EVALUATION OF THE SUITABILITY OF PROPOSED SITE FOR CONSTRUCTION OF PHOTOVOLTAIC SOLAR FACILITY AT KAKAMAS IN THE NORTHERN CAPE OF SOUTH AFRICA

Name: Tshilate Lindelani
Student Number: 11638548

Supervisor: Dr F. Amponsah-Dacosta

Co-Supervisors: Mr. Sphiwe Emmanuel Mhlongo
Mr. Confidence Muzerengi

A Dissertation submitted to the Department of Mining and Environmental Geology, University of Venda, in Fulfilment of the Requirements for the Master of Earth Sciences in Mining and Environmental Geology

February 2019

DECLARATION

I **Tshilate L,** hereby declare that this dissertation for Master of Earth Science in Mining and Environmental Geology, submitted at the University of Venda in the School of Environmental Sciences, Department of Mining and Environmental Geology, has not been previously submitted for any other degree at this or any other university. I further declare that this dissertation is my own work in designs and execution and all the reference material and cited work of others have been duly acknowledged.

....

Student's signature Date

....

Supervisor's signature Date

....

Co-Supervisor's signature Date

....

Co-Supervisor's signature Date

DEDICATION

In loving memory of my grandmother Nyamutshagole Mutshinyalo Davhula. You left finger prints of love in our lives. You shall never be forgotten.

To my Mom and Sister your love and support keeps me going. You have always been there for me in my time of need. Your selflessness is something I will cherish forever. Without you I would not be here.

AKNOWLEDGEMENTS

Without question my greatest thanks goes to my research supervisor, teacher and friend Dr. F Amponsah-Dacosta for taking me under his wing and steering me in the right direction that perpetuated success and brought more meaning and sense to this dissertation. His personality and sense of humor made working with him a pleasure.

I am also extremely grateful to my co-supervisors, Mr. Sphiwe Emmanuel Mhlongo and Mr. Confidence Muzerengi for being my field supervisors, helping me with data collection and for guiding me throughout this research. I recognize that I have been advised and educated by the some of the best in the world. The skills that you have taught me are invaluable and I will forever cherish them.

My sincere gratitude also goes to the North West university team, particularly Mr. PW Van der Venter, Cindy and Reggie for all their help in excellent supervision in the field, assistance with data collection and analysis of laboratory results. There is no way this research would have turned out like this if it weren’t for you guys.

I would like to thank my fellow mates and research assistance Reginald, Sigwa and Rudzani for their invaluable contributions and insights into the work. There is no way I would have been able to collect all that data by myself. I will always cherish the fun that we had together. I would also like to acknowledge Vintage Energy and DIT for helping fund this project, Sim lab, NWU lab and Univen lab for all their assistance with laboratory analysis of the samples.

I am sincerely thankful to my friend Tshitangano Mulamuleli for his moral support and help with the locality, magnetic and geological map of the study area.

Lastly, to family, who has given me unconditional support, happiness, selflessness and encouragement throughout this dissertation. I would never have made it this far without their love and for that I would like to thank them.

ABSTRACT

Solar energy development is experiencing significant growth due to national interest in increasing energy efficiency, reducing dependence on fossil fuels, increasing domestic energy production, and curbing greenhouse gas emissions. Northern Cape is generally known to be one of the preferred areas for the generation of solar energy in South Africa, and even in the world, due to its abundant solar radiation. Although this area has abundant potential for solar power generation, not all the areas are suitable for construction of solar plant facilities especially those that are prone to sand storm and dust accumulation. Consequently, site evaluation is very crucial for planning, design and construction of the solar facility. The main objective of this study was to determine the suitability of a proposed site at Kakamas in the Northern Cape for construction of a photovoltaic solar facility.

The specific objectives of this research were to assess and establish all the geotechnical aspects that may have an impact on the development of the site, to explore the surface conditions at the proposed site and to establish the soil properties and comment on the use of the on-site soils in the construction of the solar facility. Other specific objectives included to determine the variability of ground conditions and effects of such variability on the proposed development and to provide foundation recommendations for the design and construction of the solar facility. In order to obtain this information, methods such as desktop studies, geological survey, soil survey, magnetic survey and soil profiling were employed to obtain information about the geotechnical aspects of the study area and properties of the on-site soil. Field tests such as cone penetration and resistivity survey and laboratory tests such as foundation indicator test, California Bearing Ratio, pH and permeability test were also performed in order to determine the engineering, behavioral and hydraulic properties of the soil.

The results of the geologic and magnetic survey indicated that the study area is underlain by mainly igneous and metamorphic rocks such as gneiss, quartzite, pegmatite, gneiss and calcrete. The results of the soil profiling and the resistivity survey showed that the study area is comprised of sandy soil with either two or three horizons while the cone penetration results revealed high variable soil consistency and stiffness which ranged from very loose to very stiff soils. The particle size distribution, atterberg limits and grading modulus indicated that the study area is characterized mainly by dry, cohesionless and non-plastic to slightly plastic coarse-grained sandy soil with sand content ranging from 71- 96%. From the CBR results, it was found that the soils in the study area generally classifies as G6 material and can be used as base, sub base and backfilling material in accordance with the TRH 14 specifications.

The permeability test results indicated moderately permeable sandy silt soils with coefficient of permeability ranging between $1x10^{-3}$ to $8x10^{-3}$ cm/sec and ground water was encountered at 1.3 m depth. The material excavatability indicated variable material on site ranging from soft calcretes with soft excavation to highly competent material such as quartz and dorbank which require hard excavation while the side wall stability of trial pits indicated stable pit walls during the investigation giving an indication of stability of long pit excavations. The foundation analysis showed that driven piles and earth screws are the ideal foundation types for this site and that the site is generally suitable for construction of the solar facility provided all the recommendations are implemented.

Key words; Solar facility, renewable energy, desert, site investigation.

TABLE OF CONTENTS

Contents **Page**

LIST OF FIGURES

LIST OF TABLES

MEASUREMENT UNITS

cm	Centimeter
º	Degree
ºC	Degree Celsius
G	Gram
"	seconds
kg	kilogram
km	Kilometer
km^2	Kilometer squared
km/h	Kilometer per hour
M	Meter
mm	Millimeter
'	Minutes
%	Percentage
Ωm	Ohm meter

ABBREVIATIONS

AASHTO	American Association of State Highway and transportation Officials
ASTM	American Society for Testing and Material
CBR	California Bearing Ratio
CPV	Concentrating Photovoltaic
C_c	Coefficient of curvature
C_u	Coefficient of uniformity
DCP	Dynamic Cone Penetration
E	East
GM	Grading Modulus
GPS	Global Positioning System
IFC	International Finance Corporation
LL	Liquid Limit
MCCSSO	Moisture, Colour, Consistency, Structure, Soil type and Origin
MDD	Maximum Dry Density
PI	Plasticity Index
PL	Plastic Limit
S	South
SANS	South African National Status
TRH	Technical Recommendations for highways
USCS	Unified soil classification system
VES	Vertical electrical sounding

CHAPTER ONE: INTRODUCTION

1.1. Background of the Study

Kakamas is a semi-arid town in the Northern Cape that was founded by Dutch in 1897. This town is located just outside the Namib Desert, approximately 100 km from the Koa dunefield and was originally founded as a grazing site (Almond, 2012). However due to availability of open space, people began inhabiting this town by building houses, churches and schools, much to the engineers dismay. Ignoring the criticism of qualified engineers, these people continued to build water canals for irrigation and to supply the town and in 1912, they built a hydro-electric power station which still supplies electricity even today even though it was built without any proper site investigation (Almond, 2009). Over the years the town has grown greatly and with increasing technological advancements, solar facilities have been growing in this area.

Site investigation is a complex scientific process that is mandatory to any construction project and determines the layers of natural soil deposits that forms the foundation and their physical properties (Youssef, 2015). Civil engineering structures like buildings, solar facilities, bridges and dams are founded below or on the surface of the earth. Therefore, a suitable foundation soil is required for their stability. To establish the suitability of soil to be used as foundation or as construction materials, its properties are required to be assessed (Laska and Pal, 2012). The suitability of a site for construction purposes is therefore determined by the physical, chemical and the engineering/geotechnical properties of the soil (Kong et al., 2011). This coupled with the physiological characteristics (climate, geology, topography, hydrology and vegetation) were the factors identified by Greenwood et al. (2006) as the determining attributes of site suitability. According to Nwankwoala et al. (2014), assessment of geotechnical properties of subsoil at project site is necessary for generating relevant input data for design and construction of foundations for the proposed structures. Similarly, Nwankwoala et al. (2013) stated that proper design and construction of civil engineering structures prevent an adverse environmental impact or structural failure or post construction problems.

Soil is the most abandoned locally available construction material all over the world and together with rocks serve as foundations for structures (SAICE, 2010). However, their inherent variability results in many construction challenges such as large construction cost overruns as well as failures during or after construction resulting in damage to property and the consequential damages or even loss of life (Middleton, 2016). According to Pease et al. (2015), desert soils are not only variable from place to place, they also tend to be very poorly developed and are very thin, patchy and shallow. Zhao et al., (2017) attributed this to the harsh dusts storms which are very prevalent in these areas and the little weathering and leaching which normally yields coarse-grained soils, shallow profiles and high concentration of carbonates. However, Zu et al. (2017) argued that these soils are relatively stable and can be able to support small structures without any reinforcement, provided the subsurface conditions are good.

The subsurface condition plays an interdisciplinary role in site investigation as it includes both geophysical and hydraulic properties of the soil. Deserts or arid areas however, are characterized by very complex subsurface which are occupied by various structures and features (Grosslova et al., 2018). Van der Walls (2011) identified this in his soil study for the Proposed INCA solar power at Kakamas which revealed that the soils in this area are very jointed and comprises of several other structures such as faults and dykes and that these structures need to be thoroughly investigated prior to any construction process. This is because these subsurface conditions coupled with the soil conditions and properties determined the type of foundation needed to support the structure prior to building (Brealey et al., 2015). Similarly, Roy et al. (2017) stated that information about the surface and sub-surface features is essential for the design of structures and for planning construction techniques.

There are several types of foundations that can be used for solar plants. These include cast in situ concrete piles, drive pile foundations, rammed pile as well as concrete block foundations (Fellenius, 2009). In the history of construction, structures made of drive in or concrete foundations, non-expansive, stiff, coarse-grained, neutral pH are among the oldest and the most enduring structures/buildings (The South African National Road Agency, 2014). However, Brearly et al. (2015) argued that amongst all the foundations that can be used to support buildings, a well anchored and mounted foundation possesses properties that ensure a strong and durable structure.

Kakamas area is characterized by several solar plants such as Keren Solar Plant, PIA Solar, INCA Kakamas Photovoltaic Solar Facility and Scuitdrift Solar Project (Almond, 2012). This is because this area is characterized by extremely high temperatures that could easily be harnessed into solar energy and has a lot of open areas (Solek, 2002). Regardless the number of the solar plants in this area, poor understanding of proper site investigation, geotechnical attributes, surface exploration aspects of site investigation, subsurface exploration and the use of the on-site soils is still lacking because all the other solar plants were built based only on the information of geotechnical reports and not thorough investigation like this one. A comprehensive study that investigates all the aspects of the site is of absolute necessity because it evaluates not only the geotechnical aspect of the site such as geology, topography and soil, but also looks at the geophysical and hydraulic properties of an area which aid in better understanding of the properties of the site.

The main aim on this research was to investigate the suitability of a proposed site for construction of a photovoltaic solar facility. Consequently, the study focused on providing detailed information on site investigation, construction of a PV solar facility as well as large-scale ground-mounted PV power plants is of absolute necessity. Thus, the knowledge of these processes has both educational value and immediate commercial significance. These include bridging the knowledge gap on site investigation for PV solar facilities, arid areas soils and large-scale ground-mounted power plants. In addition, they help create employment for the local inhabitants which in turn would boost the local economy. For this reason, site investigation comprising of desktop studies, reconnaissance, field tests, soil profiling and sampling as well as laboratory analysis of the collected soil samples were conducted in order to determine the physical and engineering properties of the soil which in turn would help determine the suitability of the proposed site for construction of PV solar facility.

1.2. Problem Statement

Arid areas are ideal for solar plant construction because they have the necessary high temperatures that could be harnessed into producing solar energy (Fernandez, 2010). However, these areas tend to be very unstable. This can be attributed to certain conditions/factors such as weather and geology that are prominent in these areas (Middleton, 2017). For example, Kakamas falls within the Namaqua-natal metamorphic province which is highly dominated by calcretes and gypcretes which are very weak carbonate rocks and tend to dissolve in water (Almond, 2012). According to Middleton (2017), arid areas are predominated by windy conditions which are the principal source of dust storms which normally occur in these areas and results in the accumulation of dust particles on the surface of solar panels thereby reducing and degrading their performance.

The soil in these areas is also variable and is characterized by many different structures such as joints, faults and dykes and gives information about the subsurface conditions (Pease et al., 2015). These together with hydrology of the areas tend to have an effect of the suitability of the soil for construction (Greenwood et al., 2006). This means that these soils are characterized by different properties and as such will have different uses. According to Leurox et al. 2013, arid and semi-arid soils, particularly South African desert soils are so complex that some have not yet been discovered and are yet to be studied and classified. Therefore, proper exploration and thorough analysis of these soils needs to be conducted prior to any use.

The subsurface geometry of these areas also tends to be very complex and unpredictable and could have detrimental impacts on the stability and suitability of these areas for construction. Kaseke et al. (2018) identified this on his study on the impact of fog on soil moisture dynamics in the Namib Desert which indicated that soils next to the water table were moister than those that were further away from the water table. Consequently, this study aims to conduct a site investigations in order to obtain information about the engineering and behavioral properties of the soils at the site such as strength, consistency, stiffness and plasticity. This is because failure to properly establish these properties could results in improper use of the material on site which would ultimately results failure and collapse of buildings and structures overtime. In addition, it would also results in improper foundation recommendations which will eventually results in failure of structures.

1.3. Research Objectives

The main objective of this research was to investigate the suitability of a proposed site for construction of a solar facility at Kakamas area in the Northern Cape. The specific objectives were:

- To assess and establish all the geotechnical aspects that may have an impact on the development of the site,
- To explore the subsurface conditions at the proposed site,
- To establish the soil properties and comment on the use of the on-site soils in the construction of the solar facility,
- To determine the variability of ground conditions and the effect of such variability on the proposed development, and
- To provide foundation recommendations for the design and construction of the solar facility.

1.4. Research Questions

Base on the research objectives, the following research questions were used to guide this research:

- What are the geotechnical attributes of the proposed site? What are their impacts on the proposed development?
- What structures are present in the soil? How can they be identified, and their impact assessed?
- What are the properties and the potential use of the on-site soil?
- How does the soil condition vary from location to location? What impact does this have on the proposed development?
- Which foundation recommendations can be made for the proposed solar facility?

1.5. Research Justification

Kakamas is a host to several solar plants. These solar plants are located in different regions and as such are characterized by different features and properties. However, many people, certainly ordinary or local people do not understand the geotechnical and engineering work required for provision of solar energy. Lack of adequate site investigation would result in selection of inappropriate material for construction and foundation structures which in turn would results in failure or collapse of structures over time. According to Brearly et al. (2015), the basic components for a quality solar facility is adequate geotechnical investigation which include site research, soil investigation and load testing which in turn would lead to a site-optimized foundation design and strong reliable structures.

This research was motivated by the need to have a better understanding of all the geotechnical and engineering work that needs to be carried out to establish the suitability of Brypaal farm for construction of a solar plant. This include identification of the existing surface conditions, mapping of the subsurface features, soil profiling, field strength test as well as soil sampling and laboratory analysis to determine the physical and the engineering properties of the soil and giving foundation recommendations based on the findings. The results of this study will give a clear indication of how suitable the study area is for the construction of the solar plant and also outline optimal foundation best suited to allow erection of a strong, reliable and sustainable building. This information will play a key role in the site suitability studies and site selection of future solar projects in the area.

This research is also of benefit to construction companies as building on a strong and stable ground with high quality material reduces the disasters of cracking and falling structures. Both the properties of soil and foundation play a significant role in producing strong, reliable and lasting structures. For this reason, this research will also look at the processes such as cone penetration, compaction and the California Bearing Ratio involved in determining the physical and engineering properties of the soil at the site.

CHAPTER TWO: LITERATURE REVIEW

The aim of this chapter was to review the information relevant to the study in order to gain an understanding on the suitability of the proposed site. Consequently, it provides a general idea of previous work done by other researchers on energy demand and supply, generation of solar energy, development of solar facilities and the importance of geotechnical investigation for solar facility. In addition, it also gives a general description of the project and the study area, processes used to determine engineering properties of the soil and solar-specific foundation designs so as to identify the knowledge gaps in these works and also to provide a benchmark for comparing findings.

2.1. Energy Supply and Demand

The demand for the provision of energy is increasing worldwide and will continue to rise due to rapidly rising human population and modernization trends across the world (Fernandez et al., 2010). EIA (2015) revealed this in their study of energy demand and supply which indicated that the human demand for energy has been continuously rising with the evolution of civilization throughout the course of history and the growth is projected to rise sharply over the coming years. However, the supply of this energy has been decreasing rapidly due to increase of human population which depletion of fossil fuels people have been using to supply people with energy (IFC, 2015).

