and preparation.
- The training required to train competent operators.
- Time consuming methodology.

Due to these factors triaxial collapse potential testing is more expensive and time consuming to carry out than oedometer testing, and this is a limiting factor in the application of this test.

10. SAMPLING SITE CHARACTERISTICS

It was attempted to locate residual Archaen granitic samples with a high collapsibility, therefore the sampling sites were selected based on the criteria set out by Brink and Kantey (1961). Sites were selected that were underlain by Archaen granitic bedrock, in climatic regions with a water surplus and were situated on moderate gradients. The bedrock geology, topography and climate of the sampling sites were therefore broadly similar, largely eliminating site aspects as a potential variable. All of the four sites are below the former level of the African erosion surface, and therefore have been “pre-weathered”, the leaching and chemical alteration (particularly of plagioclase to kaolin) caused by this cyclic erosion give these soils a high probability of having formed a collapsible grain structure (Partridge and Maud, 1987).

Bedrock geology is an imperative variable in the soil formation and consequent natural hazards and geotechnical properties of the resultant residual soils, including the development of collapsible soil fabric. The residual granitic soils were collected from areas underlain exclusively by Archaen Granitoid (Basement Granite) rocks. The four localities where these residual soils were sampled from were areas close to, Bushbuckridge (Mpumalanga), Midrand (Gauteng), Paulpietersburg (KwaZulu Natal) and Tzaneen (Limpopo).

10.1. Bushbuckridge

These samples were taken from near Bushbuckridge at the co-ordinates 24°53’43.7”S and 31°06’28.8”E from an erosion gully adjacent to the road between Hazyview and Bushbuckridge in the Mpumalanga Province. The sample was retrieved on an east facing slope with a gradient of approximately 10° from between 0.10 m and 0.78 m below natural ground level (ngl) any previously overlying colluviums or hillwash covering the residual granitic material had been eroded away. An aerial photograph of the site can be seen in Figure 10.1.

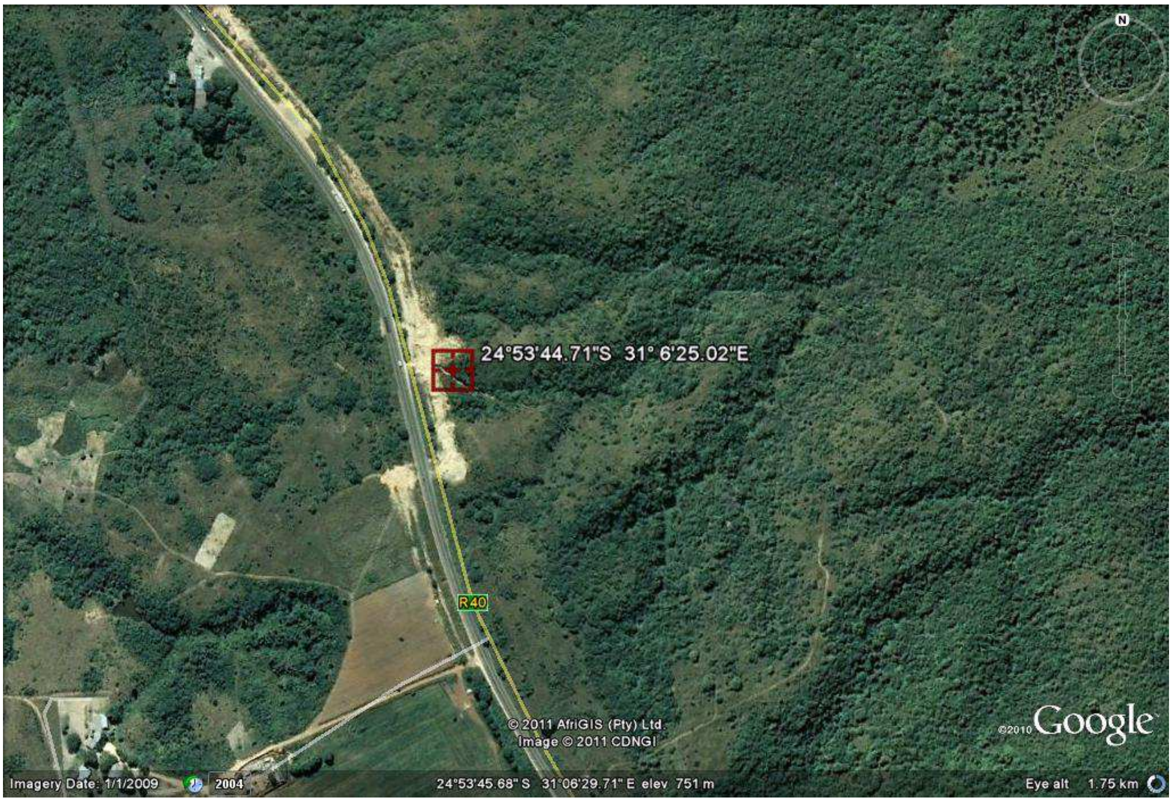


Figure 10.1: Sampling site of the Bushbuckridge sample (Google earth, 2012)

The sampled material was slightly moist, white, blotched light yellow, streaked black and mottled red, intact clayey silty sand. The material was observed to have remnant granitic structure, with the black and red staining along the remnant joint planes thought to be a result of these preferential flow paths. Despite the remnant rock structure the soil profile was seen to be micro-voided in places and have collapsible fabric, the coarser fraction of the soil was seen to be made up of sub-angular quartz, and sparse pebble to cobble sized weathered vein quartz was observed in the profile.

The samples extracted from near Bushbuckridge are composed of residual material from the Archaen granitic rocks of the Nelspruit Suite (Nelspruit Batholith), in the eastern parts of the Kaapvaal Craton. These rocks are Meso-Archaen in age (about 3105±3 Ma) and made up of a wide variety of textural classes, including gneisses, and porphyritic granites. The porphyritc granites are dominant throughout the Nelspruit Suite and grade between a granodiorite and a quartz-monzonite. The granitoid rocks of this formation commonly contain quartz, feldspar, microcline, perthite and biotite (Brandl *et al.*, 2006).

10.2. Midrand

The samples taken from the Midrand area in Gauteng were taken from near Kyalami Race Track at a road cutting on the R55 between Centurion and Sunninghill at co-ordinates 26°00’07.93”S and 28° 04’35.93”E. The area which was sampled is on a south facing slope with a gradient of approximately 5°-10° .The area drains into the nearby Juskei River, and the sample was retrieved from between 1.47 m to 2.38 m below ngl. The top 1.47 m of the soil profile consisted of hillwash and colluvium. An aerial photograph of the site can be seen in Figure 10.2.

Figure 10.2: Sampling site of the Midrand sample (Google earth, 2012)

The sampled material was slightly moist, light grey, mottled orange and yellow voided gravelly silty sand, and due to its colouration and visual inspection of material structure appeared to be leached and have a collapsible fabric. No remnant granitic rock structure was observed and the coarse soil grains were generally sub-rounded.

The bedrock geology of the samples taken from the Midrand region is Archaen granitic rock from the Johannesburg Granite dome of the central Kaapvaal Craton, and more specifically from the Lanseria Gneisses. These rocks are Paleo-Archaen in age (3340±3 Ma) and make up most of the northern half of the Johannesburg Granite Dome. The Lanseria Gneisses consist of trondhjemitic and tonalitic gneiss with minor areas of migmatite containing xenoliths of mafic and ultra-mafic material (Brandl *et al.*, 2006).

10.3. Paulpietersburg

The samples taken near Paulpietersburg were taken from a railway cutting on the Ermelo to Richards Bay Transnet Freight Rail Coal Line at co-ordinates 27°21'48.9"S and 30°53'49.5"E. The natural gradient of the site was a north west facing slope with a gradient of 10°-15°, the sample was taken from 1.20 m to 2.30 m below ngl and was directly overlain by residual granitic soils (0.15 m to 1.20 m below ngl) and by a thin layer of colluvium (0 m to 0.15 m below ngl). An aerial photograph of the site can be seen in Figure 10.3.

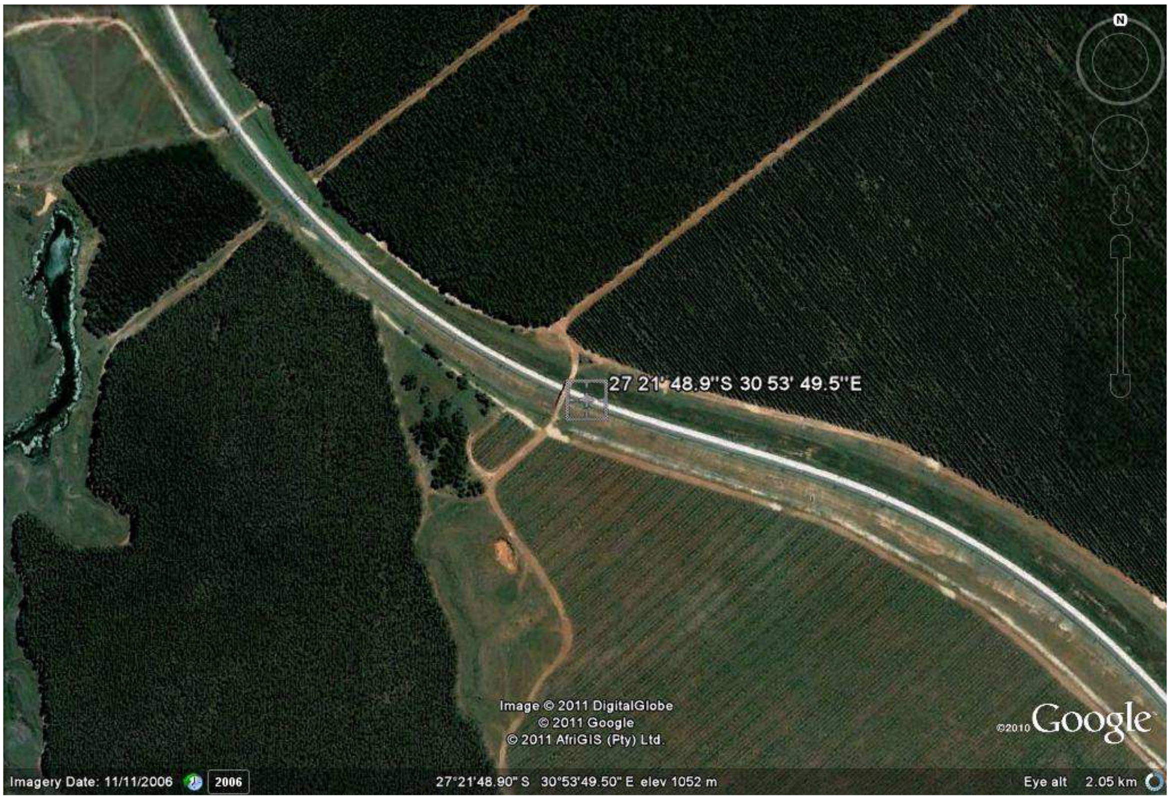


Figure 10.3: Sampling site of the Paulpietersburg sample (Google earth, 2012)

The Paulpietersburg sample consisted of moist, beige mottled orange brown and white, loose, intact, clayey very silty sand. The material had remnant granitic structure which showed slight black staining on the remnant joint planes. The coarse grained particles of the material were generally sub-angular quartz sand, the silt fraction was composed of weathered quartz and feldspar and the clay fraction was observed to consist almost entirely of kaolin clay.

The parent material of the soil samples near Paulpietersburg is part of the Anhalt granitoid suite that makes up part of the Archaen granites of the South East Kaapvaal Craton. These rocks are paleo-Archaen in age (3250±39 Ma), and have a quartz and feldspar based petrology. The intrusions making up this granitoid suite are generally tabular in shape and consist mainly of Trondhjemites, Tonalities and Granodiorites (TTG's) facies. These granitoids commonly have a well-developed fabric. The trohdjemite facies is the major occurring facies, it is normally coarse grained, and comprises of feldspar, quartz, microcline and biotite (Brandl *et al.*, 2006).

10.4. Tzaneen

The Tzaneen sample was taken from a road cutting in the Modjadi Valley at co-ordinates 23° 39' 12.27" and S 30° 16' 16.07" E. The sample was taken on a north west facing slope with a gradient of 10°-15° at a depth of 1.50 m to 2.27 m below ngl. The area drains into the Molototi River and is overlain by hillwash, and underlain by completely weathered closely jointed soft rock granitic bedrock. An aerial photograph of the site can be seen in Figure 10.4.

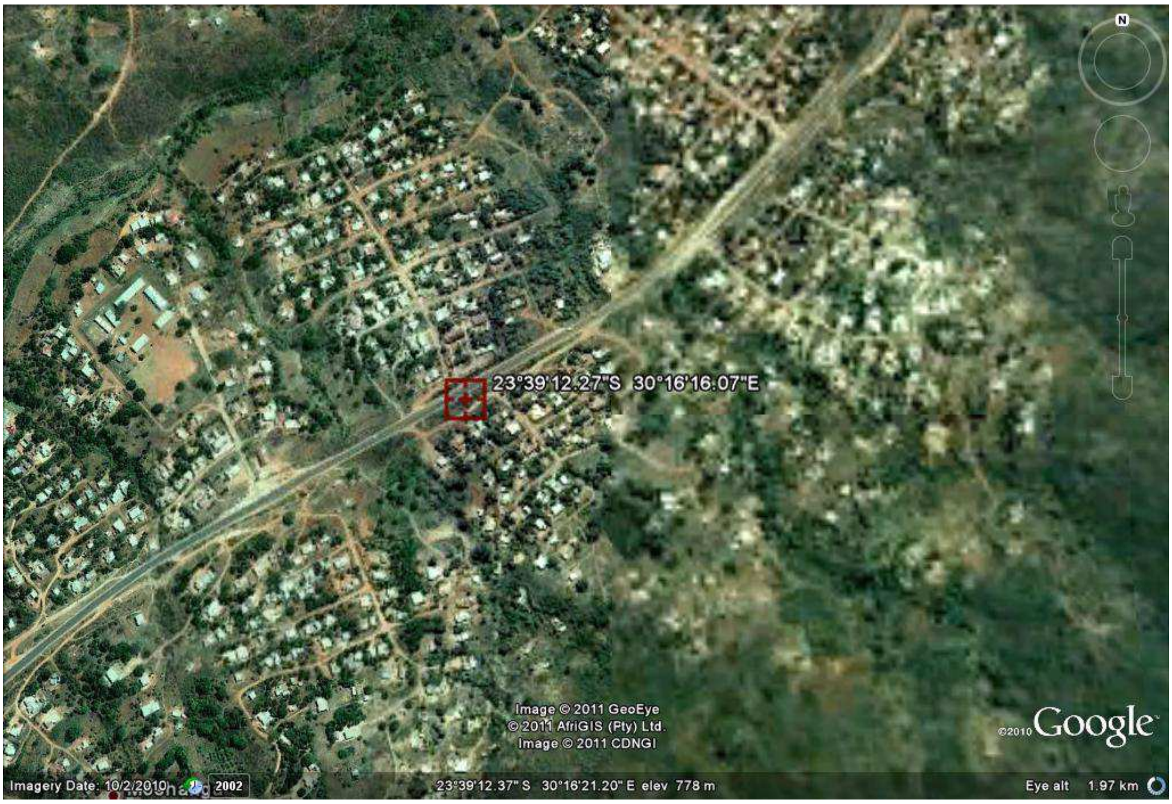


Figure 10.4: Sampling site of the Tzaneen sample (Google earth, 2012)

The material sampled in Tzaneen consisted of slightly moist, orange red, voided, medium dense clayey silty sand. The soil was observed to have no remnant granitic structure and to be highly chemically altered, with abundant sub-rounded weathered quartz grains. The soil was seen to have a very voided structure with silty to fine sandy quartz grains in a matrix of abundant kaolin clay sized and silt sized particles.

The samples in this region were taken from the Modjadi Valley within the Molototsi River catchment, the underlying geology of the area is Duivelskloof Leucogranite of the Archaen granitoids making up the north eastern Kaapvaal Craton. The Duiwelskloof Leucogranite is Neo-Archaen in age and composed of a large batholith natured intrusion stretching south from the Giyani Greenstone Belt. This body shows signs of tectonic influence, as its northern boundary with the Giyani Greenstone Belt is linear in nature although its other contacts with surrounding granite gneisses are not well defined (Brandl *et al.*, 2006). The sampling area is located in the eastern part of the Duivelskloof Leucogranite, where the granite was of medium grained texture and a beige colour. Irregular pockets of more pegmatoidal granite are also common, with the granite being much finer grained, richer in dark minerals and showing more foliation towards contacts with greenstone belt material (Brandl *et al.*, 2006). The main components of this rock is sodic plagioclase, orthoclase, quartz, microperithic microcline, biotite (some of which may have been altered to chlorite) and muscovite, and it varies in composition from a syenogranite to a monzogranite and has a peraluminous nature (Freese, 2009; Brandl *et al.*, 2006).

The soil profiles can be seen in Appendix A.

11. DISCUSSION

The discussion will focus around the collapse potential and settlement measured in the samples of residual Archaen granitic soils collected from around South African. Furthermore the testing methods used to measure the collapse and settlement will be compared and analyzed. Each parameter tested and observed in the soils during the research will be looked at and compared to the collapse and settlement characteristics of the samples. This will lead to an assessment of how various soil parameters may be used to indicate potential collapse or settlement in residual granitic soils. It is anticipated this will give an indication of how much settlement and/or collapse can be anticipated in this material and the effect of this volume change behaviour on construction on residual Archaen granitic soils in South Africa.

During the oedometer and triaxial testing all samples were tested at their in-situ moisture content or their Natural Moisture Content (NMC). The samples generally had high in-situ moisture contents which can be problematic when measuring collapse potential, as a sample with an NMC above its critical degree of saturation will not collapse upon inundation, and collapse potential will be disguised as settlement. For this reason the collapse and settlement of the samples were analyzed by looking at volume change during three stages of the tests. Firstly the collapse upon inundation at 200 kPa, then the settlement at NMC from 25-200 kPa and finally the total volume changes of the samples during the tests (from 25-800 kPa in the oedometer test and 25-600 kPa in the triaxial test). It must be considered that the oedometer samples were loaded to a higher stress than the triaxial samples, and this will affect any attempt to compare the total strain measured in the differing tests above 400 kPa.

11.1. Particle Density

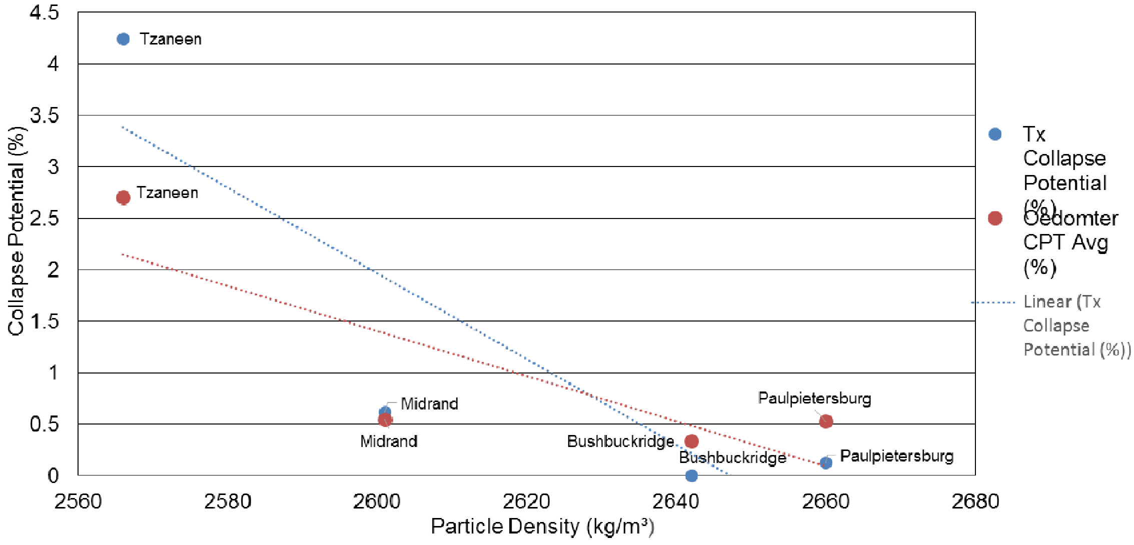
The particle density of the residual granitic soil was determined to ascertain the density of the solid portion of the sample material, to allow a more precise measurement of the samples dry density and void ratio. The particle density of the samples can be seen in Table 11.1.

Table 11.1: Particle density of samples (Soilab (Pty) Ltd)

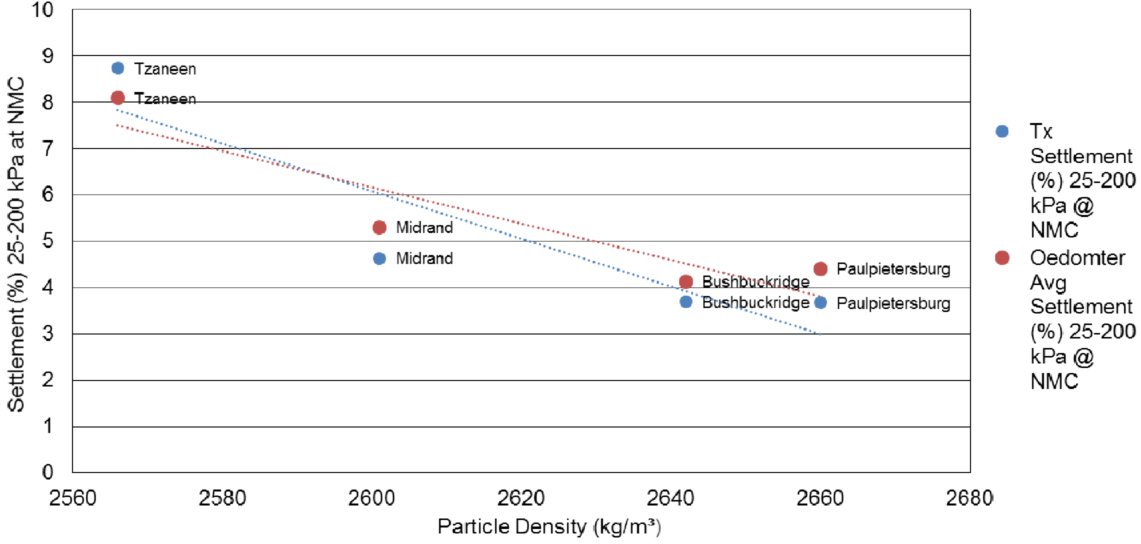
Particle Density	kg/m³
Bushbuckridge	2642
Midrand	2601
Paulpietersburg	2660
Tzaneen	2566

The particle density values attained appear to be typical of sandy material, and compare well with values found for residual granitic soils. Vermaak (2000) found values of 2610-2640 kg/m³ for residual granitic soils in Midrand. The particle densities in relation to the collapse potential and settlement are shown in Graphs 11.1 to 11.3.

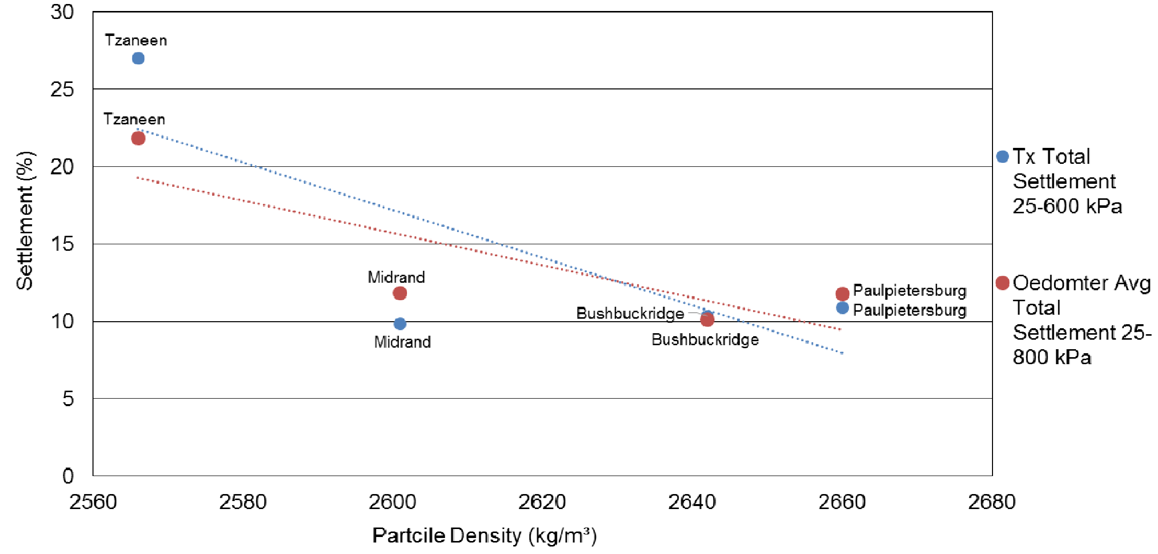
Graph 11.1: Oedometer and Triaxial Collapse Potential vs Particle Density



Graph 11.2: Oedometer and Triaxial Settlement 25-200 kPa at NMC vs Particle Density



Graph 11.3: Oedometer and Triaxial Total Settlement vs Particle Density



There is some correlation between both particle density and collapse, and particle density and settlement. The Tzaneen sample which was tested to have the highest collapse potential and settlement was found to have the lowest particle density. The Bushbuckridge, Midrand and Paulpietersburg samples were shown to have higher particle densities than the Tzaneen sample and showed lower collapse potential and settlement measurements in both the oedometer and triaxial collapse potential tests. Based on this data it appears that particle density may be inversely proportional to both collapse potential and settlement in residual granitic soils.

It is theorized that particle density may be an indicator of the amount of weathering or leaching a residual granitic soil has undergone. During weathering the denser primary granitic rock forming minerals (quartz, feldspar etc.) are weathered to less dense secondary minerals (oxides, hydroxides etc). Using this logic a low particle density soil will be more weathered and leached. Weathering and leaching are known to be essential in the formation of a collapsible grain structure in residual soils, as well as leading to highly voided soils, indicating a relationship between particle density and volume change behaviour.

11.2. Grading and Hydrometer Testing

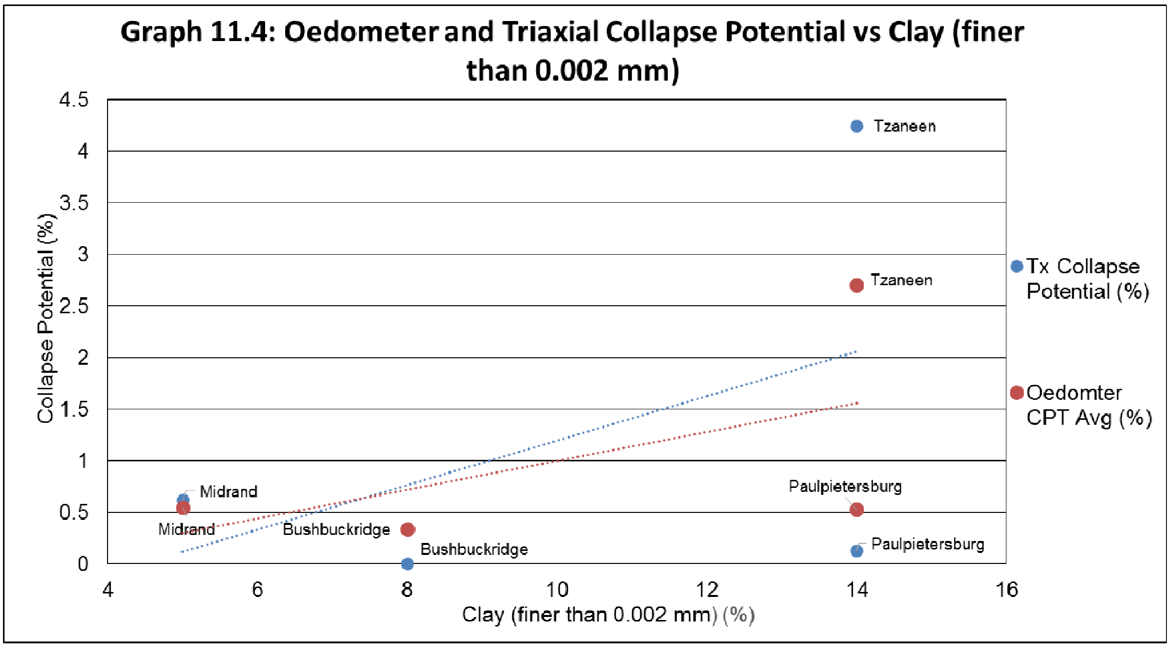
The grading of a soil plays a pivotal role in the collapse mechanism, giving an indication to relative quantities of particle sizes which are required to create a collapsible soil structure with a majority of coarse (generally sand) particles being bridged by a minority of smaller fine (generally clayey) cohesive particles. The laboratory results of foundation indicator testing undertaken on the samples are contained in Appendix B. The grading, hydrometer and Atterberg Limit laboratory results are summarized in Table 11.2.

Table 11.2: Grading and hydrometer results of samples (Soilab (Pty) Ltd)

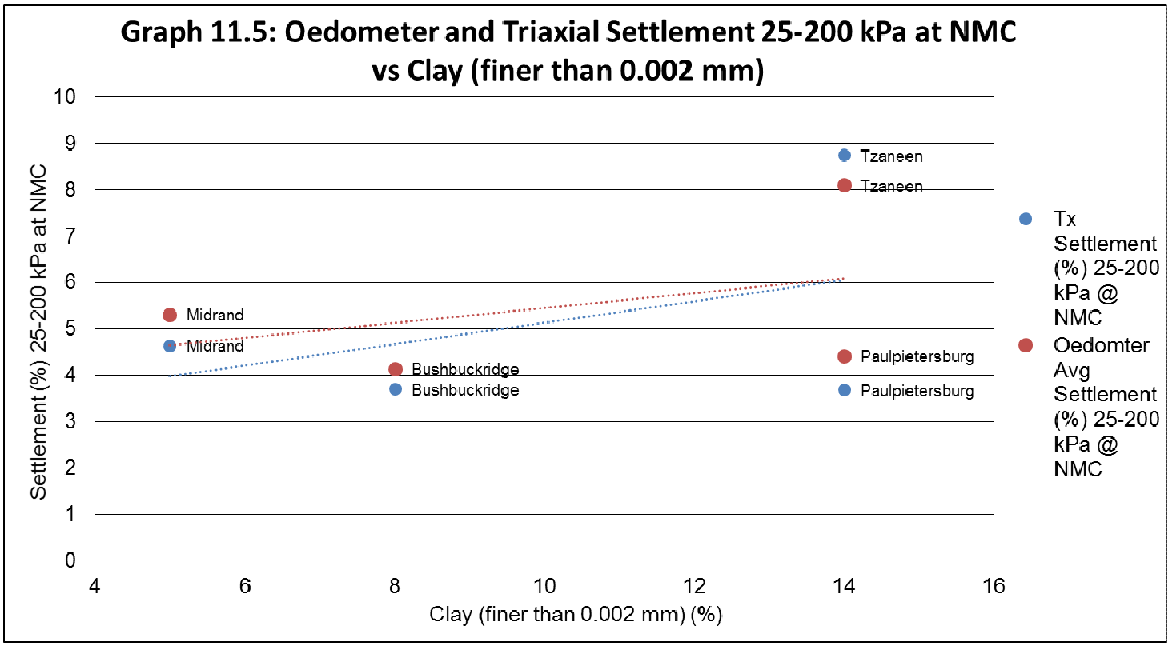
Sample	Bushbuckridge	Midrand	Paulpietersburg	Tzaneen
Clay (%)	8	5	14	14
Silt (%)	23	13	30	34
Sand (%)	64	77	49	51
Gravel (%)	5	5	7	1
Liquid Limit	31	33	41	44
Plasticity Index	7	10	13	15
Linear Shrinkage (%)	3	4	6.5	7
Unified Classification	SM	SC	SM	ML
Potential Expansiveness (Van der Merwe, 1964)	Low	Low	Low	Medium

11.2.1. Clay Content

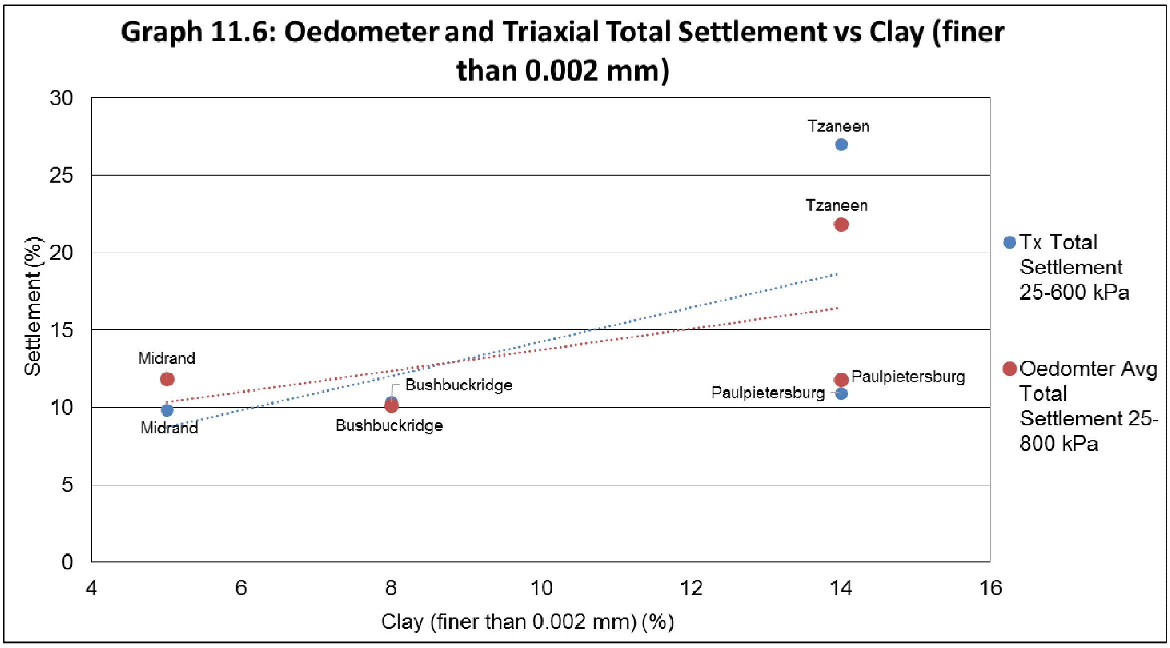
The clay content of the samples (% finer than 0.002 mm) can be seen in relation to the collapse potential and settlement in Graphs 11.4 to 11.6.



Graph 11.4: Oedometer and Triaxial Collapse Potential vs Clay (finer than 0.002 mm)



Graph 11.5: Oedometer and Triaxial Settlement 25-200 kPa at NMC vs Clay (finer than 0.002 mm)



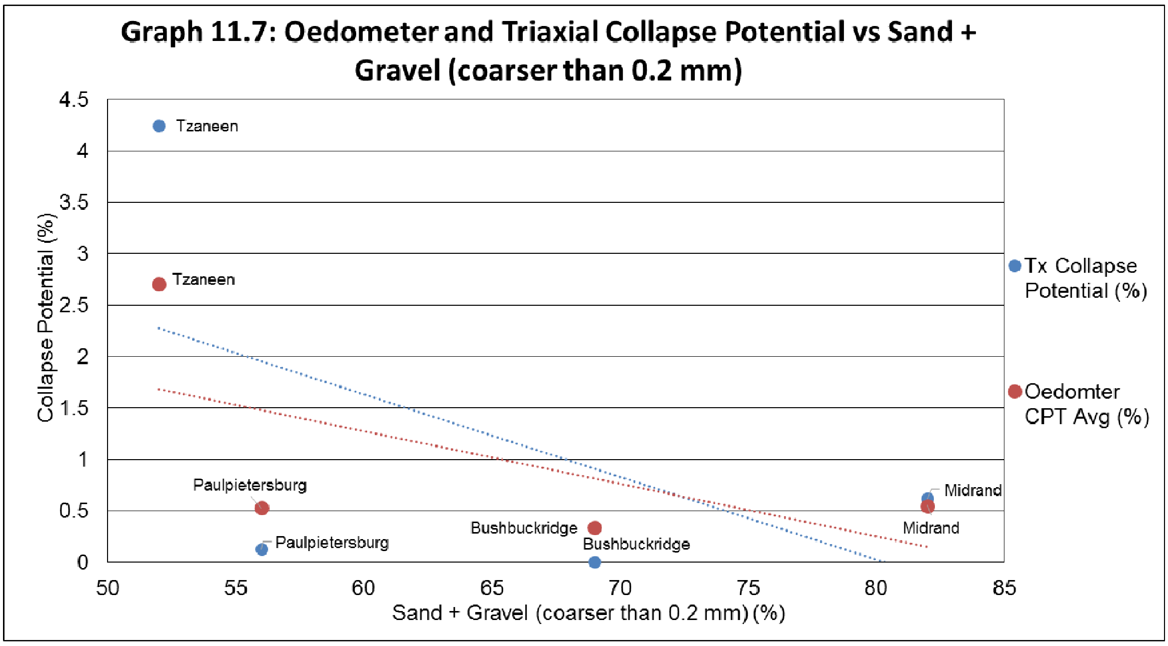
Literature has identified that the potential collapsibility of a material is generally low if there is too little (<5%) or too much (>30%) clay. The study by Dudley (1970) showed the optimum percentage of clay required to produce collapsible fabric is approximately 12%. The relatively high silt contents measured in all samples is surprising, and may be a result of the lack of mechanical dispersion of the clays in test method TMH1-A6, used by the laboratory, allowing much of the clay material to be measured as silt.