According to Muneer (2007), the world heavily relies on fossil fuels to meet its energy requirements and that fossil fuels such as oil, gas and coal provide almost 80% of the global energy demands while renewable energy such as solar and nuclear power only contribute 13.5% and 6.5% respectively of the total energy needs. Lately in South Africa, the electricity price has been increasing exponentially and other people still do not have electricity in their homes. Eskom has even introduced the issue of load shedding in an attempt to meet the electricity demand of the growing population and to also control the distribution of electricity. In a recent statement, the Department of Environmental Affairs stated that energy reserves are almost depleted and that a drastic transition in sources of energy form non-renewable to renewable energy would have to be made in order to meet the growing energy needs of the 21st Century not only in South Africa, but all over the world (DEA, 2018).

2.2. Challenges of Reliance on Fossil Fuels

The enormous amount of energy being consumed across the world is having adverse implications on the ecosystem of the planet. Fossil fuels being the main source of energy, are inflicting enormous impacts on the environment. Climatic changes driven by human activities, in particular the production of Greenhouse Gas emissions (GHG), directly impact the environment (IEA, 2015). According to World Health Organization (2003), as many as 160 000 people die each year from the side-effects of climate change and the numbers could almost double by 2020. Continuation of the use of fossil fuels is set to face multiple challenges such as depletion of fossil fuel reserves, global warming, geopolitical and military conflicts and of late, continued and significant fuel price rise, which all indicate an unsustainable situation (Muneer, 2007). The side effects range from malaria to malnutrition and diarrhea that follow in the wake of floods, droughts and warmer temperatures.

Presently employed energy systems will be unable to cope with future energy requirements because fossil fuel reserves are depleting, and predominantly the developed countries employ nuclear power (Fernandez et al., 2010). Fossil fuel and nuclear energy production and consumption are closely linked to environmental degradation that threatens human health and quality of life and affects ecological balance and biological diversity. It is therefore clear that if the rapidly increasing global energy needs are to be met without irreparable environmental damage, there will have to be a worldwide drive to exploit energy systems that should not endanger the life of current and future generations and should not exceed the carrying capacity of ecosystems (Muneer, 2007).

2.3. Alternative Sources of Energy

The adverse impacts of using fossil fuels to produce electricity has forced the world to look into other clean and non-harmful sources of energy. These clean and non-harmful sources of energy are called renewable energy sources and include wind, solar, biomass and geothermal energy. Renewable energy projects all over the world are on the rise and technologies that received limited attention in the 1970s, such as solar and wind power, are experiencing significant growth today due to the perception of national interest to increase energy efficiency, reduce dependence on fossil fuels, increase domestic energy production, and curb greenhouse gas emissions (Fernandez at al., 2010). This perception of national interest has been made evident by the range of new policies and incentives that spur renewable energy research and development.

Renewable energy sources that use indigenous resources have the potential to provide energy services with almost no emissions of both air pollutants and greenhouse gases (Muneer, 2007). Renewable energy sources such as solar energy, wind power, biomass and geothermal energy are abundant, inexhaustible and widely available. These resources have the capacity to meet the present and future energy demands of the world. According to IEA (2015), the development and use of renewable energy sources can enhance diversity in energy supply markets, contribute to securing long-term sustainable energy supplies, help reduce local and global environmental impacts and provide commercially attractive options to meet specific energy service needs, particularly in developing countries and rural areas, creating new employment opportunities. Consequently, the cost of energy generated from these renewable resources is significantly coming down while the cost of fossil fuel produced energy is in an increasing mode.

2.4. Solar Energy as Promising Renewable Source of Energy

Over the last two decades solar and wind energy systems have experienced rapid growth. This is because of several factors such as declining capital cost; declining cost of electricity generated and continued improvement in performance characteristics of these systems (IEA, 2015). Solar power however, is the cleanest, most reliable form of renewable energy and is arguably available and can be used in several forms to help power homes or businesses (Muneer, 2007). Consequently, solar energy development is experiencing significant growth. As stated by Fernandez et al. (2010), this is due to national interest in increasing energy efficiency, reducing dependence on fossil fuels, increasing domestic energy production, and curbing greenhouse gas emissions. In addition, the environmental cost and the economic and policy mechanisms needed to support its widespread dissemination and sustainable markets are rapidly evolving. According to IEA (2015), solar energy is one of the most promising renewable energy sources and a photovoltaic cell (PV) is the representative technology for utilizing solar energy and is not an exaggeration to say that we are now coming to the stage of energy transition by PV power plants.

2.5. Process of Generating Solar Energy

Solar panels, also known as modules, contain photovoltaic cells made from silicon that transforms the incoming sunlight into electricity. These solar photovoltaic cells consist of a positive and negative film of silicon placed under a thin slice of glass (IFC, 2015). The negatively charged free electrons are preferentially attracted to one side of silicon cell, which creates an electric voltage that can be collected and channeled. This current is gathered by wiring the individual solar panels together into a series to form a solar photovoltaic array. Depending on the size on the installation, multiple strings of solar photovoltaic array cables terminate in one electrical box called a fuse array combiner. Contained within the combiner box are fuses designed to protect the individual module cables, as well as the connections that deliver power to the inverter. The electricity produced at this stage is DC and is converter to AC which is suitable for use at homes (Muneer, 2007). Figure 2.1 illustrates this generation of electricity from solar panels.

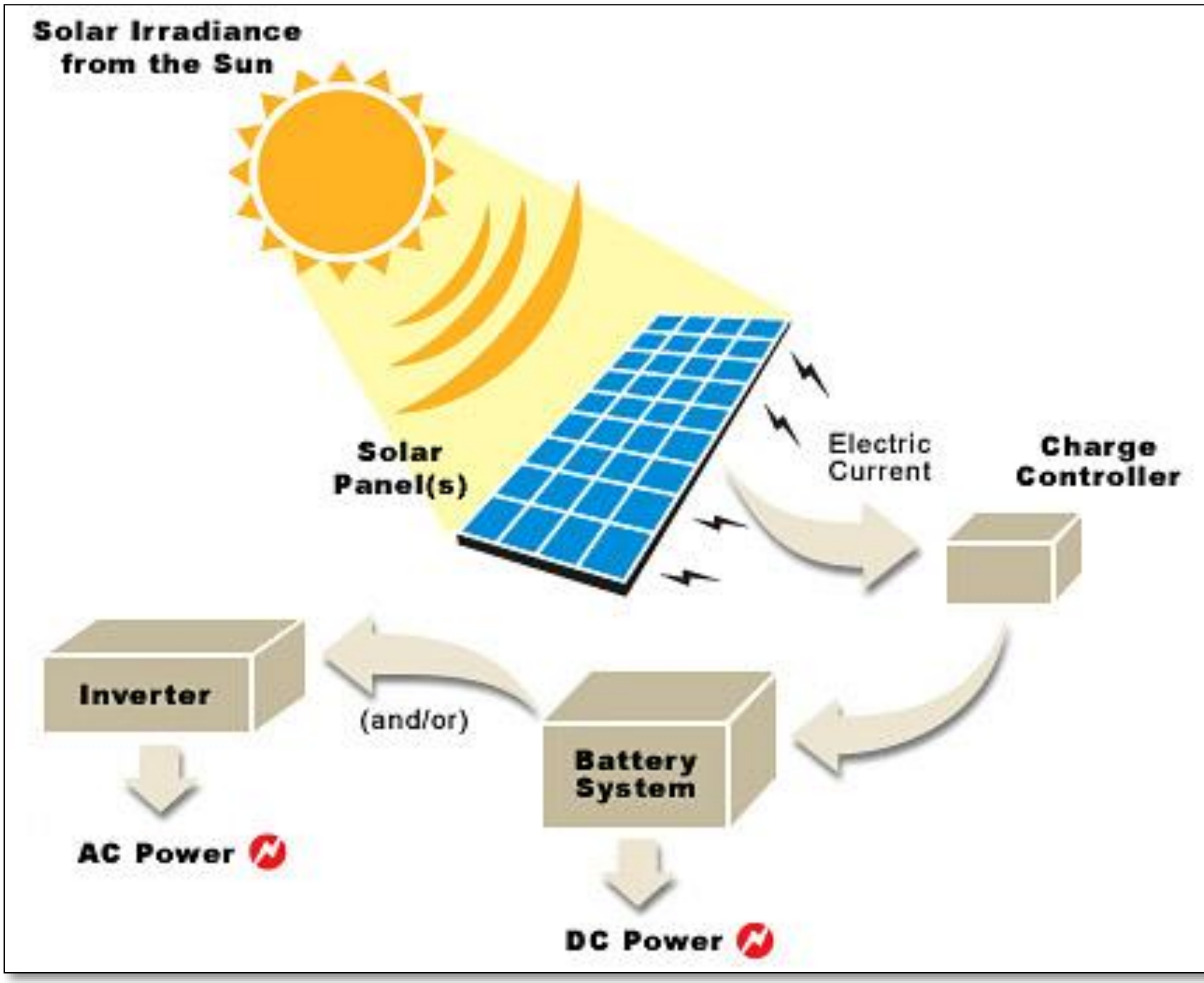


Figure 2.1: Generation of electricity from solar panels (IFC, 2015).

2.6. Development of a Solar Facility

Developing a PV solar facility is a process involving many stages and requires a multidisciplinary team of experts. According to IFC (2015), development of a solar facility as shown in Figure 2.2 is a very complex process which involves planning, site investigation, construction and operation. Project development activities are interrelated and often are carried out in parallel. As stated by Humes et al. (2012), it begins by identifying a power market that offers adequate risk-reward opportunities, then identifying a promising site and secures the land-use rights for this site. Following this is two separate rounds of technical-financial assessments (prefeasibility study and feasibility study) which are followed by obtaining all required permits and licenses, securing power purchase and interconnection agreements, arrangement of financing, and selection of teams to design and construct the solar facility (IEA, 2015).

Figure 2.2: A typical solar facility (IFC, 2015).

As the project moves from one stage to the next, the technical-financial assessments become more detailed until a final design is developed, and construction starts. According to Humes et al. (2012), it is important to emphasize the back-to-back nature of many project contracts and documents, however, this must be preceded by a grid connection agreement, construction and site access permits and land lease agreement. Throughout this process, technical, commercial, and legal/regulatory experts are involved, working in parallel on distinct yet interdependent activities. While clear responsibilities can be identified for each expert, most project activities are related and the work of one expert influences the work of other experts, hence close coordination is needed and should be emphasized throughout the project (IEA, 2015).

The key steps for developing a solar PV project are well establish, but there is no definitive detailed “road map” a developer can follow since the approach taken in each project entirely dependent on site-specific parameters and the developer’s priorities, risk appetite, regulatory requirements, and the types of financing support mechanisms available in a given market (IEA, 2015). However, in all cases, certain activities need to be completed that can broadly be organized in five stages, namely; concept development and site identification, prefeasibility study, feasibility study, permitting, financing and contracts and engineering, construction and commercial operation (IFC, 2015).

The concept development stage includes identification of the investment opportunity at a specific site and the formulation of a strategy for project development (Humes et al., 2012). It is assumed at this stage that a target market has been identified and the project developer understands any special prerequisites for investing in that specific country and power sector. These market-level decisions require a detailed assessment that carefully considers the risk–reward appetite of the project developer and potential investors. According to IEA (2015), a desirable site has favorable local climate, good solar resource, land available for purchasing or long-term leasing, an accessible grid connection or a binding regulatory commitment to connect the site to the transmission network, and no serious environmental or social concerns associated with the development of a PV project. Although many countries require that the site be part of a list pre-approved by the government, this needs to be confirmed at the outset of the site identification process.

The prefeasibility study involves assessment of different technical options, approximate cost/benefits, permitting needs and market assessment while feasibility study involves technical and financial evaluation of preferred option, assessment of financial options, initiation of permitting process and development of draft technical concept (IFC, 2015). Financing and contracts as stated by Humes et al. (2012) includes permitting, contacting strategy, supplier selection and contract negotiation and financing of the project and detailed designs include preparation of detailed design for all relevant lots, preparation of project implementation schedule and finalization of permitting process. According to (IEA, 2015), construction which involves construction supervision and commissioning which involves performance testing and preparation of as build design by an independent person.

2.7. Need and Importance of Geotechnical Investigation

A geotechnical investigation of the site is recommended prior to final selection. This is because it gives information about the geotechnical attributes such as geology, climate, topography and ground water and their impacts on the proposed development as well as investigate the physical and behavioral properties of the soil on which the construction is to take place which in turn aids optimization of foundation (Brearly et al., 2015). According to IEA (2015), the purpose of geotechnical investigation is to assess the ground conditions in order to inform the foundation design approach and right of way to ensure that the mounting structures will have adequately designed foundations. The level of detail required in the geotechnical survey will depend on the proposed foundation design.

In order to obtain this information, best practice dictates that either boreholes or trial pits be made at regular intervals, along with soil sampling and in-situ testing, at a depth (usually around 2.5m to 3m below ground level) appropriate for the foundation design in order to assess the groundwater level, the resistivity of the soil, load-bearing properties of the soil, presence of rocks or other obstructions and the suitability of chosen foundation types and drivability of piled foundations (IEA, 2015). According to Brearly et al. (2015), quality geotechnical data are key to designing a reliable and cost-effective foundation and understanding of geotechnical attributes of the study area enables prediction of possible geotechnical problems. For example, a geotechnical study of the Medupi Power Station in Lephalale enabled Eskom to identify subsurface soil anomalies, contact zones between the coal and the soil and potential zones of weaknesses simply by identifying and understanding the geotechnical attributes of the area (Inroad Consulting, 2009). This also enabled the establishment of possible solutions to some of the problems way long before they could start with operations. In addition, geotechnical investigation also allows for the establishment of the soil’s stability, potential volumetric change and the permeability of both soils and rock, as such their understanding is of paramount importance for any construction purposes (City Development Planning and Environmental Directorate, 2016).

2.8. Description of Project and the Study Area

Construction of solar facilities requires knowledge and understanding of the geotechnical aspects of the study area (Greenwood, 2006). Such aspects include size and type of the project, geology, climate, topography, vegetation, soils and ground geophysics. This section provides insight into the project, location, topography, climate, drainage pattern, pedology land use as well as the regional geology of the study area.

2.8.1. Description of the Project

The proposed solar plant is to be constructed at Brypaal farm located at Kakamas, Kai Garib Municipality in the Northern Cape and would cover an area of about 400 ha. This facility would comprise about 1400 Concentrating Photovoltaic solar panels which would generate about 100MW electricity which will be fed into the existing Escom national grid (Boscia Environmental Solutions, 2017). The CPV panels will be mounted on pedestals drilled and set into the ground. Extensive bedrock excavations are not envisaged, but some vegetation will need to be cleared from the site. Associated infrastructure includes a transformer unit, solar resource measuring station, access roads (temporary and permanent roads, 4 -6 m wide) and a temporary laydown area which includes workshops, mobile offices, mobile ablution facilities, material storage area, vehicle parking area, water tanks and a fence. The farm is currently zoned for grazing.

2.8.2. Geographical location

The proposed area of the solar plant is situated on an arid, gravelly terrain in the vicinity of Kakamas town. According to Boscia Environmental Solutions (2017), the site is located 53.23 km South of Kakamas town at 29º11'48.91" S, 20º23'19.44" E coordinates. The locality map and the proposed development areas of the solar plant are shown in Figure 2.3.

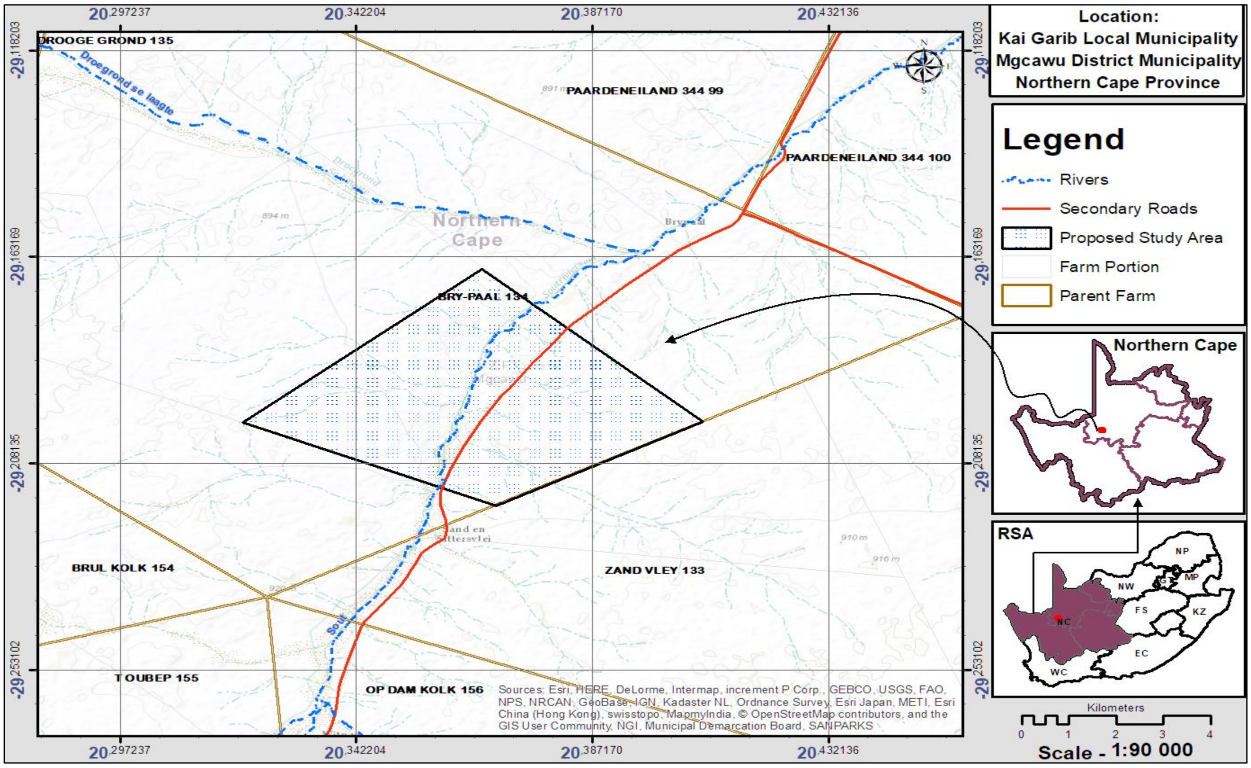


Figure 2.3: Locality map of the area for development of the solar plant.