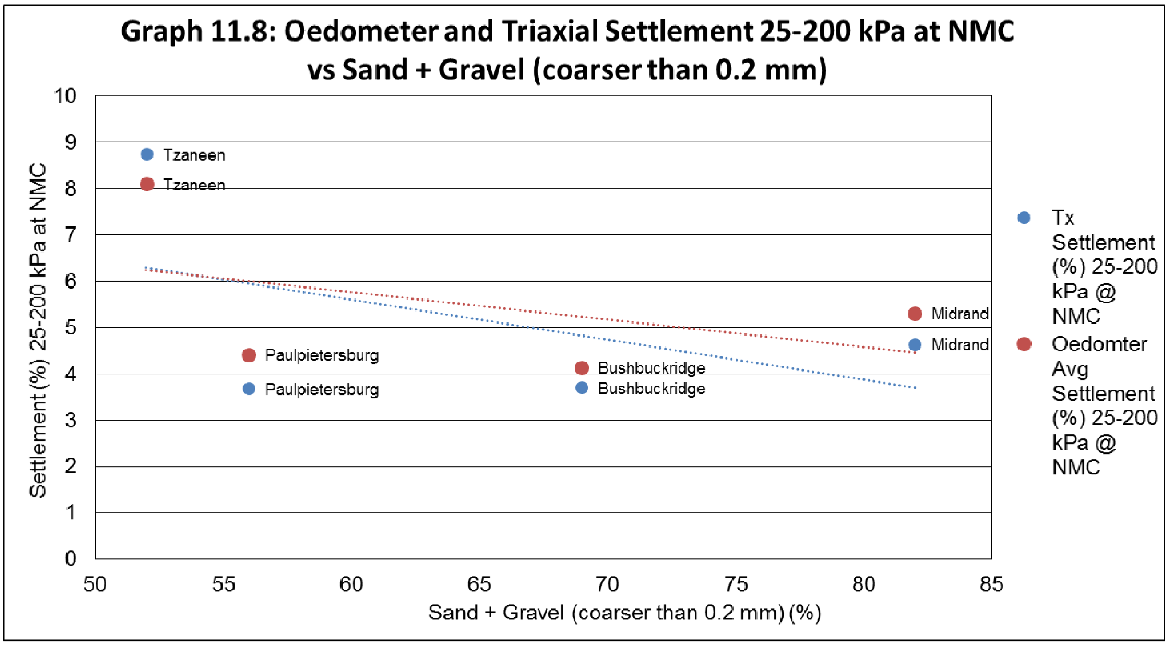
The clay percentages of the samples were relatively similar ranging between 5% and 14%, all the samples fall within the prescribed clay content for collapsible soils. The Tzaneen sample had a clay content of 14% close to the optimum clay content (12%) required for collapse as stated by Dudley (1970).This sample showed the highest collapse potential and settlement. The Paulpietersburg sample which also had a clay content of 14% showed much lower collapse potential and settlement. Based on these results no correlation can be seen between clay content and collapse potential or settlement in residual granitic soils.

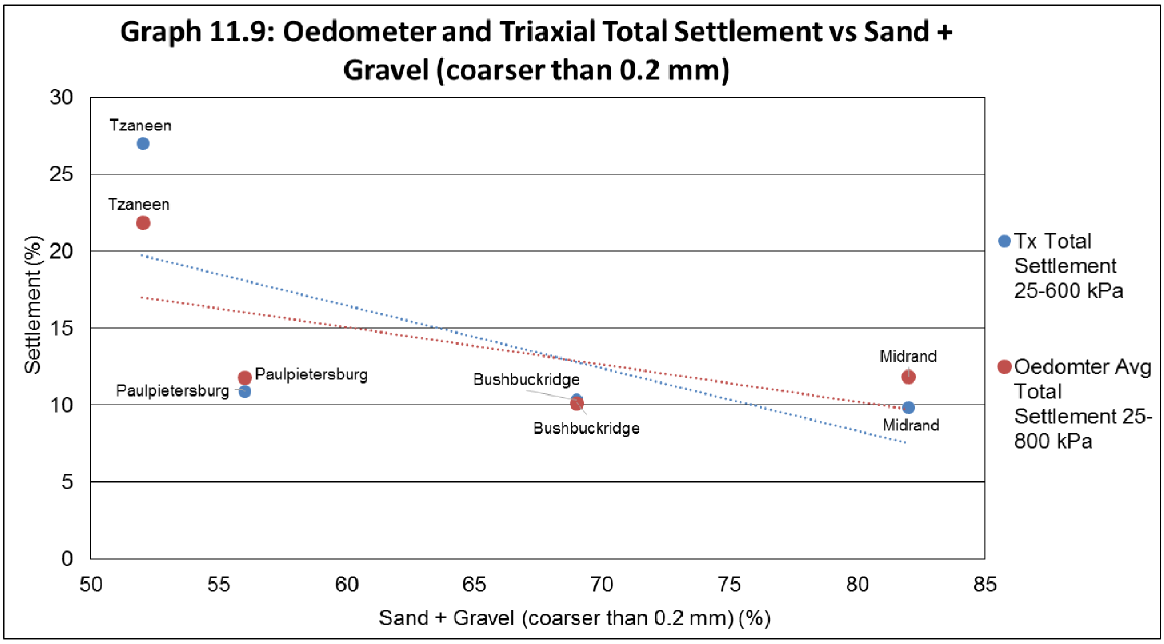
The majority of the residual granitic samples showed low potential expansiveness as was anticipated with the exception of the Tzaneen sample which was measured to have a medium expansiveness (Van der Merwe, 1964).

11.2.2. Sand and Gravel Content

The sum of the gravel and sand content of the samples (% coarser than 0.2 mm) can be seen in relation to the collapse potential and settlement in Graphs 11.7 and 11.9.







As anticipated for residual granitic soils, sand and gravel made up the majority of all four samples with 52-82% coarser than 0.2 mm. The coarser fraction content of the soil didn't correlate to the measured collapse potential or settlement in these materials.

11.3. Moisture Content and Degree of Saturation

The water content is the triggering mechanism of collapse, and therefore in-situ moisture content and degree of saturation are important factors in the testing of collapse potential. Whilst the moisture content is important in ascertaining the quantity of water in a soil, the degree of saturation is far more useful in determining the effects of water on a collapsible soil, as this value takes into account the particle density and void ratio of the soil. The average moisture contents and degrees of saturation of the collected samples can be seen in the Tables 11.3 and 11.4. All the sample were collected during the wet summer season, and were stored in wax, cling wrap and aluminium foil, to maintain in-situ moisture content until testing.

Table 11.3: Average in-situ moisture content of samples

Average In-situ Moisture Content	%
Bushbuckridge	22.04
Midrand	9.57
Paulpietersburg	22.10
Tzaneen	22.20

Table 11.4: Average in-situ degree of saturation of samples

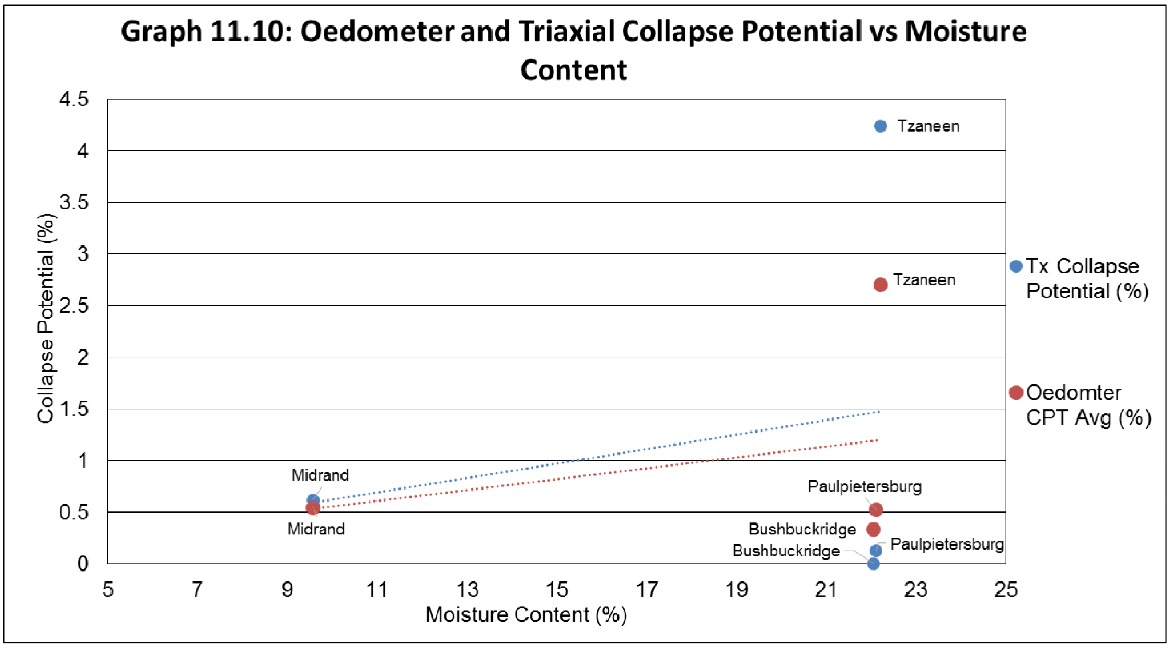
Average In-situ Degree of Saturation	%
Bushbuckridge	82.23
Midrand	44.30
Paulpietersburg	73.30
Tzaneen	46.40

All samples showed high moisture contents and degrees of saturation possibly due to the time of year (summer) in which they were collected. The Bushbuckridge, Paulpietersburg and Tzaneen samples showed very similar high moisture contents, with Midrand sample showing much drier conditions. The moisture contents of the samples are thought to be a result of the variable moisture condition (seasonal and/or precipitation event) of the various soils at the time of sampling.

The degrees of saturation for the Bushbuckridge and Paulpietersburg samples were far in excess of the identified critical degree of saturation for collapse to occur in residual granites of 52%, while the Midrand and Tzaneen samples were only marginally lower than this mark (Errera, 1977). The samples are also far in excess of the degree of saturation identified (50% to 60%) at which collapsible is known to occur in materials with similar grading (silty sands; Jennings and Knight, 1975). The samples showed silt contents of 13-35% and 19-

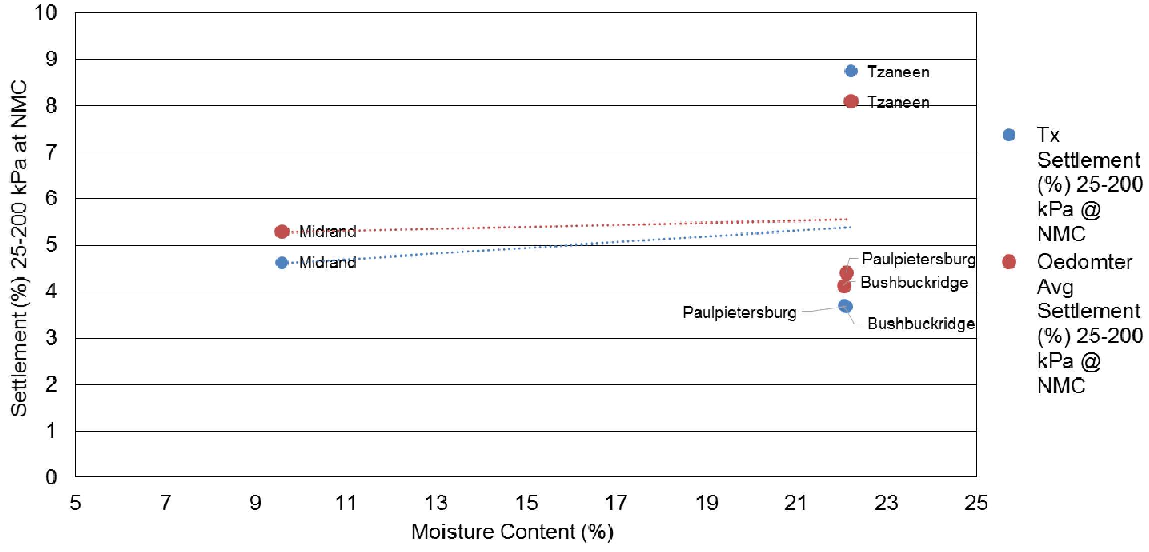
55% finer than 0.075 mm, this indicates that these materials may have slightly higher critical degrees of saturation than those prescribed in previous literature. Using the Unified Soil Classification System (USCS) the Bushbuckridge and Paulpietersburg samples were found to be silty sands (SM), the Midrand sample was found to be a clayey sand (SC) and the Tzaneen sample a low plasticity silt (ML). This indicates that the critical degree of saturation of some of the samples, particularly the Tzaneen sample may be somewhat higher than the values stated.

These findings show that the soils generally contained above or close to the amount of moisture required to trigger the collapse mechanism in residual granitic soils. The relation between the measured collapse potential and settlement against the soils moisture content or degree of saturation can be seen in Graphs 11.10 to 11.15.

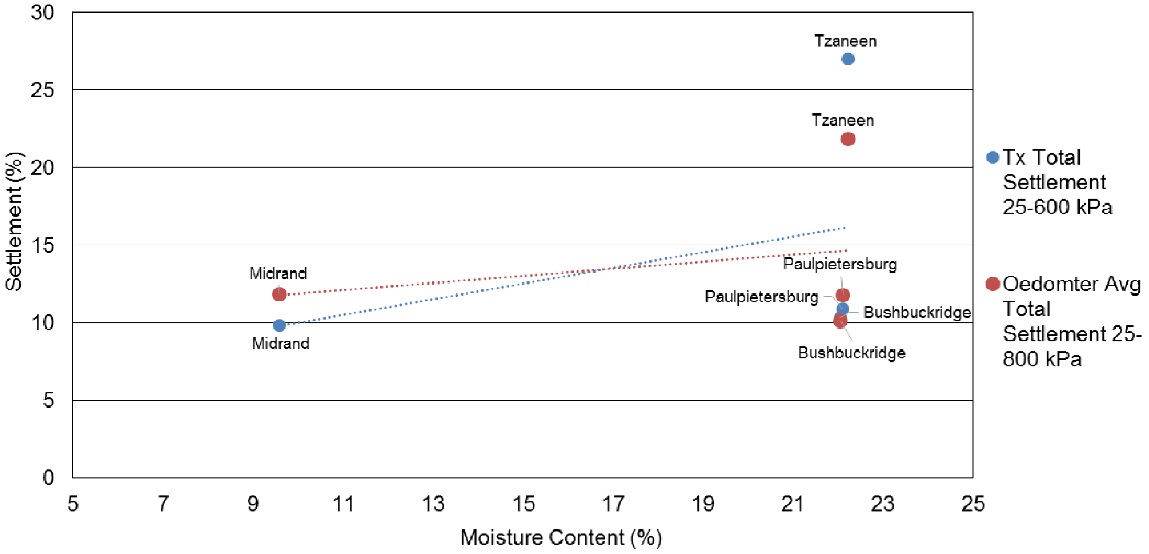


Graph 11.10: Oedometer and Triaxial Collapse Potential vs Moisture Content

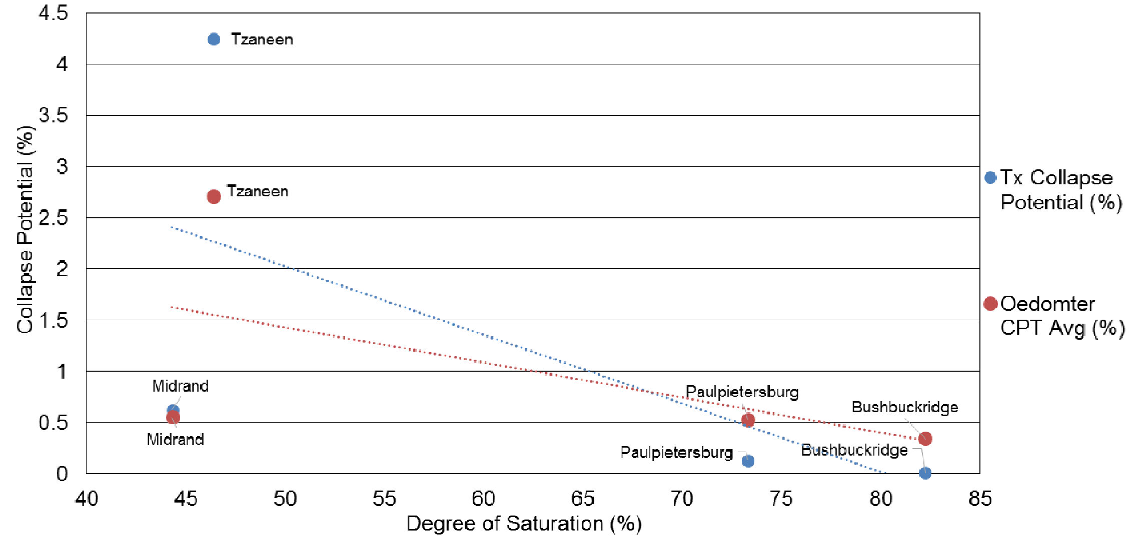
Graph 11.11: Oedometer and Triaxial Settlement 25-200 kPa at NMC vs Moisture Content



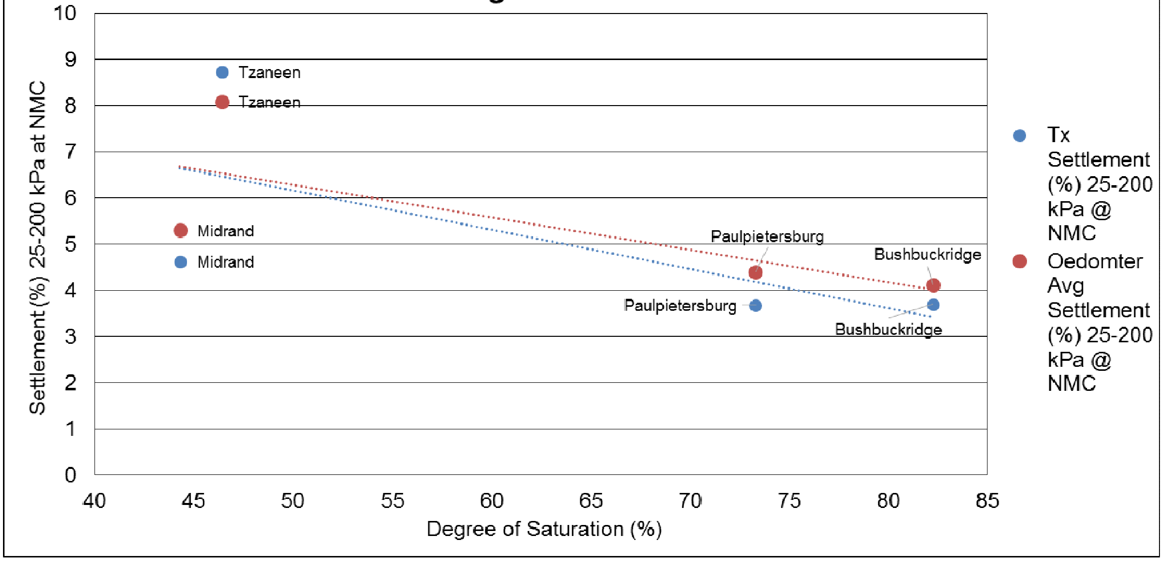
Graph 11.12: Oedometer and Triaxial Total Settlement vs Moisture Content

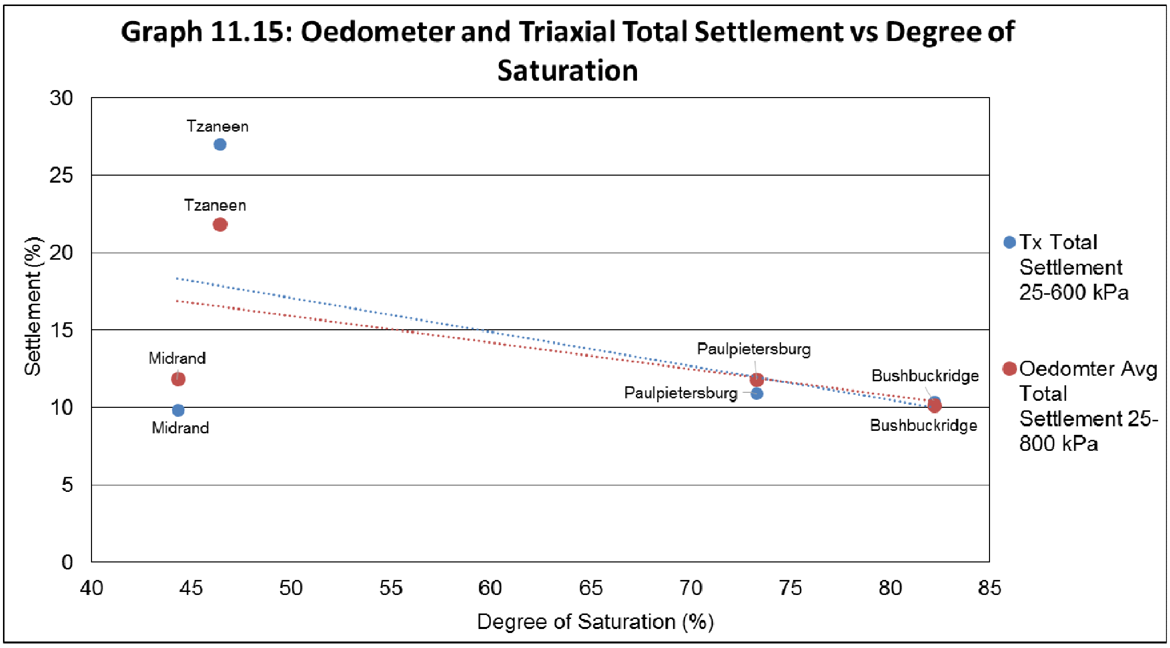


Graph 11.13: Oedometer and Triaxial Collapse Potential vs Degree of Saturation



Graph 11.14: Oedometer and Triaxial Settlement 25-200 kPa at NMC vs Degree of Saturation





The high degree of saturation of the soils during testing is thought to have lead to inflated settlement and reduced collapse potential measurements. The clay particles in the samples may have been weakened (reduced shear strength) from the commencement of the tests, and therefore much of the volume change (normally associated with collapse of the soil fabric) would have occurred before inundation of the samples and would have been measured as settlement.

It was anticipated that moisture content would show little or no correlation to collapse or settlement, as moisture is a value that does not take into account any of the physical properties of the soil. As can be seen from the test results little correlation was shown between moisture content and collapse potential or settlement in the samples.

The degree of saturation, which is used as the measure of water content required to trigger collapse, showed good correlation with the collapse potential of the samples, as the Tzaneen and Midrand samples which showed the lowest degrees of saturation also showed the highest collapse potential. This data indicates that the degree of saturation is (broadly) inversely proportional to collapse potential. Despite this, it is thought this is only true for degrees of saturation close to the critical degree of saturation for a material, as changes in this value far below or far above the critical degree of saturation will have little effect on the volume change behaviour of a soil.

11.4. XRD (X-Ray Diffraction) and XRF (X-Ray Fluorescence) Analysis

XRD and XRF analysis was used to measure the exact chemical makeup of the residual granitic soils. The XRD and XRF analysis give a good indication of both the parent material and degree of weathering of the samples, which are both thought to be important factors in the volume change behaviour of residual granitic soils. The XRD and XRF results obtained from the samples can be seen in Tables 11.5 and 11.6 respectively.

Table 11.5: XRD Analysis Results of samples (Grote, Analytical Facility, Dept Geology, University of Pretoria, 2011)

Minerals	Bushbuckridge (weight %)	Midrand (weight %)	Paulpietersburg (weight %)	Tzaneen (weight %)
Kaolinite	27.43	18.09	51.72	24.78
Microcline	2.02		6.57	
Muscovite	14.84	6.68		
Plagioclase Albite_	17.48	31.39	18.08	
Quartz_	31.06	17.25	17.06	21.28
Sepiolite_	7.16			
Palygorskite_		18.51	0.72	
Orthoclase_		8.08		
Chlorite			5.84	
Chlorite IIb-2				24.78
Gibbsite				6.97
Hematite				2.98
Rutile				3.77

Table 11.6: XRF Analysis Results of samples (Dykstra, Analytical Facility, Dept Geology, University of Pretoria, 2011)

	GSNcert	GSN	Bushbuckridge	Midrand	Paulpietersburg	Tzaneen
SiO2	65.80	65.63	74.04	73.21	61.79	53.93
TiO2	0.68	0.63	0.18	0.22	0.56	1.07
Al2O3	14.67	14.32	15.56	14.73	19.47	24.63
Fe2O3	3.75	3.90	1.55	2.26	5.95	8.45
MnO	0.06	0.05	<0.01	0.03	0.11	0.06
MgO	2.30	2.01	0.20	0.33	0.23	0.01
CaO	2.50	2.72	0.07	0.70	0.77	0.02
Na2O	3.77	3.95	1.46	3.74	3.23	<0.01
K2O	4.63	4.72	2.86	1.30	1.20	0.46
P2O5	0.28	0.23	<0.01	<0.01	<0.01	0.08
Cr2O3	0.008	0.04	<0.01	<0.01	<0.01	0.02
NiO	0.0043	0.06	<0.01	0.01	0.01	0.01
V2O5	0.01	0.01	<0.01	<0.01	0.01	0.01
ZrO2	0.03	0.02	0.02	0.01	0.01	0.02
SO3		0.03	<0.01	<0.01	<0.01	<0.01
WO3		0.06	0.01	0.02	<0.01	0.01
BaO		0.14	0.08	<0.01	<0.01	<0.01
Cl		0.07	0.08	0.11	0.11	0.08
CuO		0.03	<0.01	<0.01	<0.01	0.01
ZnO		<0.01	<0.01	0.02	<0.01	<0.01

	GSNcert	GSN	Bushbuckridge	Midrand	Paulpietersburg	Tzaneen
Rb2O		<0.01	0.02	0.01	<0.01	<0.01
SrO		0.08	<0.01	0.01	0.02	<0.01
LOI	1.32	1.32	3.87	3.30	6.54	11.12

11.4.1. Parent Material

The mineralogy of the parent material (rock) of the residual soil samples can be estimated by looking at the primary (rock forming) minerals still in the soil matrix, and the resultant secondary (chemically altered) minerals that have been derived from alteration of the primary minerals.

All samples similarly showed high quartz percentages (SiO_2 53-75% and Quartz 17-32%), and all of the samples except the Tzaneen sample showed high Feldspar percentages (in excess of 17%). These two minerals are the main rock forming minerals found in granitic rock and indicate that these soils are of a granitic origin. Many other minerals commonly associated with granitic rock were found in the soils, such as muscovite and microcline.

All the samples showed a high percentage of Kaolinite (18-52 weight %), which is a (1:1) clay mineral normally derived from the kaolinization (chemical alteration) of Feldpar minerals such as Albite. This indicates that the parent material of all of these samples had a high Feldspar content. The high concentration of quartz, feldpar and the associated secondary minerals is a good indication of the granitic nature of the parent material of the samples.

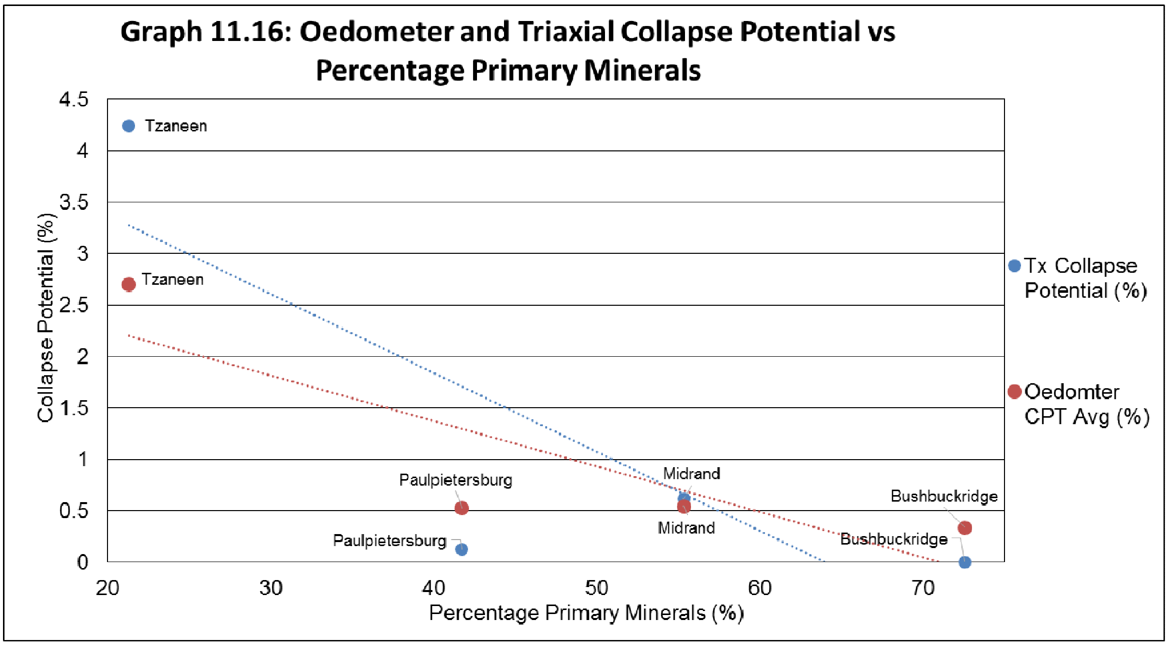
11.4.2. Degree of Weathering

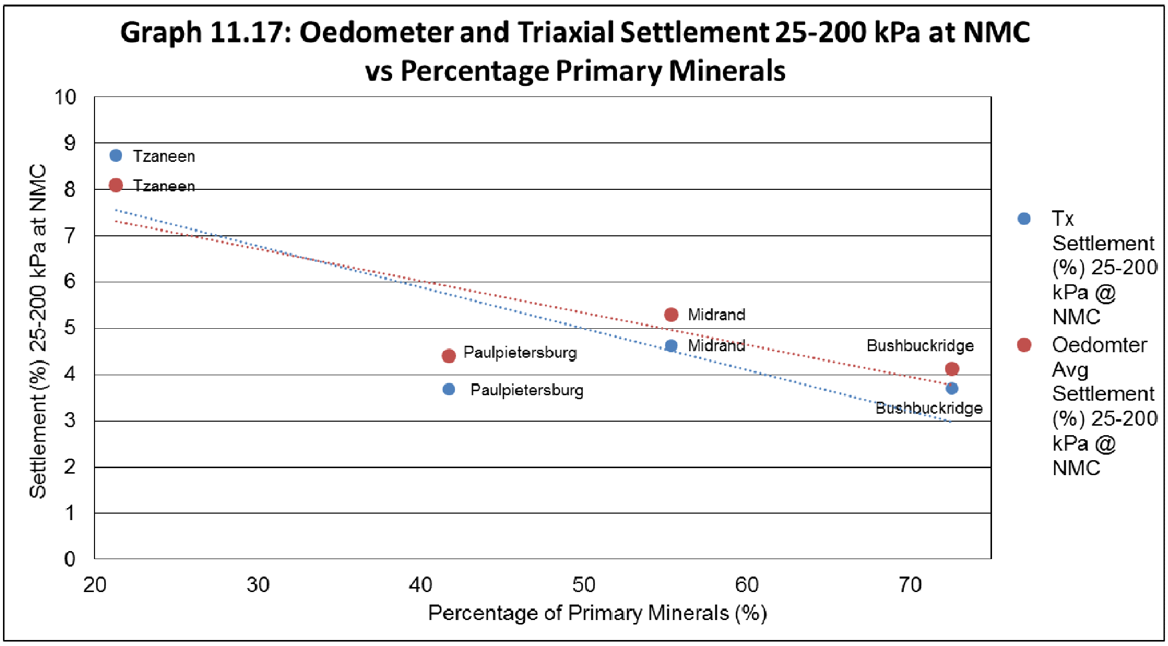
Schwartz (1985) identified the role played by chemical decomposition and leaching in the formation of a collapsible residual soil. The percentage of primary minerals (Quartz, Sepiolite, Muscovite, Rutile and Feldpsar) to secondary and tertiary minerals (Kaolinite, Palygorskite, Chlorite, Gibbsite and Hematite) is anticipated to give a good indication of the degree of weathering (decomposition) of each of the residual granitic soil samples. The percentage of primary minerals remaining in the samples can be seen in Table 11.7.

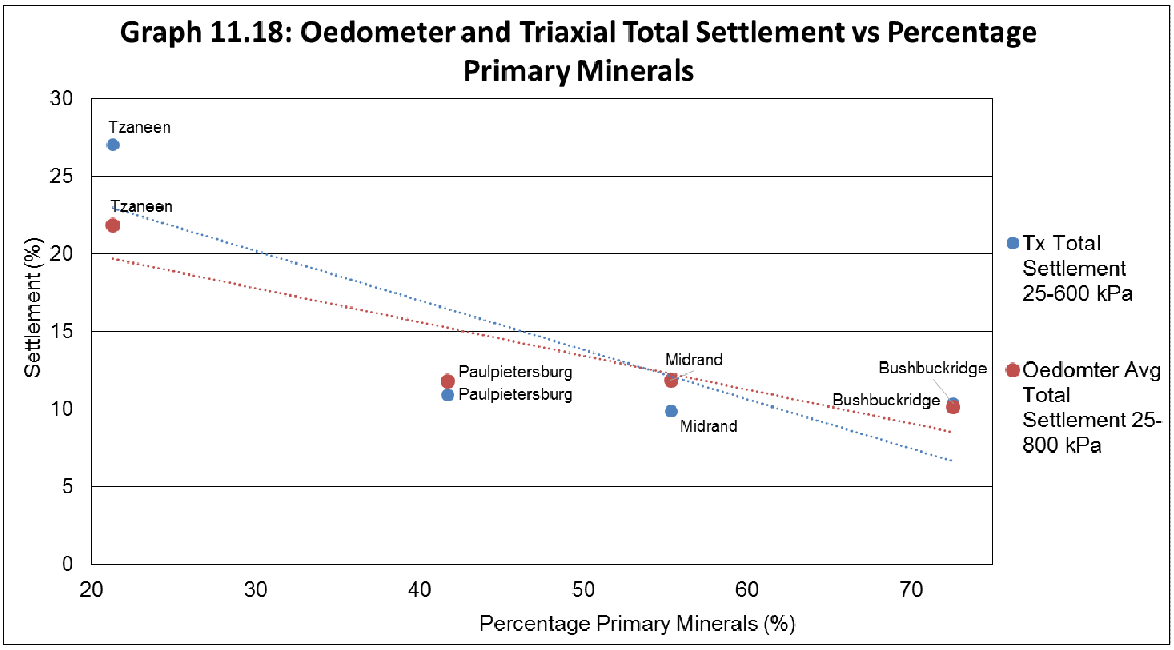
Table 11.7: Percentage of primary granitic minerals in each sample

Primary Minerals	%
Bushbuckridge	72.56
Midrand	55.32
Paulpietersburg	41.71
Tzaneen	21.28

The Bushbuckridge, Midrand and Paulpietersburg samples all showed reasonable amounts of weathering with 40%-70% being made up of primary minerals, whilst only 21% of the Tzaneen sample was found to be made up primary minerals. This indicates that the Tzaneen sample was far more weathered than the other three samples. The percentage of primary minerals can be seen compared to the measured collapse potential and settlement of the samples in Graphs 11.16 and 11.18.