2.8.3. Climatic conditions

The site lies within the dry, arid Nama-Karoo climatic conditions, comprising of very hot summers and very cold winters (Dold et al., 2006). This climate is largely continental and temperatures are subject to great variation both seasonal and diurnal (Almond, 2012). Summers are long, and winters are cold with maximum average temperatures ranging between 46ºC and 32ºC in January, with the corresponding average minimum temperatures of 17ºC and -10ºC in June respectively (Boscia Environmental Solutions, 2017). The annual precipitation of this area normally ranges between 80mm and120mm, with highest precipitation (61mm) falling in summer, particularly in February and least precipitation (0mm) in winter between June and July (Dold et al., 2006).

The rainfall and other precipitation have no distinct peak month and evaporation levels exceed the annual rainfall (Almond, 2012). The land area is not cultivated and most of the natural vegetation is still intact and frost is common on the interior plateau in winter (Boscia Environmental Solutions, 2017). The climate is classified as a subtropical desert (low latitude desert), with a subtropical desert scrub biozone and September is on average the month with most sunshine (Pulmit Mining, 2003).

Climate determines the mode weathering and the rate of weathering. This effect of climate on the weathering process (i.e. soil formation) is almost entirely determined by the climatic N value defined by Weinert (Pulmit Mining, 2003). The N-value for the area is 12, which implies a deficiency of water. The general implication of this is that the soil profile is likely to be shallow, and comprise physically disintegrated residual soils (SMEC Consulting, 2013).

2.8.4. Regional geology

The study area is underlain by the metallogenically significant granites and the gneisses of the Namaqua-natal metamorphic province of the Mid Proterozoic (Mokolian) age (Almond, 2012). These basement rocks are estimated to be between 1and 2 billion years and do not contain any fossils and are mantled with a spectrum of coarse to fine grained superficial deposits such as rocky soils, downwashed gravels and colluvium (Englington, 2006). The alluvial gravels come from the Orange River and are of Mio-cene and Younger age. The geology of this complex is dominated by Mokolian gneisses such as those of the Hartbees complex and the Younger Eendoorn Suite (Minaar, 2006). Schist and quartzite of the Bushmanland Group are also significant in these areas. There is also quartz, calcrete and dorbanks, at many places of the study area (Almond, 2009).

The Namaqua metamorphic province occupies an area of about 80000 km^2 and was deformed and metamorphosed during the Namaquan orogeny and is endowed with various deposits such as volcanogenic massive sulphides, Broken hill type, Magmatic Ni-Cu (Honderkloof), Intrusive Cu (Okiep), Cu-Mo porphyries (Haib), Fe-Oxide hosted Cu&Au, Vein hosted rare earth elements (Th-Cu-Fe), base precious metals, vein alluvial, marine diamonds, varies industrial minerals and uraniferous granites (Mossom et al,. 2006). The Namaqua-Natal metamorphic province is divided into 3 subprovinces namely, the Richtersveld Bushmanland, Gordonia and khei subprovinces, of which our site falls under the Bushmanland subgroup (Sithole, 2013).

The bushmanland terrain is located east-west of Kakamas terrain and is characterized by numerous fault systems and shear zones which are used to delineate the tectostratigraphic boundaries with the other subprovinces (Shunqukela, 2014). It is bound on the east by the Hartbees River thrust and consists of intensely deformed high-grade gneisses, granulites and granutoids (Boelema, 1994). These rocks are overprinted by the west coast belt, the Vanrhysdrorp Group and the Karoo super group of the Kaapval craton (Cornell et al., 2006). The most distinctive and economically important lithological assemblage on this subprovince is the Quartzite-aluminous schist which is wide spread throughout this terrain (Almond, 2012).

The study area is also host to the underground Salt River which is host to several deposits of economic importance such as the Geelvloer uranium-gypsum deposits and several Volcanic Massive Sulphides (McCung, 2008). This river is located in the Supracrustal rocks of the Garies terrain and is hosted by the Geelvloer sequences and the Kenhardt subgroup of the bushmanland group, which are characterized by immature Metasedimentary rocks (Biotite schist/gneiss, para-amphibolite/calc-silicaterocks, feldspathic quartzites and cordierite gneisses) (Boelema, 1994). All the rocks in this terrain are dated 1700-1600ma ago and are associated with the regional metamorphism that occurred 1200 ma years ago (Englington, 2006).

2.8.5. Topography and drainage pattern

The study area is situated on a flat plain sloping from east to west (Boscia Environmental Solutions, 2017). The Kamiesberg mountain range forms an escarpment from Garies in the southeast to Springbok in the northeast and elevation above sea level ranges from about 250 m to 750 m in this area (Sithole, 2013). This mountain functions as an important rain catchment area and is characterized by granite and gneiss rock formations and steep rocky slopes that are separated by sandy plains and lowland areas (Chidley et al., 2011). The lowest lying areas of the district are situated along the coastal plain belt and these areas are relatively narrow (Almond, 2012).

The area of the proposed solar plant is situated in a region that is drained by the perennial Orange River (Minaar, 2006). The majority of the rivers in these areas are non-perennial, dendritic, shallow and all intermittently drain northward into the Orange River during rainy seasons (Almond, 2012). The site is also host to an underground Salt River which is a tributary of the Hartbees river (McCung, 2008). Ground water levels on site are in the order of 1450masl and drainage is generally in the form of sheet wash. The Orange River flows through a wide, flat, cultivated valley to form the famous Augrabies fall which form part of the heritage of Kakamas town (Osbun, 2011). From the 56 m high falls it flows into a deep, 100 m wide gorge which extends 18 km and the main incision of the peneplain to form the Orange River gorge and the evolution of the Augrabies Falls are associated with the continental uplift of the late Tertiary (Lange, 2006).

The dramatic increases in the flow rate of water experienced from time to time are not the result of local climatic conditions but originate upstream (Almond et al., 2009). Especially high-water flow rates were experienced in 1988 (7,800 m^3/s - the highest in recent time), 2006 (1,983m^3/s), February 2010 (2, 936 m^3/s), January 2011 (4, 779 m^3/s) and after it subsided somewhat it increased again to 4, 102 m^3/s on 2 February 2011(Lange, 2006). The highest water flow rate after February 2011 was on 18 June 2011 at 1, 237 m^3/s with the average being roughly 50 to 70 m^3/s (Almond, 2012).

2.8.6. Effects of site conditions on suitability, construction and performance of the solar facility

Site attributes refers the geology, topography, hydrology, climate, vegetation and seismology of the study area (Greenwood et al., 2006). Of all these factors and attributes the climate of the study area is of paramount importance because it has an overbearing influence on all the other factors such as the geology, the resultant soil as well as the hydrology of the study area (Lucian, 2006).

The climate of the areas determines the type of weather that occurs within that specific area and therefore also influence the type and the amount of precipitation in that area which in turn influences the whole hydrology and water systems of the area. The climate also plays a vital role in influencing the type of weathering that occur in the rocks within the area, thereby influencing the rock and soil formation in the area since soil is a by-product of weathered rocks. For example, areas that are characterized by extremely hot days and extremely cold nights are predominated by exfoliation due to thermal expansion during the day and contraction at night.

Climate in turn is the primary determinant of key mechanical and chemical weathering processes and the evolution of hillslopes and thus regolith and soil formation (Pease et al., 2015). According to Zhang et al. (2017), arid and semi-arid areas are characterized by hot and dry climate, consequently the formation of soil is slow and clay and organic matter contents are often low. Therefore, the soils have a naturally loose structure, a shortage of aggregates and are prone to wind erosion, especially under tillage. Recently Lia et al., (2018) revealed that the environment in arid and semi-arid areas is characterized by a series of harsh conditions such as poor soils, extreme drought, high salinity, pH, radiation, large temperature variation and are accustomed to wind and sand storms.

The geology of the site is also of paramount importance when evaluating the suitability of an area for construction. According to SAICE (2010), the site is primarily characterized by the regional geology and geomorphology of the terrain within which it occurs. This together with the climate were identified by Greenwood et al. (2006) as factors that influence soil formation and the hydrology of an area. For example, soils are by-product of weathered rocks and the soil type in turn determines the movement of water in it and also the amount of water stored in the soil.

Generally arid areas are characterized by carbonate rocks such as calcretes, ferricretes and gypcretes (Wicander and Monroe, 2002). Al-Khafaji et al. (2013) also stated that gypseous rocks and soils exist mainly in arid and semi- arid region. Gypseous soils are one of the most complex materials that challenge the geotechnical engineers. Consequently, they are classified as one of the problematic soils due to their complex and unpredictable behavior (Paige, 2002). In Iraq it has been reported that several structures have experienced different patterns of cracks and uneven deformations generated primarily from the exposition of the supporting gypseous soils to water (Khafaji et al., 2013). Brink et al. (2002) referred to these rocks as “indicators” in the South African guidelines for soil and rock profiling and that once encountered, they have to be removed and the soil backfilled before any use. In addition, Middleton et al. (2017) revealed that rocks in these areas are highly weathered, foliated and faulted due to thermal expansion and cooling and results in surfaces that are highly unstable and not suitable for construction without any proper reinforcement. Despite this, Pease et al. (2015) argued that some arid and semi-arid areas are characterized by very competent rock such as quartz and quartzite which results in the formation of very competent soils that are stable and good for construction. Similarly, Almond (2009) revealed that structures (the hydropower station and water canals) built in 1912 in this area are still standing strong regardless the geology and the soil condition.

The hydrology of the area is also very important because it influence the moisture content of the soil. According to Du et al. (2018), desert soils are extremely dry because precipitation in these areas is very rare or little. In addition, streams are mainly dry and non-perennial and exhibit a complex composition of both single threaded and multi-headed channels which do not exhibit consistent channel adjustment relative to hydraulic inputs (Hamada et al., 2016). The lack of precipitation or well drained streams together with extremely high temperatures in these areas results in soils which are very dry, have loose structure, no cohesion, uniform particle size making it very difficult to compact (Du et al., 2018). Consequently, it must be strengthened before use as foundation for roadbeds and other buildings.

Other factors that are of paramount importance during site investigations are the subsurface features and soil profile since they provide valuable information about the subsurface of the study area (IAEA, 2004). This is because these factors reveal information about all the subsurface features and geological structures present in the ground, hence it is imperative that all these factors be clearly identified and fully comprehended for successful construction to take place. According to Brearly et al (2013), understanding of geotechnical attributes of the study area enables prediction of possible geotechnical problems.

For example, a geotechnical study of the Medupi Power station in Lephalale enabled Eskom to identify subsurface soil anomalies, contact zones between the coal and the soil and potential zones of weaknesses simply by identifying and understanding the geotechnical attributes of the area (Inroad Consulting, 2009). This also enabled the establishment of possible solutions to some of the problems way long before they could start with operations. In addition, geotechnical attributes also allow for the establishment of the soil's stability, potential volumetric change and the permeability of both soils and rock, as such their understanding is of paramount importance for any construction purposes.

2.9. Influence of Site-specific Soil Characteristics on the Project

The soils in the study area are reddish, moderately shallow, sandy and often overlay layers of calcrete of varying depth and thickness (Boscia Environmental Solutions, 2017). These soils are typically weakly structured with low organic content and changes to alluvial and aeolian sand near the river (Almond, 2012). According to Brink et al. (2002), aeolian soils have collapsible fabric, are very mobile and have poor compaction characteristics while alluvium soils also have collapsible fabric, are dispersive characteristic, incompressible and are susceptible to flooding. Similarly, Boscia Environmental Solutions (2017) stated that soils in this area drain freely, which results in a soil surface susceptible to erosion, especially wind erosion when vegetation cover is sparse. Most soil in the study area are derived in situ under arid conditions from sedimentary rocks, igneous intrusives and lime-rich evaporates and are generally bas-rich, weakly structured and skeletal (Osburn, 2011). The most common soils in this area are red and yellow sands to non-swelling clays generally freely drained. These soils are also rich in arenosols (ar) - soils with sandy or loamy sand texture (Dold et al., 2006).

The A-horizon of the soils is orthic, as is typical of arid areas in South Africa (Ahrens et al., 2003). Duribank areas are also widespread and the coarse soils associated with them are generally high in most plant nutrients, particularly potassium (Almond, 2012). In flat areas, shallow, coarse sand to sandy loam soils of high nutrient contents are associated with dorbanks and calcrete hardpan (Pulmit Mining, 2003). According to Dold et al. (2006), the dominating soil forms in Kakamas area are Hutton and Mispah (Coarse sandy) and are shallow (0.06-0.18m). A study conducted by Van der Waals (2011) for the proposed INCA Kakamas Photovoltaic Solar Facility revealed that Kakamas area consist of shallow rocky soils dominantly of Mispah (Orthic A-horizon/ Hard rock) and Glenrosa (orthic A-horizon/ Lithocutanic B-horizon) forms along with minor occurrences of Hutton

(Orthic A-horizon/ Red Apedal B-horizon/ Unspecified-usually hard or weathering rock), Dundee (Orthic A-horizon/ Stratified alluvium), Brandvlei/Coega (Orthic A-horizon/ Soft or hard Carbonate B-horizon) and Knersvlakte (Orthic A-horizon/ Dorbank horizon).

In arid and semi-arid ecosystems, the distribution of the soil has been commonly associated with the structure and the spatial arrangement of the vegetation (Megliolo et al., 2017). The aeolian sands in these areas have loose structure, no cohesion, uniform particle size and poor self -stabilizing making it very hard to compact (Duo et al., 2018). Therefore, it must be strengthened before use as foundation for buildings. Similarly, Zhang et al. (2017) revealed the same thing on his study of Soil Macropore Characteristics on Native Desert Soil in Ecotone China that the evolution of soils in arid areas is slow due to hot and dry climate and clay and organic matter contents are often low. Consequently, these soils have a naturally loose structure, a shortage of aggregates and are susceptible to wind erosion, especially under tillage. In general, soil in arid areas are alkaline with high salinity and are exposed to high radiation together with extreme day/night and seasonal temperature fluctuation.

2.10. Investigation of Subsurface Conditions

For many years people have been investigating subsurface conditions using geophysical methods. Geophysics comprises indirect methods for the detection or inference of the presence and position of geological structures, as well as the imaging or mapping of the physical properties of the earth (Beaurez et al., 2009). According to Milson and Erikson (2011), geophysics is a multidisciplinary science that seeks to investigate the nature of the earth and its environment through the application of the principles of physics, mathematics and chemistry. However, since the results of the geophysical survey are not subject to direct visual verification, geophysical exploration requires boreholes or other direct geological exploration for references and control of measurements (Ariffin, 2001). Consequently, geophysical explorations complement core drilling, test pits, or other direct methods of sub-surface exploration and can provide a rapid evaluation of certain geologic conditions (Mariita, 2007).

Geophysical methods of data acquisition are divided into surface and subsurface methods wherein surface geophysical methods obtain information about the subsurface by conducting measurements on or above the ground surface, and subsurface methods that require the drilling of boreholes to obtain information about the subsurface (Dobrin, 1976). The application of both surface and borehole geophysical techniques to ground water exploration has been a usual practice since 1916, with some proving to be more effective in locating groundwater than others (Moore, 2012). This is because groundwater is characterized by a set of physical parameters such as porosity, permeability and conductivity that most geophysical techniques cannot determine (Bernard, 2003). The application of a particular geophysical technique in locating subsurface targets is controlled primarily by the physical properties of the target under investigation (Beaurez et al., 2009). The most widely used surface geophysical methods for investigating the subsurface conditions and groundwater exploration are magnetic survey, electrical resistivity and seismic methods. However, the high cost associated with seismic methods limits its application.

2.10.1. Magnetic survey

Magnetic survey rely on the magnetism of objects is to investigate subsurface geology on the basis of the anomalies in the earth's magnetic field resulting from the magnetic properties of the underlying rocks (Ariffin, 2001). According to Beaurez et al. (2009), magnetic survey looks for variations in the magnetic field of the earth that are caused by changes in the subsurface geologic structure or by differences in the magnetic properties of near-surface rocks. This inherent magnetism of a rocks is called the magnetic susceptibility. The most common applications of magnetic survey are to map the geologic structure or basement rocks (the crystalline rocks that lie beneath the sedimentary layers) as well as to detect magnetic minerals directly (Mariita, 2007).