Graph 11.16: Oedometer and Triaxial Collapse Potential vs Percentage Primary Minerals
Collapse Potential (%)
4.5
4
3.5
3
2.5
2
1.5
1
0.5
0
20
30
40
50
60
70
Percentage Primary Minerals (%)
Tzaneen
Tzaneen
Paulpietersburg
Paulpietersburg
Midrand
Midrand
Bushbuckridge
Bushbuckridge
Tx Collapse Potential (%)
Oedomter CPT Avg (%)


Graph 11.17: Oedometer and Triaxial Settlement 25-200 kPa at NMC vs Percentage Primary Minerals
Settlement (%) 25-200 kPa at NMC
10
9
8
7
6
5
4
3
2
1
0
20
30
40
50
60
70
Percentage of Primary Minerals (%)
Tzaneen
Tzaneen
Midrand
Midrand
Paulpietersburg
Paulpietersburg
Bushbuckridge
Bushbuckridge
Tx Settlement (%) 25-200 kPa @ NMC
Oedomter Avg Settlement (%) 25-200 kPa @ NMC



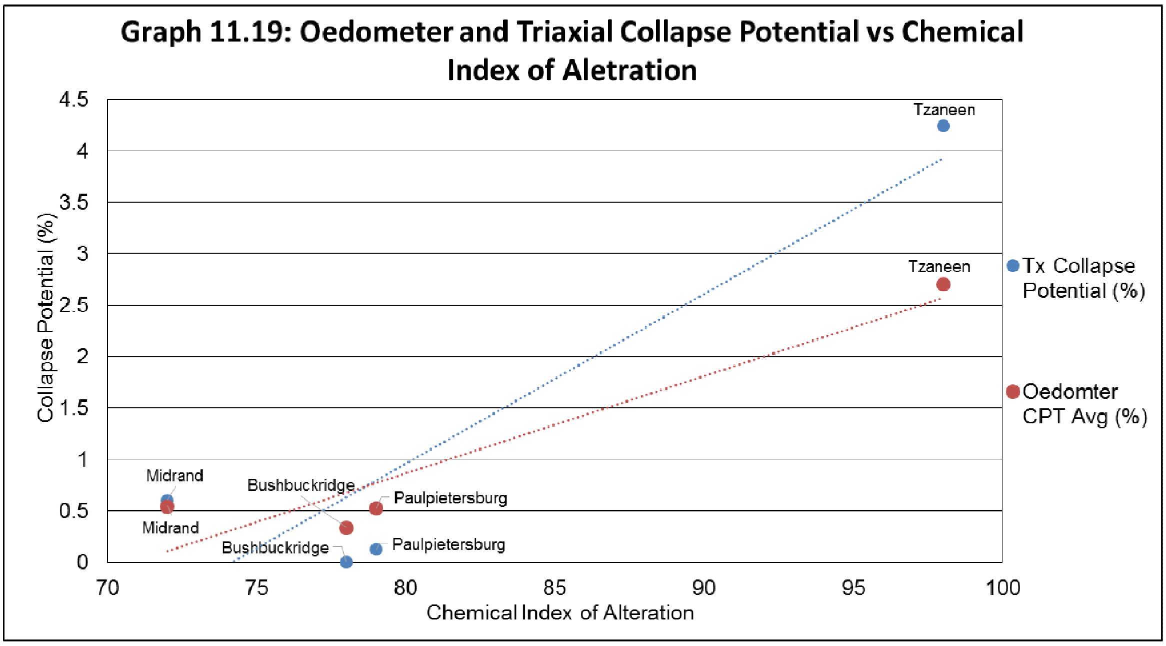
From the comparison is can be seen that the Tzaneen sample which contained the least primary minerals (the most weathered) showed the highest collapse potential and the highest settlement. Besides this the other three samples showed little correlation between the content of primary granitic minerals and collapse or settlement. Despite this it is thought that the content of primary minerals may be a good indication of collapsible residual soils. Visual inspection of the primary granitic minerals in a soil may prove to be a valuable indicator of potentially collapsible soils.

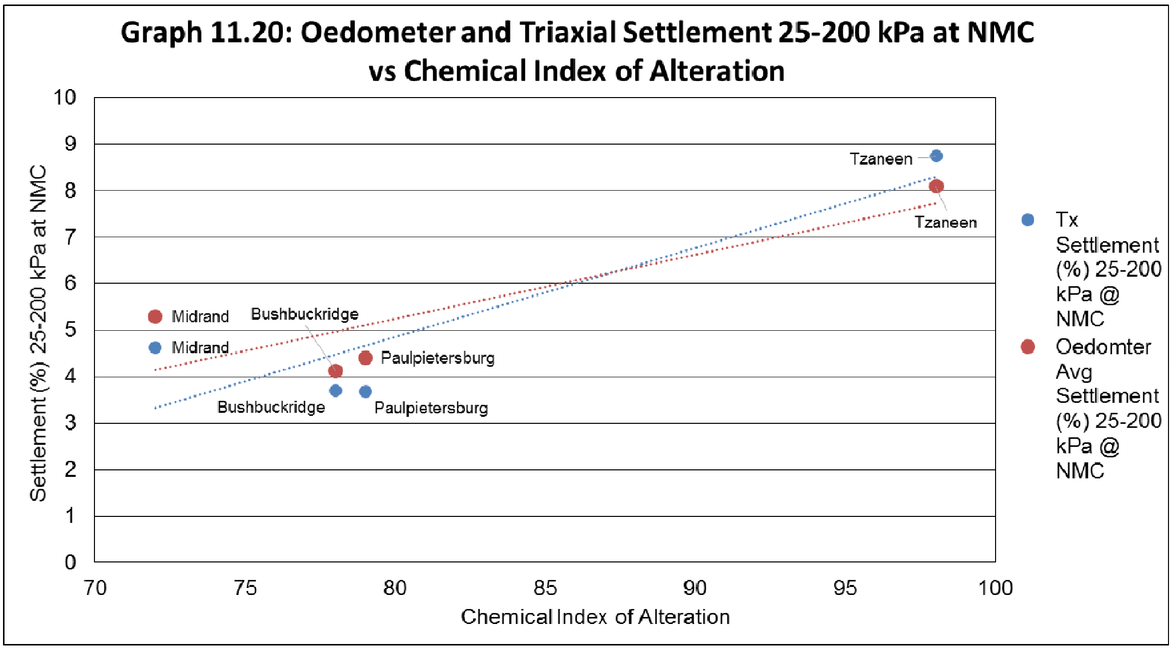
Using the XRF analysis results the degree of weathering of the samples was also calculated using Chemical Index of Alteration (CIA) (Nesbitt and Young, 1982). This index is universally accepted as a good indicator of weathering, having a completely weathered material with a value of 100 and values given for unweathered granite of 45-55 (Nesbitt and Young, 1982). The CIA of the samples is shown in Table 11.8.

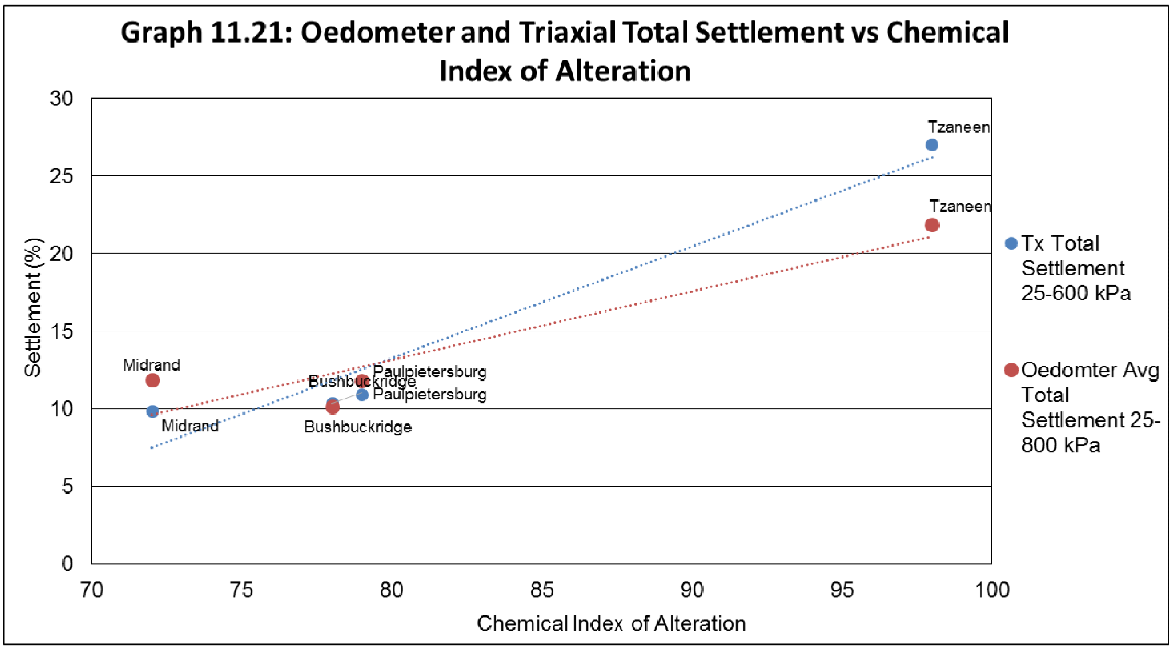
Table 11.8: Chemical Index of Alteration of samples

Chemical Index of Alteration	CIA Value
Bushbuckridge	78
Midrand	72
Paulpietersburg	79
Tzaneen	98

From these results it can be seen that the Bushbuckridge, Midrand and Paulpietersburg samples are at a very similar degree of weathering (72-79), whilst the Tzaneen sample is almost completely weathered (98). The CIA of the samples was compared to the collapse potential and settlement shown in Graph 11.19 to 11.21.


Graph 11.19: Oedometer and Triaxial Collapse Potential vs Chemical Index of Aletration





The comparison of these results shows a very good correlation between the CIA and the measured collapse and settlement. The Bushbuckridge, Midrand and Paulpietersburg samples which showed very similar CIA values also showed very similar volume change behaviour both in collapse and settlement. The Tzaneen sample which had a high CIA showed much higher collapse potential and settlement. This data therefore indicates that the CIA correlates well with collapse potential and settlement in residual granitic soils. Thus it shows that the degree of weathering (calculated by means of the CIA) of a residual granitic soil may be a good indicator of both collapse potential and settlement in residual granitic soils. Although the degree of weathering is potentially a good indicator of settlement behaviour, a condition of internal drainage must exist in the soil to leach the horizon, and produce the void space into which the soil can collapse or settle.

11.5. Pore Fluid Suction

The pore fluid suction of a soil is a value that quantifies the strength of the forces binding the finer portion of the soil together. These forces are vital in determining the critical degree of saturation at which a collapsible soil loses it's shear strength and undergoes volume change (collapse). The in-situ pore water suction of each sample was measured using the filter paper method of Chandler and Gutierrez (1986). The measured pore water suction of the samples can be seen in Table 11.9.

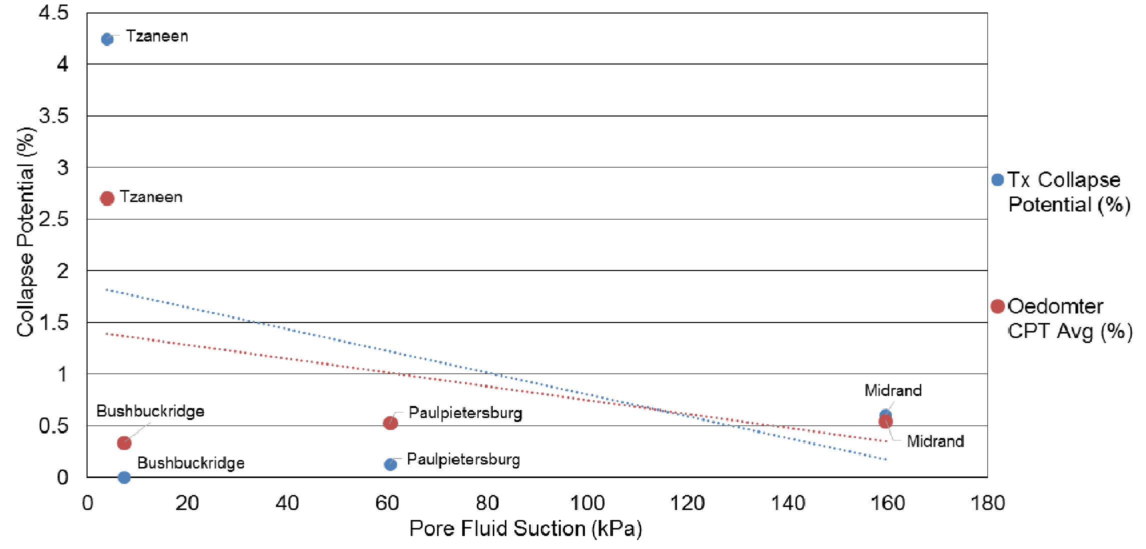
Table 11.9: Pore Fluid Suction of samples

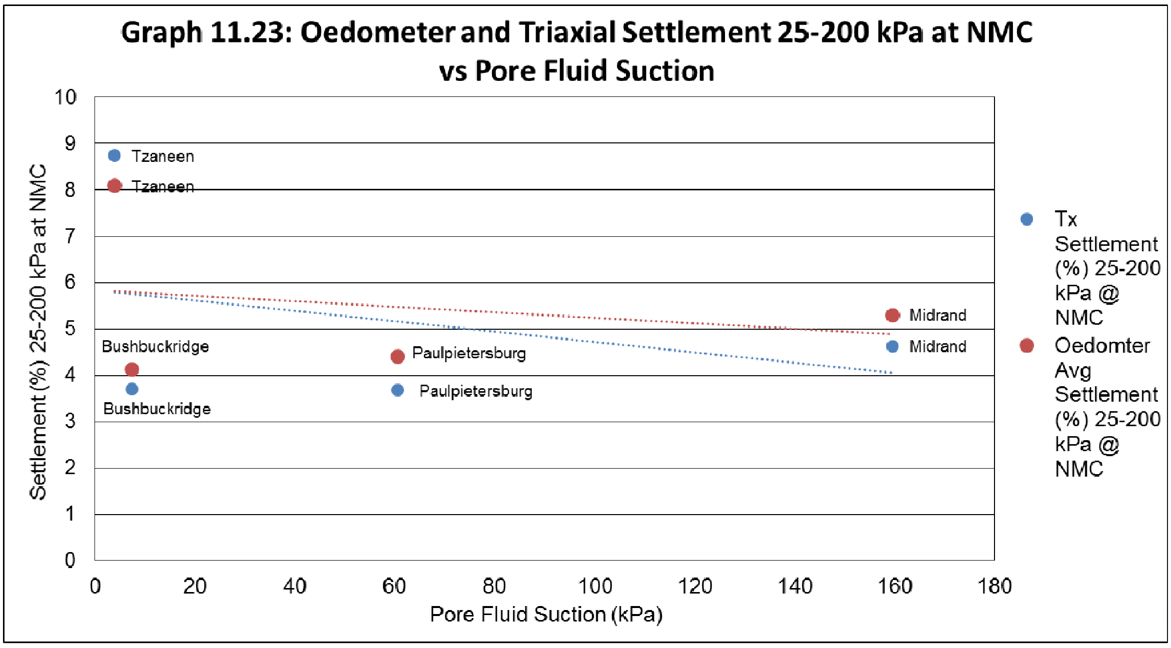
Pore Fluid Suction Average	kPa
Bushbuckridge	7.33
Midrand	159.56
Paulpietersburg	60.51
Tzaneen	3.86

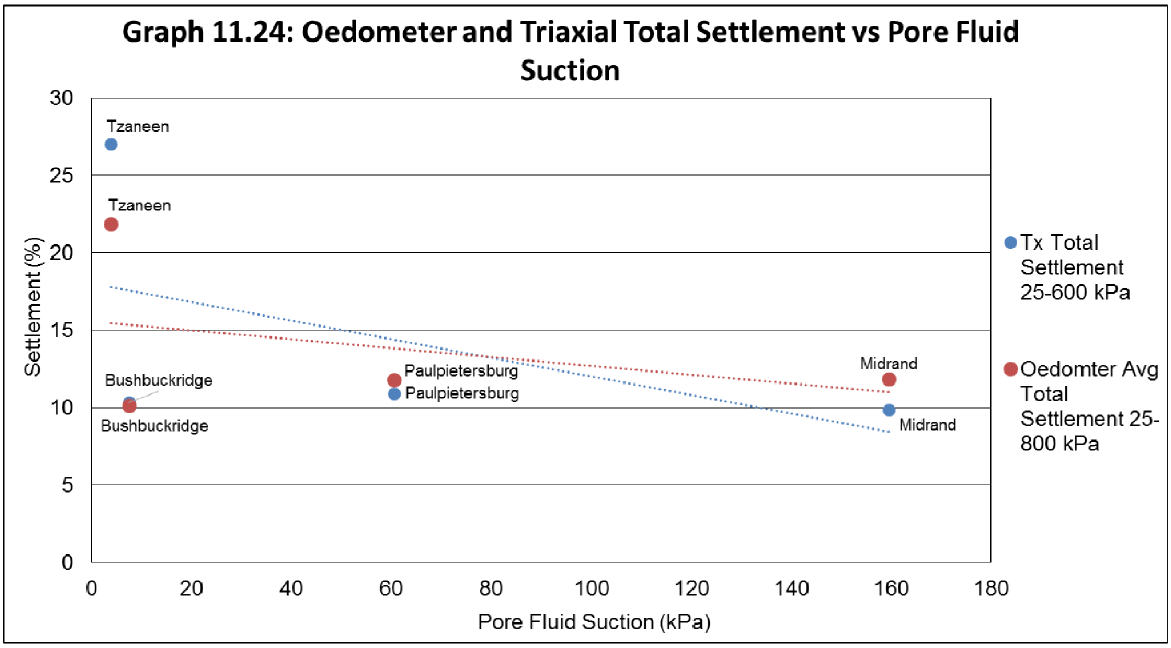
The samples were found to generally have low pore water suctions as was anticipated based on the high moisture contents and degrees of saturation of the samples. At pore water suctions of less than 80 kPa the filter paper method becomes unreliable (Chandler and Gutierrez, 1986). Three of the samples were found to have pore water suction of less than 80 kPa, and the values measured for these samples must be accepted as unreliable.

It has been found in soils of similar origin (compacted gneissic soils) that collapse tends to occur at pore water suctions of 90-10 kPa, with the majority of volume change behaviour at 30-10 kPa (Jose et al., 2000). Based on this it is thought that the Bushbuckridge, Paulpietersburg and Tzaneen samples have minimal pore water suction approaching 0 kPa. The Midrand sample was found to have a pore water suction of approximately 160 kPa which shows that this material had some cohesion. The pore water suction values are compared against the collapse potential and settlement measured in the samples in Graphs 11.22 to 11.24.

Graph 11.22: Oedometer and Triaxial Collapse Potential vs Pore Fluid Suction







These results indicate that the samples tested had little to no cohesion. Furthermore this suggests that the samples should show low collapse potential as the majority of volume change associated with collapse, in drier conditions, would occur prior to inundation of the sample (at 200 kPa) and would be measured as settlement. The low pore water suctions indicate the fact that all the samples were close to above the critical degree of saturation and therefore would collapse little, with the majority of volume change measured as settlement. Therefore the low pore water suctions measured correlate well with the low collapse potential measured in the samples.

11.6. Void Ratio

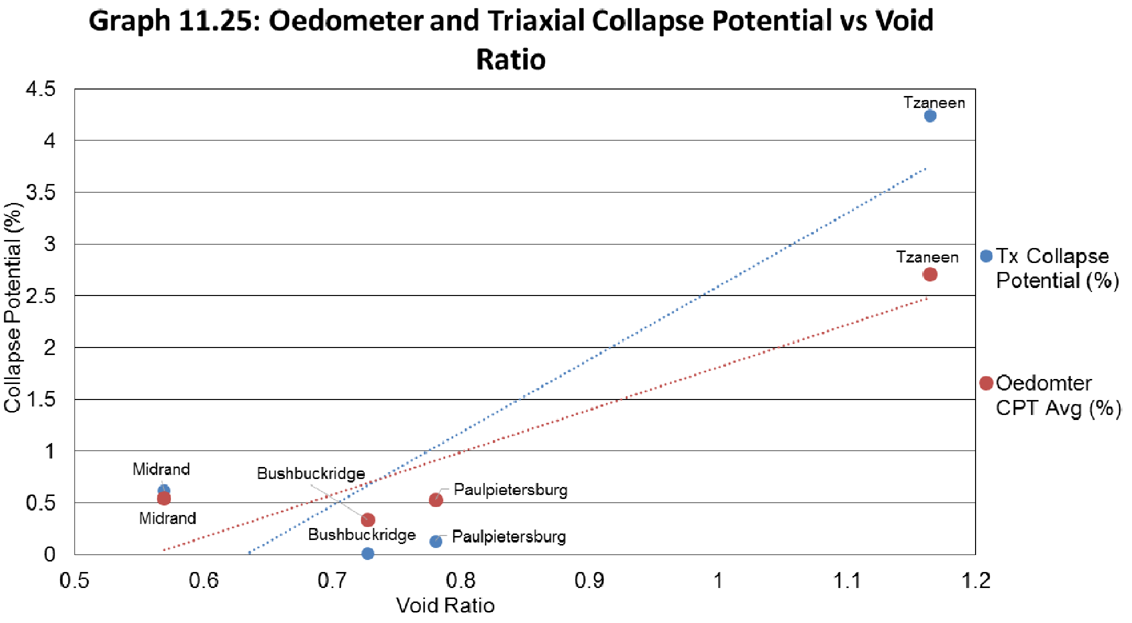
As stated by Schwartz (1985) formation of a collapsible fabric is essential to collapse settlement occurring. This is because a solid medium (with few voids) will not collapse, as there is no void space for the soil grains to move into, and thus lose volume when stress is applied. It can therefore be seen that void spaces associated with a collapsible fabric are crucial to the collapse phenomenon. Theoretically the higher the void ratio the higher the potential volume reduction in a collapsing soil. Void ratio is also important as it is inter-related to porosity, and therefore water will be dispersed quickly and thoroughly through a soil profile with a high void ratio, allowing faster, deeper and more extensive collapse than in soil profile with lower void ratio. Void ratio is generally seen in the geotechnical industry as the most important material property in determining the potential volume loss of a soil during collapse. The average in-situ void ratios for the various sampled residual Archaen granitic soils are shown in Table 11.10.

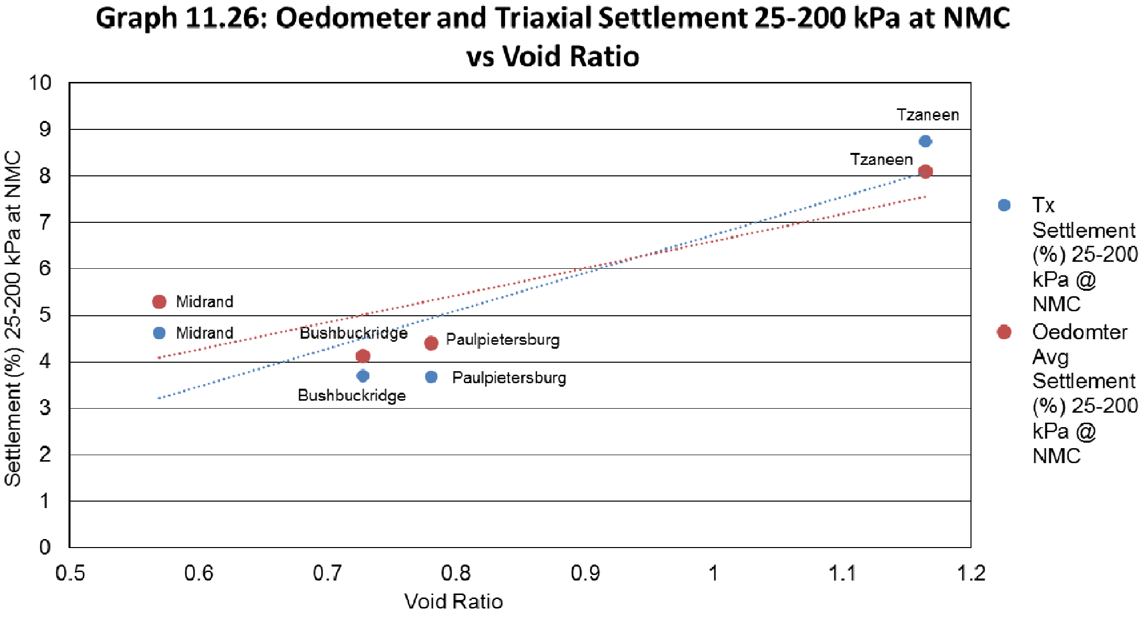
Table 11.10: Average in-situ void ratio of samples

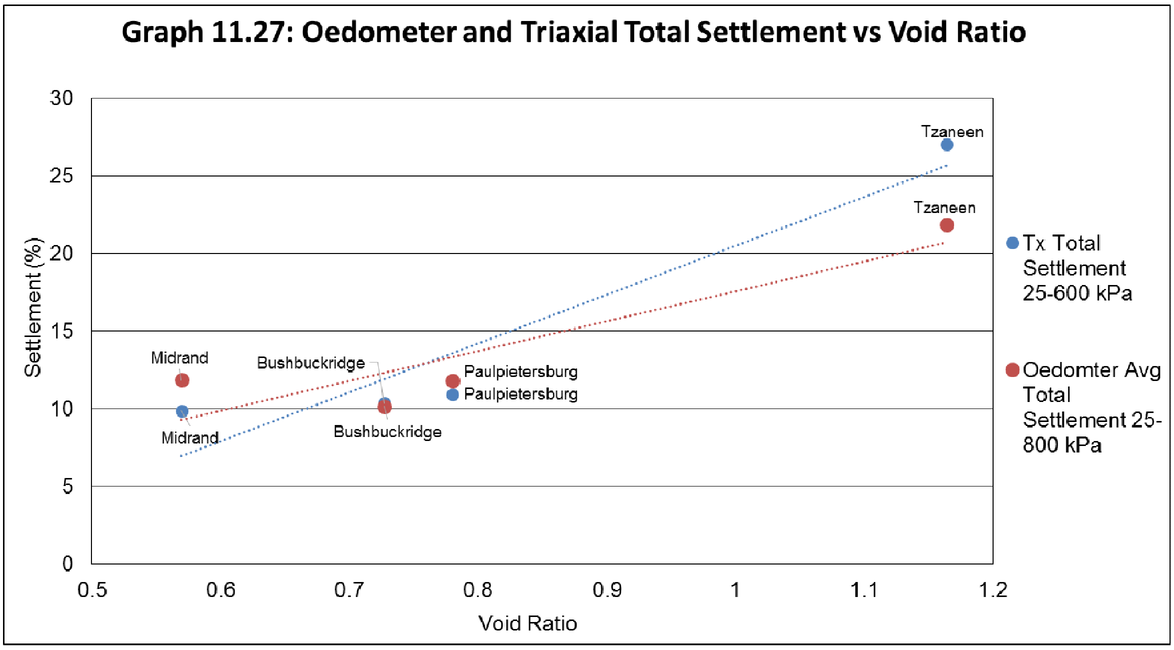
Average In-situ Void Ratio	Void Ratio
Bushbuckridge	0.73
Midrand	0.57
Paulpietersburg	0.78
Tzaneen	1.16

By both international, and South African standards an in-situ void ratio of 0.8 or above is seen as characteristic of a collapsible fabric (Lin, 1994; Rust et al., 2005). It can be seen that the Midrand sample has a void ratio far below that prescribed for collapsible materials. The values for the samples from Bushbuckridge and Paulpietersburg are only slightly below

0.8 and would be classified as possibly collapsible using only in-situ void ratio. The high void ratio of the Tzaneen sample is indicative of a highly leached and very collapsible material. Graphs showing the average void ratios of the samples versus the measured collapse potential and settlement are shown in Graph 11.25 to 11.27.







Graph 11.27: Oedometer and Triaxial Total Settlement vs Void Ratio

From the graphs it can be seen that there is a good correlation between void ratio and collapse potential. The only discrepancy being that the Midrand sample, which had the lowest void ratio, and demonstrated a higher measured collapse potential than the Bushbuckridge and Paulpietersburg samples using both test apparatuses. This may be a result of the lower degree of saturation of the Midrand sample compared to the other samples, which would lead to higher collapse potential measurement for this material. The data presented in Graph 11.27 shows a very good correlation between total settlement and void ratio, as would be expected. This data serves to prove that high void ratio can be a good indicator of collapsibility, but may be misleading in some circumstances depending on the degree of saturation, and other factors must be taken into account when identifying collapsible soils. The void ratio alo sjows good correlation to settlement measurement.

11.7. Dry Density

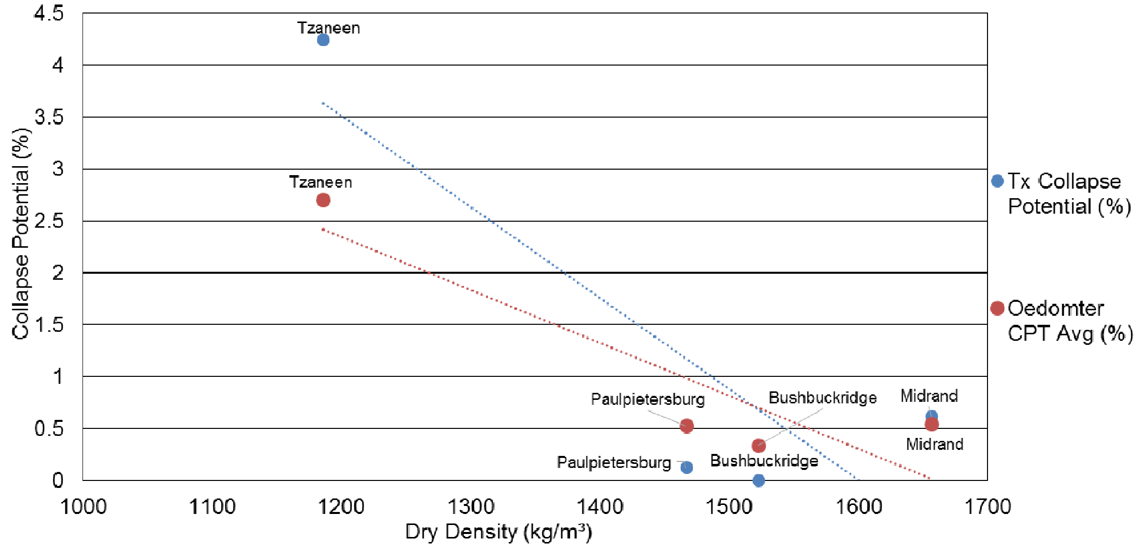
Extensive literature on collapse settlement has recognized dry density as an indicator or even method of identifying collapse. Jennings and Knight (1975) warned against assuming collapsible fabric based on low dry densities, and assuming low or no collapse in soils with high dry densities. Nevertheless dry density is still used by geotechnical professionals as a key indicator of potential collapse. The average in-situ dry densities for the sampled residual Archaen granitic soils are given in Table 11.11 below.

Table 11.11: Average in-situ dry density of samples

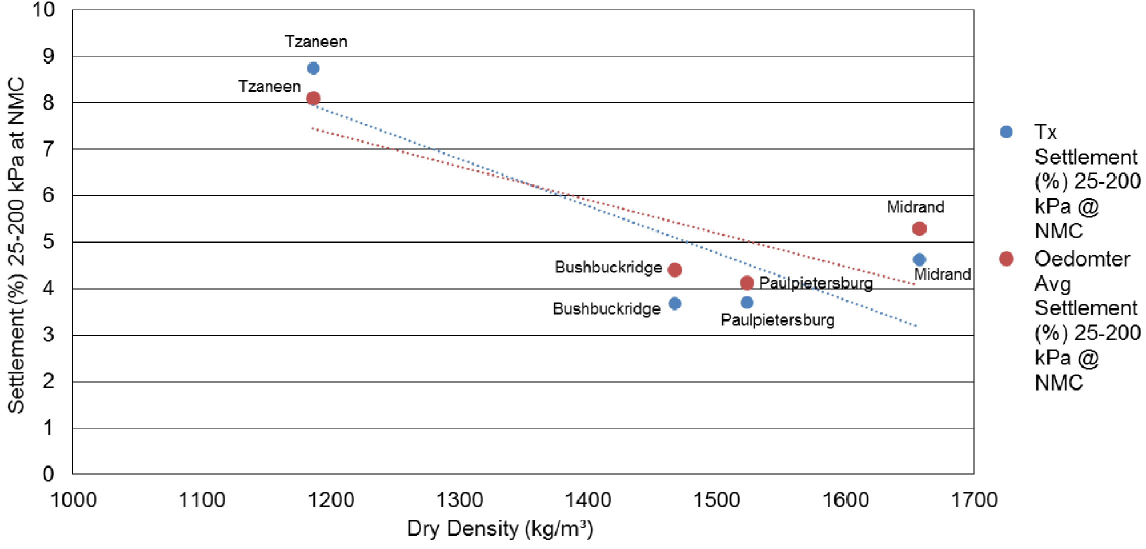
Average In-situ Dry Density	kg/m³
Bushbuckridge	1523
Midrand	1657
Paulpietersburg	1467
Tzaneen	1186

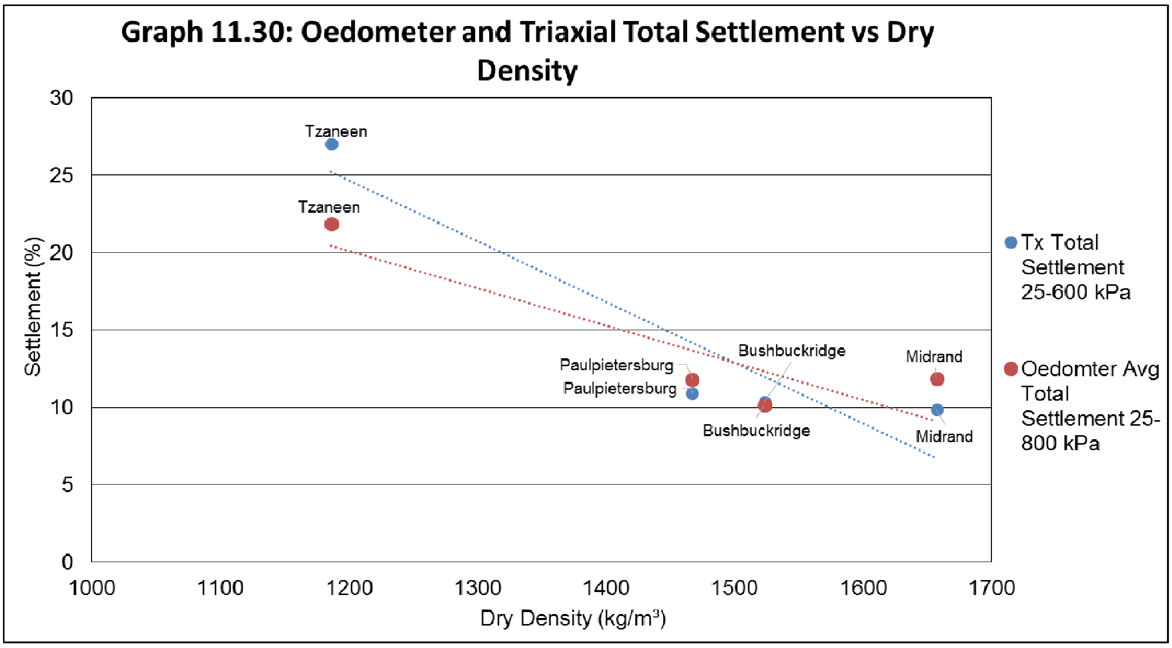
The majority of literature identifies the upper end of dry densities in which collapse can be expected to occur as approximately 1600-1700 kg/m³. The Bushbuckridge, Midrand and Paulpietersburg samples fell within this mark and based primarily on dry density would be considered as potentially collapsible soils. The dry density of the Tzaneen sample is extremely low and this would be considered as a potentially highly collapsible material. The dry densities of the samples are compared to the measured collapse potential and settlement in Graphs 11.28 to11.30.