In general, the magnetic susceptibility of rocks is extremely variable depending on the type of rock and the environment it occurs within. According to Ariffin (2001), mafic rocks have higher magnetic susceptibility than felsic rocks. This is because these rocks are comprised of heavy and dense minerals such as magnetite and iron. The most common causes of magnetic anomalies include dykes, faults and lava flows (Mariita et al., 2015).

However, Beaurez et al. (2009) argued that anomalies in the earth's magnetic field are caused by induced or remanent magnetism.

Induced magnetic anomalies are the result of secondary magnetization induced in a ferrous body by the earth’s magnetic field. The shape, dimensions, and amplitude of an induced magnetic anomaly is a function of the orientation, geometry, size, depth, and magnetic susceptibility of the body as well as the intensity and inclination of the earth's magnetic field in the survey area. A magnetic high anomaly is where the measured field strength is higher than the value predicted by the global model, and a magnetic low is where the measured field strength is lower than the value predicted by the global model (Beaurez et al., 2009). Possible causes for magnetic highs include the presence of magnetically charged rocks in the subsurface (Ariffin, 2001).

Generally, sedimentary rocks have a very small magnetic susceptibility compared with igneous or metamorphic rocks, which tend to have a much higher magnetite (a common magnetic mineral) content (Murdock et al., 2013). According to Hunt et al. (1995), sedimentary rocks tend to have very low magnetic susceptibility values of 0-25 nT while igneous and metamorphic rocks are characterized by very high values which range from 0 to 270 000 nT. A study conducted in Nigeria by Nsals et al. (2013) revealed that sedimentary rocks had a magnetic susceptibility of 232 nT which was very low as compared to that of the igneous rocks in the area which was 31490 nT.

2.10.2. Electrical Resistivity survey

The utilization of the electrical resistivity methods and their merits in the exploration of the subsurface follows the work of Conrad Schlumberger in the periods from 1912 to 1914 (Zohdy et al., 1990). Due to his early interests in earth sciences, he made a realization that metal ores should be discernable from their surrounding material by measuring their electrical conductivity. During that period, the resistivity method was mainly used for prospecting of conductive ore bodies (Parasnis, 1997). According to Breusse (1963), the main application of these methods to subsurface mapping and groundwater exploration began during World War II. Since then, the use of traditional sounding surveys for quantitative interpretation has become a common practice and the rapid advances in computer software over the last few years have resulted in the increased application of the resistivity method in groundwater exploration and subsurface mapping (Jatau et al., 2013).

In recent years, resistivity methods have been used for the detection of subsurface cavities and fractures, for monitoring pollution in the ground and in delineating archaeological features. In addition, the electrical resistivity method has been widely used for groundwater exploration in both sedimentary terrains and areas underlain by crystalline basement rocks. In the investigation of groundwater, subsurface structural and rock variations, evidence has shown that the resistivity method is the most accurate and reliable means of all surveying methods (Emenike, 2001). Consequently, it is the most commonly used geophysical technique in the exploration of groundwater and subsurface mapping and has proven to be the most effective technique in locating water-bearing formations.

Soil resistivity is the key factor that determines what the resistance of a grounding electrode will be, and to what depth it must be driven to obtain low ground resistance (Megger, 2003). The resistivity of the soil varies widely throughout the world and changes seasonally. Soil resistivity is determined largely by its content of electrolytes, which consist of moisture, minerals and dissolved salts. A dry soil has high resistivity if it contains no soluble salts (Beaurez, 2009). In a dry state, most rocks and minerals are naturally insulators, however in nature, most earth materials hold some interstitial water which enhance their ability to conduct electrical current. Since the resistivities of rocks and soils is controlled primarily by their pore water content, similar rocks and soils can have a wide range of resistivities depending on their pore water content (Mahmoud et al., 2009).

The presence of water between the pores of soil and rock fractures renders them electrolytic conductors, with an ionic conductivity that depends largely on the nature of the dissolved salts and degree of saturation of the material (Parasnis, 1997). Sedimentary rocks such as sandstones tend to have lower resistivity values due to their porous nature, while crystalline rocks possess little or no primary intragranular porosity, therefore they are characterized by high resistivity values. Low resistivity values may be recorded in crystalline rocks if they have water-filled fractures and voids. In addition, resistivity has a direct impact on the degree of corrosion in underground pipelines. For example, a decrease in resistivity relates to an increase in corrosion activity and therefore dictates that protective treatment to be used.

The results of an electrical resistivity survey are usually presented in a form of resistivity curves and resistivity pseudo-sections. Generally, the interpretation of resistivity data is guided by available knowledge of the geology of the study area and the objectives of the survey. In resistivity curves, the measured apparent resistivity values are plotted as a function of half-electrode spacing on a log-log graph as shown in Figure 2.4. The qualitative inspection of the plotted VES curves enables the interpreter to postulate the number of layers that would give rise to a curve of that specific appearance (Parasnis, 2007). In general, the number of turning points correspond to the number of detectable layers in a resistivity curve. For example, the curve in Figure 2.4 suggests that the first layer must be underlain by a layer of low resistivity as the curve is descending in the beginning, which is then followed by a layer of higher resistivity.

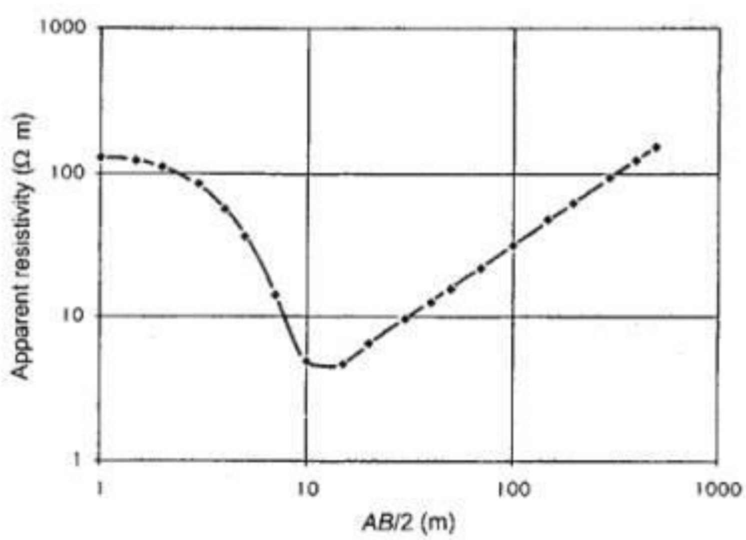


Figure 2.4: Typical VES curve (Zohdy et al., 1990).

If the subsurface consists of three layers of resistivities ρ1, ρ2 and ρ3, the sounding curves may be described according to the relation between ρ1, ρ2 and ρ3. Possible variations in resistivity curves exist for a three-layered ground, the most common curve types include the ones shown in Figure 2.5.

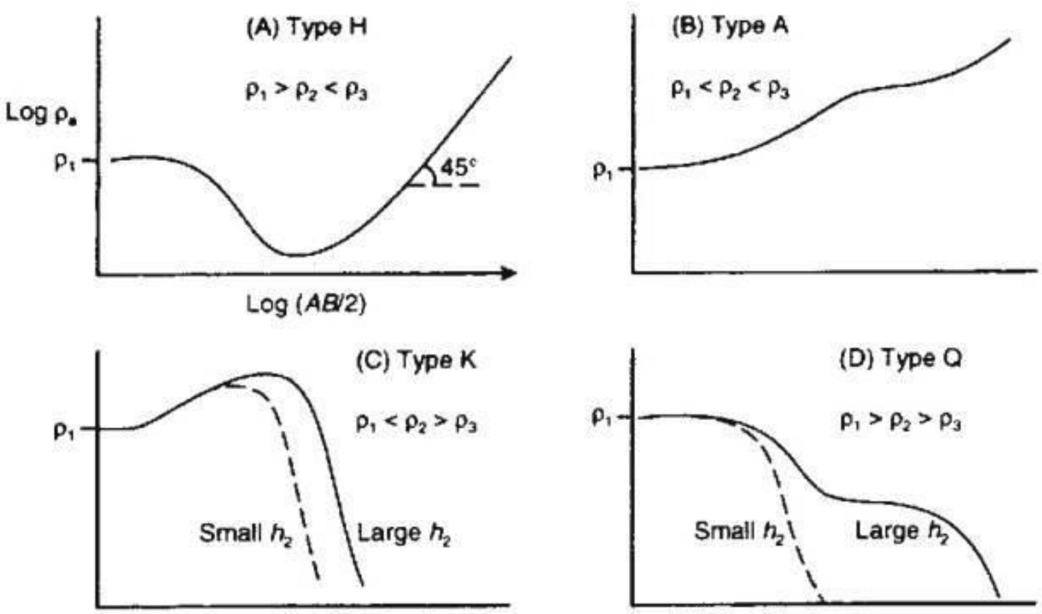


Figure 2.5: Common sounding curves for a three-layered ground (Zohdy et al., 1990).

If the ground consists of more than three layers of resistivities ρ1, ρ2 ρ3 and ρ4, the sounding curves are described according to the relationships between the resistivities of the layers. The letter A, K, H and Q are used in combination to describe the variations of resistivity with depth. According to Raju et al. (2012), H-type of sounding curve are not normally in hard rock formations and it consists of dry top soil of high resistivity as the first layer, water saturated weathered layer of minimum resistivity as second layer and compact hard with very high resistivity as the third layer while A-type of curves are obtained in hard rocks with conductive topsoils. K-type of curves results from different environment and have maximum peak and are occupied by low resistivity values while Q-type of curves are commonly obtained in coastal regions because of saline water.

There are eight possible combinations for a four-layered ground and this include HA-type curve where $\rho1>\rho2<\rho3<\rho4$, HK-type curve $\rho1>\rho2<\rho3>\rho4$, AA-type curve where $\rho1<\rho2<\rho3<\rho4$, AK-type curve where $\rho1<\rho2<\rho3>\rho4$, QQ-type curve where $\rho1>\rho2>\rho3>\rho4$, QH-type curve $\rho1>\rho2>\rho3<\rho4$, KQ-type curve where $\rho1<\rho2>\rho3>\rho4$ and KH-type curve where $\rho1<\rho2>\rho3<\rho4$.

In 2011, Nwankwo carried out a 2D electrical resistivity survey for water exploration at the MADGAS observatory which revealed the presence of 3 geo-electric layers in the study area wherein the top lateritic soil had resistivity values ranging from 200 to 500 Ωm and thickness of 3 to 7m. The weathered basement had resistivity values form 10 to 500Ωm with thickness of 10 to 30 m while the fresh basement layer had resistivities greater than 900Ωm and the analysis of the thickness maps revealed that the western parts of the study have relatively low resistivities in the fractured weathered rocks coupled with thickness of more than 25 m which is consistent with the aquiferous layer, and such areas are suitable for groundwater tube drilling.

Similarly, Sivasubramanian et al. (2012) carried out geo-electrical resistivity survey using Vertical Electrical Sounding in order to assess the subsurface geology and groundwater potential zones in Medak district India and discovered 3-5 layer subsurface wherein the second layer was considered to be a good aquifer as it consisted of fractured/weathered rock formation. The vertical Electrical Sounding for groundwater Prospecting in Uppodai of Tambaraparani river carried out by Balasubramanian (2012), indicated A, K, AK and KH type of curves. The resistivity of the first, second and third layers were varied from 3.6 Ωm to 256 Ωm; 65 Ωm to 2022.3 Ωm and 161.5 Ωm to 2500 Ωm respectively. The layer thickness of the first and second layer were found as 0.7m to 41.5m and 8.3 to 65m. The third layer was weathered, and its thickness ranged from 2 to 12m.

Kaboli et al., (2011) revealed four major geo-electric layers in his Ground water exploration using VES in Southeast Iran. From these results the first layer was interpreted to be near-surface layer had highly variable resistivity ranging from about 3 to 800 Ωm and a thickness of less than 15m. The second layer was dry alluvium layer with a resistivity of less than 100 Ωm and a thickness of more than the surface layer. The third geo-electric layer that corresponded to the saturated layer had a resistivity of less than

10 Ωm and a depth of less than 30m while the fourth layer was interpreted to be a bedrock of slate with a resistivity of 60 Ωm.

In 2017, Abdelrahman et al. indicated the presence of three geoelectrical resistivity layers in his ground water exploration at al-Amar region. The top most layer showed variable resistivity that ranged from 15.115 Ωm to 500 Ωm. the second layer had relatively low resistivity values that varied between 3.549 Ωm to 9.8 Ωm while the third layer had high resistivity values that reached 500 Ωm. on the other hand, the resistivity values along the geoelectrical profile b showed that the first geoelectrical layer ranged from 35.86 Ωm to 270 Ωm. The second layer had resistivity values varying from 9.5 Ωm to 72.28 Ωm while the third layer had resistivity ranging from 500 Ωm to 6516 Ωm. Present results indicate that the second layer has the potential to be groundwater-bearing aquifer in the area of study. The thickness of this layer varies on both profiles which reach more than 100m.

2.11. Determination of Profile of the Natural Soil Deposits

A soil profile is a cross section of the soil structure from the top soil to the bedrock. After careful and thorough study of soil properties, Jennings et al. (1973) developed guidelines to determine the natural soil profiles which became very renowned in South Africa. This classification system is called it the MCCSSO classification system. This classification system describes/ characterizes soil based on their moisture, colour consistency, structure, soil type as well as soil origin. In 2002, Brink et al reviewed Jennings's MCCSSO classification system and modified it to also include rocks in soil profiles as well. Recently, a lot of work on this MCCSSO classification system has been done by Fey (2010) and has discovered that there are 73 soil forms in South Africa and classified them into 14 groups which are: organic, humic, vertic, melanic, silicic, calcic, duplex, podzolic, oxidic, gleyic, cumulic, lithic and anthropic.

A soil profile usually consists of several layers or horizons that are distinguished by changes in moisture content, colour, consistency, structure, soil type and soil origin (Brink et al., 2002). Observations of a soil profile take place in trenches, test pits or large-diameter auger holes in order to provide geological information to depths in excess of 3m (SMEC Consulting, 2013).

The individual layers are identified, and their depths/thicknesses are recorded. The layers are then described by the MCCSSO method on the basis of moisture content, colour, consistency, structure, soil type and origin (Jennings et al., 1973).

The moisture content as defined by Das (2010) is the amount of water in the soil and varies with time and rainfall events. The moisture content is described as dry, slightly moist, moist, and very moist or wet (Brink et al., 2002). According to Das, (2010) wet soils are generally situated below the groundwater or perched groundwater levels and tend to be very weak as compared to dry soils because water tends to weaken the structure of the soil by breaking the intermolecular forces between the soil particles. According to Das (2010), the moisture content must be interpreted in terms of grain-size. For example, sand with a water content of five to ten per cent may be described as 'wet', while a clay with the same water content may be described as 'slightly moist'. The term 'moist' usually describes a soil with a water content close to optimum. Dry and slightly moist soils require water if they are to be compacted effectively (Das, 2010). Likewise, very moist and wet soils require drying before they can be effectively compacted (Brink et al., 2002).

The colour of soil is recorded to allow correlation of the same layer in different holes in the investigation area and depends on its moisture content (Turner, 2013). The colour is therefore described 'in profile' at natural water content in both wet and dry condition (Fey 2010). However, the proper description of colour as observed in the soil profile is difficult. The most satisfactory basis for the description is provided by comparing the colours with those standards laid down in the Munsell colour charts, but these standards are difficult to use in confined and frequently dirty conditions encountered in trial holes (Brink et al., 2002).

Soil consistency is a measure of the hardness or toughness of soil. For example, the effort required to excavate or mould the soil with the fingers (Brink et al., 2002). It is also a rough measure of the soil's strength and density. According to Das (2010), the consistency of a soil depends on its moisture content, particularly with regard to cohesive soils. Soil consistency is described in different terms for granular and cohesive soils (Fey, 2010). The terms relating to consistency of granular and cohesive soils and their meanings are presented in Table 2.1 and 2.2 respectively.

Table 2.1: Consistency of granular soils (Jennings et al., 1973).

Term	Description	Typical density(kg/m^3)
Very loose	Crumbles very easily when scraped with a geological pick	<1450
Loose	Slight resistance to penetration with sharp end of a geological pick	1450-1600
Medium dense	Considerable resistance to penetration with the sharp end of a geological pick	1600-1750
Dense	High resistance to penetration with sharp end of geological pick, requires many blows of the pick for excavation	1750-1925
Very dense	High resistance to repeated blows of geological pick; requires power tools for excavation	>1925

Table 2.2: Consistency of cohesive soils (Jennings et al., 1973).

Term	Description	Unconfined compressive strength(kN/m^2)
Very soft	Pick head can easily be pushed in up to shaft of handle, easily remoulded by fingers	<35
Soft	Easily penetrated by thumb, sharp end of pick can be pushed in by 30-40mm, moulded by fingers with some pressure	35-75
Firm	Indented by thumb with effort, sharp end of pick can be oushed in up to 10mm	75-150
Stiff	Slight indentation with sharp end of pick, requires a hand pick for excavation	150-300
Very stiff	Slight indentation produced by blow of pick point, requires power tools for excavation	>300

The structure of the soil as defined by Brink et al., (2002) refers the presence or absence of discontinuities in the soil and their nature. Non-cohesive soils exhibit a granular structure and since this is an invariable feature it is usually not recorded. Contrary to this, cohesive soils exhibit several types of structural characteristics summarized in Table 2.3.

Table 2.3: Identification of typical soil structures (Brink et al., 2002).

Term	Identification
Intact	Structureless, no discontinuities identified
Fissured	Soil contains discontinuities that may be open or closed, stained or unstained and of variable origin
Slickensided	Describes discontinuity surfaces which are smooth or glossy and possibly striated
Shattered	Very close to extremely closely spaced continuities resulting in gravel sized soil fragments which are usually stiff to very stiff and difficult to break down
Micro-shattered	As above, but sand-sized fragments
Stratified, laminated &foliated	Used to describe sedimentary structures in transported soils structures in residual soils
Pinholed	Pinhole-sized voids or pores (<=2mm) which require a hand lens to identify
honeycomb	Similar to pinholed but voids and pores >2mm
Matrix-supported	Clasts supported by matrix
Clast-supported	Clasts touching (matrix may or may not be present)

The soil type is described on the basis of grain size and can either be sandy, silt or clay. (Brink et al., 2002). According to Das, (2010) sandy soil comprises of particles between 0.06mm and 2mm, silt comprises of particles between 0.06mm and 0.002mm and clay consists of all the particles<0.002mm. Particles >2mm are regarded as gravel and their shapes should be described as either well-rounded, rounded, sub-rounded, sub-angular or angular. Packing in these particles is also of paramount importance and should be described as densely packed, rounded and subrounded quartz, coarse gravel and cobbles in a matrix of silty medium and coarse sand' or 'Loosely packed nodular ferricrete in a matrix of silty fine and medium sand' (Brink et al., 2002). Following this convention, consistent descriptions of soil profiles are possible.

The origin of transported soils is described by the mode of transport and can either be hillwash, fine or coarse colluvium, alluvium, lacustrine, aeolian or beach deposits. A pebble marker usually occurs at the base of transported soil (Fey, 2010). The origin of residual soils is described in terms of the parent rock and the name of the formation of the parent rock should be included (Ahrens et al., 2003).

2.12. Determination of Engineering Properties of Soils

Engineering properties of soil comprises of physical properties, index properties, strength parameters (shear strength parameters), permeability characteristics, consolidation properties, modulus parameters, dynamic behavior of the soil. These properties are typically determined through in-situ testing from field exploration, laboratory testing and back analysis based on site performance data. In essence, this section seeks to review the engineering properties of the soil.

2.12.1. Physical properties of the soil

The physical or index properties of the soil include the particle size, grain shape and the plasticity. To determine the particle size and the grain shape, sieve analysis is used. This is a process of determining the particle size distribution of the soil by placing soil samples on stacks of sieves which are then shaken on a mechanical shaker to determine the percentage passing in each individual sieve. After sieving the soil, the percentage passing in each sieve together with the grain sizes are used to plot a gradational or particle distribution curve. According to Das (2010), these graphs can either be well grade, uniformly graded or gap graded. A well graded curve contains various particle sizes in it and the uniform graded contains particles of the same size while gap graded only contains some particle sizes but misses others.

Apparao and Rao, (1995) explained that the grain size analysis is widely used in classification of soils. The data obtained from grain size distribution curves is used in the design of filters for earth dams and to determine suitability of soil for road construction, and air fields. Raj (2012) stated that the particle size of sands and silts has some practical value in design of filters and in the assessment of permeability, capillarity, and frost susceptibility while Bowles (2012) stated that particle size is one of the suitability criteria of soils for roads, airfield, levee, dam, and another embankment construction. Information obtained from particle-size analysis can be used to predict soil-water movement, although permeability tests are more generally used. According to Dafalla (2013), the sand shape whether rounded, subrounded, or angular will affect the shearing strength of soil. Angular grains provide more interlock and increased shear resistance. The gradation and size of the sand affect the shear resistance. Well-graded materials provide more grain to grain area contact than poorly graded materials. Porosity and spaces available for clay within the sand is important when considering the mixtures of clays and sands.

The sieve analysis was based on Das (2004) geotechnical soil classification. The geotechnical soil classification classifies soils into groups with similar behavior in terms of simple indices in order to provide geotechnical engineers with a general guidance on the engineering properties of the soils. It classifies soil into two commonly known classification schemes which are the Unified Soil Classification System (USCS) and the American Association of the State Highway and Transportation Officials System (AASHTO) (Das, 2004). The AASHTO classifies into seven major groups viz are A-1 through to A-7. Materials classified under groups A-1, A-2, and A-3 are granular materials of which 35% or less of the particles pass through the No. 200 sieve while those with more than 35% pass through the No. 200 sieve are classified under groups A-4, A-5, A-6, and A-7. These materials are mostly silt and clay-type materials (Das, 2006).

The classification system is based on three material properties which are grain size, plasticity and the group within which the material falls under (Ahrens at al., 2003). The grain size of the material has been divided into three groups. These include gravel, sand and fines (silt and clay) wherein gravel materials are those fractions passing the 75 mm sieve and retained on the 2 mm sieve and sand are those fractions passing the 2 mm sieve and retained on the No. 200 (0.075 mm) sieve while fines are those fractions that passes through the No. 200 sieve (Das, 2006). To determine the plasticity of the soil, linear shrinkage or plastic limit is used. Linear shrinkage is the lowest moisture content at which the soil can be rolled into threads of 3 mm diameter without crumbling or the decrease in length of the soil sample when oven dried.

To determine this, Das, 2004 mixed soils that passed the 0.425 mm sieve size are mixed with water and moulded and rolled them into balls. The ball was placed on the glass plate and rolled by the fingers into a uniform thread of 3 mm in diameter, referencing by a metallic rod. The thread is then kneaded together into a ball and placed on the glass plate to be re-rolled into a thread and this process of kneading and rolling is repeated until the thread 36 starts crumbling just before reaching a thread of 3 mm diameter. The crumbled threads are collected and placed into a container of known mass and weighed to record its mass. The container with the collected threads is placed in an oven to dry. After oven drying the mass of the soil is weighed and recorded on a spread sheet in order to calculate the water content. The plastic index of the soil samples is then calculated using Equation 3.3.

The plasticity of the materials in this classification system is categorized into two groups wherein a PI<10 refers to silty soil if the material is fine and that which is greater than 11(PI>11) refers to clayey soils when the material is made up of fine fractions. Regarding the groups, materials have been classified into eight groups, A-1 to A-8 wherein A-1, A-2 and A-3 are the major groups and represent the coarse-grained materials. Consequently, A-4, A-5, A-6, A-7 and A-8 represent fine grained materials and are identified by visual inspection. The A-8 group is a group that represents organic soils, these soils cannot be used for any engineering purpose due to the fact that materials classified under this group do not meet any standard of engineering or re-use potentials (Das, 2006).

The plasticity index is used in soil classification and in various correlations with other soil properties as a basic soil characteristic. Based on the plasticity index, the soils were classified by Atterberg, shows the correlations between the plasticity index, soil type, degree of plasticity and degree of cohesiveness (Prakash and Jain, 2002). Skempton (1953) observed that the plasticity index of a soil increases linearly with the percentage of the clay-sized fraction. Laskar and Pal (2012) found that plasticity depends on grain size of soil. With the increase of sand content plasticity index of soil decreases, which might be due to decrease of inter molecular attraction force. Due to decrease of attraction force, liquid limit of the soil decreases and accordingly plasticity index decreases. But as the clay content increases inter molecular attraction force increases and liquid limit increases.

Table 2.4 shows how the AASHTO material classification system is used. Carter and Bentley (1991) explain that this system does not provide as clear a border of the classes as the Casagrande system, because some materials that may be classified as granular material may yet contain large quantities of clay or silt. They also emphasize that the aim of the AASHTO System is not to classify soil, but rather to classify the material suitability for use as pavement.

Table 2.4: The AASHTO material classification system (Das, 2006).

General Classification	**Granular Materials (35% or less passing the 0.075 mm sieve)**							**Silt-Clay Materials (>35% passing the 0.075 mm sieve)**			
Group Classification	A-1		A-3	A-2				A-4	A-5	A-6	A-7
	A-1-a	A-1-b		A-2-4	A-2-5	A-2-6	A-2-7				A-7-5 A-7-6
Sieve Analysis (% passing)											
2 mm	50 max										
0.425 mm	30 max	50 max	51 min								
0.075 mm	15 max	25 max	10 max	35 max	35 max	35 max	35 max	36 min	36 min	36 min	36 min
Characteristics of fraction passing 0.425 mm											
Liquid Limit				40 max	41 min	40 max	41 min	40 max	41 min	40 max	41 min
Plasticity Index	6 max		NP	10 max	10 max	11 min	11 min	10 max	19 max	11 min	11 min[1]
Significant constituent materials	stone fragments gravel sand		fine sand	silty or clayey gravel sand				silty soils	clayey soils		
General rating as a subgrade	Good							Fair to poor			

The USCS was developed for the purpose of airfield construction and can also be applicable to dams, foundations and other construction and classifies soils into two broad categories, namely: coarse- and fine-grained soils (Das, 2006). Coarse-grained soils that are gravelly and sandy in nature with less than 50% passing through the No. 200 sieve. The group symbols start with a prefix of **G** or **S. G** stands for gravel or gravelly soil, and **S** for sand or sandy soil and is shown in Table 2.5 (Das, 1997). Fine-grained soils are with 50% or more passing through the No. 200 sieve. The group symbols start with prefixes of **M**, which stands for inorganic silt, **C** for inorganic clay, or **O** for organic silts and clays. The symbol **Pt** is used for peat, muck, and other highly organic soils (Das, 1997).

The classification of gravel and fine-grained material is shown in Table 2.6. Gravel materials are identified primarily on the basis of particle grain size and are divided into two groups namely, Sandy-gravel & Gravel. Particles having diameter larger than 4.75 mm are classified as Gravel while those having diameter ranging between 4.75 mm to 75 microns are identified as Sandy-gravel. This classification is done on the basis of its gradation (well or poor), particle shape (angular, sub-angular, rounded or sub-rounded). Gravel material exhibit a good load bearing capacity, however the engineering properties are controlled by the grain size of the particles and their structural arrangement (Das, 1997).

Fine-grained material are those in which more than half of the material passes a Number 200 sieve. The fine-grained materials are not classified by grain size but according to plasticity and compressibility. Fine-grain material may either be silt or clay (Das, 1997). Classification of fine-grained material is done on the basis of its dry strength, dilatancy, dispersion and plasticity. Fine-grained material exhibits a poor load bearing capacity. Strength changes with change in moisture condition. Laboratory classification criteria are based on the relationship between the Liquid limit (LL) and the Plastic index (PI) (Das, 2010).

Table 2.5: Overall unified soil classification system definitions (Das, 2010).

First Letter	Definition	Second letter	Definition
G	Gravel	**P**	Poorly graded (Uniform particle size)
S	Sand	**W**	Well graded (diversified particle sizes)
M	Silt	**H**	High plasticity
C	Clay	**L**	Low plasticity
O	Organic	**Pt**	Peat

Table 2.6: Classification of gravel and fine-grained materials according to the USCS (Das, 1997).

Material type	Group symbol	Group description
Gravel	**GW**	Well graded gravel, gravel-sand mixture, with little or no fines.
	GP	Poorly graded gravel, gravel-sand mixture, with little or no fines
	GM	Silt gravel, or gravel-sand-silt mixture
	GC	Clayey gravel or gravel-sand-clay mixture.
Silt/clay with LL< 50	**MI**	Inorganic silt and very fine sand, rock flour, silt or clayey fine sands or clayey silt with slight-plasticity.
	CI	Inorganic clay of low to medium plasticity, gravel- clay, sand-clay, silt or clean clay.
	OI	Organic silt and organic silt clay of low plasticity.
Silt/clay with LL>50	**MH**	Inorganic silt, micaleous or diatomaceous fine-sandy or elastic silt.

The grading test performed by Outeniqua Geotechnical Service, 2015 in Eastern Cape for a proposed subsidy housing project in Nieubethesda indicated that the soils in the areas could be classified USCS as CL (Inorganic clays of low to medium plasticity), SM-SC (silty/ clayey sands with low plasticity) and SP which is poorly graded sands or sands with little or no fines. The test also confirmed that the soils were dominated by fine grained sands and silts although some gravelly soils were encountered, and the plasticity index was low to medium with low clay content.

In 2002 HC Petersen and Associates performed a foundation indicator test for roads at the Coves Hartebeespoort dam and found out that the aeolian sands classified mainly as SM soil type, these being silty sands and poorly graded sand-silt material. The grading modulus averaged 0.93 in the range 0.89 and 0.99 which reflects its fairly fine nature. It was also non-plastic or slightly plastic which indicate it has had very little fines. The calcrete gravels classified as GC and GP-GM which are clayey gravels or gravel-sand-clay mixtures and poorly graded silty gravel or gravel-sand mixtures. The grading moduli ranged from 2.18-2.29 indicating a relatively coarse nature of the material. The plasticity indices ranged widely from slightly plastic to 20 which is typical of calcretes.

SMEC (2016) used a particle size distribution/ foundation indicator test for the Sibaya Bulk service project which showed that the majority of the site soil consisted of clayey and silt sands of negligible low plasticity denoted as SP-SM and SM-SC in terms of the USCS. The potential expansiveness according to van der Merwe (1964) for such material was negligible.

2.12.2. Determination of the strength, consolidation and behavioral properties of the soil

The soil strength as defined by Balasubramanian (2017) is the capacity of a material to withstand axially directed compressive forces (the unconfined compressive strength of the soil). This strength is a function of the stresses/load applied as well as the behavioral properties of the soil. According to Mike et al. (2012), the soil’s strength is the internal resistance per unit area that the soil can handle before failure and is expressed as a stress. The test involves a soil specimen subjected to an axial load until failure while also being subjected to confining pressure that approximates the in-situ stress conditions. Brearly et al. (2015) stated that knowledge of strength of soils is necessary to determine the bearing capacity of foundations, the lateral pressure exerted on retaining walls, and the stability of slopes.

These tests are performed on high quality, relatively undisturbed in-situ specimens. However, according to Balasubramanian (2017) it is difficult and frequently impossible to sample, transport, extrude and set-up testing for granular, cohesionless soils such as sand and gravel without excessively disturbing or completely obliterating the soil specimen. In addition, Teymur (2015) stated that it is difficult to obtain good strength values through laboratory testing of disturbed specimens since the soil matrix (i.e., cohesion/ bonding of soil particles) is destroyed and the in-situ density and moisture content are very difficult to recreate. Hence, strength, consolidation and permeability tests must only be performed on undisturbed soil samples.

The soil's strength is the most important engineering property required for foundation designs (Youseff, 2015). This is because it reveals the resistance, competency, max dry density as well as the moisture content of the soil (CBR). There are several methods developed to determine the soil strength both in the field and in the laboratory. These include but not limited to cone penetration, consolidation test, compaction and the California Bearing Ratio (CBR).

Cone penetration is a "field-strength test" done by measuring the number of blows from a 65kg hammer dropping 760mm to advance a standard split spoon sampler through 6 increments of 75mm (Mayne, 2007). According to Rogers (2010), a standard cone penetration test (CPT) involves hydraulically pushing a probe with a conical tip and tubular sleeve into the ground while measuring the force required for this penetration. The total force of penetration is converted into the cone or tip resistance and the sleeve friction resistance respectively and the results are presented as tip resistance, sleeve friction and friction ratio (tip to sleeve) with depth of penetration (Bailey, 2007).

Generally, the number of blows increases with the soil strength (Rogers, 2010). According to SAICE, (2015) cone penetration is also conducted in order to establish the stiffness of the soil and the consistency of the subsoil underlying the site at shallow to moderate depth. Consequently, the more the number of blows the greater the stiffness and vice versa. According to SMEC SA (2013), cohesive soils with between 5 and 12.5 number of blows are regarded as soft and those that have between 12.5 and 20 number of blows are considered as firm. Furthermore, those that have 20-38 number of blows are stiff while those with 38-75 number of blows are considered to be very stiff.

In 2013, SMEC SA carried out a DCP test for Olien PV solar pant in Lime Acres in Northern Cape which revealed that the soil had highly variable consistency with no consistent pattern emerging from DCP test. Similarly, the Outeniqua Geotechnical Services (2015) performed a standard cone penetration on the soil of Nieubethesda in the Eastern Cape for a proposed subsidy housing project and the results indicated a highly variable soil consistency with no general pattern emerging from the insitu cone penetration test. Lastly the geotechnical investigation for Sibaya Bulk Service Project

carried out by SMEC, (2016) involved performance of DCP on the soil and the results indicated that the site profiles may be described as medium dense with a coefficient of variation equal to 40%, which is typical for DCP tests.
One of the greatest advantages of this geotechnical method is the ability to record the dissipation of the excess pore water pressures around the cone tip at the end of each rod change, usually 1m intervals (Mayne, 2007). In addition, the dissipation data is used to determine the consolidation parameters for the soils, as well as the nature of the seepage regime in the test area, especially the depth of the phreatic surface (Duarte et al., 2014). Furthermore, it can also be used to classify and characterize soils. However, cone penetration has limited application in soils that are dense/stiff or gravelly (Mayne, 2007).

The compaction test was developed in the 1950s to determine the strength and the consolidation of the soil in the laboratory. According to Reynolds (2013), compaction is the process of increasing the density of a soil by packing the particles closer together with a reduction in the volume of air. Compaction is the application of mechanical energy to a soil so as to rearrange its particles and reduce the void ratio. There is usually no change in the water content and in the size of the individual soil particles. Compaction is also used to prepare a level surface during construction of buildings. The purpose of compaction are to increase soil shear strength and therefore its bearing capacity, to reduce subsequent settlement under working loads and to reduce soil permeability making it more difficult for water to flow through (Roy et al., 2017).