Graph 11.28: Oedometer and Triaxial Collapse Potential vs Dry Density



Graph 11.29: Oedometer and Triaxial Settlement 25-200 kPa at NMC vs Dry Density




Graph 11.30: Oedometer and Triaxial Total Settlement vs Dry Density

The data shows that there is a good correlation between dry density and collapse. The Bushbuckridge, Midrand and Paulpietersburg samples showed relatively similar dry densities (1467-1567 kg/m³) and show very similar low collapse potential (0.01-0.62%) and more importantly comparable settlement (3.68-5.30%). Whilst the Tzaneen sample was measured to have a very low dry density and had relatively high collapse (2.71-4.25%) and high settlement (8.09-8.74%). Based on this data it appears dry density shows a good inversely proportional relationship to both collapse and settlement.

11.8. Microscopy Observations

Micrographs of the residual granitic soil were taken using a Weiss optical microscope at magnifications of X1 to X100 to observe the structure of the soils in both in-situ (unconsolidated) and collapsed (consolidated) state. Representative micrographs of each sample can be seen in Figures 11.1 to 11.8.

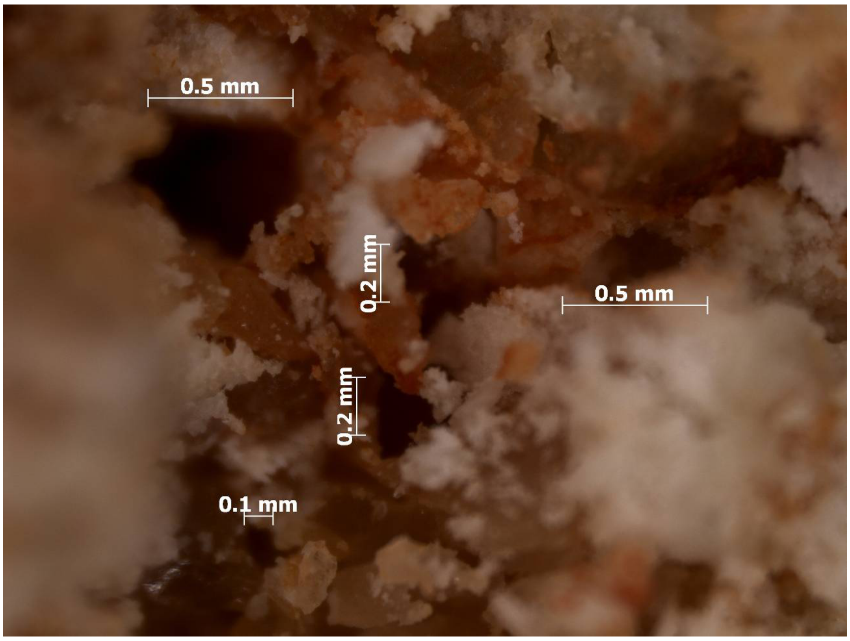


Figure 11.1: Bushbuckridge sample in-situ (un-compressed) at X50 magnification

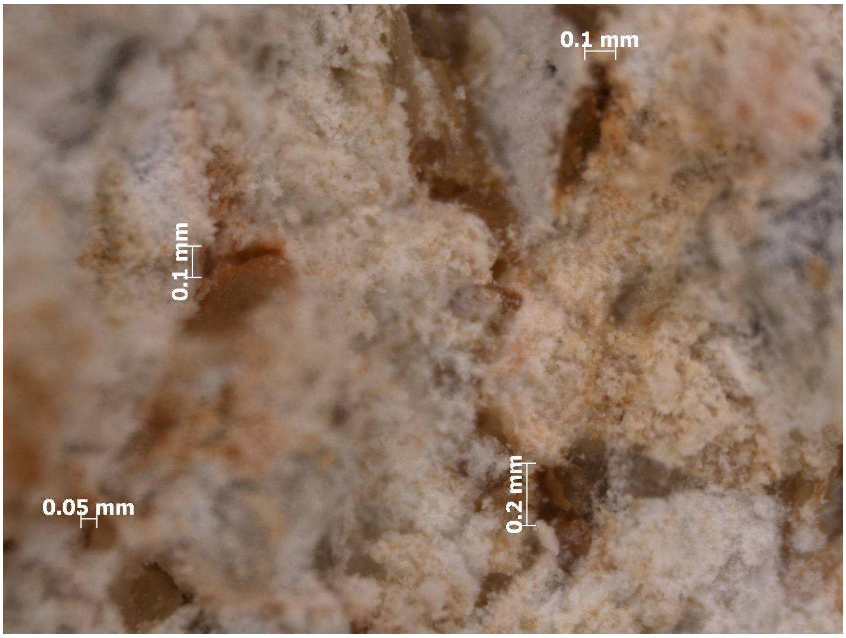


Figure 11.2: Bushbuckridge sample after testing (compressed at 600-800 kPa) at X50 magnification

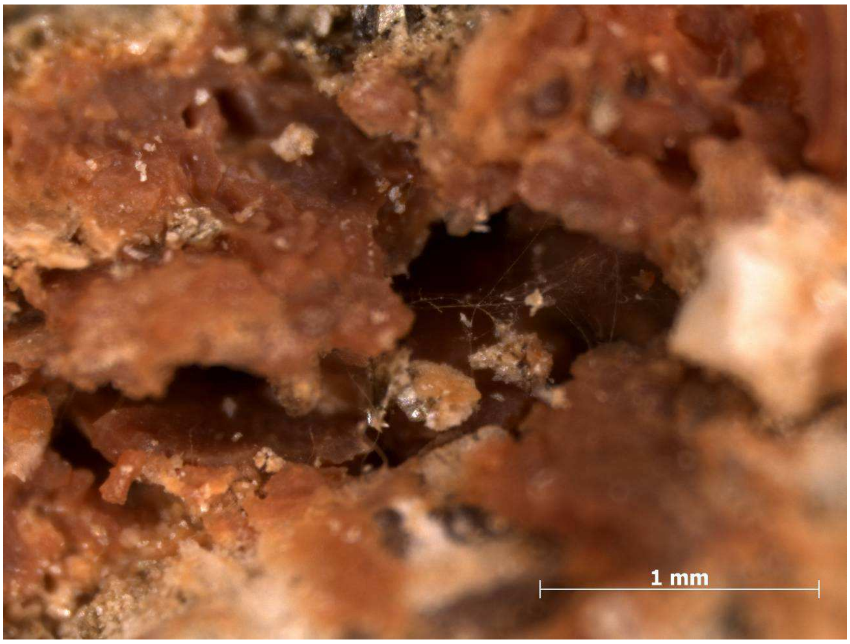


Figure 11.3: Midrand sample in-situ (un-compressed) at X50 magnification

Figure 11.4: Midrand sample after testing (compressed at 600-800 kPa) at X50 magnification

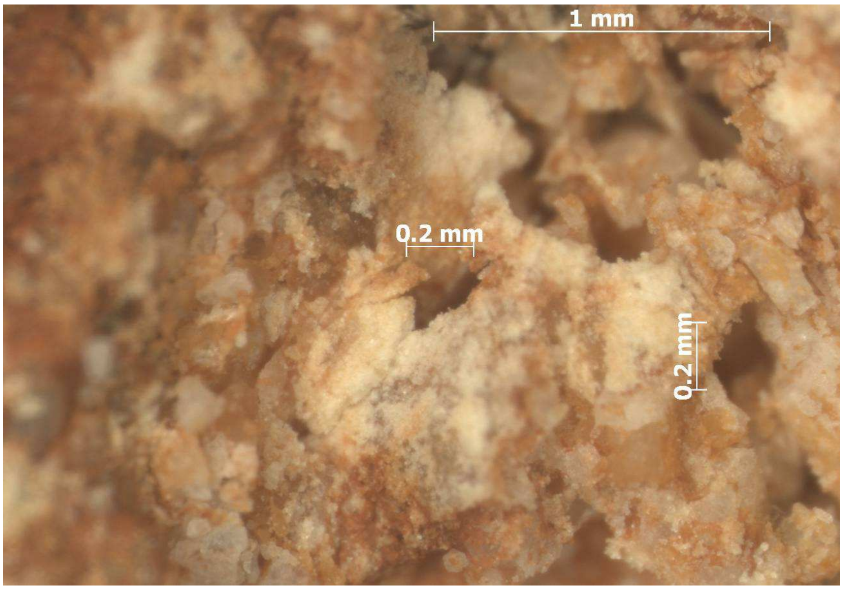


Figure 11.5: Paulpietersburg sample in-situ (un-compressed) at X50 magnification

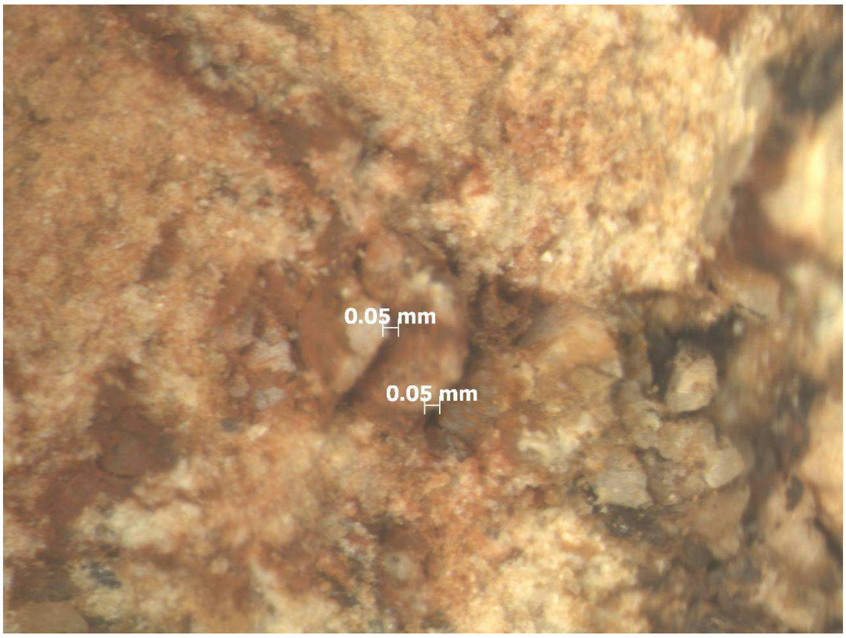


Figure 11.6: Paulpietersburg sample after testing (compressed at 600-800 kPa) at X50 magnification

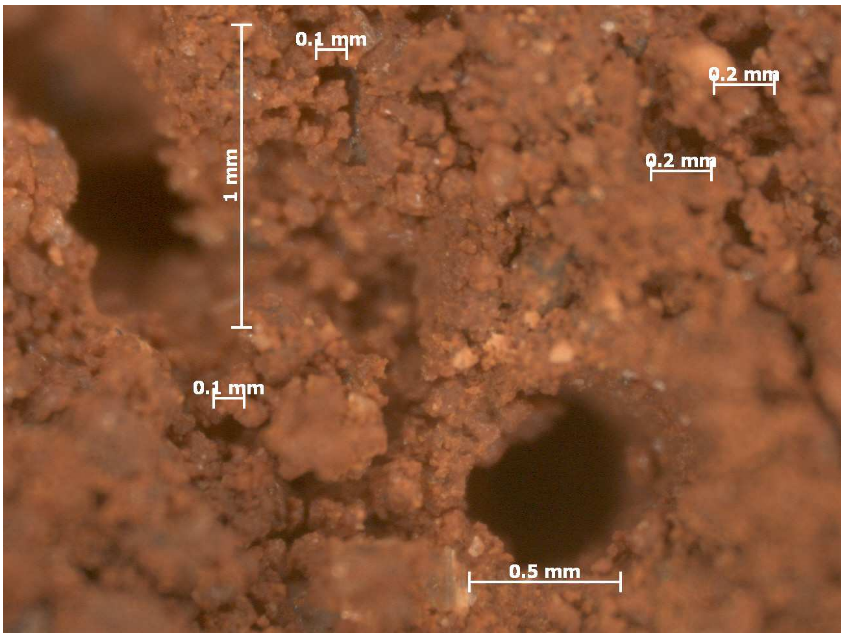


Figure 11.7: Tzaneen sample in-situ (un-compressed) at X50 magnification

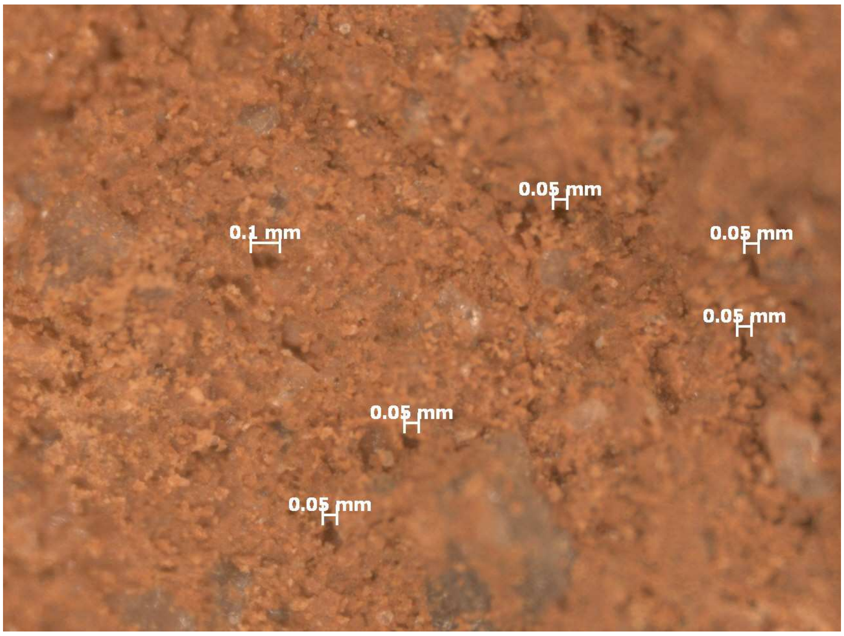


Figure 11.8: Tzaneen sample after testing (compressed at 600-800 kPa) at X50 magnification

Observation of the micrographs in an unconsolidated state showed that the sample that collapsed and settled the most (namely the Tzaneen sample) showed much larger void sizes and more voids than the other samples. The Bushbuckridge, Midrand and Paulpietersburg samples showed very similar amounts of voids but the Midrand sample generally showed larger voids than the Bushbuckridge and Paulpietersburg samples. The Midrand sample was measured to collapse more than the Bushbuckridge and Paupietersburg samples, even though it had a lower void ratio and based on the data this is thought to be a result of the larger voids observed in the Midrand sample.

A number of evaluations can be made from viewing micrographs of the samples. Firstly it can be seen, as should be anticipated, there is a large decrease in the volume of voids in the samples from unconsolidated state to the consolidated state. Voids large enough to be seen at the magnifications used are clearly visible after consolidation (at 600-800 kPa), indicating there is still propensity of the soils to consolidate under a higher stress. It appears from the images that the amount (number) of voids at in-situ condition and after compression decreases slightly, but more importantly it appears the size of voids decreases dramatically during consolidation. From these results it is thought that maximum void size may be a good indicator of volume change behaviour.

11.9. Collapse Potential and Settlement Measurements (Oedometer and Triaxial Tests)

Part of this research consisted of reviewing and comparing the laboratory methods used to measure and/or identify collapse. Oedometer testing was evaluated using Jennings and Knight (1975) collapse potential test, whilst triaxial cell collapse testing was evaluated using the triaxial collapse potential test (Heymann, 2000). The values obtained for collapse and settlement in residual Archaen granitic soils are also presented as these may give insight into what sort of volume change behaviour may be anticipated in this material. It should be noted that one triaxial collapse potential test and two oedometer collapse potential test were undertaken on each material. The collapse potential, settlement at NMC and total settlement of the samples measured in the oedometer tests and triaxial tests are presented in Table 11.12 and 11.13 respectively.

Table 11.12: Collapse potential and settlement measured using the oedometer collapse potential test

Sample	Test Number	Oedometer (%)
Bushbuckridge	CPT 1 Collapse Potential	0.20
	CPT 2 Collapse Potential	0.47
	CPT 1 Settlement (25-200 kPa)	4.60
	CPT 2 Settlement (25-200 kPa)	3.66
	CPT 1 Settlement Total (25-800 kPa)	10.39
	CPT 2 Settlement Total (25-800 kPa)	9.94
Midrand	CPT 1 Collapse Potential	0.61
	CPT 2 Collapse Potential	0.48
	CPT 1 Settlement (25-200 kPa)	6.87
	CPT 2 Settlement (25-200 kPa)	3.72
	CPT 1 Settlement Total (25-800 kPa)	13.59
	CPT 2 Settlement Total (25-800 kPa)	10.11
Paulpietersburg	CPT 1 Collapse Potential	0.25
	CPT 2 Collapse Potential	0.80
	CPT 1 Settlement (25-200 kPa)	4.16
	CPT 2 Settlement (25-200 kPa)	4.66
	CPT 1 Settlement Total (25-800 kPa)	11.78
	CPT 2 Settlement Total (25-800 kPa)	11.89

Sample	Test Number	Oedometer (%)
Tzaneen	CPT 1 Collapse Potential	2.41
	CPT 2 Collapse Potential	3.00
	CPT 1 Settlement (25-200 kPa)	8.61
	CPT 2 Settlement (25-200 kPa)	7.56
	CPT 1 Settlement Total (25-800 kPa)	25.34
	CPT 2 Settlement Total (25-800 kPa)	18.37

Table 11.13: Collapse potential and settlement measured using the triaxial collapse potential test

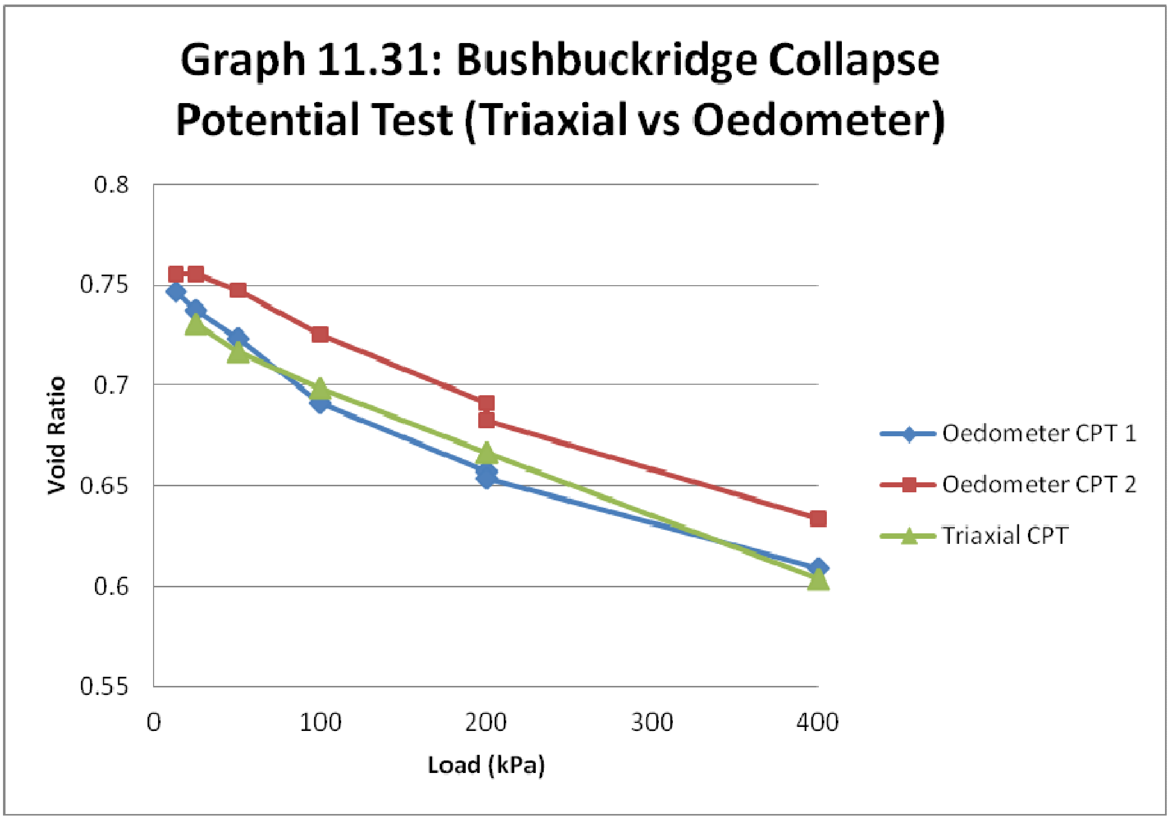
Sample	Test Number	Triaxial (%)
Bushbuckridge	Collapse Potential	0.01
	Settlement at NMC (25-200 kPa)	3.70
	Settlement Total (25-600 kPa)	10.35
Midrand	Collapse Potential	0.61
	Settlement at NMC (25-200 kPa)	4.63
	Settlement Total (25-600 kPa)	9.89
Paulpietersburg	Collapse Potential	0.13
	Settlement at NMC (25-200 kPa)	3.68
	Settlement Total (25-600 kPa)	10.91
Tzaneen	Collapse Potential	4.25
	Settlement at NMC (25-200 kPa)	8.74
	Settlement Total (25-600 kPa)	27.06

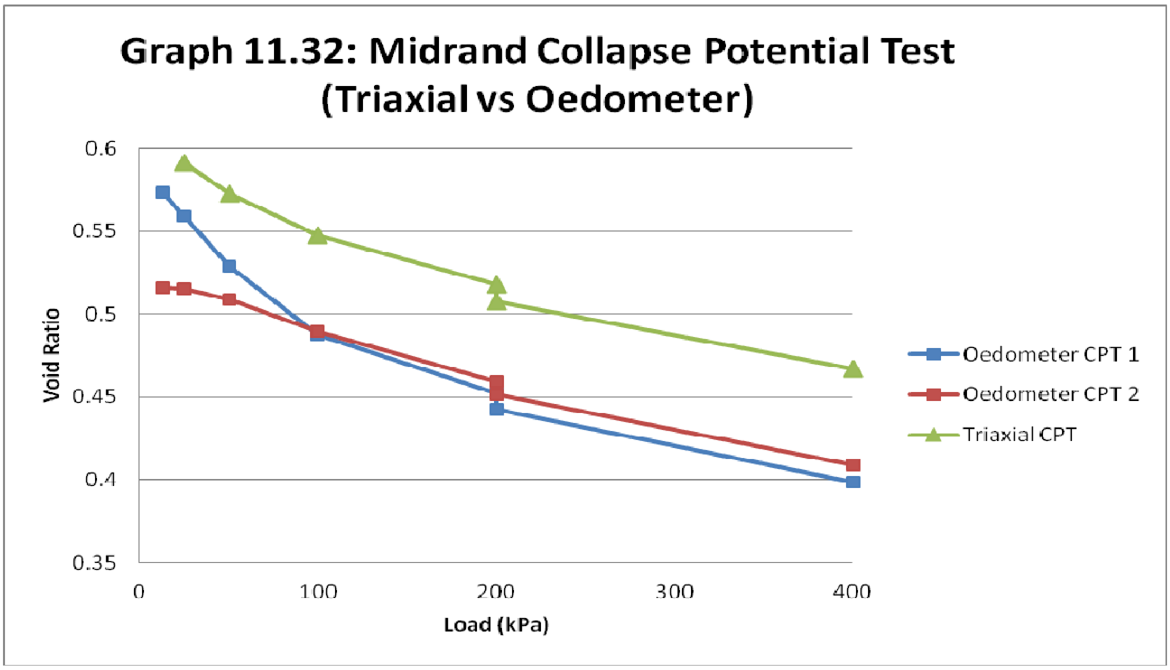
These results show that the majority of the residual granitic samples showed low collapse potential, with the Bushbuckridge, Midrand and Paulpietersburg samples, measured (using both test methods) as having less than 1% collapse potential. The Tzaneen sample showed a significantly higher collapse potential of 2.41-4.35%. These results appear to contradict the assumptions of Brink and Kantey (1961) that a residual granitic soil in an area of water surplus (precipitation>evaporation) with a condition of internal drainage are collapsible.

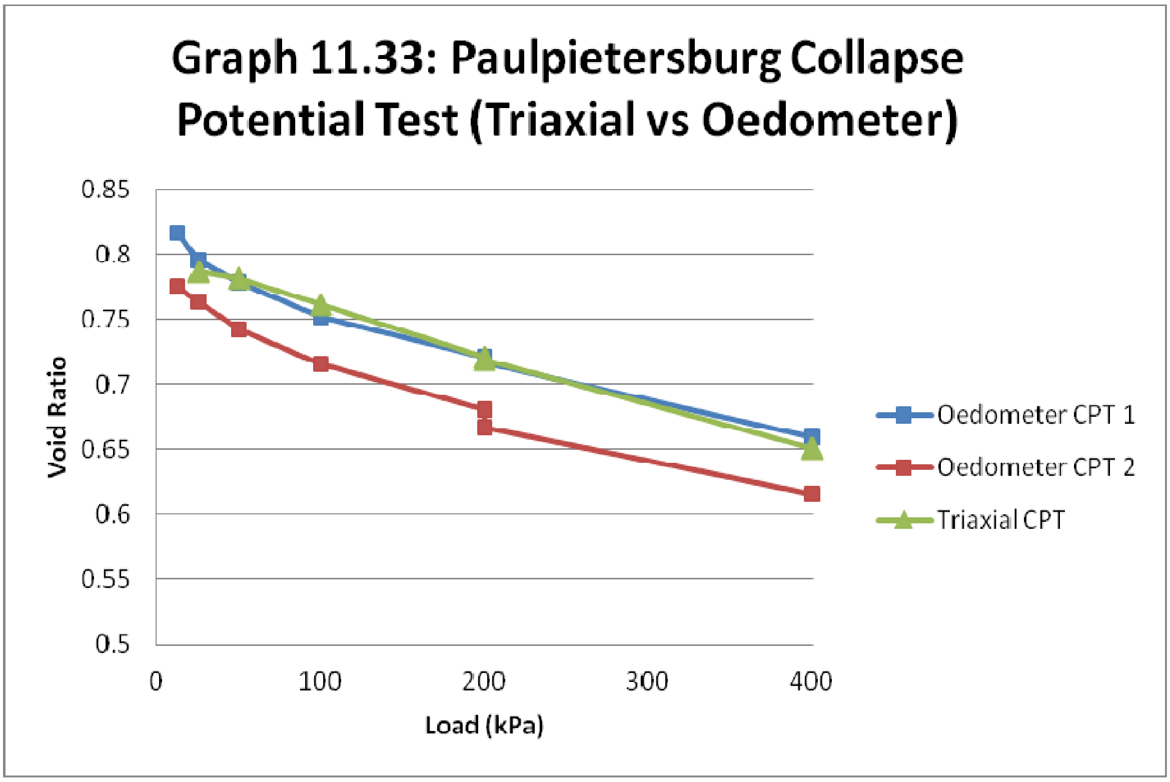
The collapse potential test results show that the Bushbuckridge, Midrand and Paulpietersburg samples have a ‘no problem’ severity of collapse (with CP<1%), while the

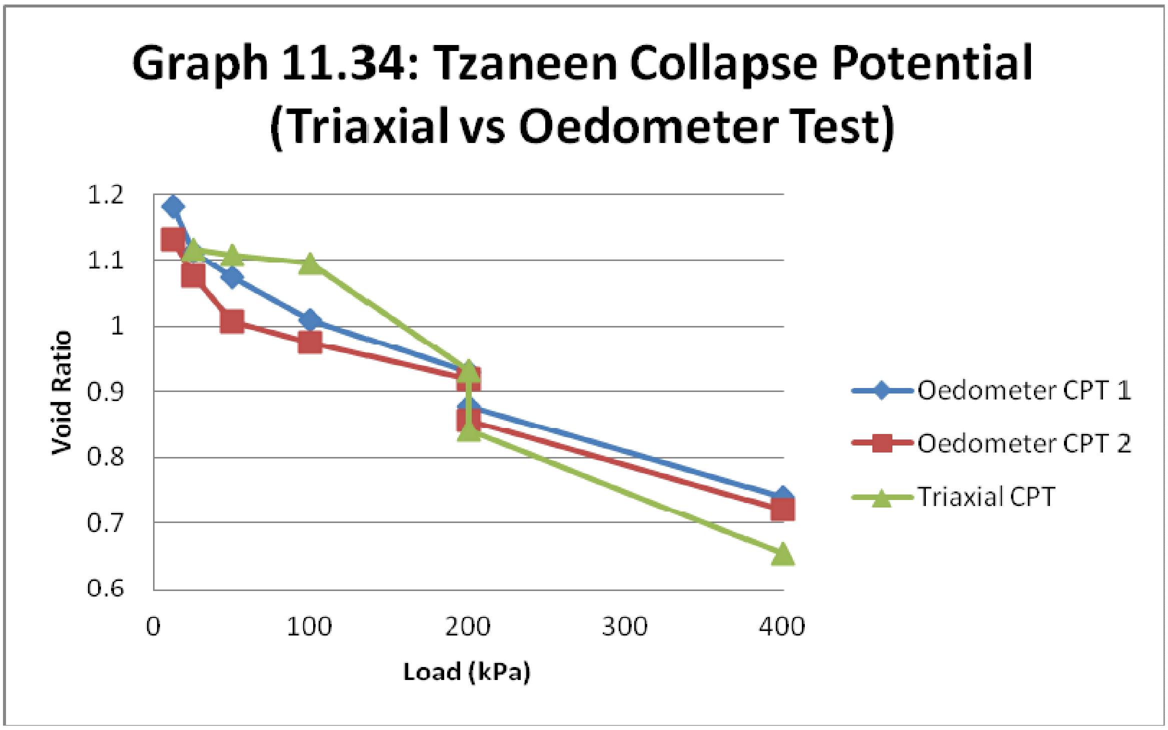
Tzaneen sample was shown to have a ‘moderate trouble’ severity of collapse (with CP 1%-5%), as per the prescribed standard set out by Jennings and Knight (1975). This indicates that little collapse may be experienced by structures built on these soils in this condition.

The settlements measured showed very similar volume change behaviour of the Bushbuckridge, Midrand and Paulpietersburg samples with settlement at 25-200 kPa at NMC of 3.66-6.87%, total settlements of 9.89-13.59%. The settlement of the Tzaneen sample was again substantially higher with settlements at NMC (25-200 kPa) of 7.56-8.74% and total settlement (25-600/800 kPa) of 18.37-27.06%. From these results it can be seen that moderate settlement can be anticipated in the Bushbuckridge, Midrand and Paulpietersburg samples and that high settlement can be anticipated in the Tzaneen material. Graphs 11.31 to 11.35 show the oedometer and triaxial results of void ratio vs load.




Graph 11.32: Midrand Collapse Potential Test (Triaxial vs Oedometer)


Graph 11.33: Paulpietersburg Collapse Potential Test (Triaxial vs Oedometer)



11.10. Appraisal of Laboratory Test Methods Used To Identify Collapse

The oedometer collapse potential test (Jennings and Knight, 1975) has been identified as a good indicator test of collapse potential, despite this it is advised that collapse potential measured in the oedometer cell is not used as design input for structures. The inaccuracy of the oedometer apparatus in collapse measurement is thought to be a result of the bedding errors and potential lateral strain identified to be inherent in this test method. Heyamnn, (2000) developed the triaxial collapse potential test to eliminate the errors identified in the oedometer test and create a collapse potential test that can measure collapse and settlement more accurately and possibly produce design parameters for structures constructed on collapsible soils. They found that triaxial collapse potential test generally measured collapse potential at approximately one third the value measured in the oedometer collapse potential test, and the triaxial results showed far less scatter (variation) than those attained from the oedometer apparatus.

The following problems were identified with residual granitic soils in collapse potential tests :

- Non-homogenous samples

- Variable degree of saturation

The following problems were identified with the oedometer collapse potential test:

- Bedding errors
- Lateral strain

The following problems were identified with the triaxial collapse potential test:

- Principal stress ratio (during testing strain occurs in three dimensions but is only measured in one dimension)
- Sampling and sample preparation
- The high sensitivity of the LVDT's (Linear Variable Differential Transducers)
- Cost (machinery required, sampling and training/skills required)
- Time consuming methodology

In this study the oedometer and triaxial collapse potential tests showed very similar collapse and settlement measurements for each sample tested. The Bushbuckridge and Paulpietersburg samples were both measured to have a slightly higher collapse potential using oedometer testing than that found using tri-axial testing, whilst the Midrand and Tzaneen samples both showed lower collapse potential in oedometer testing. Similarly there was no general trend of one test showing generally higher or lower values than the other for settlement, and results gathered using both tests were similar in all samples. This data contradicts previous work done using this apparatuses.

The fact that measurements of collapse and settlement were similar in both the oedometer and triaxial apparatus (for all samples) indicates that bedding errors and lateral strain in the oedometer cell had little effect on the results. This indicates that there is little advantage in using the triaxial cell over the oedometer cell apparatus for residual granitic soil samples.

11.10.1. The Effect of Bedding Errors

Bedding errors associated with oedometer collapse potential testing were found to be low in this study, as compared with those found in previous research. Bedding errors, as described in Section 9.2.1, are theorized to lead to an over estimation of volume change.

The difficulties posed by bedding errors in oedometer testing are thought to be less substantial in soils with a high percentage of fines. The asperites which form on the surface of a sample cut into an oedometer cell are thought to be proportional to the grading of a material, as a generally coarse grained material will generally have more, larger grains which form more and/or larger asperites on the surface of the sample leading to higher bedding errors. Whilst soils with a large fines percentage are thought to have smaller asperites leading to a more continuous contact with the oedometer loading plate and thus lower bedding errors.

By this logic the residual granitic soils tested in this study, which had 19-55% finer than 0.075 mm material should show lower bedding errors than the coarse sands found at the Mozal Smelter site with approximately 20% finer than 0.075 mm (McKnight, 1999). This is also indicated by the settlement results of the Midrand sample which had the lowest percent of material passing 0.075 mm (19%), and showed the highest variation in measured settlement (25-200 kPa at NMC) in oedometer collapse potential test (CPT 1=6.87%, CPT 2=3.72%). This data indicates that further study is required into the affect of soil grading on bedding errors and the subsequent consequences to collapse potential and settlement testing.

11.10.2. The Effect of Lateral Strain

This research indicates that the measurement errors in the oedometer collapse potential test were low in this research. Lateral strain is a leading factor causing error in collapse and settlement measurements in the oedometer apparatus, and generally leads to over estimation of volume change as described in Section 9.2.2. It appears little or no lateral strain occurred in the oedometer testing for this research.

The effect of lateral strain can be mitigated by correct sample preparation. This entails the cutting the sample into the oedometer ring as ‘tightly’ as possible, maintaining a continuous contact between the sides of the ring and the sample.

Similarly to bedding errors it is thought that the grading of the sample is a critical factor in the lateral strain that can be expected in an oedometer collapse potential test. As a sample with a finer grading will result in less and/or smaller asperites forming between the sample and the sides of the oedometer ring and therefore less space/voids at the contact into which the sample may strain laterally. The finer grading of the residual granitic soils in this research is speculated to have resulted in less lateral strain and measurement error in the oedometer test as compared errors measured on similar tests done on the coarser Mozal sands by Rust et al., (2005).

11.10.3. The Effects of Moisture Content on the Identification of Collapse

Moisture is the triggering mechanism of collapse settlement and measurement of a soils volume change behaviour is highly dependent on this parameter. Moisture content is a less significant parameter in determining the moisture required to cause collapse, and it can be seen that the degree of saturation is a more helpful value, as it takes into account soil parameters such as particle density and void ratio. It is therefore recommended that degree of saturation be used to identify the moisture condition of the soil with regards to volume change behaviour.