Soil compaction is one of the ground improvement techniques. It is a process in which by expending compactive energy on soil, the soil grains are more closely rearranged. Compaction increases the shear strength of soil and reduces its compressibility and permeability (Roy et al., 2017). Murthy (2002) explained that when soil is properly compacted, the shear strength of the material is increased, and soil becomes more stable and since the soil becomes dense, its permeability decreases which in turn decreases the seepage loss of the water stored. Furthermore, the settlement of the soil also decreases due to the increase in the density of the materials. According to Prakash and Jain (2002), compaction of soils increases the density, shear strength, bearing capacity but reduces their void ratio, porosity, permeability and settlements.

Compaction is normally done simultaneously with the CBR test and involves subjecting soil under a compactive force at different moisture contents. The results are useful in the stability of field problems like earthen dams, embankments, roads and airfields. The moisture content at which the soils are compacted in the field is controlled by the value of optimum moisture content determined by the laboratory proctor compaction test and the compaction energy applied in the field is also controlled by the maximum dry density determined in the laboratory (Reynolds, 2013).

Durgunoglu et al. (2003) used heavy dynamic compaction method for the compaction of foundation subsoil of Carrefoursa Hypermarket and Trade Center in Bursa, Turkey in order to increase the bearing capacity of the foundations sub soils as well as to control the total and differential settlements underneath the foundations. A dynamic compaction test carried out in Lephalale for the proposed New Eskom General landfill on the aeolian sands and calcretes by In Road Consulting (2009) and the results indicated that they had a maximum density of 2122 and 1972 kg/m^2 respectively at optimum moisture contents of 7.3 and 11.9.

A mega water (208 MLD) supply project was undertaken by Guwahati Metropolitan Development Authority funded by Japan International Cooperation Agency for central Guwahati region, India wherein mechanical compaction of foundation soil using road roller were observed to be adequate for the construction of foundation (Gogoi and Laskar, 2015). In 2015, SMEC SA performed a compaction test for Olien PV Solar Plant in Lime Acres, Northern Cape which revealed that the coarse-grained soils, mainly sands as defined by ASTM D2487 should be compacted to at least 95% relative compaction and the fine-grained soils to at least 90% relative compaction.

The CBR test was developed in 1952 to assess the strength of pavement subgrades (Reynolds, 2013). According to Paige et al. (2009), CBR test was developed in order to establish the shear strength of pavement material and has been used widely as pavement material test throughout the world since the 1930s. Breytenbach (2009) defined the California Bearing Ratio as a measure of the shear strength of material and according to Reynolds (2013), the CBR test was developed because the use of grading and Atterberg limits alone were not sufficient in qualifying materials for road construction use due to the fact that such materials behave differently under different moisture and density conditions. As such the California Division of Highways developed the CBR and the swell test using the Proctor's original compaction technique. According to Reynolds (2013), CBR was developed to determine the shear strength of materials while the swell test was developed to indicate the materials (potentially worst state) post-compaction behavior like when the soil is saturated.

Reynold (2013) also defined the California Bearing Ratio as the ratio of force required to cause a circular plunger of 1932 mm^2 area to penetrate the material for a specified distance expressed as a percentage of a standard force. Similarly, the Indian Institute of Technology Gandhinagar (2002) defined the CBR as the ratio expressed in percentage of force per unit area required to penetrate a soil mass with a circular plunger of 50mm diameter at the rate of 1.25mm/min to that required corresponding penetration in a standard material. Essentially, it is a measure of the shear strength of a material at a known density and moisture content (Breytenbach, 2009).

In view of this, Reynolds (2013) stated that this shear strength should generally be considered in terms of Coulomb s Law, as discussed by Breytenbach (2009) failure of a soil occurs when individual grains move relative to one another as described in fundamental soil mechanics. This strength is dependent on the frictional component and the cohesion of the material (Breytenbach, 2009). The frictional component depends on the friction and interlock between the soils grains, is a function primarily affected by the particle size distribution (grading) of the material (Paige, 2009). This component is also affected by an applied stress normal to the shear plane (Reynolds, 2013). The impact of cohesion is influenced by the grain size (distribution), the affinity of the particles to moisture (plasticity) and the moisture content (Das, 2010).

CBR involves subjecting materials passing 19mm sieve to a compactive effort of 596 kJ/m^3 and testing them in either soaked or unsoaked condition (Reynolds, 2013). According to Amoudi et al. (2002), CBR is considered as one of the most important tests used to assess earth backfills. Talukdar (2014) stated it is one of the most important engineering properties of soil for design of sub grade of rural roads. This is because it allows for the determination of CBR Maximum Dry Density and CBR Optimum Moisture

Content as well as the optional determination of swell and post penetration moisture content and can also be used to simulate climatic conditions and estimate the potential swell behavior of expansive clays in its natural state (Reynolds, 2013). Consequently, CBR value of soil may depends on many factors like maximum dry density (MDD), optimum moisture content (OMC), liquid limit (LL), plastic limit (PL), plasticity index (PI), type of soil, permeability of soil (Talukdar, 2014).

In 2009, a compaction and CBR test were carried out on the aeolian sand and calcretes of Lephalale for the proposed Eskom general landfill and the results revealed that they had maximum densities of 2122 and 1972 kg/m^2 relatively at optimum moisture contents of 7.3 and 11.9 (EGA Consulting). The CBR carried out on the soils indicated that the aeolian sand and calcrete classified as G8 and G6 materials respectively in accordance with the TRH 14 materials specifications.

Similarly, HC Petersen and Associates (2002) conducted a CBR test as part of the geotechnical investigation for roads at the Coves-Hartebeespoort dam and discovered that the maximum dry density was 1773 kg/m. However, the CBR was relatively low i.e. 9 at 93%Mod AASHTO and the latter was the reason for the TRH14 classification of G7 despite how good the material looked visually. In addition, the material broke down under compaction to form a fine soil (silt) with low strength characteristics and the grading modulus was highly variable, ranging from 0.3 to 1.16.

Conversely the geotechnical investigation of lime acres, Northern Cape conducted by SMEC SA, (2015) indicated that the insitu roadbed was generally free drained and had poor quality in terms of the CBR. Subsequently SMEC, (2016) carried out a CBR test on the Berea formation and recent aeolian materials of Sibaya Bulk service project and deduced that the materials may be classified as G7-G8 in terms of the COLTO classification while the grading moduli were determined to be less than 1.

2.12.3. Determination of hydraulic characteristics of the soil

The amount, distribution, and movement of water in soil have an important role on the properties and behavior of soil. The hydraulic behavior of the soil is explained by Darcy's law. Water pressure is always measured relative to atmospheric pressure, and water table is the level at which the pressure is atmospheric. Soil mass is divided into two zones with respect to the water table which are below the water table (a saturated zone with 100% degree of saturation) and just above the water table (called the capillary zone with degree of saturation ≤ 100%) (Raj, 2012).

Data from field permeability tests are needed in the design of various civil engineering works, such as cut-off wall design of earth dams, to ascertain the pumping capacity for dewatering excavations and to obtain aquifer constants (Raj, 2012). The permeability of soils has a decisive effect on the stability of foundations, seepage loss through embankments of reservoirs, drainage of subgrades, excavation of open cuts in water bearing sand, and rate of flow of water into wells (Murthy, 2002). Prakash and Jain (2002) explained that water flowing through soil exerts considerable seepage forces, which have direct effect on the safety of hydraulic structures.

The rate of settlement of compressible clay layer under load depends on its permeability and the quantity of stored water escaping through and beneath an earthen dam depends on the permeability of the embankment and the foundation respectively (Roy et al., 2017). Consequently, the rate of drainage of water through wells and excavated foundation pits depends on the coefficient of permeability of the soils and the shear strength of soils also depends indirectly on its permeability, because dissipation of pore pressure is controlled by its permeability (Murthy, 2002).

According to U. S. Bureau of Reclamation, soils whose k (coefficient of permeability) is less than 10-6 cm/sec are classified as impervious while those whose k is between 10-6 to 10-4 cm/sec are classified as semi-pervious. Lastly soils whose k is greater than 10-4 cm/sec are classified as pervious. The typical permeability coefficients of different soil types are shown in Table 2.7. In 2011, EGA performed a permeability test for the proposed Mount Signal Solar Farm and Associated structured by falling method and identified low permeable clayey soils and ground water at 3-5 m depth. Similarly, the permeability test for the proposed subsidy housing project at Nieubethesda revealed fine grained soils (silt and fine sands) with low permeability ad a perched water table at depths ranging from 1.7m to 3m (Outeniqua Geotechnical Services).

Table 2.7: Typical permeability coefficients for different soils (Ishbashi and Hazarika 2011).

Relative permeability	Coefficient of permeability k(cm/sec)	Typical soils
Very Permeable	$>1\times10^{-1}$	Coarse gravel
Medium permeable	1×10^{-1} to 1×10^{-3}	sand, fine, sand
Low permeable	1×10^{-3} to 1×10^{-5}	Silty sand, dirty sand
Very low permeable	1×10^{-5} to 1×10^{-7}	Silt, fine sandstone
Impervious	$<1\times10^{-7}$	Clay

2.14. Assessment of the Impact of the Soil on Buried Metal Objects

For decades now, scientist have been using pH and electrical conductivity to assess the impact of the soil on buried metal objects, metal fittings and even foundations. This is mainly influenced by the chemistry of the soil, but other factors such as degree of saturation also play a role since soil conducts electricity through electrolysis using ions in the soil (Akpila et al., 2013). Soil pH is a measure of the amount of acidity or alkalinity (basicity) that is present in soil. The water pH was determined by weighing approximately 100g of each sample and mixing it with at least 100 ml of deionized water. The mixture was then stirred intermittently for 30 mins and let to stand for about an hour whilst pH buffers 4, 7, 9 and 10 were used to calibrate the pH bench top meter (Orion Star A215). The pH electrode was then immersed in each of the soil samples and sample analysis were performed with the results being recorded on a spread sheet.

The electrical conductivity (EC) is determined in the laboratory by measuring the electrical resistance of a 1as to 5-water suspension. The 1:5-water suspension is prepared by weighing 10g of air-dried soil (<2 mm) into a sample bottle and adding 50ml of deionized water, mechanically shaking the mixture at 15rpm for at least an hour to dissolve soluble salts, rinsing a conductivity cell and then immersing in in the sample and then taking measurements. The observed readings were recorded in µS/cm.

According to NSW Agriculture (2000) these values are regarded as fairly neutral with the exception of the H values >9 which are slightly corrosive and alkaline and may results in some plant nutrients being unavailable. Similarly, the pH test performed by SMEC (2013) and (2016) for Olien PV Solar Plant in Northern Cape and for Sibaya Bulk Service Project also indicated neutral soil conditions with a pH range of 6.30-6.83 and 6.8-7.3 respectively. However, the in-situ soil corrosivity testing conducted by EGA Consultants (2011) for Proposed Mount Signal Solar Farm and Associated structures presented results which correlated with the field resistivity survey in showing that the soils were considered to be severely corrosive toward ferrous metals. In 2015, Outeniqua Geotechnical Services performed soil pH test which also indicated mildly soil conditions for the proposed subsidy housing project at Nieubethesda in the Eastern Cape with pH ranging from 8.66-9.38.

The pH and electrical conductivity results for this study area are similar to those expressed by SMEC (2016) for Sibaya Bulk Serve Project which revealed neutral and non-corrosive conditions with a typical range of EC between 0.03-0.07mS/m. Nevertheless, the conductivity test for Olien PV Solar Plant showed the soils to be corrosive towards steel (SMEC, 2013) while that of the proposed subsidy Housing project in Nieubethesda indicated mildly soil conditions with EC range of 216-568mS/m indicating slightly corrosive to corrosive soils (Outeniqua Geotechnical Services, 2015). In addition, the in-situ soil corrosivity testing conducted by EGA Consultants (2011) for Proposed Mount Signal Solar Farm and Associated structures presented results which correlated with the field resistivity survey in showing that the soils were considered to be severely corrosive toward ferrous metals.

2.15. Solar-specific Foundation Design

Large-scale ground-mounted PV power plants are a relatively recent phenomenon. As a result, solar-specific geotechnical engineering is in its infancy compared to geotechnical engineering for more conventional applications such as vertical construction, buildings, bridges or dams (Sindhu et al., 2017). The key to designing a reliable and cost effective solar-specific foundation is quality geotechnical data and adequate site characterization (Brearly et al, 2015). The better you understand these attributes, the more effectively you optimize and design the foundation.

The ultimate goal of a solar-specific geotechnical analysis is to use site research, soil investigation and empirical load test data to optimize the foundation for the specific site (Sindhu et al, 2017). For example, site research might give you an idea about the basic distribution of soil types. Soil investigation can be used to verify soil classification and map distribution more accurately. The collected load-test data for these soil types can be used to correlate the results to areas across the site with analogous soil conditions. These data ca be analyzed and used to optimize PV power plant foundation designs in terms of foundation type and geometry, embedment depth, corrosion control, mounting-system geometry, material costs as well as the installation costs.

There are two basic geometries used for PV power plants and these include center-post foundations and double-post foundations (Fellenius, 2009). In a center-post foundation, a single row of foundations supports each mechanical array section or table while two rows of foundations, a north row and a south row support each table in a double-post foundation (Brearly, 2015). Typically, vertical and horizontal loads are greater with center-post designs than with double-post designs (Fellenius, 2009). Each center-post foundation usually supports a relatively large surface area, and a comparatively longer lever arm applies horizontal forces to the foundation. In contrast, double- post foundations typically support a smaller surface area, and the structural design shortens the lever arm (Brearly, 2015).

Some foundation types and geometries better suit specific soil or site conditions than other. On smaller projects, it often makes sense to design around a single foundation type to simplify project logistics. However, an optimized design for larger sites often eschews a one-size-fits-all approach in favor of multiple pile profiles, embedment depths or even foundation types. For example, structural engineers almost always use double-post foundations with helical anchors and specify longitudinal bars between the rows to reduce horizontal loads.

According to Brearly et al. (2015), there are several types of solar specific foundations and these include the driven pile, earth screws, helical anchors and ballast. From a foundation optimization standpoint, driven-pile foundations are appealing because they generally offer the most attractive price point while providing good lateral and vertical bearing. SMEC SA (2013) stated that they are also the most appropriate in areas where soils are firm and compacted, with enough fine-grain materials silt or clay to offer high skin friction. Softer soils require deeper embedment depths and larger cross-sectional profiles.

Contrary to this, driven piles are problematic in soils that resist installation, such as soils with very coarse gravel or rock fragments, very hard soils or bedrock and are typically constrained to slopes less than15° (Brearly et al., 2015). Compared to driven piles, earth screws can adapt to a wider range of soil and site conditions (SMEC SA, 2013). They can also be installed in rocky soils and even bedrock if (pilot holes are predrilled). While drilling pilot holes typically increases the foundation cost compared to driven piles, using earth screws may increase the deployable area. For example, earth screw installation is feasible on slopes up to roughly 30° (Brearly et al., 2015).

Earth screws can be installed in softer soils without pilot holes. However, this requires soils deeper embedment depths. For sites with high refusal rates, earth screws may be more economical than driven piles, simply because of the high costs associated with using drilled and grouted piles whenever refusals are encountered (IEA, 2015). Earth screws offer good pullout resistance. Although the screws offer good lateral resistance in firm soils, engineers may need to find ways to increase lateral bearing in softer soils and to adapt their mounting-system design (Brearly et al., 2015).

Helical anchors are generally less economical than driven piles or earth screws. However, they suit soft soils such as clean sand or weak saturated soil especially well (Brearly et al., 2015). The anchor consists of a helical bearing plate welded near the bottom of a narrow central shaft. The surface area of the bearing plate provides high pullout resistance, even in loose soils. However, the narrow shaft offers minimal lateral bearing capacity. As is the case with earth screws, a construction equipment with an auger attachment must be used to drive helical anchors into the ground. While helical anchors are ideal for sites with poor soil cohesion, they are not well suited to hard soils and soils with very coarse gravel or rock fragments (IEA, 2015). A structural engineer needs to ensure that design elements minimize horizontal loading and may also need to adapt the mounting system design to use a helical anchor foundation.

Precast or pour-in-place concrete ballast foundations best suit sites where soil penetration is undesirable or impractical (SMEC SA, 2013). For example, project developers often deploy ballast foundations at PV power plants installed over landfills or brownfields (Sindhu et al., 2017). Sites with bedrock, a high-water table or unconsolidated soils with high refusal rates may also benefit from a ballasted foundation. The mass of the ballast material resists the applied load, and the foundation distributes these loads across a large bearing surface (Brearly et al., 2015). The best application for drilled and grouted piles in PV power plants is as an engineered foundation used in case of pile rejection (Fellenius, 2009). Drilled and grouted piles are otherwise prohibitively expensive, as they require drilling and concrete equipment. Further, the concrete needs to cure before you install the mounting system. However, drilled and grouted piles are suitable for most soil types and provide good load resistance.