The moisture contents and degrees of saturation of all the samples were relatively high, and this lead to the assumption that all the samples were either above or close to the critical degree of saturation from the commencement of testing. This assumption was confirmed by measurement of the pore water suction of the samples which showed low pore water suction (approaching 0 kPa for the majority of the samples). Collapse potential measurements were generally small and much lower than settlement measurements for all the samples as a result of the high in situ degree of saturation of the samples, at which they were tested. The low measured collapse potential of the samples indicates the sensitivity of volume change behaviour to the degree of saturation.

A major problem with the identification of collapse is that samples tested with a moisture content above the critical degree of saturation will show little collapse and higher conventional settlement prior to inundation of the sample. If the samples were at a moisture content below the critical degree of saturation much higher collapse would be realized. A geotechnical professional may not identify collapse in these conditions (high degree of saturation), and structures may be designed to mitigate the measured settlement and not take into account the potential collapse.

Whilst many authors promote collapse potential testing at in-situ moisture content, based on these findings, this is seen as misguided as the in-situ moisture content is a parameter that is both seasonally variable and sensitive to specific precipitation events. Variability of the moisture content can lead to misrepresentation of the volume change of a soil, and this variability should be minimized. Based on this it is recommended that a sample preparation procedure is developed for collapse potential testing in which undisturbed samples are dried prior to testing to give the most conservative collapse potential measurements (worst case scenario).

In this research little variation was found between volume change behaviour measured in the oedometer apparatus and the triaxial apparatus.

These results along with analysis of previous research indicate that grading may affect bedding errors and lateral strains in the oedometer collapse potential test, and that finer graded soils are less susceptible to these variations. Furthermore the advantages of the triaxial collapse potential test may only apply to the testing of coarser soils. Due to the cost, sampling and sample preparation required for the triaxial test, the oedometer collapse potential test may be most applicable test for testing collapse potential in finer graded material, such as residual Archaen granitic soils. A statistical analysis of collapse measured in both tests in soils of different grading is required to conclusively prove this supposition.

Collapse potential testing is highly sensitive to the degree of saturation of the samples, and it is recommended that collapse is measured from dried samples to give geotechnical professionals the most conservative (worst case scenario) collapse potential, and prevent this hazard from being overlooked as conventional settlement.

11.11. Quantification and Identification of Collapse in Residual Granitic Soils

The collapse potential measured for the residual granitic samples were generally low (with the exception of the Tzaneen sample) and these results appear to contradict the findings of Brink and Kantey (1961). The low values measured have been largely attributed to the high degree of saturation of the samples, and under drier conditions these materials may show significantly higher collapse potential.

It is believed that the collapse potential measured in residual granitic soils for this research is misleading due to the high in-situ degree of saturation of the samples. Based on the settlements measured at natural moisture content (NMC) to 200 kPa it is anticipated that the Bushbuckridge, Midrand and Paulpietersburg samples may show collapse potential in excess of 5% and the Tzaneen sample in excess 10% upon inundation at 200 kPa, tested from a dry condition. This would confirm that residual granites are potentially moderately to highly collapsible and a hazardous founding material in this respect.

Based on the data gathered in this research the following parameters were seen to show good correlation to the collapse potential measured:

- Particle density (inversely proportional)
- Degree of saturation (inversely proportional)
- Degree of weathering, chemical index of alteration (CIA)(directly proportional)
- Pore fluid suction (directly proportional)
- Void ratio (directly proportional)
- Dry density (inversely proportional)
- Maximum void size (directly proportional)

The following parameters showed little correlation to the measured collapse potential;

- Clay content (% finer than 0.002 mm)
- Sand and gravel content (% coarser than 0.2 mm)

- Moisture content

- Percentage of primary granitic minerals

It must also be noted that the Tzaneen which exhibited the highest collapse potential and settlement was found to have a medium potential expansiveness (Van der Merwe, 1964), whilst all other samples were found to have low potential expansiveness. This indicates that a moderately collapsible material may exhibit medium potential expansiveness.

These findings along with analysis of previous literature show that certain parameters are very important in the identification of collapse within residual granitic soils. The findings relating the degree of weathering and maximum void size to collapse potential and settlement are of particular interest as little research has been carried out into these parameters. The findings show that certain soil parameters can be of great value to the geotechnical professional in identifying the volume change behaviour of residual Archaen granitic soils. It is recommended that further research is conducted to ascertain the relationship between each of these parameters and collapse and/or settlement.

11.11.1. The Effects of Void Ratio and Maximum Void Size on Collapse and Settlement

It has been identified that the pore size of collapsible soils plays a crucial role in the volume change behaviour of these materials, and that identifying volume change behaviour purely from void ratio may be misleading. Dudley (1970) found that soils with larger pore spaces collapsed more than soils with the same void ratio but smaller pore spaces. The size of the pore spaces in the undisturbed residual granitic samples were assessed using optical microscopy at X1 to X100 magnification, at in-situ state (un-consolidated) and post collapse (consolidated). Observation of the void sizes of the samples along with the measured collapse and settlement showed that the samples with larger voids collapsed and settled more than the samples with smaller void sizes. The data also showed that a soil with larger voids would collapse and settle more than a soil with a slightly higher void ratio and smaller voids.

Representative Micrographs of the samples before and after testing can be seen in Figures 11.1 to 11.8, and the all the micrographs taken can be seen in Appendix D.

11.11.2. The Effects of Weathering on Collapse and Settlement

The effects of weathering and leaching have been identified as a main cause of collapsible fabric development (Schwartz, 1985; Brink and Kantey, 1961). Despite this little research has been aimed at quantifying the degree of weathering and using this parameter as a means of identifying or quantifying collapse. This study compared the Chemical index of Alteration (CIA) against the volume change behaviour of the samples. The CIA values showed very good correlation to both collapse potential and settlement measured in the samples. This parameter is anticipated to be useful in identifying collapse and settlement in residual soils. The degree of weathering is speculated to be less useful in identifying collapse and settlement in collapsible colluvial and Aeolian material as the weathering of these soils prior to deposition is unknown and may not be uniform. Further research into the relationship between weathering and collapse or settlement may produce interesting results.

A (inversely proportional) correlation was also found between particle density and both collapse and settlement. It is speculated that a lower particle density results from the alteration of denser primary minerals (silicates etc.) to less dense secondary minerals (oxides etc.) and indicates a higher degree of weathering and leaching of the soil. Based on this data it is hypothesized that this parameter may be useful in the identification of collapse. A soil with a higher degree of weathering and leaching is anticipated to result in a greater propensity to collapse and settle.

11.11.3. The Effects of Biological Action on Collapse and Settlement

The Tzaneen sample was measured to have the highest collapse potential and settlements at in-situ condition. From the micrographs of the Tzaneen sample it was observed that biological influences had greatly affected the structure of the soil. Voids found within the Tzaneen soil were observed to be large, cylindrical in shape and often contain residual biological material (roots), as shown in Figure 11.9.

Based on these observations it is thought that biological influences in soil can play a large role in the development of a collapsible grain structure and resultant high collapse potential and settlements. Furthermore piping in the soil profile caused by biological activities can lead to preferential drainage paths which will result in faster weathering of the soil profile, as well as faster wetting of the soil during precipitation. This will lead to higher infiltration, and possibly result in a soil reaching its critical degree of saturation quicker and resultant rapid collapse.

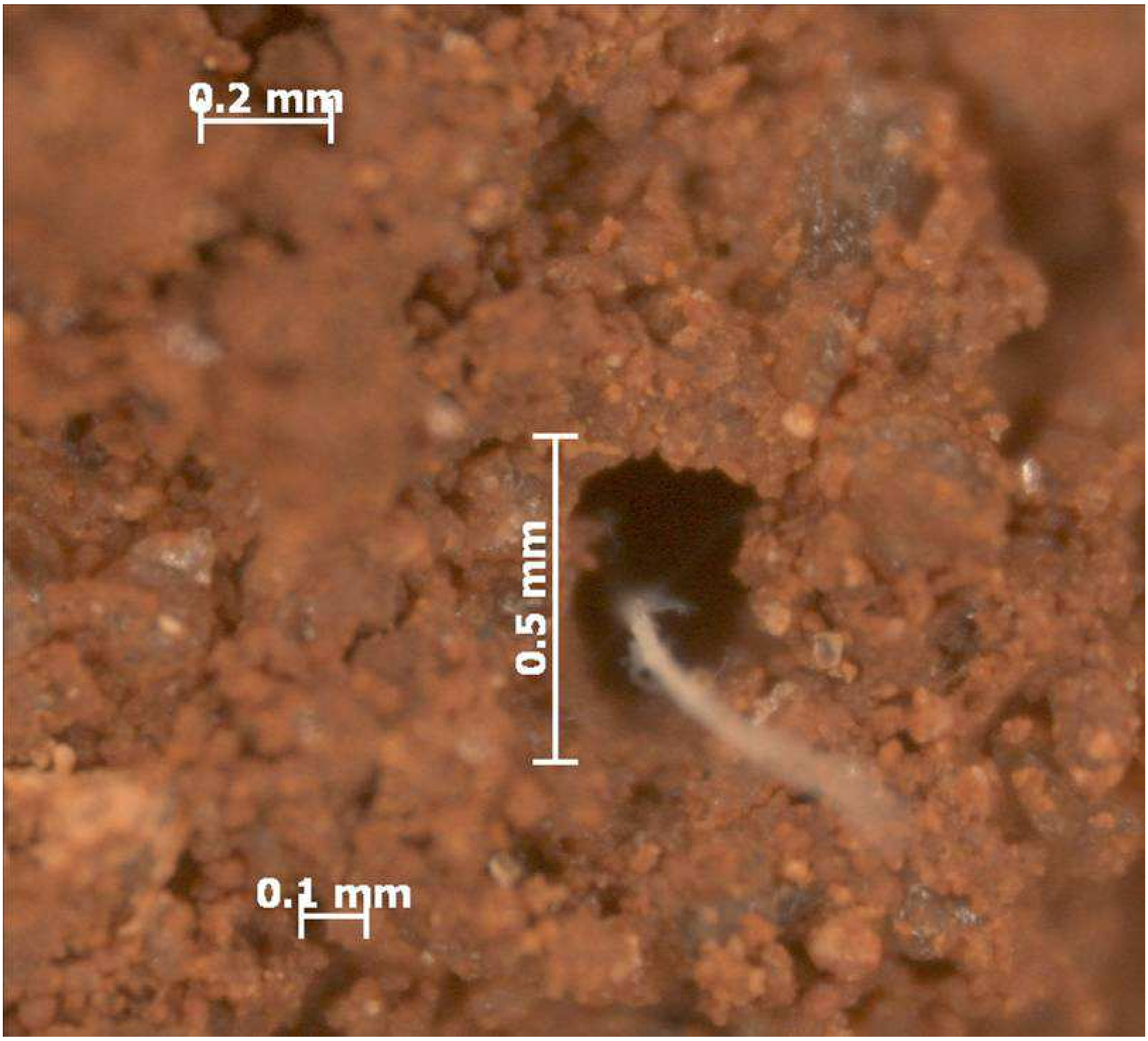


Figure 11.9: Tzaneen sample in-situ (un-consolidated), large cylindrical void formed by visible root

11.11.4. Relevance to Construction on Residual Granitic Soils

From this data it is thought that the propensity for collapse to occur is highly dependent on the soil moisture conditions (and indirectly seasonal rainfall variation) at the time of construction. Therefore it is also recommended that moisture content of a collapsible soil is monitored constantly during construction.

The volume change behaviour of the residual granitic soils shows that in some instances it may be favourable that the natural moisture content of a potentially collapsible soil is higher than the critical degree of saturation for collapse to occur. This is due to the fact that a soil wetted past the critical degree of saturation may gradually settle as construction takes place and higher loads are placed on it. It is theorized that the soil would therefore be pre-collapsed by construction of the founding slab, prior to the construction of the rigid structure and collapse settlement would be mitigated in these conditions. This theorized method of construction would require constant evaluation of the soil’s degree of saturation, dry density and void ratio beneath the foundation slab during construction to confirm the volume change behaviour of the soils.

12. CONCLUSIONS

This research involves the quantification of collapse in residual Archaen granitic soils in South Africa using updated laboratory test methods and an evaluation of the various soil parameters of this material and their use in identifying collapse. The research also looks at the laboratory methods used to identify and quantify collapse, specifically the oedometer and triaxial collapse potential tests. It should be considered that the conclusions, findings and assumptions of this research are drawn from a limited data set, taken from four sampling localities.

Laboratory Test Methods Used to Identify and Quantify Collapse

- Many problems pertaining to collapse testing in both the oedometer and triaxial collapse potential tests were identified during this research.

- The non-homogeneity of residual granitic soils was found to lead to problematic sampling and sample preparation for undisturbed testing. This characteristic is also believed to cause variation in volume change measurements.

- Problems associated with oedometer collapse potential testing include bedding errors and lateral strain.

- Problems identified with the triaxial collapse potential test include the unknown (untested) principal stress ratio of the sample, sampling and sample preparation of triaxial samples, the high sensitivity of the LVDT's and the high cost of this test (machinery required, sampling and sample preparation methodology, training/skills required and time consuming methodology).

- The oedometer and triaxial collapse potential tests showed very similar results in both collapse and settlement measured in each sample tested. Neither of the tests showed results consistently higher or lower than those of the other.

- This data indicates that the oedometer and triaxial apparatus measures volume change behaviour in the same way and indicates that bedding errors and lateral strain within the oedometer apparatus were small.

- The low bedding errors and lateral strains observed during oedometer testing suggest that the samples had a 'tight' continuous contact with both the load plate and the oedometer ring sidewall. It is speculated that the good contact achieved is due to the higher fines content of the residual granitic samples, and that this resulted in smaller aspirites on the edges of the samples in contrast to the higher bedding errors and lateral strain observed during tests on the coarser Mozal sands of Mozambique.

- It is hypothesized that the grading of samples may play a crucial role in extent of bedding errors and lateral strain in the oedometer apparatus and this topic requires further investigation.

- The moisture content was observed to affect greatly the volume change behaviour of samples. This begs the question whether it is wise to test soils for collapse at natural moisture content as this value is seasonally variable and sensitive to specific events (precipitation etc.). To minimise the effect of moisture content as a variable and give the most conservative measurement of collapse it is advised that collapse potential testing take place using dried samples, and that a sample preparation procedure be devised to dry undisturbed samples effectively.

The Quantification and Identification of Collapse in Residual Archaen Granitic Soils

- The sampled residual Archaen granitic soils, were taken from areas of water surplus on side slopes exhibiting internal drainage. These soils were measured to have low to moderate collapse potential. The Bushbuckridge, Midrand and Paulpietersburg samples measured to have collapse potential of 0-0.8%, and the Tzaneen sample 2.41-4.25%.

- The collapse potential measured was thought to be misleading due to the in situ high degree of saturation of the samples, and observation of the measured settlement indicate that the samples may exhibit moderate to high collapse potential in a drier state.

- Many soil parameters were found to show good correlation to collapse and settlement in residual granitic soils namely:

    - Particle density (inversely proportional)
    - Degree of saturation (inversely proportional)
    - Degree of weathering, chemical index of alteration (CIA) (directly proportional)
    - Pore fluid suction (directly proportional)
    - Void ratio (directly proportional)
    - Dry density (inversely proportional)
    - Maximum void size (directly proportional)

- Some soil parameters showed little correlation to collapse and settlement data, these were:

  - Clay content (% finer than 0.002 mm)
  - Sand and gravel content (% coarser than 0.2 mm)
  - Moisture content
  - Percentage of primary granitic minerals

- It is thought that the soil parameters showing good correlation to collapse and settlement can be used to assist geotechnical professionals in identifying potential collapse and high settlement in residual Archaen granitic soils. It is advised that multiple soil parameters are used in this identification, and these are not a guaranteed method of indicating or quantifying collapse. Specialized laboratory testing should be undertaken in any residual granitic soil anticipated to be collapsible from measured soil properties and field observations.

- The Tzaneen sample, which showed the highest collapse potential and settlement was found to have medium potential expansiveness, and this shows that a material with an elevated potential expansiveness is not precluded from being potentially collapsible.

- Through observation of the samples using optical microscopy it was seen that maximum void size is an important parameter in analysing collapse and settlement in residual granitic soils. It was found that soils with a larger maximum void size collapsed and settled more than soils with a higher void ratio and smaller maximum void size.

- The degree of weathering of the samples was assessed by calculating the CIA which showed a good correlation to collapse and settlement of the samples. The samples with a higher CIA were measured to collapse and settle more than those with a lower CIA.

- Little research has been carried out on the relationship between collapse and settlement within residual soils and the degree of weathering. It is hypothesized based on this research, that the degree of weathering is a valuable tool with which to identify problematic collapse and settlement in residual soils. The degree of weathering is generally calculated from the proportion of molecular or mineralogical elements found in a horizon but does not take into account the quantity and type of materials that may have been leached from the material. A condition of internal drainage must exist in the soil to leach the horizon, and produce the void space into which the soil can collapse or settle .The degree of weathering must be taken into account with the degree to which the material has been leached, making it a difficult parameter to calculate precisely.

- Observation of the structure of the Tzaneen sample (which was found to exhibit collapse and settle) through optical microscopy showed many large cylindrical voids containing root material. This shows the biological influence in the formation of collapsible soil fabric and resultant high collapse and settlement that can be anticipated in soils heavily altered by biological activity. This action is speculated to result in a more voided soil, with larger voids and the creation of preferential drainage paths which lead to higher infiltration and the soil reaching the critical degree of saturation faster. It is thought intense biological action will lead to a higher propensity to collapse or settle, less precipitation required for collapse to occur and more rapid collapse during precipitation events.

- Due to the low collapse measured in residual Archaen granitic soils tested at or close to the critical degree of saturation it is theorized that it may be advantageous to construct structures on a collapsible founding medium when the medium is above the critical degree of saturation. This theorized method must allow time for settlement of the foundation slab prior to erection of the rigid structure in order to pre-collapse the founding medium. This methodology will require meticulous measurement of the founding layers' degree of saturation and void ratio through construction to confirm the hypothesized volume change behaviour.

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Google earth V 6.2.2.6613, imagery of South Africa, Bushbuckridge Mpumalanga (01/01/2009), Midrand Gauteng (05/02/2007), Paulpietersburg KwaZulu Natal (11/11/2006) and Tzaneen Limpopo (10/02/2010) Provinces, © 2011 GeoEye, © 2011 AfriGIS, © 2011 CDNGI, © 2011 Europa Technologies, © 2011 DigitalGlobe, Images taken at eye altitude of 1.75-2.10 Km, http://www.earth.google.com.

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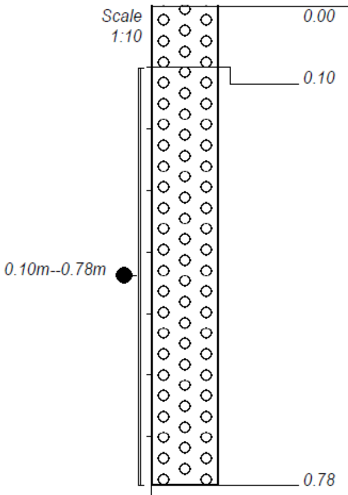
APPENDIX A: SOIL PROFILES

Bushbuckridge

HOLE No: 1
Sheet 1 of 1

JOB NUMBER: 000



Depth	Description
0.00–0.10	Dry, light yellow to white, blotched red and orange, loose, voided, gravel, talus, scattered cobbles pebbles and gravel.
0.10–0.78	Slightly moist, white, blothed light yellow, streaked red and black, mottled red, medium dense, intact with micro voids in places, remnant granitic structure, sandy clayey silty gravel. residual granodiorite, red and black staining on remnant joint planes.

NOTES

1) Profile taken at a road cutting.

2) No water seepage observed.

3) Sample taken at 0.10m--0.78m.

CONTRACTOR :
MACHINE :
DRILLED BY :
PROFILED BY : RYAN FREESE
TYPE SET BY :
SETUP FILE : STANDARD.SET

INCLINATION :
DIAM :
DATE :
DATE : 16/02/2011
DATE : 07/03/13 17:15
TEXT : ..\000\BRPROF~1.TXT

ELEVATION :
X-COORD : 24 53 43.7
Y-COORD : 31 06 28.8

HOLE No: 1

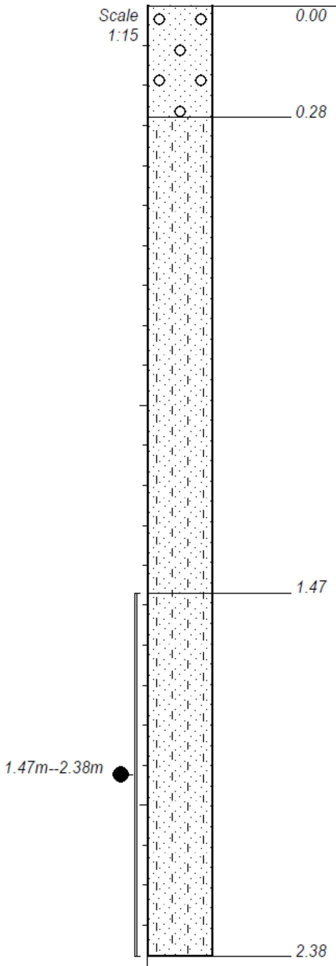
D06C D48

dot.PLOT 5008 J&W

Midrand

HOLE No: 1
Sheet 1 of 1

JOB NUMBER: 000



0.00 – Slightly moist, light brown, mottled white and yellow, loose, voided, gravelly SAND. Hillwash. Derived fom granitic origin.

0.28 – Slightly moist, light orange brown, with spckled white and black, mottled yellow, medium dense, voided, gravelly silty SAND. Colluvium. Derived from granitic origin.

1.47 – Slightly moist, Light grey, mottled orange and yellow, speckled black white and brown. medium dense, voided, gravelly silty SAND. Residual granitic soil.

2.38

NOTES

1) Profile taken at a road cutting.

2) No water seepage observed.

3) Sample taken at 1.47m--2.38m.

CONTRACTOR :
MACHINE :
DRILLED BY :
PROFILED BY : RYAN FREESE

TYPE SET BY :
SETUP FILE : STANDARD.SET

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DIAM :
DATE :
DATE : 15/01/2011

DATE : 07/03/13 17:19
TEXT : ..\000\MRPROF~1.TXT

ELEVATION :
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Y-COORD : 28 04 35.93 E

HOLE No: 1

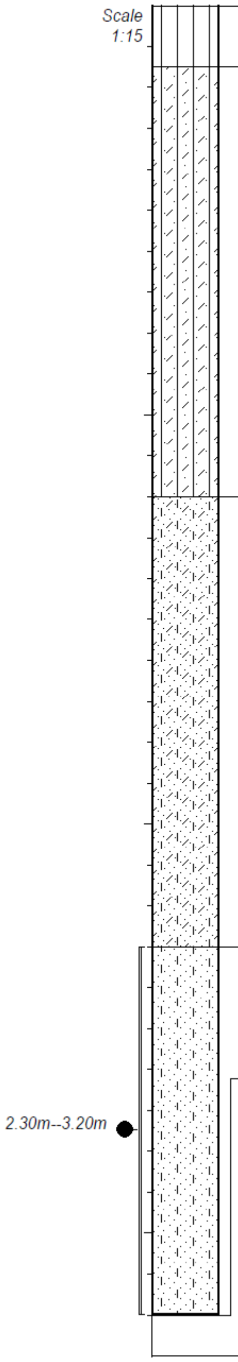
D06C D48

dot.PLOT 5008 J&W

Paulpietersburg

HOLE No: **1**
Sheet 1 of 1

JOB NUMBER: **000**



0.00

Moist, brown, soft, intact, clayey sandy SILT: Colluvium.

0.15

Moist, light orange mottled orange brown and beige, soft, intact, CLAYEY very SANDY SILT: Residual granite.

1.20

Moist, pale orange and beige mottled orange brown and white, loose, intact, CLAYEY very SILTY SAND: Residual granite.

2.30

Slightly moist, pale orange mottled light orange brown, medium dense, intact with relict jointing, clayey SILTY fine to medium grained SAND: Residual granite.

3.20

NOTES

1) No refusal.

2) No water table.

3) Sample taken from 2.30m--3.20m.

4) Photo.

CONTRACTOR :
MACHINE :
DRILLED BY :
PROFILED BY : R FREESE

TYPE SET BY : R FREESE
SETUP FILE : STANDARD.SET

INCLINATION :
DIAM :
DATE :
DATE : 14/12/2010

DATE : 07/03/13 17:20
TEXT : ..\000\PPPROF~1.TXT

ELEVATION :
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Y-COORD : 30d53'49.5"E

HOLE No: **1**

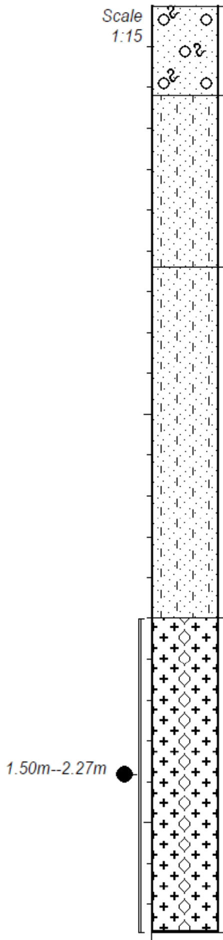
D06C D48

dot.PLOT 5008 J&W

Tzaneen

HOLE No: 1
Sheet 1 of 1

JOB NUMBER: 000



Depth	Description
0.00	Slightly moist, darkbrown, loose, voided, gravelly SAND. Hillwash. Roots, termites.
0.22	Slightly moist, dark redbrown with blotched orange and black, medium dense, voided, clayey silty fine SAND. Hillwash.
0.64	Slightly moist, orange red, medium dense, voided, clayey silty SAND, residual granite, with abundant quartz pebbles and cobbles residual weathered quartz veins.
1.50	Orange, blotched white, completely weathered, soft rock, closely jointed, medium grained, Duiwelskloof Leucogranite, GRANITE, multiple variable jointing.
2.27	

NOTES

1) Profile taken at a road cutting.

2) No water seepage observed.

3) Sample taken at 1.50m--2.27m.

CONTRACTOR :
MACHINE :
DRILLED BY :
PROFILED BY : RYAN FREESE

TYPE SET BY :
SETUP FILE : STANDARD.SET

INCLINATION :
DIAM :
DATE :
DATE : 08/01/2011

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TEXT : ..\000\TZPROF~1.TXT

ELEVATION :
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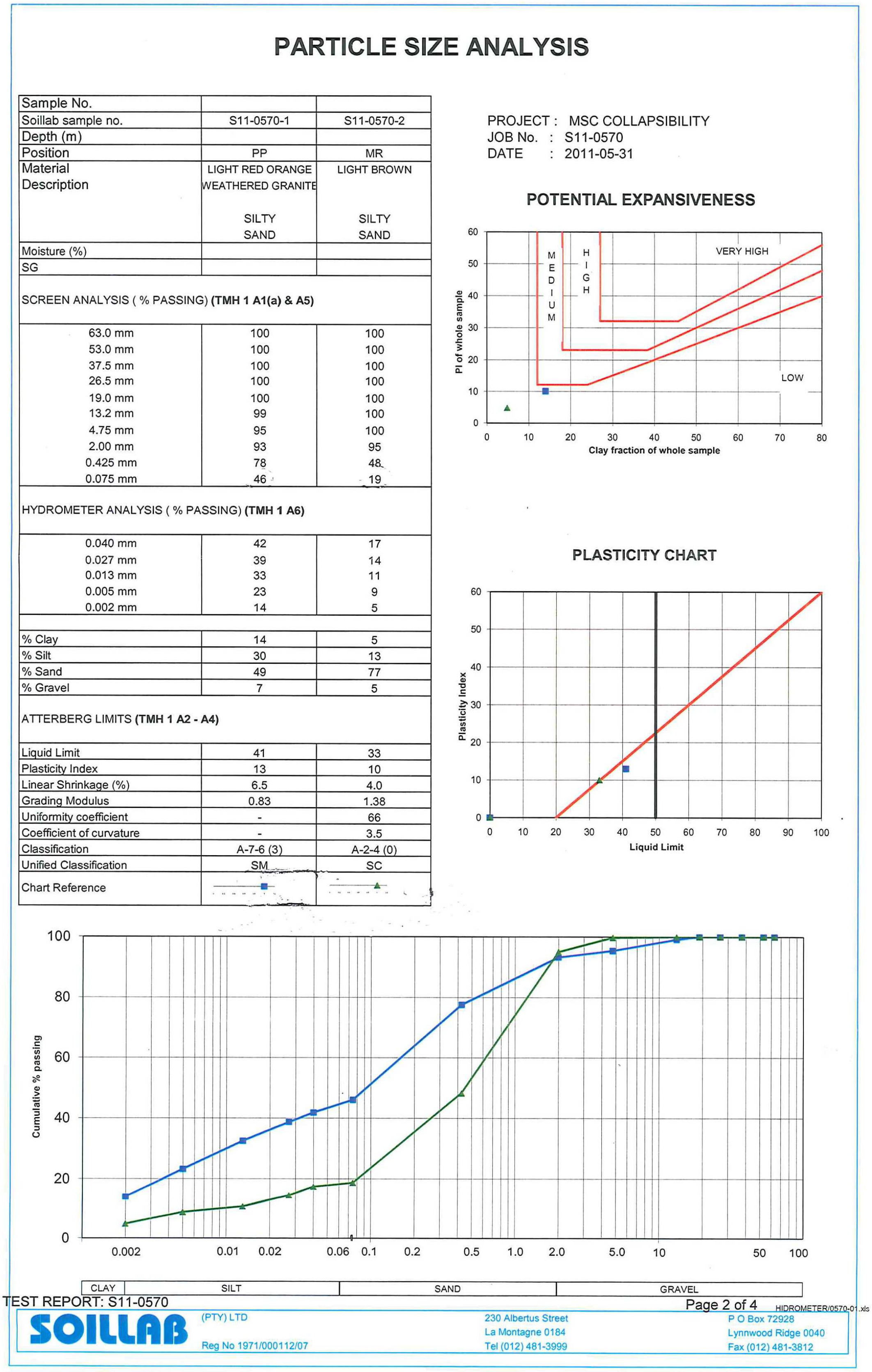
Y-COORD : 30 16' 16.07" E

HOLE No: 1

D06C D48

dot.PLOT 5008 J&W

APPENDIX B: LABORATORY RESULTS (SOILAB (PTY) LTD)



PARTICLE SIZE ANALYSIS

PROJECT : MSC COLLAPSIBILITY
JOB No. : S11-0570
DATE : 2011-05-31

Sample No.		
Soillab sample no.	S11-0570-1	S11-0570-2
Depth (m)		
Position	PP	MR
Material Description	LIGHT RED ORANGE WEATHERED GRANITE SILTY SAND	LIGHT BROWN SILTY SAND
Moisture (%)		
SG		
SCREEN ANALYSIS (% PASSING) **(TMH 1 A1(a) & A5)**		
63.0 mm	100	100
53.0 mm	100	100
37.5 mm	100	100
26.5 mm	100	100
19.0 mm	100	100
13.2 mm	99	100
4.75 mm	95	100
2.00 mm	93	95
0.425 mm	78	48
0.075 mm	46	19
HYDROMETER ANALYSIS (% PASSING) **(TMH 1 A6)**		
0.040 mm	42	17
0.027 mm	39	14
0.013 mm	33	11
0.005 mm	23	9
0.002 mm	14	5
% Clay	14	5
% Silt	30	13
% Sand	49	77
% Gravel	7	5
ATTERBERG LIMITS **(TMH 1 A2 - A4)**		
Liquid Limit	41	33
Plasticity Index	13	10
Linear Shrinkage (%)	6.5	4.0
Grading Modulus	0.83	1.38
Uniformity coefficient	-	66
Coefficient of curvature	-	3.5
Classification	A-7-6 (3)	A-2-4 (0)
Unified Classification	SM	SC
Chart Reference	■	▲

POTENTIAL EXPANSIVENESS

PLASTICITY CHART

TEST REPORT: S11-0570

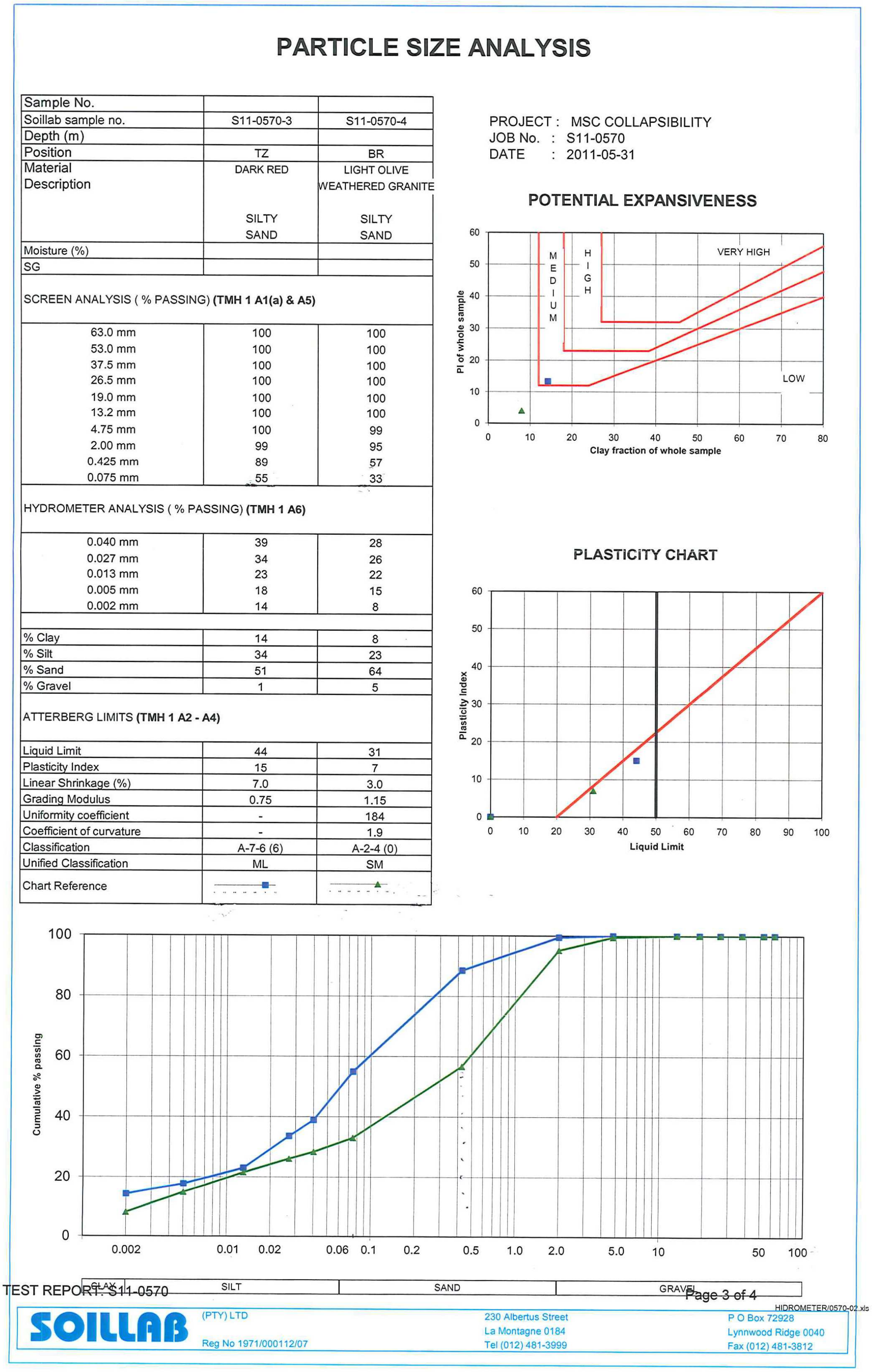
Page 2 of 4

HIDROMETER/0570-01.xls

SOILLAB (PTY) LTD
Reg No 1971/000112/07

230 Albertus Street
La Montagne 0184
Tel (012) 481-3999

P O Box 72928
Lynnwood Ridge 0040
Fax (012) 481-3812



PARTICLE SIZE ANALYSIS

Sample No.		
Soillab sample no.	S11-0570-3	S11-0570-4
Depth (m)		
Position	TZ	BR
Material Description	DARK RED SILTY SAND	LIGHT OLIVE WEATHERED GRANITE SILTY SAND
Moisture (%)		
SG		
SCREEN ANALYSIS (% PASSING) **(TMH 1 A1(a) & A5)**		
63.0 mm	100	100
53.0 mm	100	100
37.5 mm	100	100
26.5 mm	100	100
19.0 mm	100	100
13.2 mm	100	100
4.75 mm	100	99
2.00 mm	99	95
0.425 mm	89	57
0.075 mm	55	33
HYDROMETER ANALYSIS (% PASSING) **(TMH 1 A6)**		
0.040 mm	39	28
0.027 mm	34	26
0.013 mm	23	22
0.005 mm	18	15
0.002 mm	14	8
% Clay	14	8
% Silt	34	23
% Sand	51	64
% Gravel	1	5
ATTERBERG LIMITS **(TMH 1 A2 - A4)**		
Liquid Limit	44	31
Plasticity Index	15	7
Linear Shrinkage (%)	7.0	3.0
Grading Modulus	0.75	1.15
Uniformity coefficient	-	184
Coefficient of curvature	-	1.9
Classification	A-7-6 (6)	A-2-4 (0)
Unified Classification	ML	SM
Chart Reference	■	▲

PROJECT : MSC COLLAPSIBILITY
JOB No. : S11-0570
DATE : 2011-05-31

POTENTIAL EXPANSIVENESS

PLASTICITY CHART

TEST REPORT: S11-0570

Page 3 of 4

HIDROMETER/0570-02.xls

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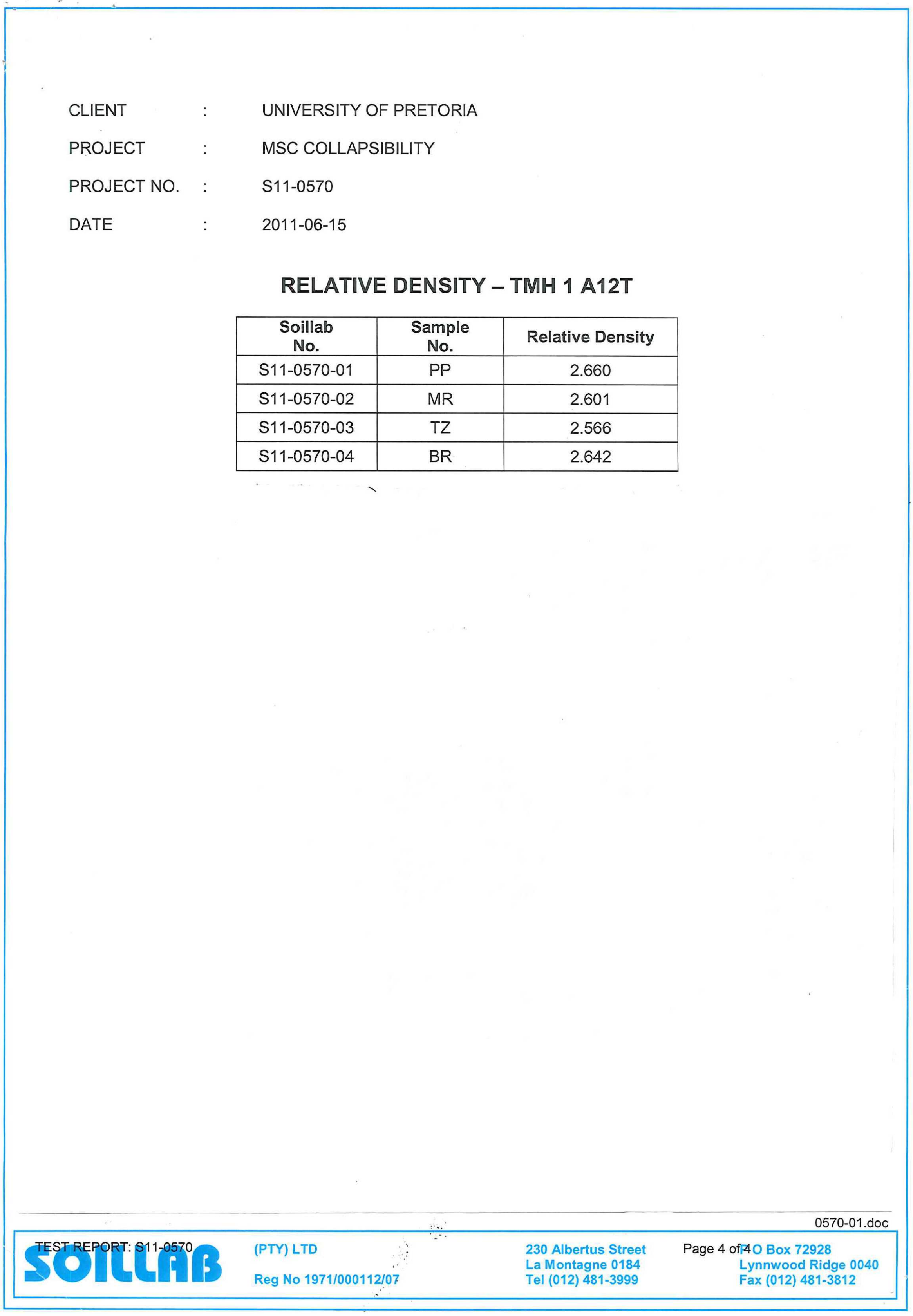

CLIENT : UNIVERSITY OF PRETORIA
PROJECT : MSC COLLAPSIBILITY
PROJECT NO. : S11-0570
DATE : 2011-06-15

RELATIVE DENSITY – TMH 1 A12T

Soillab No.	Sample No.	Relative Density
S11-0570-01	PP	2.660
S11-0570-02	MR	2.601
S11-0570-03	TZ	2.566
S11-0570-04	BR	2.642

APPENDIX C: XRF AND XRD RESULTS

100
1908 - 2008

UNIVERSITEIT VAN PRETORIA
UNIVERSITY OF PRETORIA
YUNIBESITHI YA PRETORIA

**Faculty of Natural &
Agricultural Sciences
XRD & XRF Facility**

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CLIENT: Ryan Freëse

DATE: 1 March 2011

SAMPLES: 4 Samples

ANALYSIS: Qualitative and Quantitative XRD

The samples were prepared for XRD analysis using a back loading preparation method.

They were analysed using a PANalytical X'Pert Pro powder diffractometer with X'Celerator detector and variable divergence- and receiving slits with Fe filtered Co-Kα radiation. The phases were identified using X'Pert Highscore plus software. Graphical representations of the qualitative result follow below.

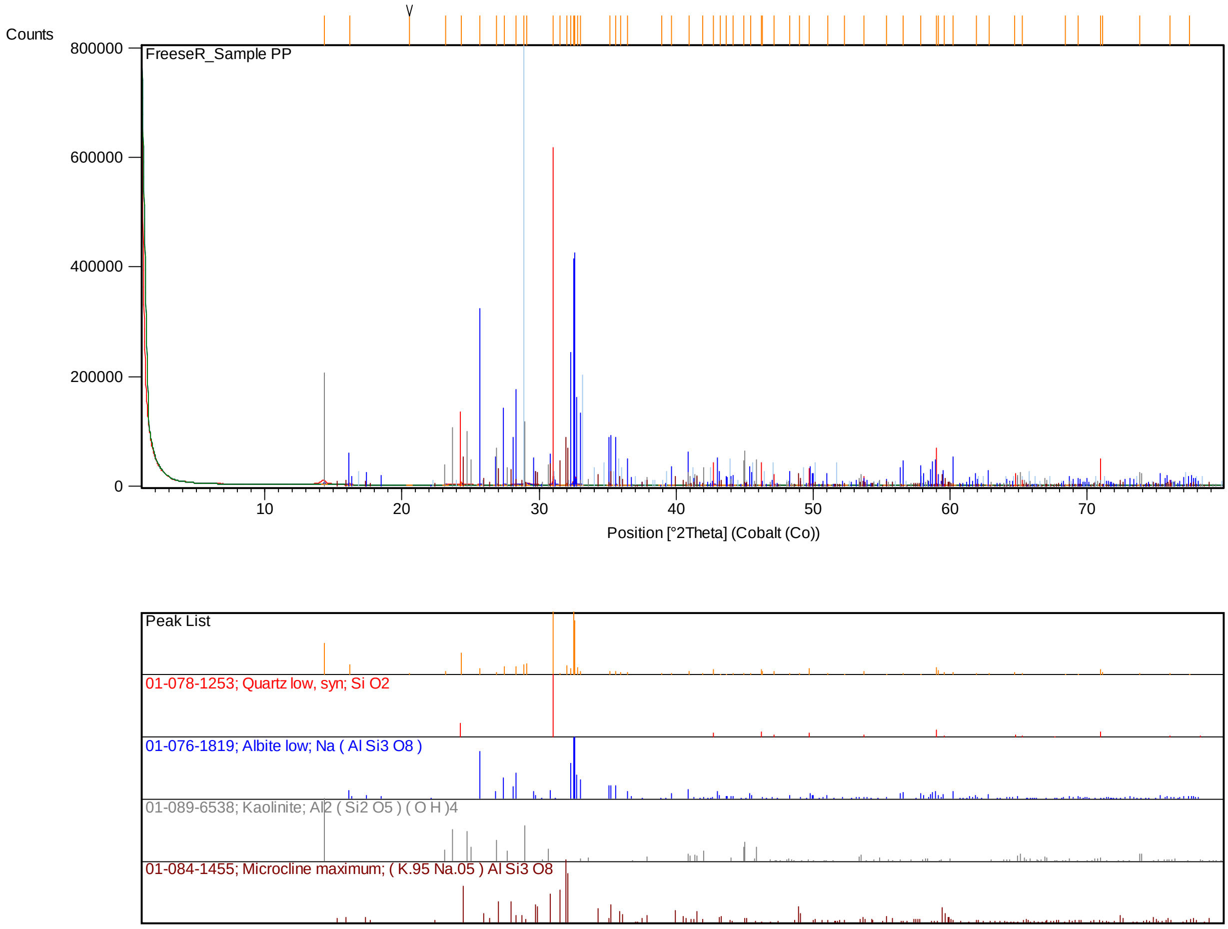
The relative phase amounts (weight %) were estimated using the Rietveld method (Autoquan Program). Errors are on the 3 sigma level in the column to the right of the amount. Amorphous phases, if present were not taken into consideration in the quantification. The quantitative results are listed below.

If you have any further queries, kindly contact the laboratory.

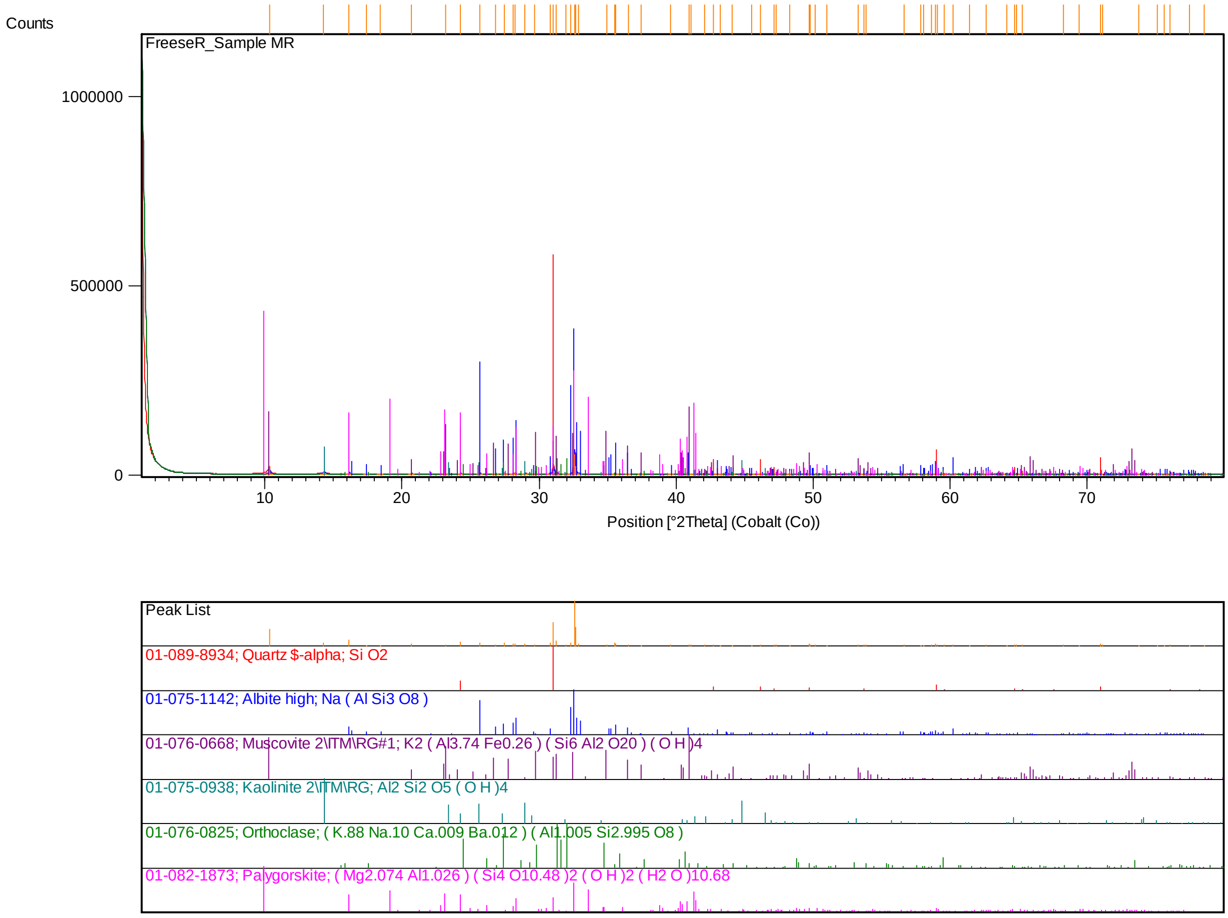
Analyst:

Wiebke Grote

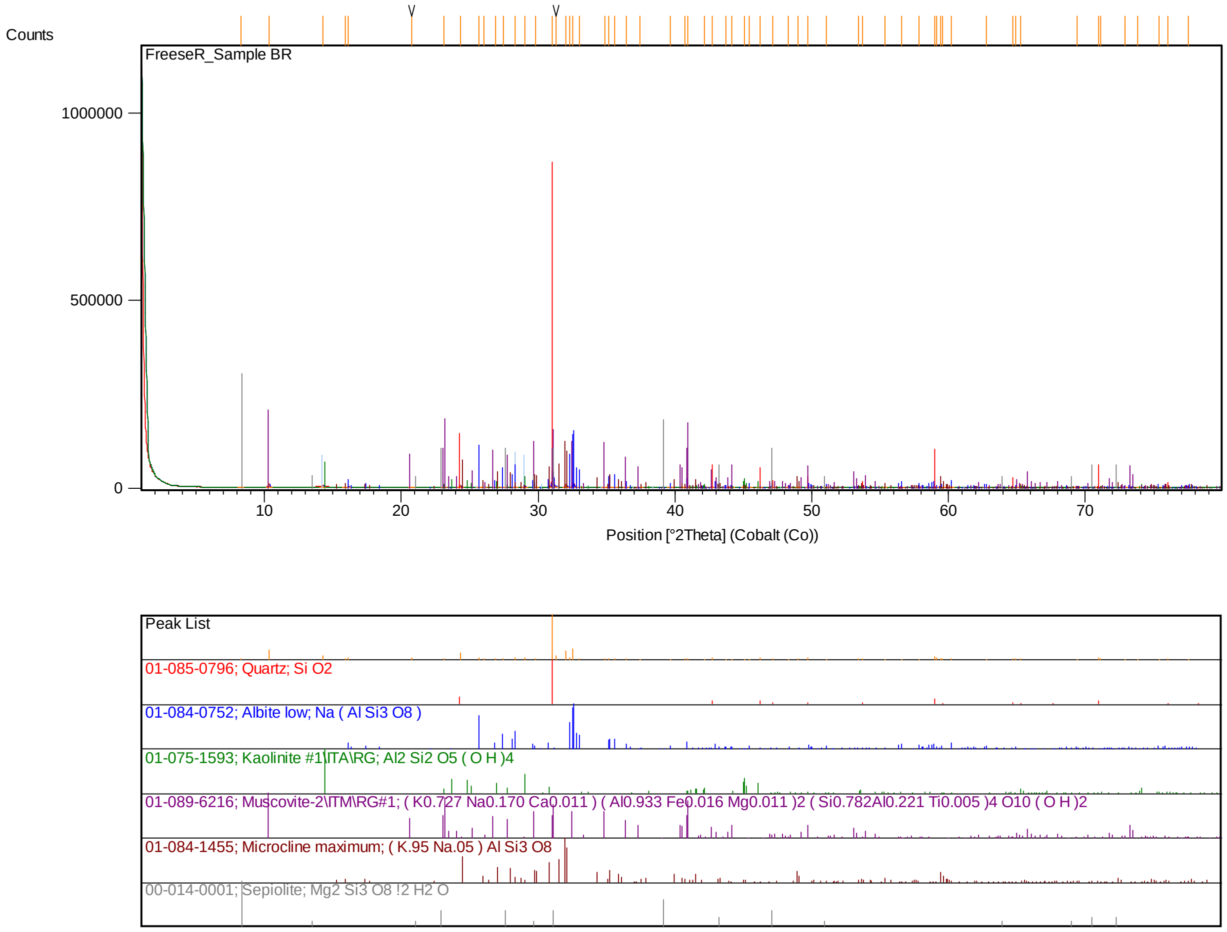
(Grote, Analytical Facility, Dept Geology, University of Pretoria, 2011)



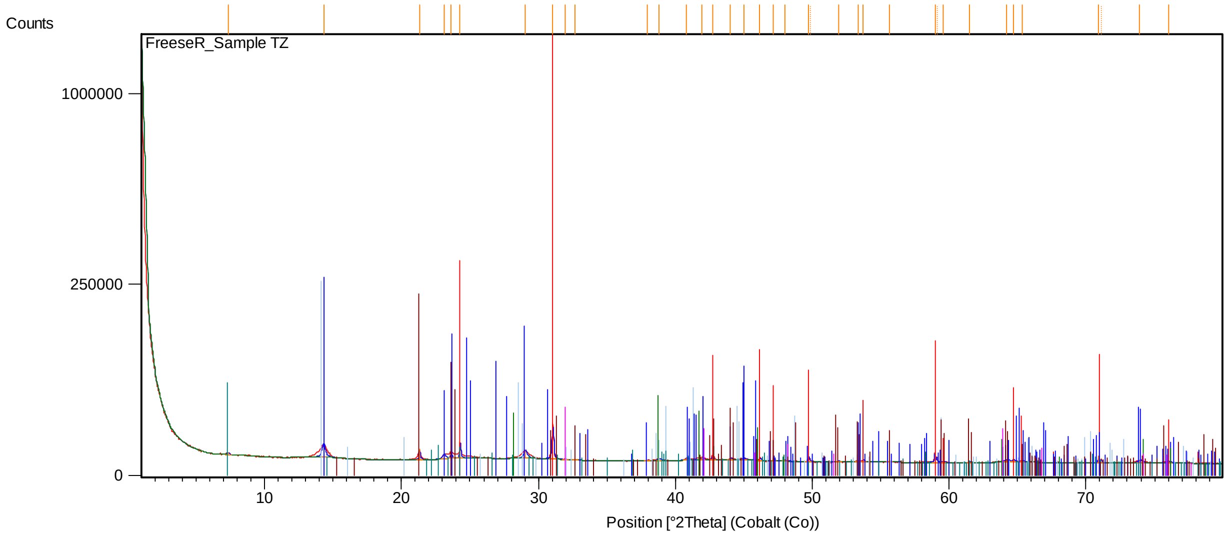
(Grote, Analytical Facility, Dept Geology, University of Pretoria, 2011)

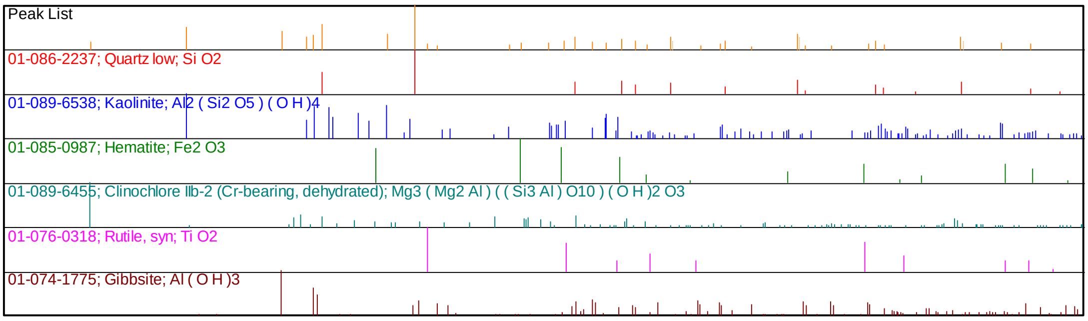


(Grote, Analytical Facility, Dept Geology, University of Pretoria, 2011)



(Grote, Analytical Facility, Dept Geology, University of Pretoria, 2011)





(Grote, Analytical Facility, Dept Geology, University of Pretoria, 2011)

Quantitative results

Sample BR		
	weight%	3 σ error
Kaolinite	27.43	1.17
Microcline	2.02	0.33
Muscovite	14.84	0.96
Plagioclase Albite_	17.48	1.02
Quartz_	31.06	1.08
Sepiolite_	7.16	1.14

Sample MR		
	weight%	3 σ error
Kaolinite	18.09	2.67
Muscovite	6.68	1.62
Orthoclase_	8.08	1.83
Palygorskite_	18.51	1.92
Plagioclase Albite_	31.39	1.98
Quartz_	17.25	1.35

Sample PP		
	weight%	3 σ error
Chlorite	5.84	1.44
Kaolinite	51.72	1.77
Microcline	6.57	0.9
Palygorskite_	0.72	0.66
Plagioclase Albite_	18.08	1.14
Quartz_	17.06	0.87

Sample TZ		
	weight%	3 σ error
Chlorite IIb-2_	24.78	1.65
Gibbsite_	6.97	0.93
Hematite_	2.98	0.33
Kaolinite	40.21	1.74
Quartz_	21.28	0.81
Rutile_	3.77	0.69

(Dykstra, Analytical Facility, Dept Geology, University of Pretoria, 2011)

Sample BR XRF			
	GSNcert	**GSN**	**BR**
SiO_2	**65.80**	65.63	74.04
TiO_2	**0.68**	0.63	0.18
Al_2O_3	**14.67**	14.32	15.56
Fe_2O_3	**3.75**	3.90	1.55
MnO	**0.06**	0.05	*<0.01*
MgO	**2.30**	2.01	0.20
CaO	**2.50**	2.72	0.07
Na_2O	**3.77**	3.95	1.46
K_2O	**4.63**	4.72	2.86
P_2O_5	**0.28**	0.23	*<0.01*
Cr_2O_3	**0.008**	0.04	*<0.01*
NiO	**0.0043**	0.06	*<0.01*
V_2O_5	**0.01**	0.01	*<0.01*
ZrO_2	**0.03**	0.02	0.02
SO_3		0.03	*<0.01*
WO_3		0.06	0.01
BaO		0.14	0.08
Cl		0.07	0.08
CuO		0.03	*<0.01*
ZnO		*<0.01*	*<0.01*
Rb_2O		*<0.01*	0.02
SrO		0.08	*<0.01*
LOI	**1.32**	1.32	3.87

(Dykstra, Analytical Facility, Dept Geology, University of Pretoria, 2011)

Sample MR XRF			
	GSNcert	**GSN**	**MR**
SiO_2	**65.80**	65.63	73.21
TiO_2	**0.68**	0.63	0.22
Al_2O_3	**14.67**	14.32	14.73
Fe_2O_3	**3.75**	3.90	2.26
MnO	**0.06**	0.05	0.03
MgO	**2.30**	2.01	0.33
CaO	**2.50**	2.72	0.70
Na_2O	**3.77**	3.95	3.74
K_2O	**4.63**	4.72	1.30
P_2O_5	**0.28**	0.23	*<0.01*
Cr_2O_3	**0.008**	0.04	*<0.01*
NiO	**0.0043**	0.06	0.01
V_2O_5	**0.01**	0.01	*<0.01*
ZrO_2	**0.03**	0.02	0.01
SO_3		0.03	*<0.01*
WO_3		0.06	0.02
BaO		0.14	*<0.01*
Cl		0.07	0.11
CuO		0.03	*<0.01*
ZnO		*<0.01*	0.02
Rb_2O		*<0.01*	0.01
SrO		0.08	0.01
LOI	**1.32**	1.32	3.30

(Dykstra, Analytical Facility, Dept Geology, University of Pretoria, 2011)

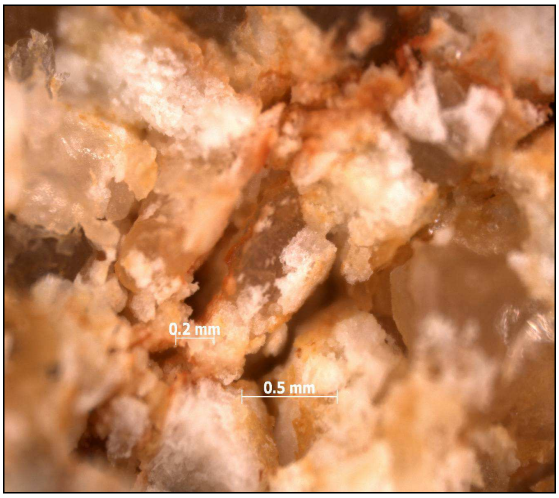
Sample PP XRF			
	GSNcert	**GSN**	**PP**
SiO_2	**65.80**	65.63	61.79
TiO_2	**0.68**	0.63	0.56
Al_2O_3	**14.67**	14.32	19.47
Fe_2O_3	**3.75**	3.90	5.95
MnO	**0.06**	0.05	0.11
MgO	**2.30**	2.01	0.23
CaO	**2.50**	2.72	0.77
Na_2O	**3.77**	3.95	3.23
K_2O	**4.63**	4.72	1.20
P_2O_5	**0.28**	0.23	*<0.01*
Cr_2O_3	**0.008**	0.04	*<0.01*
NiO	**0.0043**	0.06	0.01
V_2O_5	**0.01**	0.01	0.01
ZrO_2	**0.03**	0.02	0.01
SO_3		0.03	*<0.01*
WO_3		0.06	*<0.01*
BaO		0.14	*<0.01*
Cl		0.07	0.11
CuO		0.03	*<0.01*
ZnO		*<0.01*	*<0.01*
Rb_2O		*<0.01*	*<0.01*
SrO		0.08	0.02
LOI	**1.32**	1.32	6.54

(Dykstra, Analytical Facility, Dept Geology, University of Pretoria, 2011)

Sample TZ XRF			
	GSNcert	**GSN**	**TZ**
SiO_2	**65.80**	65.63	53.93
TiO_2	**0.68**	0.63	1.07
Al_2O_3	**14.67**	14.32	24.63
Fe_2O_3	**3.75**	3.90	8.45
MnO	**0.06**	0.05	0.06
MgO	**2.30**	2.01	0.01
CaO	**2.50**	2.72	0.02
Na_2O	**3.77**	3.95	*<0.01*
K_2O	**4.63**	4.72	0.46
P_2O_5	**0.28**	0.23	0.08
Cr_2O_3	**0.008**	0.04	0.02
NiO	**0.0043**	0.06	0.01
V_2O_5	**0.01**	0.01	0.01
ZrO_2	**0.03**	0.02	0.02
SO_3		0.03	*<0.01*
WO_3		0.06	0.01
BaO		0.14	*<0.01*
Cl		0.07	0.08
CuO		0.03	0.01
ZnO		*<0.01*	*<0.01*
Rb_2O		*<0.01*	*<0.01*
SrO		0.08	*<0.01*
LOI	**1.32**	1.32	11.12