Solar foundations present unique design problems although people have been specializing in solar-specific geotechnical analysis and foundation design since 2004 (Sindhu et al., 2017). For example, PV Power plants tend to have a very high number of relatively small piles and people tend to underestimate the skills required to use small piles effectively, because the design loads are very low compared to those for a high-rise building or a bridge (Brearly et al., 2015). Furthermore, the climatic effect influences the first 2 meters of soil can lead to plastic deformation of soils and structural failure of the piles (Fellenius, 2009). In essence, a well-designed solar foundation needs to be cost-effective without sacrificing reliability and while the design loads associated with ground-mounted PV systems may be small compared to those for other structures, the foundation still needs to support considerable dynamic loads because any foundation system can fail over time when subjected to these forces, and foundation system failures are expensive to mitigate (Brearly et al., 2015).

According to Boscia Environmental Solutions (2017), concrete footing will be used to support the PV panels, depending on the voltage levels and the site conditions. In 2002, Solek recommended driven piles and earth screws for Scuitdrift Solar Project located approximately 100 km Northwest of Kakamas town in the Northern Cape. This was done in order to minimize the environmental impacts of the facility and to also accommodate the relatively flat topography which is entirely dominated by reddish alluvial/Aeolian sandy soil with minor gravel and cobbles. These foundation types were also chosen because they imposed a lower cost over the lifetime of the project and offered high equity returns for investors and the operational risks were limited due to proven performance and track record.

The preliminary geotechnical investigation for proposed Solar Plant, Lost Hills, Kern County, California by URS (2010) recommended deep foundation type of concrete piers supported by steel beams or shallow cast-in-drilled hole or Driven steel sections because of the presence of potentially expansive, collapsible and corrosive clay at the site with carbonate cementation. Geotechnical investigation for Olien PV Solar Plant in Lime Acres, Northern Cape suggested cast in situ concrete piles and concrete block foundations due to the presence of strongly cemented calcrete occurring over a flat terrain on site (SMEC SA, 2013).

Preliminary geotechnical investigations for proposed Mount Signal Solar farm, Seeley, California recommended driven steel sections or H-piles instead of cast-in–place caissons over alluvium, lacustrine and fill/crop soils because of less soil disturbance, less concrete, limited access, shorter construction time and target load capacity of representative samples (EGA, 2011). They also recommended underpinning of the solar module array using galvanized steel H-pile or Helical anchors. Furthermore, they recommended Mat slab foundation for the inverter and associated structures due to the presence of heavy loads and expansive soils.

CHAPTER THREE: MATERIALS AND METHODS

Understanding the site investigations required prior to the construction of a solar plant requires a good, detailed and precise description of the procedure required to acquire the engineering properties of the soil. As such, this chapter seeks to describe all the research methodologies that were adopted to acquire all the geotechnical, geological and geophysical data of the area. In addition, it also seeks to describe all the materials and equipment that were used to acquire the data. The flow chart in Figure 3.1 summarizes all the methodologies that were adopted in order to obtain the data.

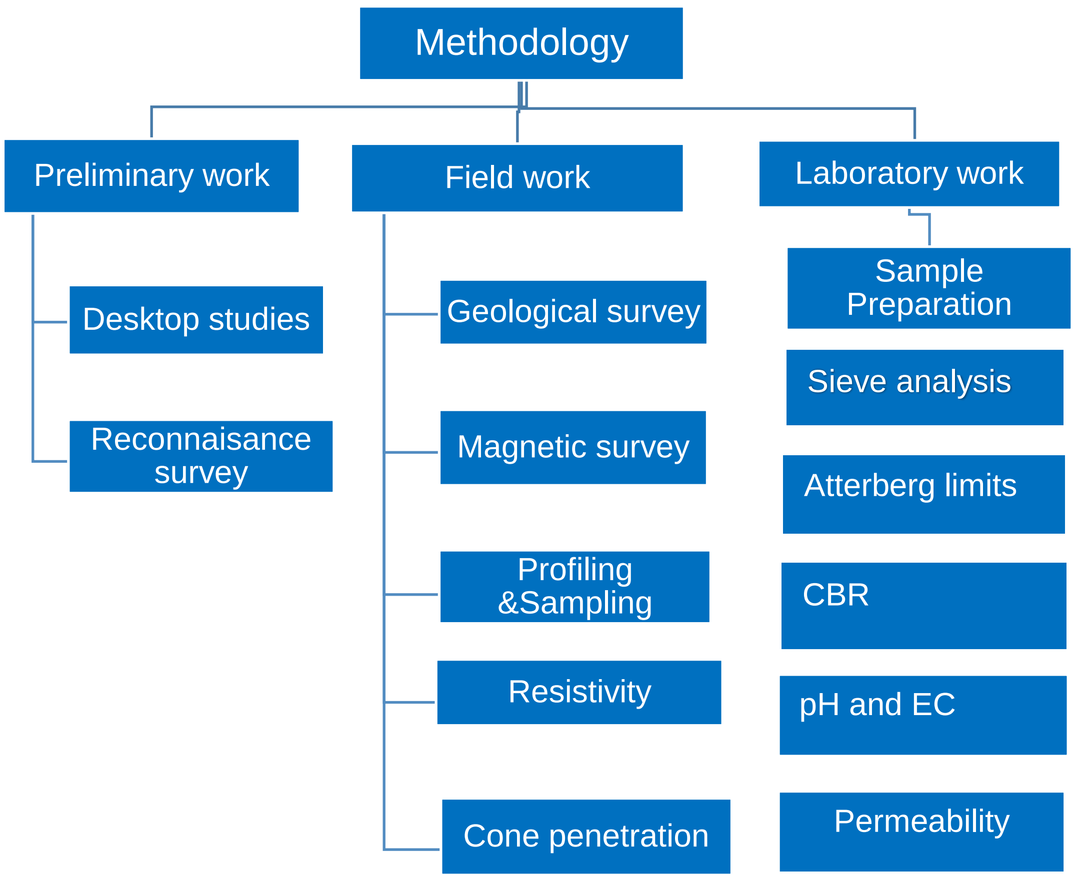


Figure 3.1: Summary of the research methodologies that were adopted.

3.1. Preliminary Studies

The site characterization of Brypaal farm commenced by conducting preliminary work that comprised desktop studies and reconnaissance survey.

3.1.1. Desktop studies

The desktop study was conducted prior to the site visit and it involved analyzing the general geology, soil types and landscape features from different maps and satellite images. Extensive reading and analysis of documents relating to site investigation and construction of solar facilities was also carried out as part of the desktop study. This assisted in gathering the necessary facts about the study area and the problem under investigation.

3.1.2. Reconnaissance survey

Reconnaissance survey was conducted in order to examine the study area and observe the types and distribution of the geographical features such as landforms and topographical reliefs, vegetation cover as well as the nature of the soil within the vicinity of the area. The main purpose of this survey was to verify the actual conditions which were revealed during the desktop study. The survey was also intended at identifying obstacles that might be encountered in the detailed investigation or survey. This included the analysis of the accessibility of the area for soil sampling purposes.

The reconnaissance involved gathering information about the evidence of the presence of geological structures and rocks with problematic engineering properties such as carbonaceous rocks. This was done through geological field mapping in an attempt to establish the overall geology of the study area. Analysis of the drainage pattern of the study area was also conducted during reconnaissance survey. The drainage pattern analysis was done through identification of the major stream systems in the area and using them to establish the general drainage pattern of the area.

Information from this phase of the investigation together with that from the desktop study provided preliminary assessment of the conditions underlying the site and identified areas necessary for further investigation. Based on the outcome of the preliminary work, a detailed site investigation plan was developed, and the equipment needed for the actual field work was assembled.

3.2. Actual Field Work

The actual field work was done following the reconnaissance survey. This was done to obtain information about the actual ground on site and also to justify the data obtained from desktop study and the reconnaissance survey. In order to achieve the objectives of this research, the main task of the site investigation was to perform soil sampling in the major pits and to conduct field tests of the soils so as to obtain information about the engineering properties of the on-site soil.

3.2.1. Site assessment and mapping

Site assessment and mapping in this study was done through soil survey, magnetic survey and geological survey.

Soil Survey

Soil survey involved random collection of soil samples around the field with a shovel and placing it in a sample bag for further laboratory analysis. The random collection of soil was done to eliminate biasness in collecting soil samples and also to ensure that all collected soil samples were representatives of the soils all over the study area. The main aim of soil survey was to provide sufficient samples for laboratory analysis in order to determine the physical, chemical as well as the engineering and behavioral properties of soil. This method was chosen mainly because it is cheap and can be conducted over a large area very quickly. Consequently, disturbed soil samples were collected using a shovel and placed in sample bags which were immediately closed to prevent moisture from entering and contaminating the soil sample and then taken to the laboratory for analysis.

Geological Survey

The geological survey was done simultaneously with the soil survey and was achieved by mapping all the rock units in the area through systematically designed traverses. To attain this, a sample would be collected, using a geological hammer and the sample would be named using a permanent marker and placed into a sample bag for safe keeping and correlation purposes during the production of the geologic map. The outcrop's co-ordinates and elevation would then be determined using a GPS and they would then be jotted down in a field notebook and location of the outcrop would then be marked or coloured on a topographic/ base map for secondary interpretation and map production.

The full knowledge of the geology of the area was intended to assist in ensuring increase in the strength, stability and durability of the foundation of the proposed solar plant since rocks are the most common materials used in the construction of foundations. The main reason for having selected this method was that it is cheap and that all the apparatus required to carry it out were readily available. However, geological survey is quite time consuming and require very thorough, skeptical and accurate correlation of the rock units in order to produce the geological map.

Magnetic Survey

The magnetic survey normally forms part of the geophysical exploration, however in this study, it was used as a criteria for sampling and excavation of trial pits for soil profiling, hence it formed part of site assessment and mapping. The magnetic survey was done by measuring the magnetic susceptibility of the area in traverses using a Proton Precession Magnetometer to investigate possible anomalies due to geological structures such as dykes or faults. The reason for having selected this method was purely based on the fact that the equipment was available and that it is very accurate. Since variations in the magnetic field can be caused by changes in the subsurface geology or by difference in magnetic properties of near-surface rocks, this survey was intended to compliment geotechnical and electrical resistivity data of the area.

Contrary to being very accurate, magnetic survey is very complex and requires designation of traverses to take measurements along and the magnetometer is very receptive to high temperatures and results in errors in measurements. In an attempt to reduce the errors, all measurements were taken before noon. The results of the magnetic survey were used to produce a magnetic map with areas with high magnetic values being represented by a red colour and those with low magnetic valued being represented by a green colour. This together with the geological and soil survey results were used as a criteria for sampling and excavation of test pits shown in the site plan in Figure 3.2.

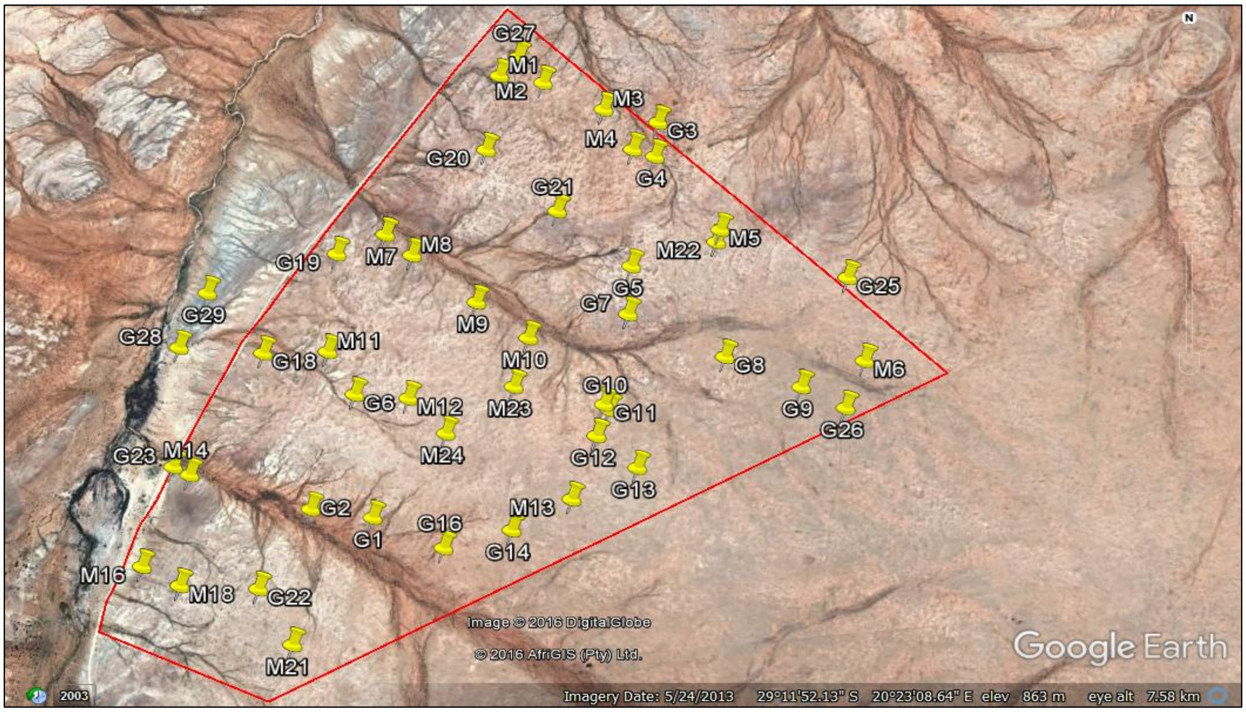


Figure 3.2: A site plan showing all the samples points in the field.

3.2.2 Trial pit excavation, profiling and sampling

Trial pit excavation entailed setting out and excavating 46 test holes employing a Tractor-Loader-Backhoe (TLB) excavator shown in Figure 3.3a. The reason for having selected this method was that it is a cheap method of investigation to shallow depth and that it allows visual inspection of the strata or soil layers. Consequently, trial pits, referenced G1 and M1 to G27 and M23 respectively, were excavated by a TBL excavator shown in Figure 3.3a at the positions shown on the site plan in Figure 3.2 and were positioned so as to cover the footprint of the entire site. These trial pits were excavated on the basis of the geological and magnetic field survey. For example, on different rock units and on areas with magnetic anomalies.

The test pits were excavated up to a convenient depth (average depth of 1.5 to 2 m) and each of the holes were profiled in accordance with the standard methods (MCCSSO) prescribed in the document Guidelines for Soil and Rock Logging in South Africa (1990). The profiling involved identification of different soil layers and their depth and thickness of these were recorded. The setup of the trial pit during the profiling process can be seen in Figure 3.3b. Subsequently properties such as colour, moisture, consistency, structure, soil type and origin were used to describe the different soil horizons. This method however makes it difficult to collect undisturbed samples because granular soils tend to collapse very easily and cannot be used below the water table. Therefore, on-site test such as cone penetration and resistivity survey were conducted on undisturbed samples in order to complement the soil profiling results.

The rocks encountered along the pits were described according to the rock profiling section of the MCCSSO method and the results were recorded in a profile description paper shown in Appendix-A. The colour of the soils and the soft rocks however were described using the Munsell colour Chart, which provided the colours of the soil and soft rock in both dry and wet state as well as the year within which they were formed. During the profiling exercise, both rock and soil samples of different soil horizons were collected and taken to the laboratory for further analysis. The collected data about the soil horizons in pits was processed using Strata10®software to produce logs shown in Figure 3.3c.

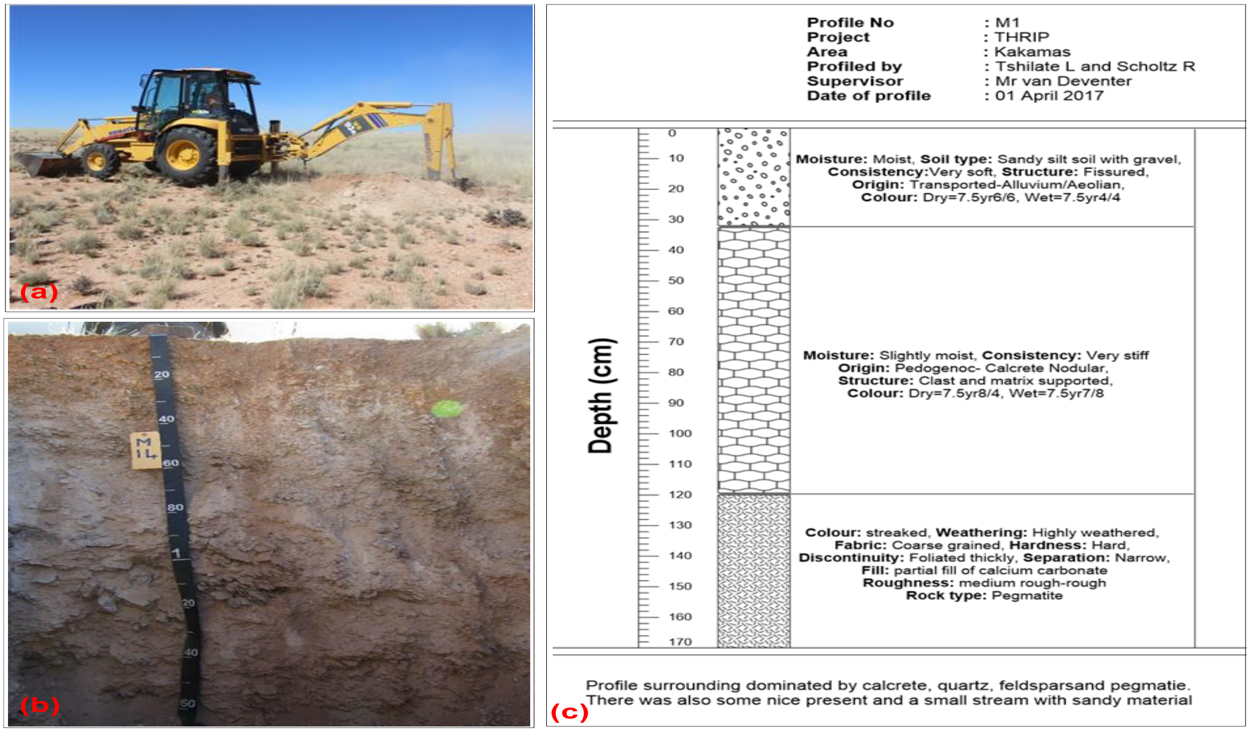


Figure 3.3: Excavation and profiling of trial pits. (a) illustration of the pits excavation process, (b) the pit setup during profiling and sample collection and (c) is an illustration of the profiling data presentation.