APPENDIX D: SAMPLE MICROGRAPHS

<u>Bushbuckridge Un-consolidated Sample at X1 to X100 Magnitude</u>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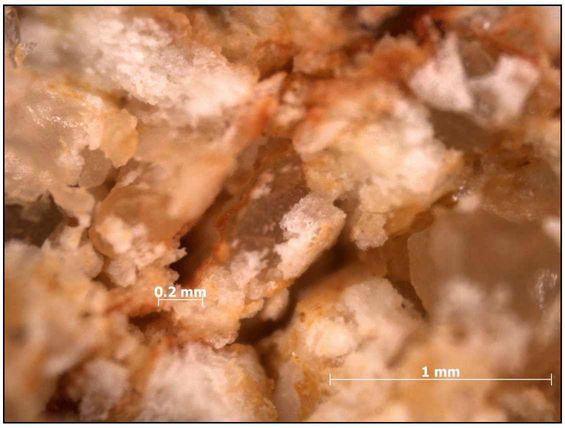
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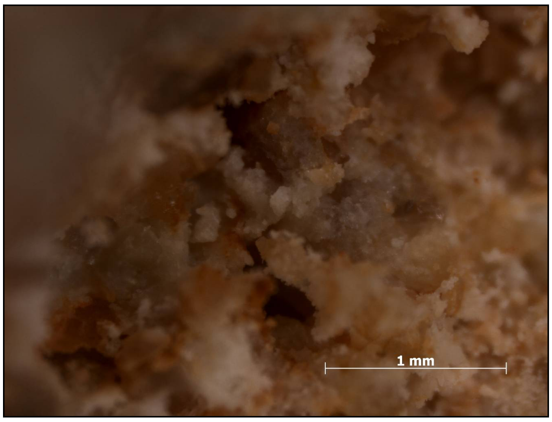
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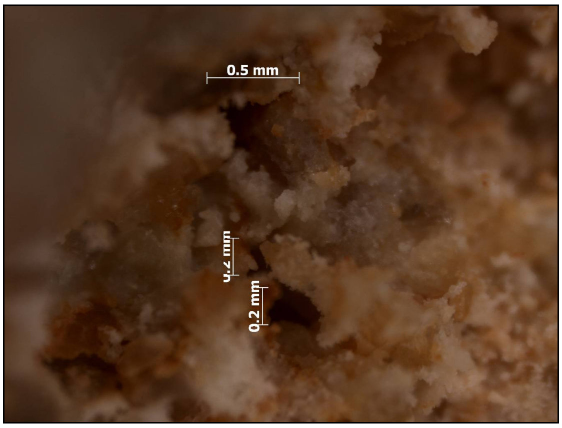
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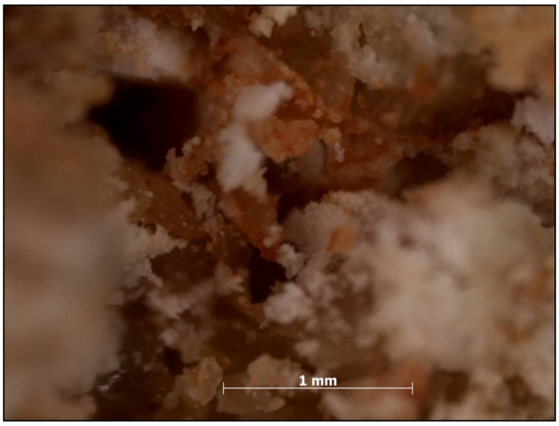
0.2 mm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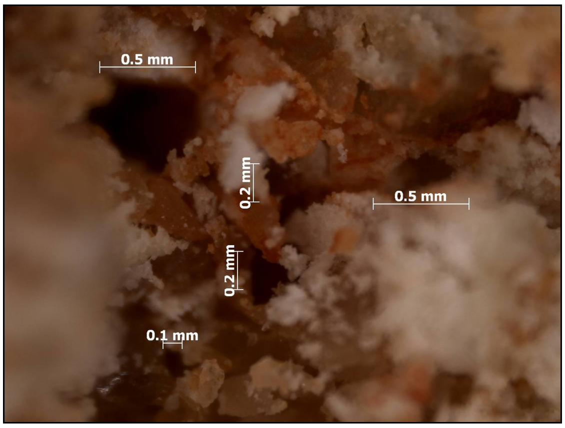
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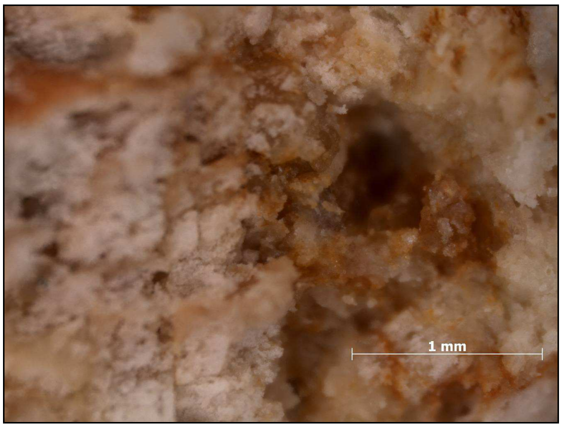
0.2 mm
0.5 mm
0.2 mm
0.1 mm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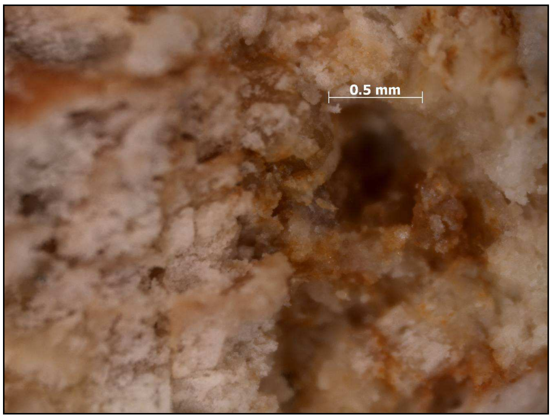
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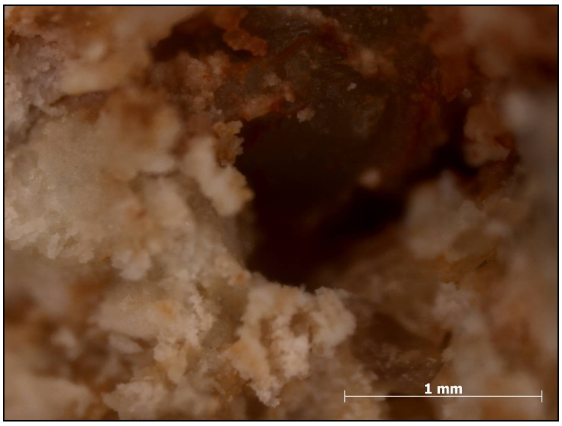
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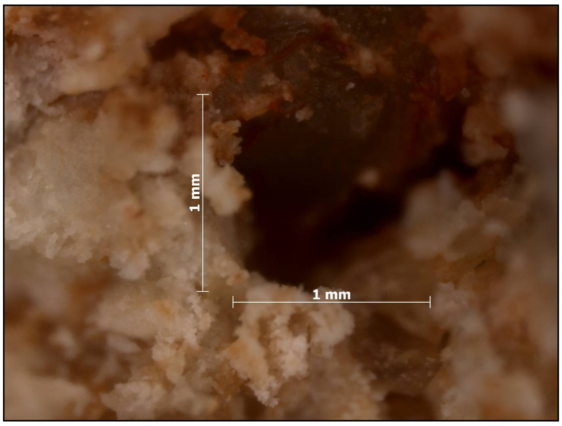
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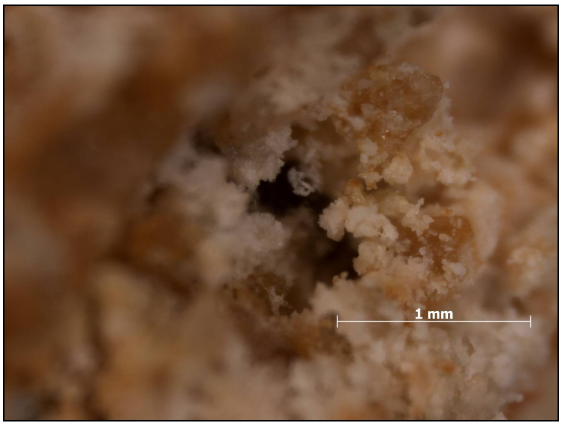
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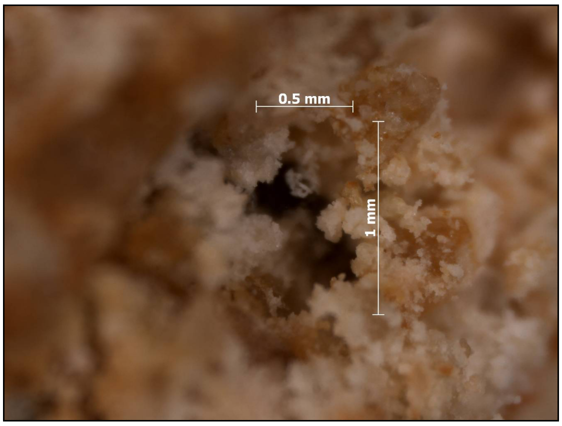
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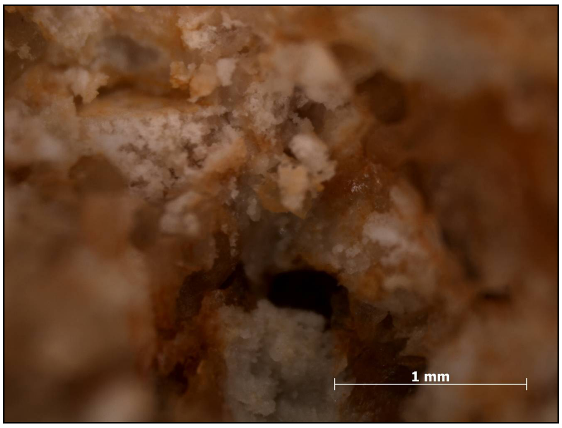
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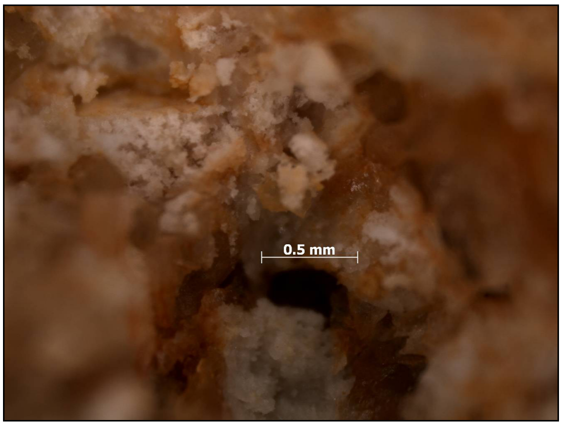
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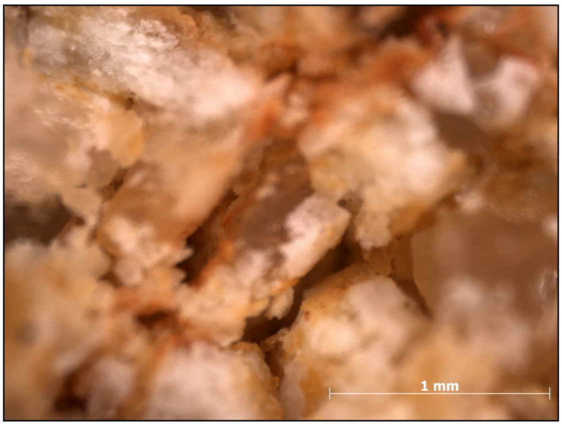
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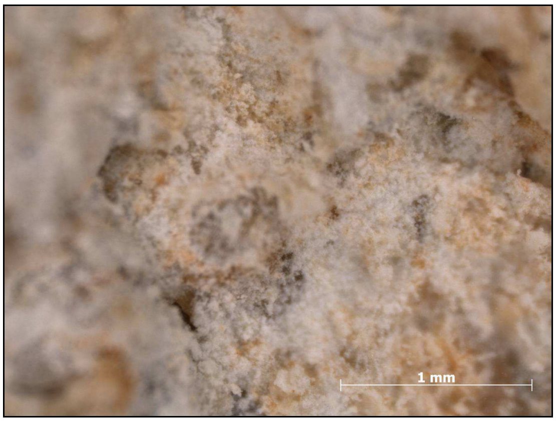
Bushbuckridge Consolidated Sample at X1 to X100 Magnitude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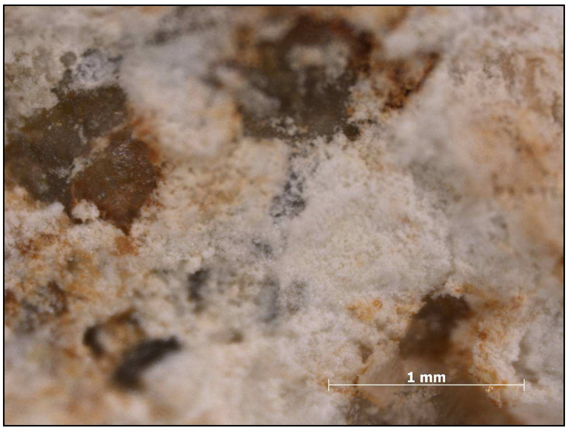
0.05 m
0.1 mm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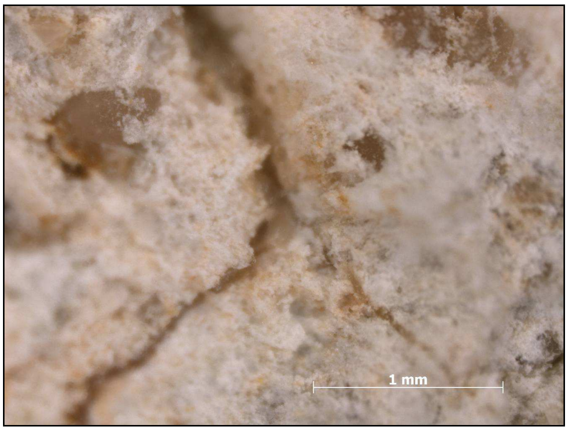
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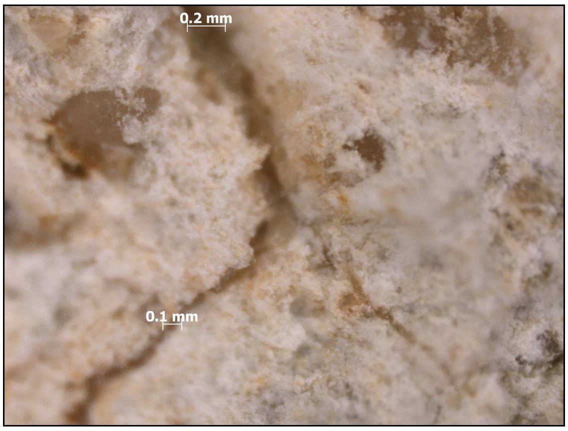
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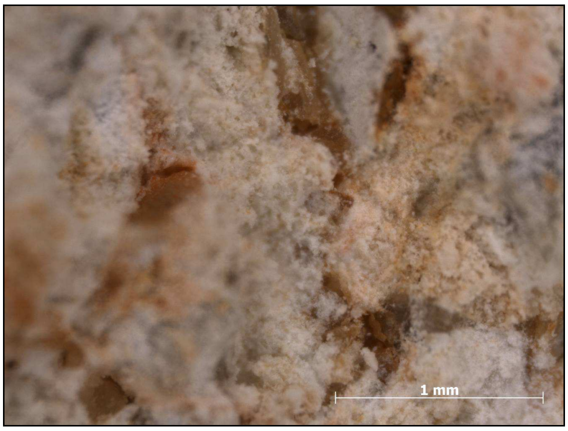
0.1 mm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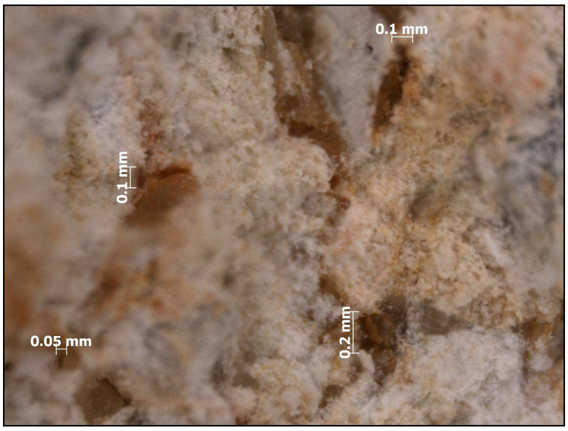
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0.1 mm
0.05 mm
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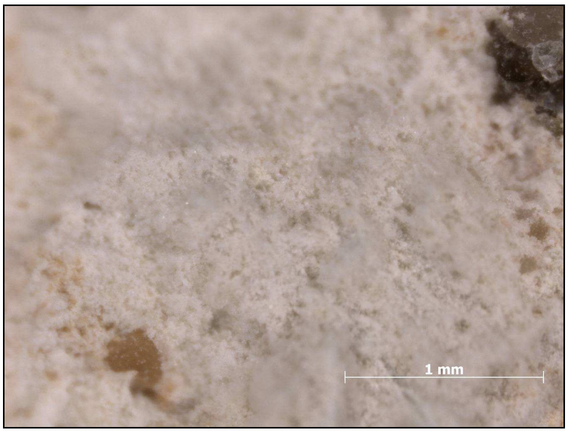
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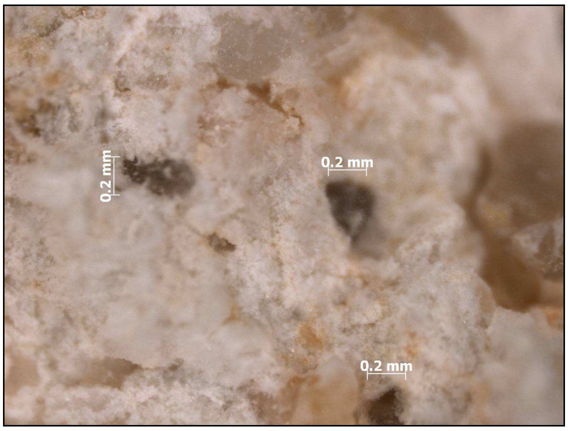
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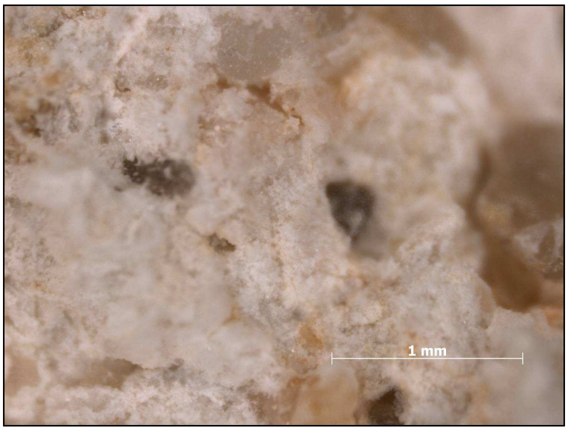
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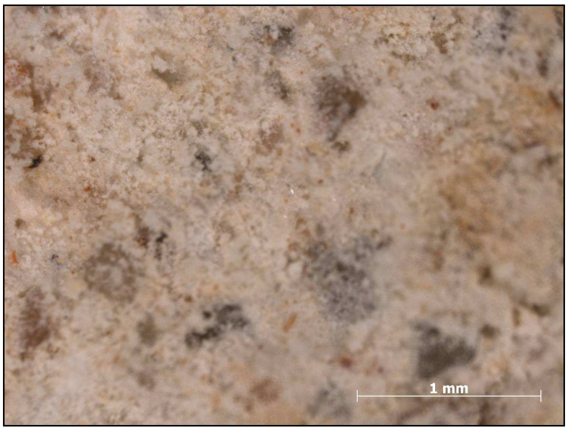
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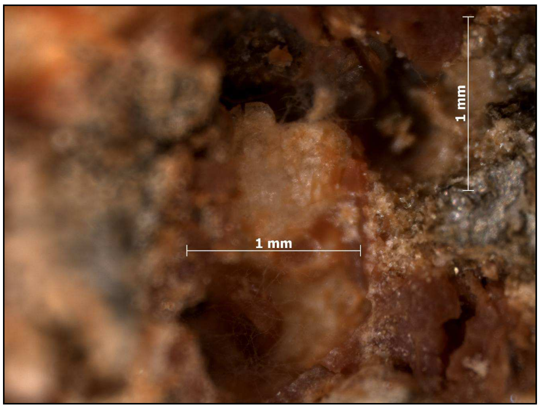
Midrand Un-consolidated Sample at X1 to X100 Magnitude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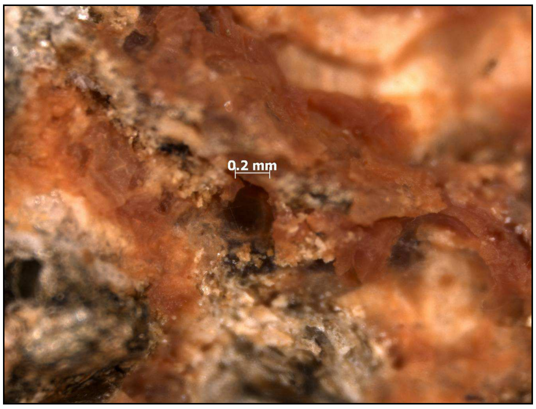
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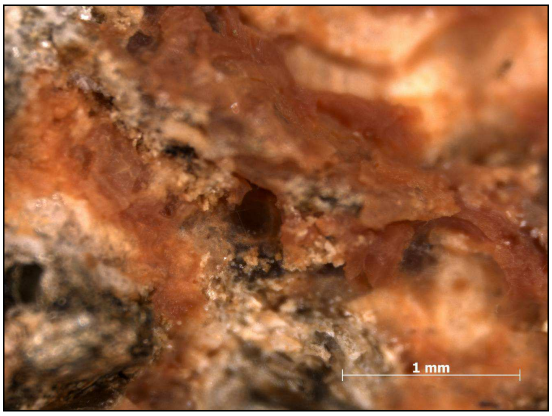
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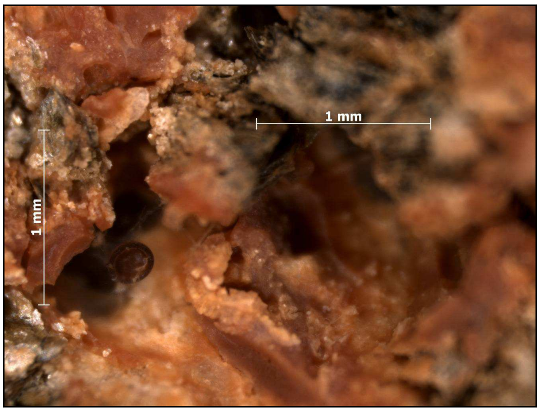
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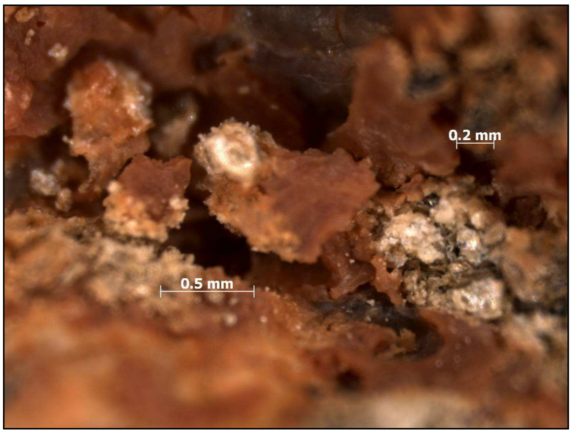
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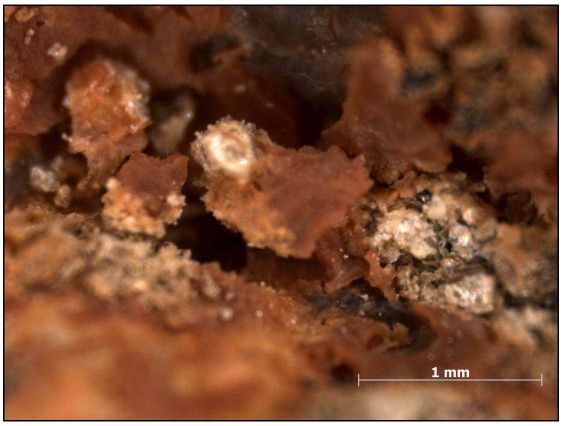
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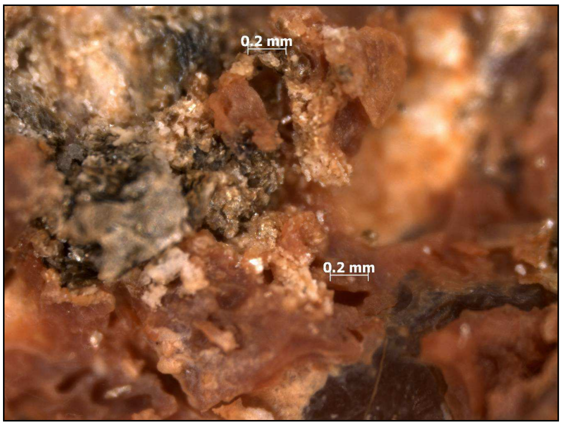
2 mm


1 mm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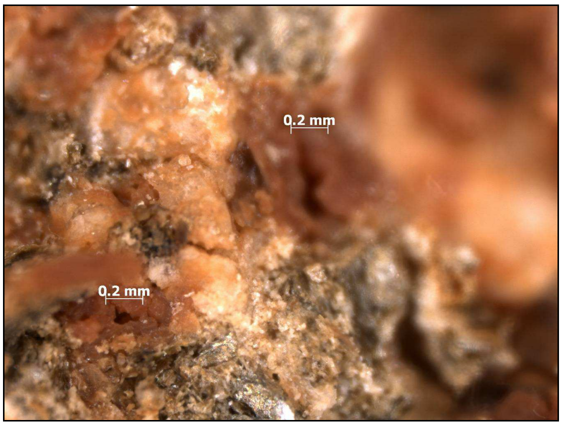
0.2 mm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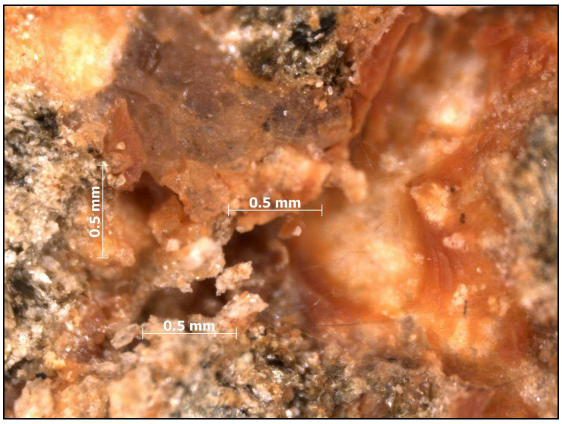
0.2 mm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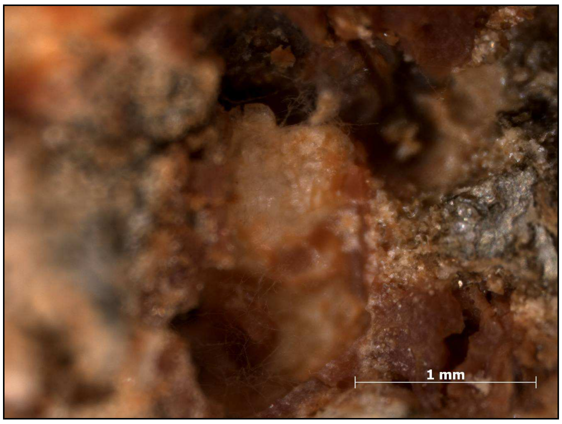
0.5 mm
0.5 mm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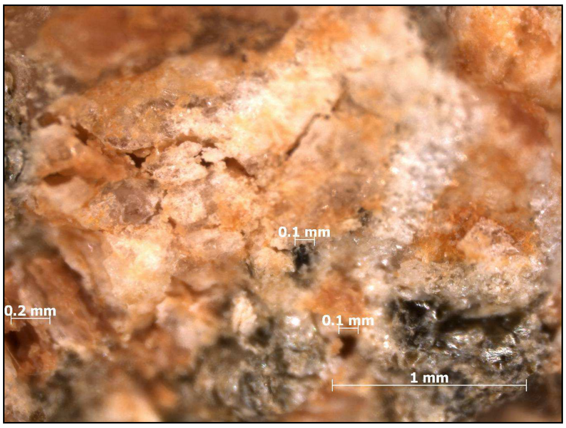
1 mm

Midrand Consolidated Sample at X1 to X100 Magnitude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0.2 mm
0.1 mm
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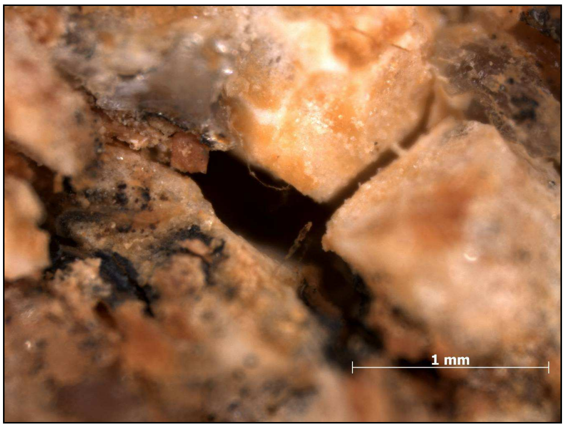
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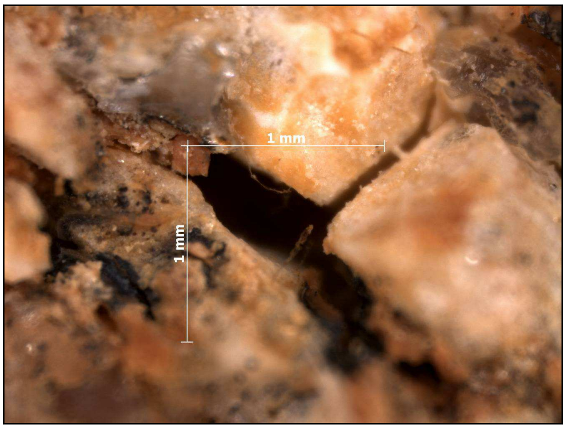
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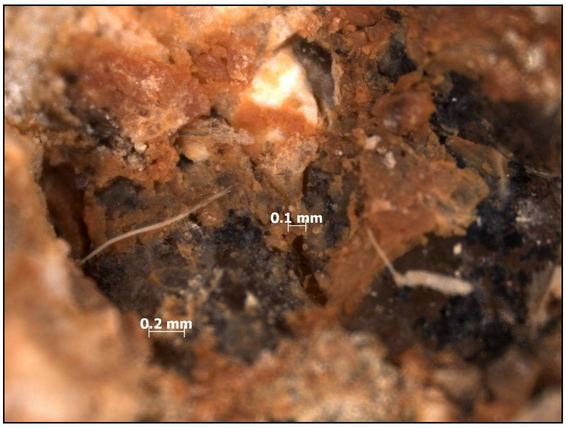
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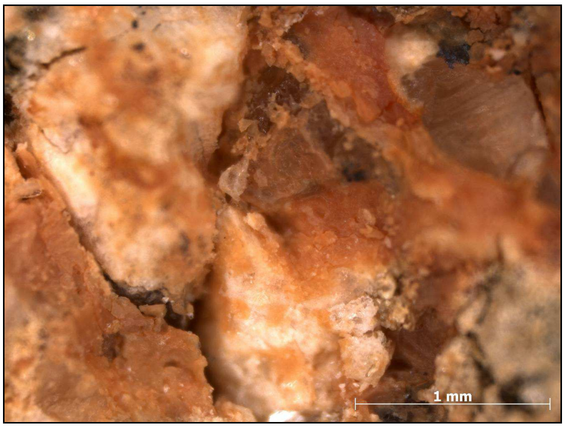
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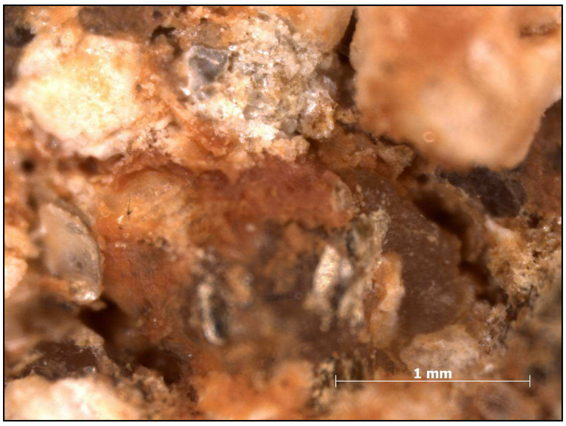
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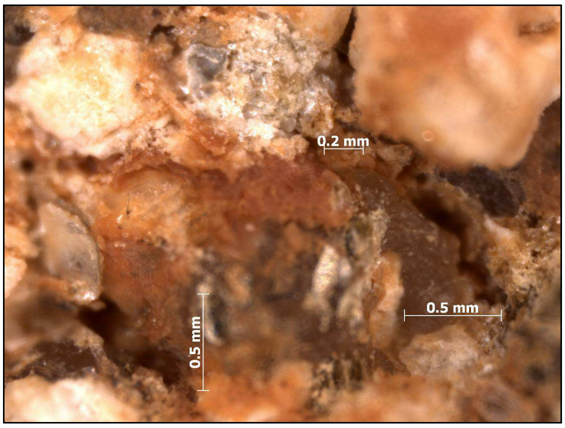
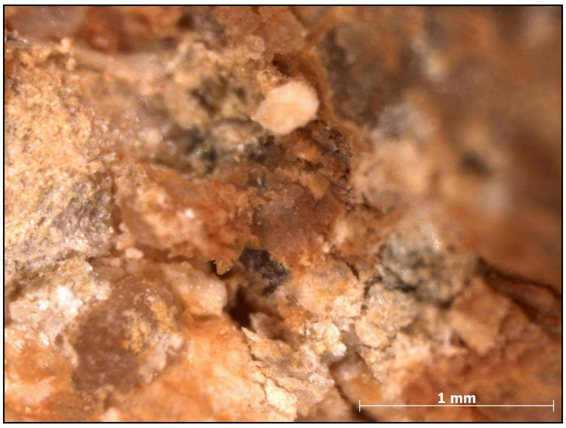
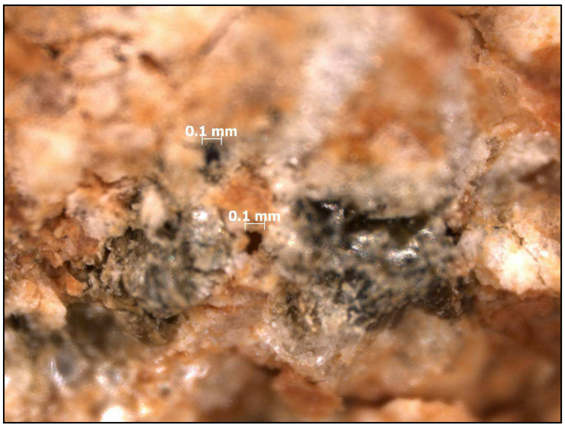
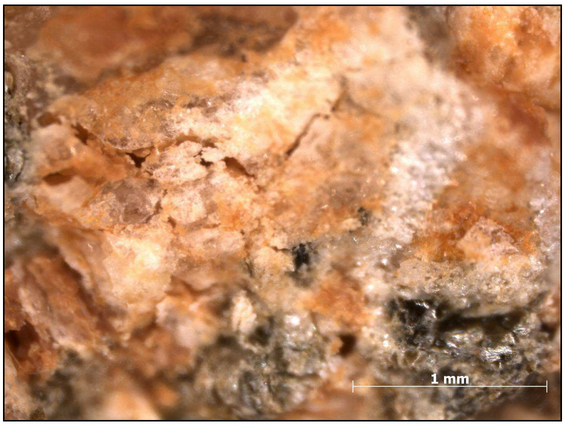

0.2 mm
0.5 mm
0.5 mm

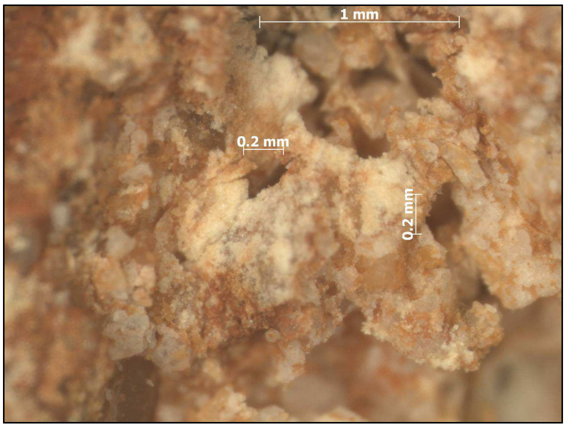

1 mm

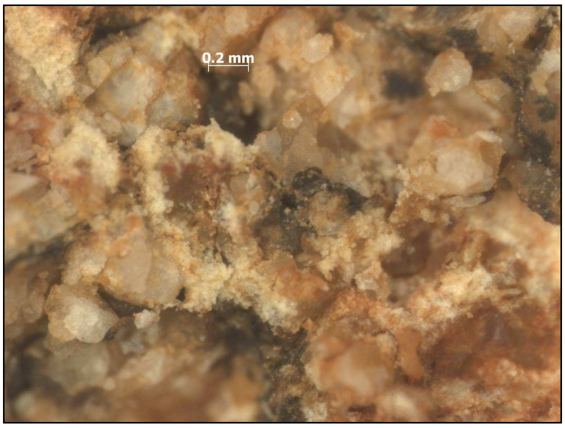

0.1 mm
0.1 mm


0.2 mm


1 mm

Paulpietersburg Un-consolidated Sample at X1 to X100 Magnitude


1 mm
0.2 mm
0.2 mm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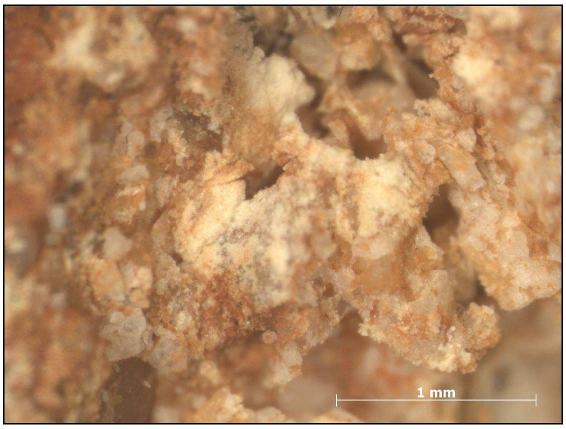
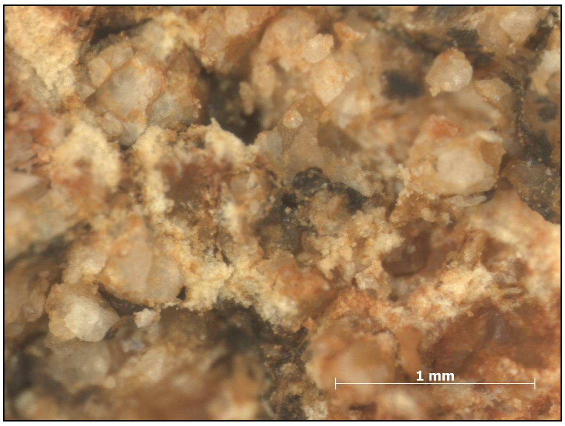
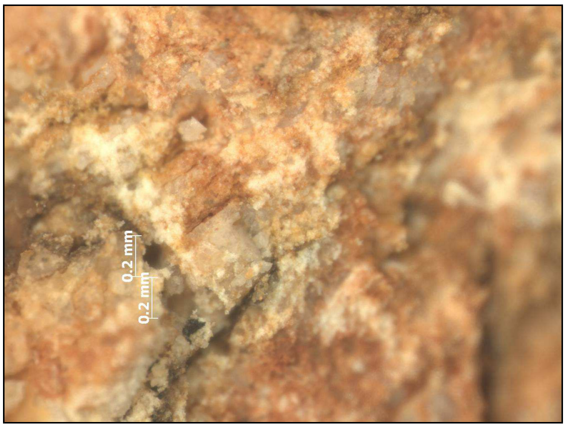
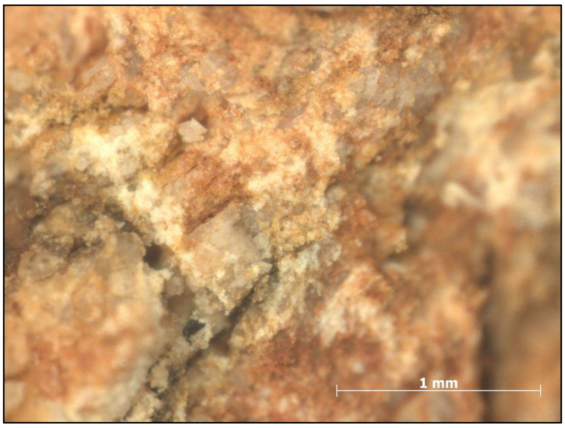
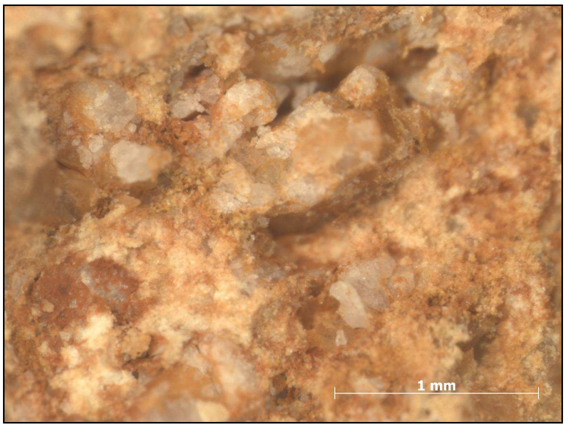

1 mm


1 mm


0.2 mm
0.2 mm


1 mm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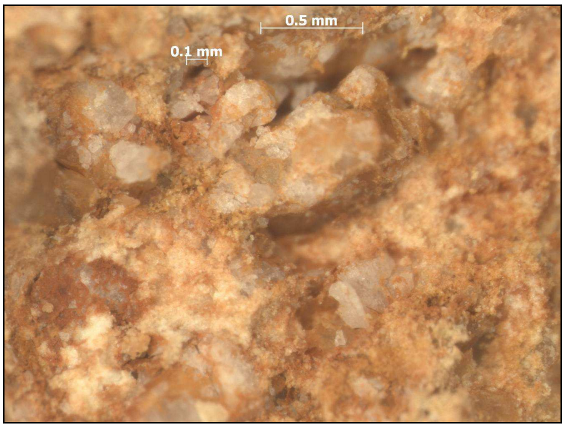
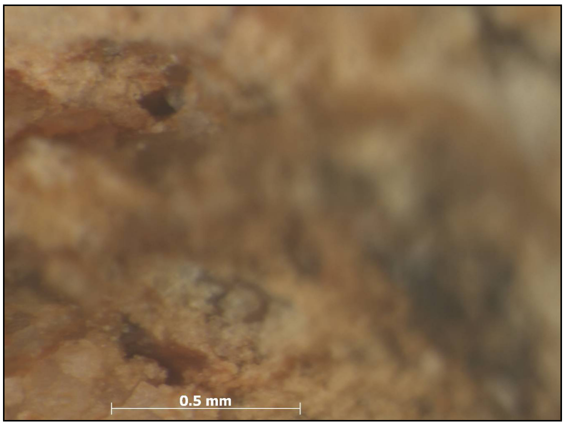

0.5 mm
0.1 mm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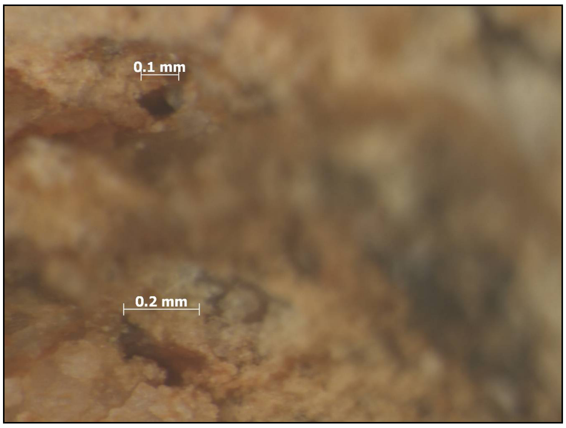
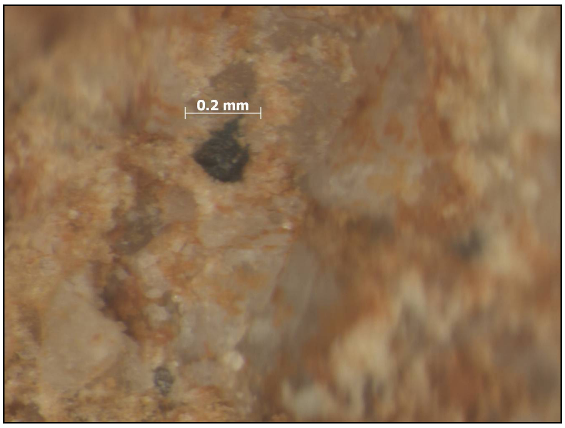
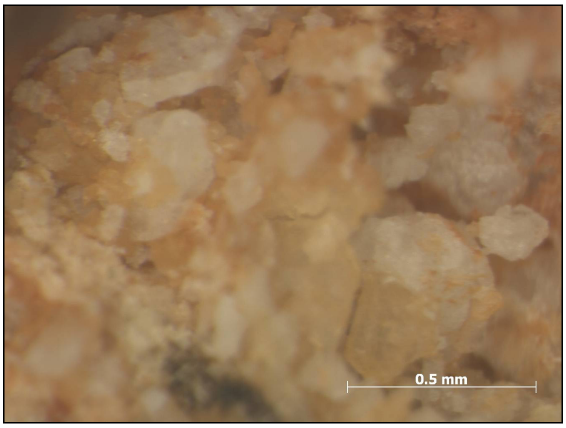

0.1 mm
0.2 mm


0.2 mm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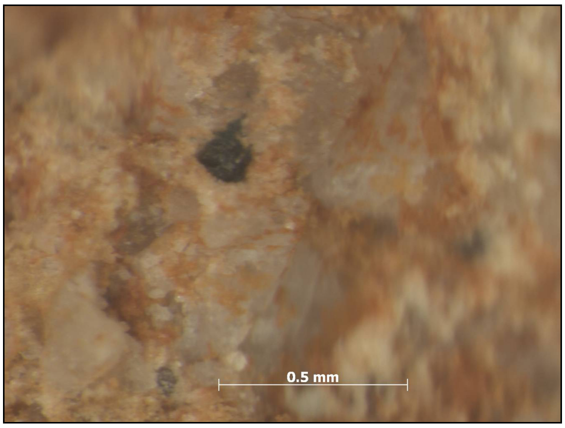
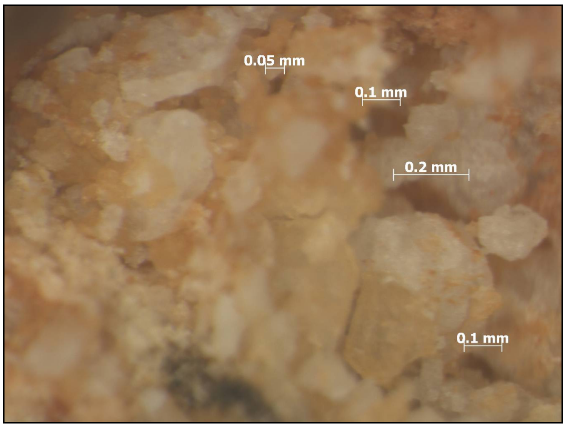
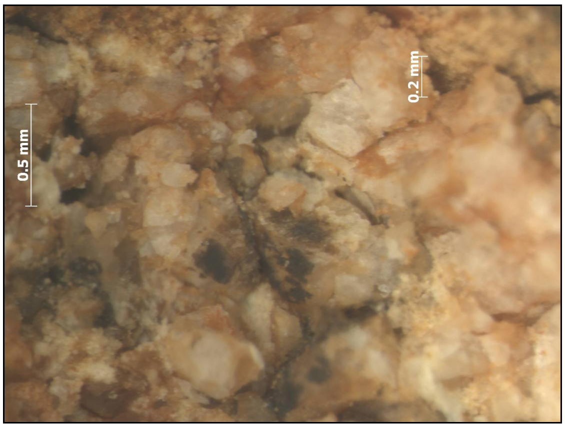
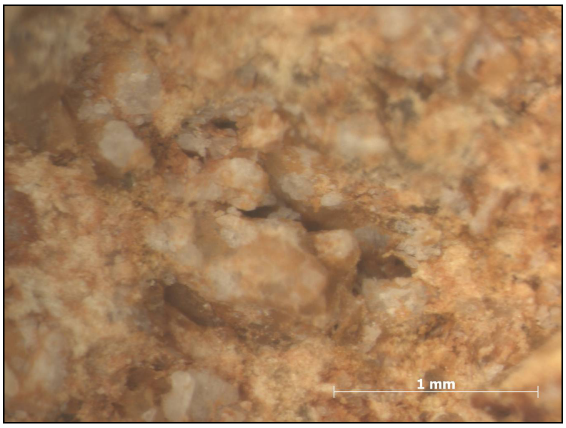
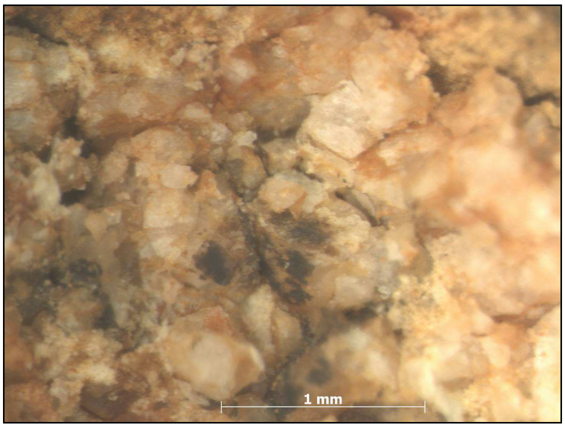
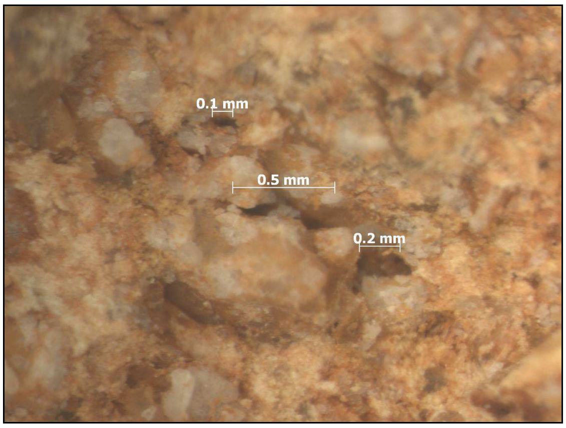

0.5 mm


0.05 mm
0.1 mm
0.2 mm
0.1 mm


0.5 mm
0.2 mm

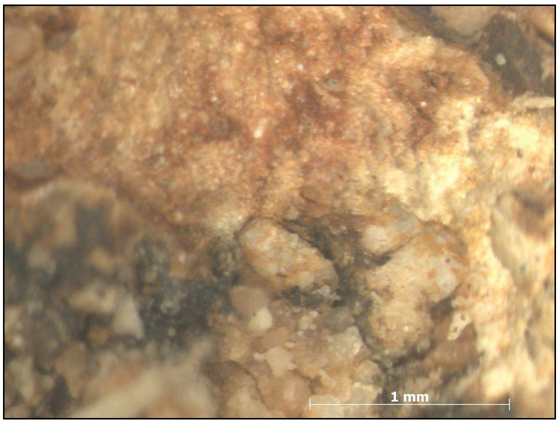

1 mm

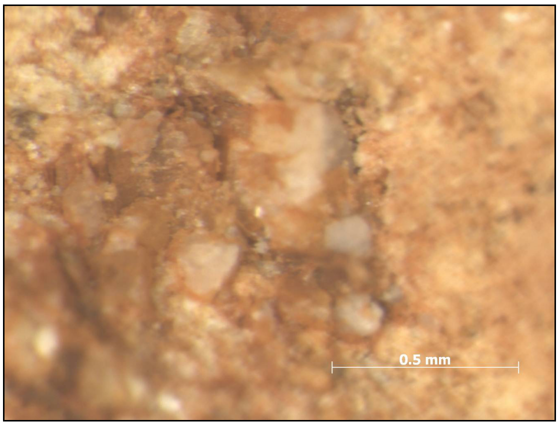

1 mm

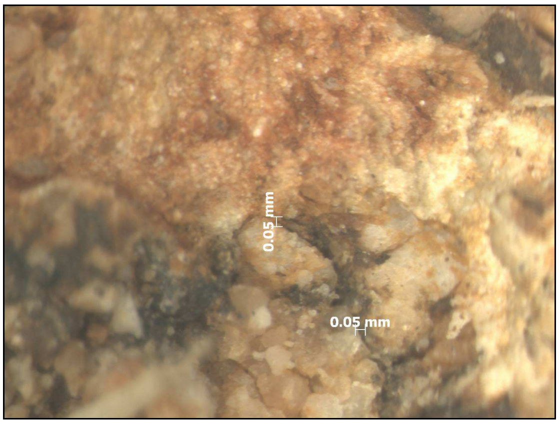

0.1 mm
0.5 mm
0.2 mm

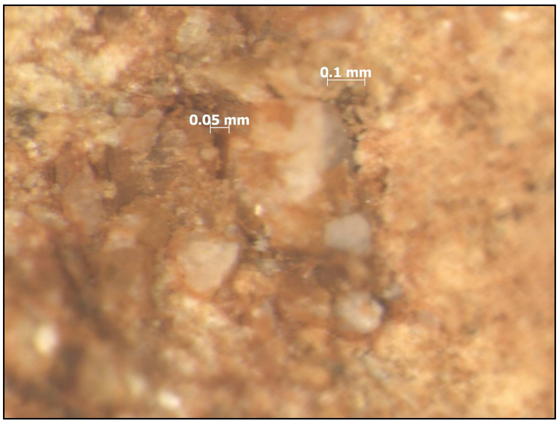
Paulpietersburg Consolidated Sample at X1 to X100 Magnitude

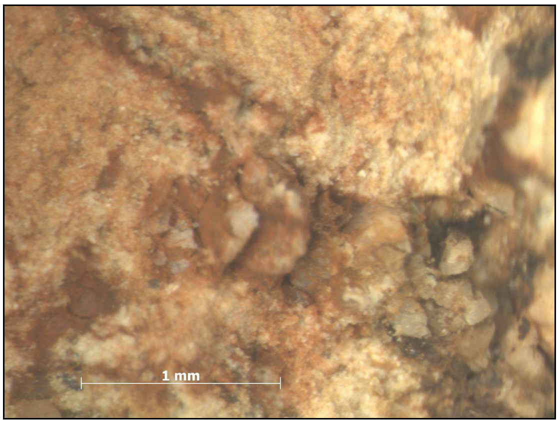

1 mm

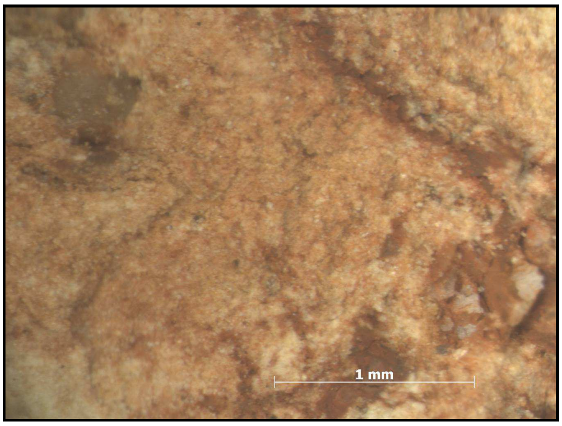

1 mm

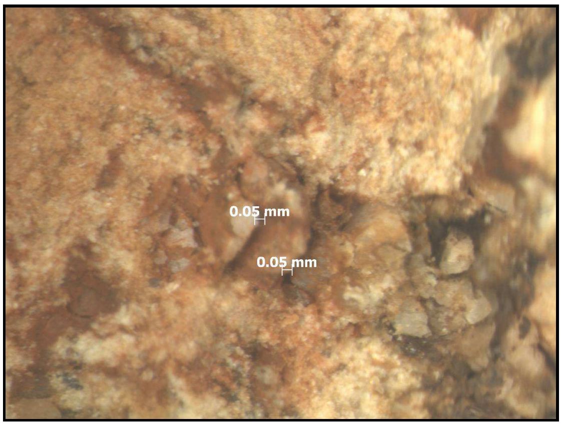

0.5 mm

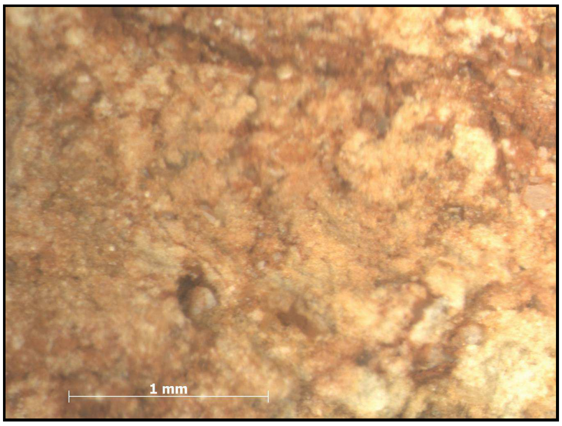

0.05 mm
0.05 mm

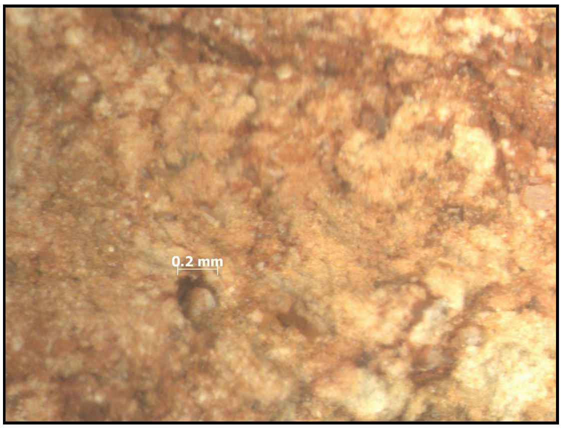

0.1 mm
0.05 mm

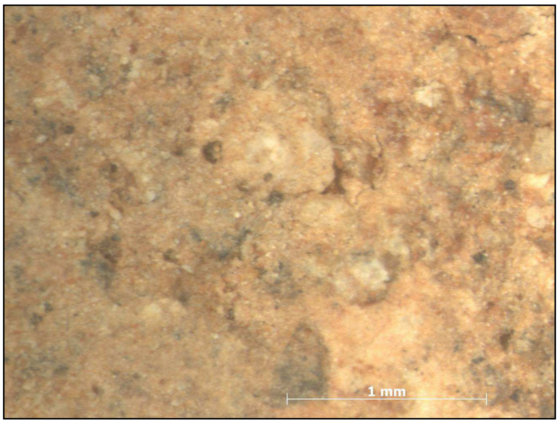

1 mm

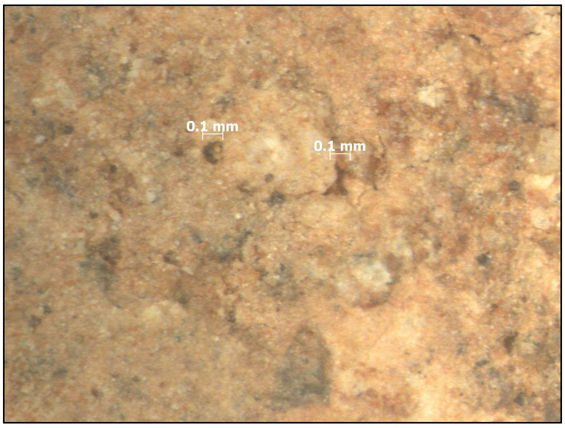

1 mm


0.05 mm
0.05 mm

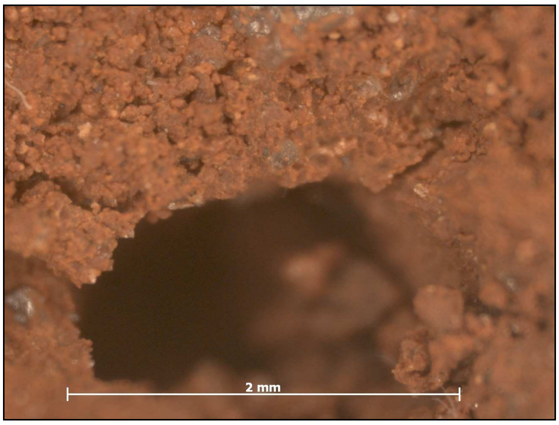

1 mm

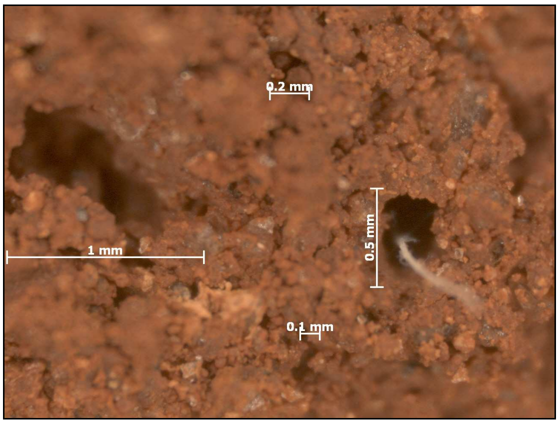

0.2 mm

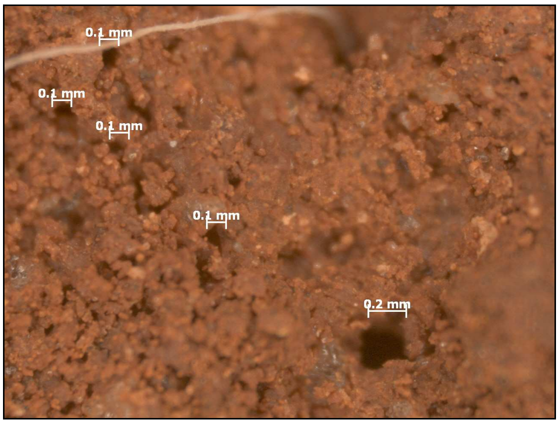

1 mm

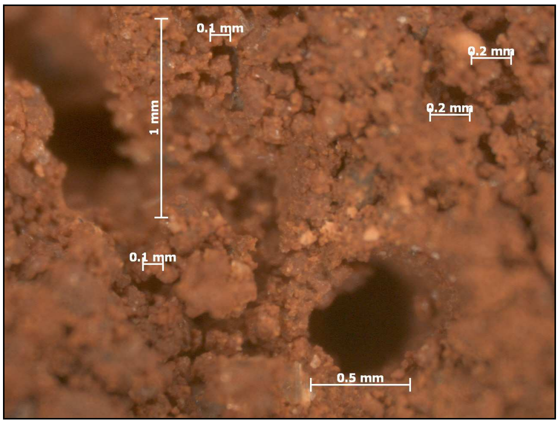

0.1 mm
0.1 mm

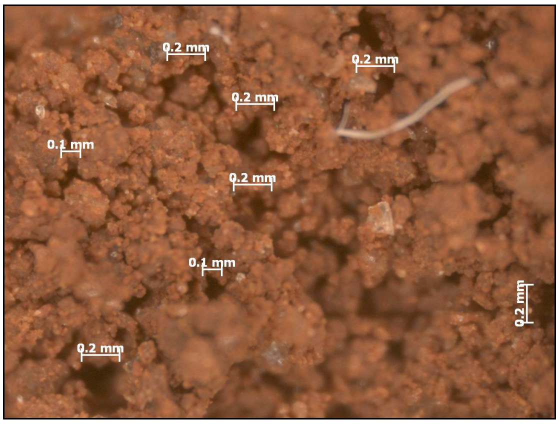
Tzaneen Un-consolidated Sample at X1 to X100 Magnitude

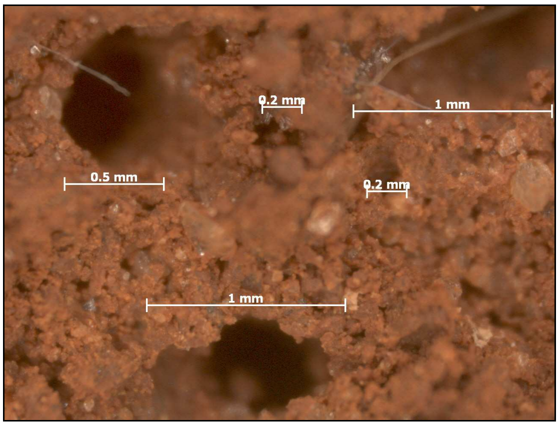

2 mm

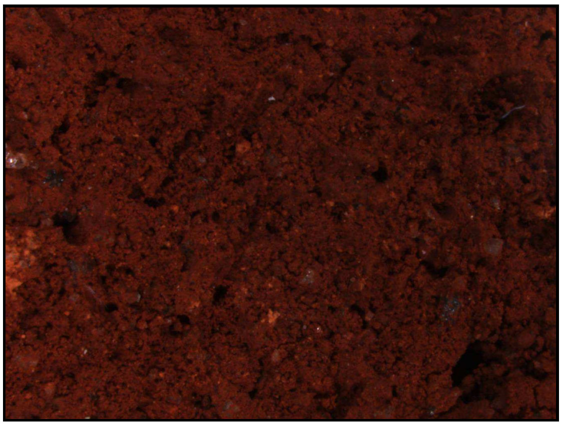

0.2 mm
1 mm
0.5 mm
0.1 mm

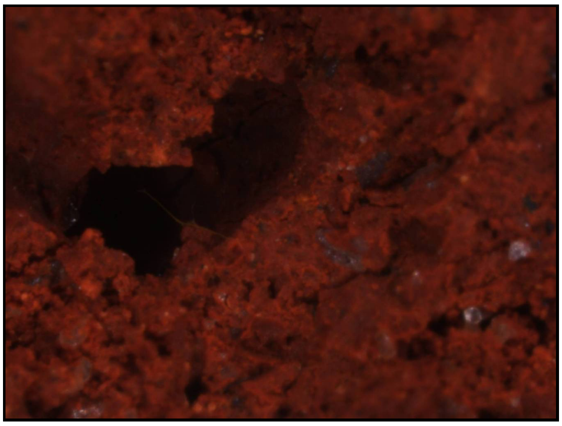

0.1 mm
0.1 mm
0.1 mm
0.1 mm
0.2 mm

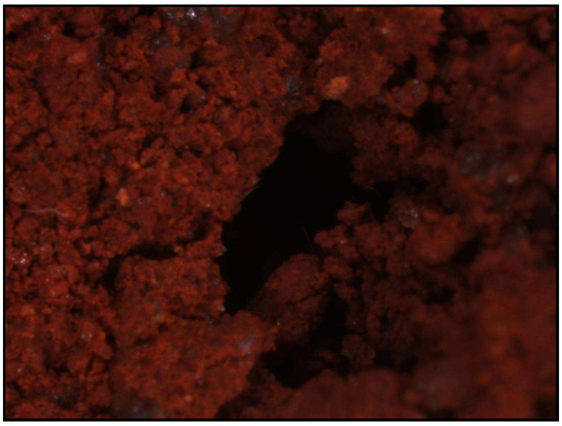

0.1 mm
0.2 mm
0.2 mm
1 mm
0.1 mm
0.5 mm

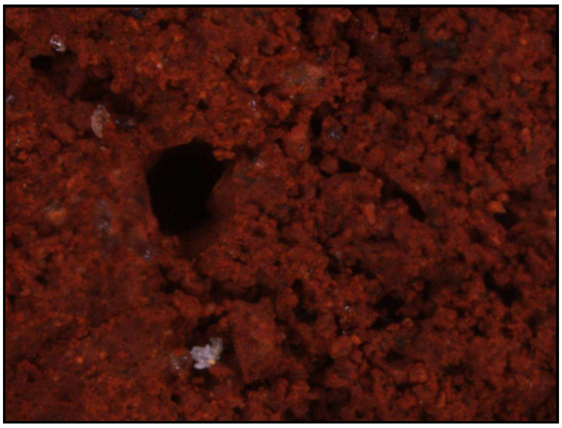

0.2 mm
0.2 mm
0.2 mm
0.1 mm
0.2 mm
0.1 mm
0.2 mm
0.2 mm

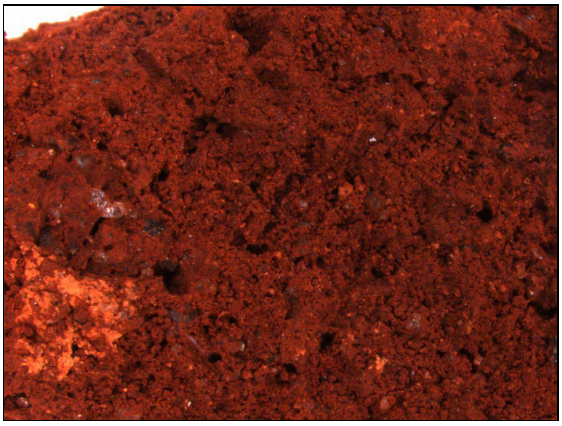

0.2 mm
1 mm
0.5 mm
0.2 mm
1 mm

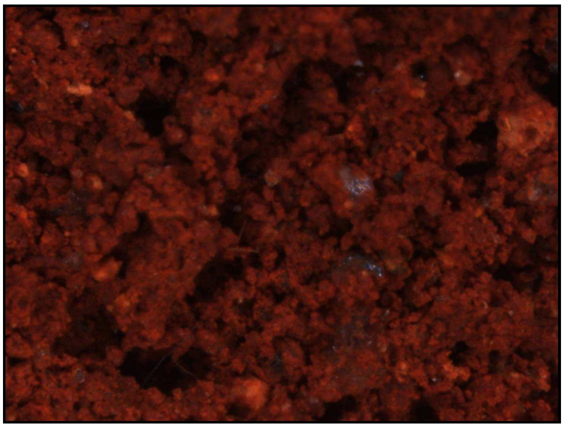


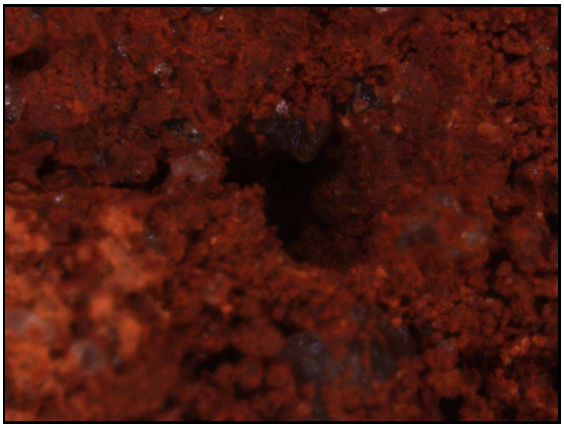


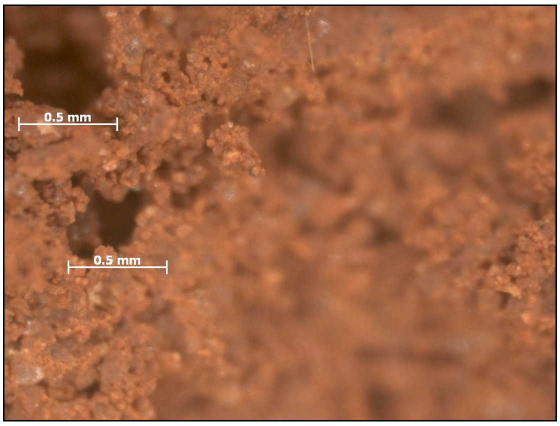




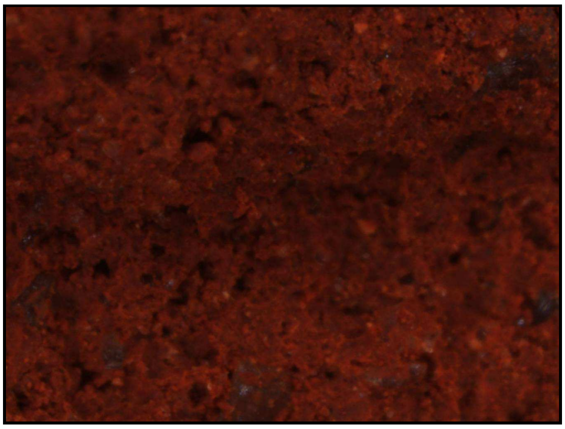


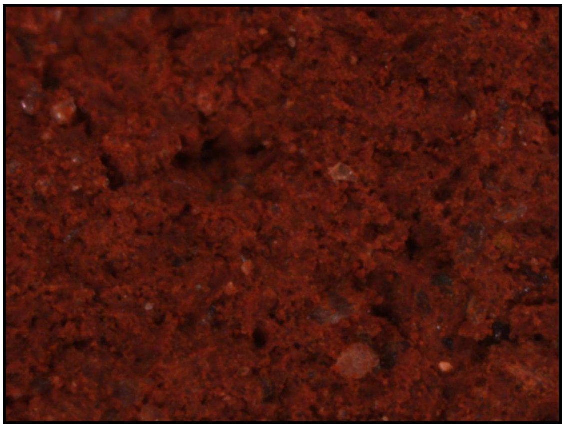


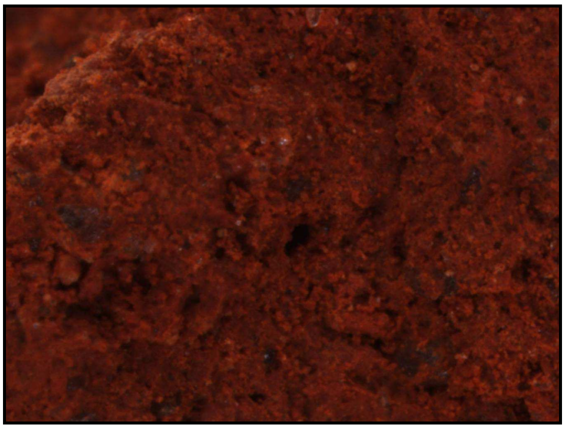


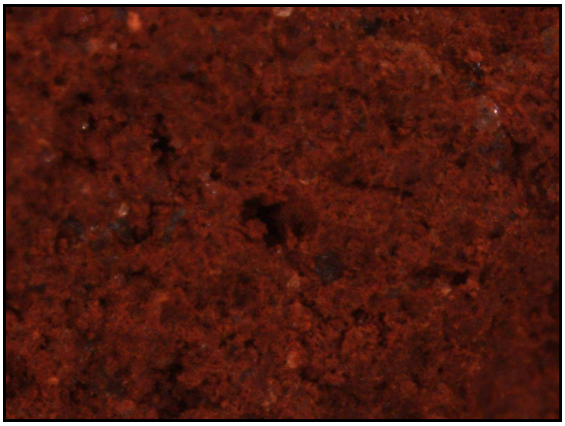

0.5 mm
0.5 mm

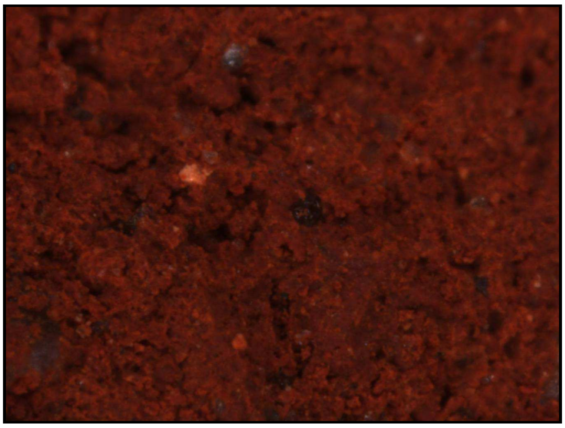
Tzaneen Consolidated Sample at X1 to X100 Magnitude

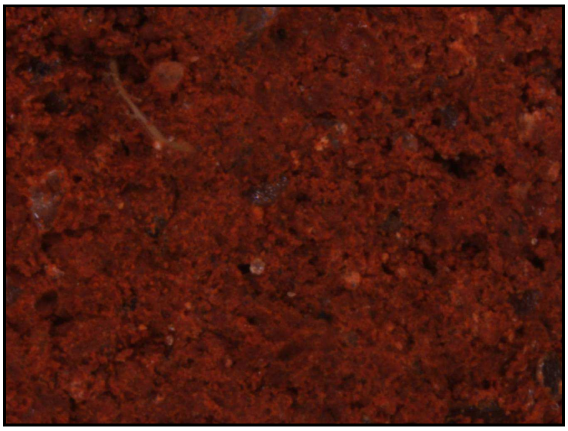


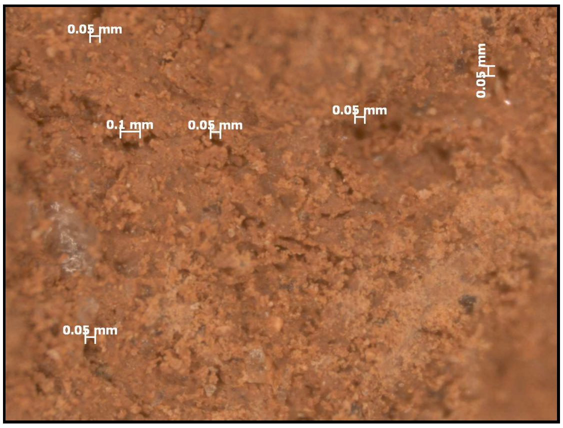


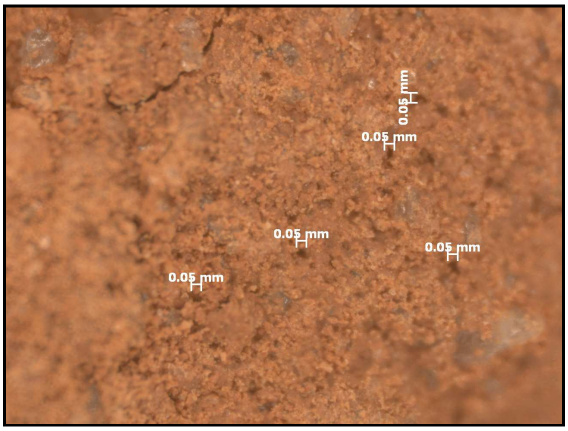


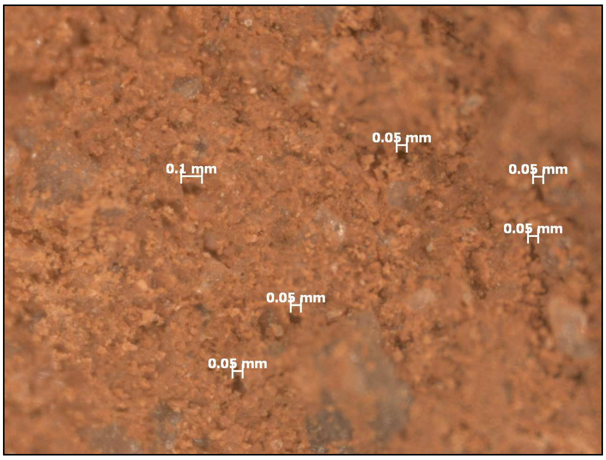







0.05 mm
0.05 mm
0.1 mm
0.05 mm
0.05 mm
0.05 mm


0.05 mm
0.05 mm
0.05 mm
0.05 mm
0.05 mm


0.05 mm
0.1 mm
0.05 mm
0.05 mm
0.05 mm
0.05 mm