3.2.3. Resistivity survey

Field resistivity surveys were conducted using a GF resistivity meter, a 12 V car battery, cables, electrodes and cases. This was meant to investigate the subsurface formations as electrical conduction in soil is essentially electrolytic. As such the soil's resistivity depends on several factors such as moisture content, salt content, mineralization, bedrock and geological phenomena. The resistivity survey was conducted in the field by Wenner array around 26 profiled trial pits. Two current and two potential electrodes were used with electrode spacing being 2.5 m, 5 m and 10 m respectively. A 12 V car battery (capacity 60 Ah) was used to generate current into the ground using the electrodes potential and current cables. Figure 3.4 shows the GF resistivity meter that was used to determine the resistivity of the soil.

The selection of this method was largely because the equipment was readily available and that it is relatively quiet simple and that it can also be used for ground water exploration. In addition, the method is non-disturbing, meaning that measurements were taken on undisturbed samples which are true representative of the on-site soil, field equipment is potable and inexpensive and detects both lateral and horizontal changes. Upon completion of the measurements, the measured values (injected current, measured voltage, apparent resistivity, standard deviation and the measured point number) as displayed on the resistivity meter were downloaded to a PC. The individual resistivity on the pits was plotted on an excel spreadsheet and the results were used to plot trends summarizing the general resistivity of the study area.

However, interpretation of the data requires sound knowledge of the geology of the area and buried metallic objects cause interference of results Consequently all the metallic objects encountered were kept away from the equipment and a detailed geological survey of the entire study area was conducted prior to the resistivity survey in order to acquire comprehensive geological knowledge of the area to help with the interpretation of the results.

Figure 3.4: The GF resistivity meter used to record the resistivity of the subsurface in the study area.

3.2.4. Cone penetration

In general, cone penetration is a dynamic method of measuring the in-situ resistance to penetration of the soil. This method was chosen because it is very quick, fairly non-destructive and that it is the only on-site method that can determine strength on undisturbed soil samples. The field dynamic cone penetration test was performed on the subgrade soil in 26 selected trial pits in the study area to determine the stiffness of the ground at the proposed site. The reason for having performed the cone penetration on only 26 trial pits was that the other untested 27 pits had the same properties as the ones that were selected. Figure 3.5 shows the standard cone penetrometer that was used to determine the resistance of the soils to penetration in the study area. Consequently, a probe with a conical tip and tubular sleeve was hydraulically pushed into the ground until it reached the bedrock.

The pushing of the cone into the ground was undertaken at a rate of 1 to 2 cm/sec and involved dropping a 65 kg hammer at a height of 760 mm to achieve penetration. The instrument also had a lower shaft on an anvil and a cone (55º to 60º) that helped with the penetration. The total force that was used to push the cone into the ground was converted to cone or tip resistance and the sleeve friction resistance respectively and the results were presented as tip resistance, sleeve friction and friction ratio (tip to sleeve) with depth of penetration. However, this method is mainly suited for silt and clay soils which are cohesive and not suited for sandy soil because it is very loose, cohesionless and tends to be easily disturbed. Therefore, it is important to note that the upper 150 mm portion of the top soil was removed prior to the test as it was considered to be "disturbed material". In addition, a hydraulically push piston was used in order to minimize disturbance especially in sandy soil.

Figure 3.5: The standard cone penetrometer that was used to determine the soil's resistance to penetration.

3.3. Laboratory Analysis of Soil Samples

Laboratory work comprises all the analysis that were conducted on the disturbed samples from the field in the laboratory. In order to achieve the objectives of the study soil samples were collected from the different trial pits and were taken to the laboratory for moisture, and sieve analysis, linear shrinkage, maximum dry density and optimum moisture contents, California bearing ratio of untreated soils and gravel, pH and electrical conductivity so as to establish the engineering and behavioral properties of the on-site soil.

3.3.1. Procedure of sieve analysis

Sieve analysis is a practice or procedure used to assess the particle size distribution of the soils particle and was conducted as per SANS 3001-GR2. The main reason for having chosen this method is that it is cheap, and the equipment was readily available in the laboratory. The objective was to divide particles into separate ranges of sizes and determine the relative proportion by weight of each size range. In order to achieve this, samples of soil were weighed on a scale balance and their weights recorded and then poured on a stack of sieves which were then placed on a mechanical shaker for an hour to allow sieving to take place. Each of the sieves and pan were also weighed and their mass recorded. The sieves were stacked on top of each other in a decreasing sequence of their size and a soil sample was poured into the top sieve.

The larger lumps were broken into smaller pieces before they were passed through the sieve. These stacks of sieves were then placed on a mechanical sieve shaker and a cover was tightly closed to reduce sample loss during shaking. The shaker was turned on for 60 minutes at amplitude of 60. After shaking the soil, the sieves and pan were removed and each of sieves were weighed together with the retained fraction of the soil and recorded. The soil fines were then used for the mechanical analysis and for the determination of the atterberg constants.

The results were represented by semi-logarithmic graph to produce a gradational curve that shows the distribution of different particles of the soil sample and the soil was classified according to USCS or ASTM D2487-06. Particle size distribution curves were then plotted whilst the diameter (D10), diameter corresponding to 30% passing (D30) and diameter corresponding to 60% passing (D60) were determined. The coefficient of uniformity Cu and coefficient of curvature Cc were then calculated using the equation 3.1 and 3.2 respectively while the grading modulus was calculated using equation 3.3.

$$\text{Coefficient of uniformity} = C_u = \frac{D60}{D10} \tag{3.1}$$

$$\text{Coefficient of curvature} = C_c = \frac{D230}{D60^{\times}D10} \tag{3.2}$$

$$\text{Grading Modulus} = \frac{\text{P2.00mm+P0.425mm+P0.075mm}}{100} \tag{3.3}$$

Where: D_{10} is the grain size corresponding to 10% passing, D_{30} is the grain size corresponding to 30% passing, and D_{60} is grain size corresponding to 60% passing. P2.00 mm is the % retained on 2mm sieve, P0.425 is % retained on 0.425 mm sieve and P0.075 mm is the % retained on 0.075 mm sieve.

3.3.2. Linear shrinkage determination

Linear shrinkage is the decrease in length of a soil sample when oven-dried, starting with a moisture content of the sample at its liquid limit (Das, 2010). The linear shrinkage was performed in accordance with the SANS 3001-GR10 and involved placing at least 300 g of the material passing the No 36 B.S. sieve (0.425 mm) as per sieve analysis results in the mixing bowl and thoroughly mixed with deionized water using the spatula. This was done until the mass became a thick homogeneous paste. More water was added into the paste to bring it to a consistency equal to or slightly wetter than the liquid limit. This was done so that when the sample is tested in the liquid limit machine, the groove should close with between 15 and 25 blows. The method was chosen because it is cheap and the material was readily available in the laboratory.

The inside of a clean shrinkage mould was then greased with Vaseline and the wet soil paste was carefully placed inside the mould to ensure that all air bubbles from each layer were removed by lightly tapping the base of the mould when placing the soil. The excess material were removed with a spatula to level off the soil in the mould. The soil specimen were then allowed to dry at room temperature for about 24 hours until a distinct change in colour could be noticed and then placed in an oven to dry overnight between 105 °C and 110 °C. After being oven dried, the specimen were allowed to cool and then measure its longitudinal shrinkage Ls to the nearest millimeter. If the specimen had cracked into pieces, its individual cracked pieces were firmly held together, and the linear shrinkage was measured. If the specimen curled in the mould, it was carefully removed, and the length of the top and bottom surfaces were measured and the mean of the two lengths was subtracted from the internal length of the mould to obtain the shrinkage. The percentage LS of the soils were calculated using equation 3.4 and the results were interpreted in accordance with the TRH14 and COLTO requirements.

$$LS(\%) = \frac{L_S}{L} X100 \qquad (3.4)$$

Where: L = Length of the mould (mm) and Ls = Longitudinal shrinkage of the specimen (mm).

3.3.3. Moisture content and dry density

The maximum dry density and optimum moisture content are determined by establishing the moisture-density relationship of the material when prepared and compacted at the Modified AASHTO compaction effort at different moistures. The maximum dry density of the soil is defined as the maximum dry density of a material for a specific compactive effort is the highest density obtainable when the compaction is carried out on the material at varied moisture contents and the optimum moisture content of the soil is defined as the optimum moisture content for a specific compactive effort is the moisture content at which the maximum density is obtained. The choice of selection of this method was governed by its accuracy and also its ability to determine multiple engineering properties such as the maximum dry density, optimum moisture content as well as the CBR of the soil simultaneously. In addition, it is cheap and relatively quite simple.

Soils normally contain a finite amount of water, which can be expressed as the "soil moisture content." This moisture exists within the pore spaces in between soil aggregates (inter-aggregate pore space) and within soil aggregates (intra-aggregate pore space). The moisture was determined by weighing the filed samples and drying in a drying oven at 110^{O}C overnight. The sample were then allowed to cool at room temperature before weighing to obtain difference in weight. The total percent moisture was calculated using the following equation;

$$\%\ \mathrm{H_20} = \frac{\mathrm{Wet.Weight - Dry.Weight}}{\mathrm{Wet.Weight}} \mathrm{X100} \qquad (3.5)$$

Soil density and unit weight are basic to many soil and foundation problems, as the weight of soil involved can eclipse that of other materials used to build structures. To assess the degree of compaction, it is necessary to use the dry unit weight, which is an indicator of compactness of solid soil particles in a given volume. The laboratory testing was meant to establish the maximum dry density that can be attained for a given soil with a standard amount of compactive effort. In the test, the dry density could not be determined directly, and as such the bulk density and the moisture content were obtained first to calculate the dry density.

The soil samples were compacted at different water contents, and a curve was drawn with axes of dry density and water content. As water was added to the soil at low moisture contents, it became easier for the particles to move past one another during the application of compacting force. The particles came closer, the voids were reduced, and this caused the dry density to increase. As the water content increased, the soil particles developed larger water films around them. This increase in dry density continued till a stage was reached where water starts occupying the space that could have been occupied by the soil grains. Thus, the water at this stage hindered the closer packing of grains and reduces the dry unit weight. The maximum dry density (MDD) occurred at an optimum water content (OMC), and their values were obtained from the plot of data. On the contrary, MDD is not suitable for cohesionless soil/ sand and therefore other methods such as TMH1 11T was used. In addition, MDD has moderate repeatability and thus did not give exact answer, so the test were carried out strictly in accordance with the SANS 3001-GR20 procedure in order to get the most accurate results.

3.3.4. Determination of California Bearing Ratio of the soil

The California Bearing Ratio (CBR) of a material is determined by measuring the load required to allow a standard piston to penetrate the surface of a material compacted according to method used to determine the maximum dry density of the soil and its optimum moisture content. The CBR was carried out in accordance with the SANS 3001-GR40 and the determination of the CBR-density relationship and swell of the material was also included. It is in essence a simple penetration test developed to test and evaluate the strength of soil. The selection of this method was purely due to the fact that it is a cheap and accurate way of determining the strength of disturbed soil samples.

To determine the CBR of the in-situ soil, an empty mould was weighed and fixed to the base plate with a spacer disc and a filter paper and attached with a collar. The specimen was then added with water and compacted according to Standard proctor test or modified proctor test. After compaction, the collar was removed, and the surface was levelled with a cutting edge. Following this, the base plate was detached, and the spacer disc removed. The weight of the mould plus the compacted specimen was then taken and its bulk density was determined. Thereafter the sample was taken for moisture content determination and then dry density. After having determined the dry density, the filter paper was placed on a perforated base plate and the mould was fixed upside down to the base plate so that the surface of the specimen was turned upwards to allow the penetration test to be performed (for unsoaked conditions).

For soaked condition, the adjustable stem and the perforated plate are fixed on the compacted soil specimen in the mould along with a 2.5 kg surcharge load. The above set up was then placed in the soaking tank for four days. After four days, the swell reading was measured together with the % swell with the help of dial gauge reading. The mould was then removed from the tank and water was allowed to drain and the specimen was placed under the penetration piston and placed with total surcharge load of 4 kg (2.5 kg during soaking+ 1.5 kg during testing). The load and deformation gauges were set to zero and the load was applied to the plunger into the soil at a rate of 1.25 mm per minute to allow readings of the load to be taken at penetrations of 0.5, 1.0, 1.5, 2.0, 2.5, 4.0, 5.0, 7.5, 10.0 and 12.5 mm. The plunger was then removed, and the water content of the soil was determined. The results were then used to plot load versus deformation curve and the soil was classified as per TRH14 and COLTO specifications.

Despite its advantages, the method is very long and time consuming, therefore multiple people are required to perform the test to avoid fatigue and reading of wrong results. Furthermore, CBR values tend to increase with increased compaction and thus a marginally substandard CBR were improved by calling a higher than normal compaction and the use of intermediate categories were considered in areas where there was a shortage of suitable material. Since CBR increase with increasing compaction, it is also not suitable for cohesionless material (sand) and therefore other methods such as TMH1 11T were adopted.

3.3.5. Soil pH and electrical conductivity

Soil pH is a measure of the amount of acidity or alkalinity (basicity) that is present in soil. The reason for selection of this method was that it is cheap, accurate and the material were readily available in the laboratory. In this research, two types of pH were determined, namely; water pH (H_2OpH) and potassium Chloride pH (KpH). The water pH was determined as per TMH1 A20 by weighing approximately 100g of each sample and mixing it with at least 100ml of deionized water. The mixture was then stirred intermittently for 30 mins and let to stand for about an hour whilst pH buffers 4, 7, 9 and 10 were used to calibrate the pH bench top meter (Orion Star A215). The pH electrode was then immersed in each of the soil samples and sample analysis were performed in accordance to the TRH14 in duplicate with the results being recorded on a spread sheet.

The Potassium Chloride pH was determined by weighing 10 g of air-dried mineral soil (<2 mm) or 2 g of organic soil into a 30 mL beaker and adding it into 20 mL of 0:01 M CaCl2.the duplicate quality control samples were also included in the batch. The mixture was then intermittently stirred for 30 mins and left to stay for about an hour. The pH combination (Glass membrane and the porous salt bridge) electrode was immersed into the soil sample and the pH was recorded once the reading was constant.

The electrical conductivity (EC) was determined in the Laboratory as per TMH1 A21T by using a conducting cell by measuring the electrical resistance of a 1as to 5-water suspension. The 1:5-water suspension was prepared by weighing 10g of air-dried soil (<2mm) into a sample bottle. 50ml of deionized water was added and the mixture was mechanically shaken at 15rpm for at least an hour to dissolve soluble salts. The conductivity cell of an Orion Star A215 Conductivity Bench top meter was rinsed, and its temperature was adjusted to ± 20ºC. The Conductivity cell containing electrode was immersed in the sample. The observed readings were recorded in µS/cm. This was repeated twice by rinsing the electrode before sample analysis to ensure reliability of results and all the results were subsequently recorded on a spread sheet for interpretation in accordance with the TRH14 specifications.

3.3.6. Determination of the hydraulic properties of the soil

The hydraulic properties of the soil were determined through the permeability test. Permeability refers to the ease with which water can flow through the soil. The reason for selection of this method was purely based on the fact that the equipment was readily available in the laboratory and the fact that it is a very accurate method of measuring the hydraulic properties of the soil. In order to determine the permeability of the soil, the permeability test was performed on the collected soil through the constant head method. This is because all the soil collected from the study area were coarse grained and the constant head method is specifically suited for these type of materials.

In order to determine the permeability of the soil, the mass of the weighing pan was determined along with the dry mass of the soil. The cap and the upper chamber of the permeameter were removed and lifted off the tie rods and the inside diameter of upper and lower chambers were measured to allow calculation of the average diameter of the permeameter and a porous stone was placed on the inner support ring in the base of the chamber. Consequently, the soil was mixed with sufficient quantity of distilled water to prevent segregation of particle sizes during placement into the permeameter, poured into the lower chamber using a scoop in circular motion to fill it to a depth of 1.5 cm and then compacted using a tamping device in order to close up void spaces in the soil. The placement of the soil was continued until the level was about 2 cm and the upper porous stone was placed on it. Compression springs were then placed on the porous stone and the chamber cap and its sealing gasket were replaced.

After the placement of compression springs and a chamber cap, the sample lengths were measured at four locations around the circumference of the permeameter to allow calculation of the average length and the level of the funnel was adjusted to allow a constant water level in it to remain a few centimeters above the top of the soil. The flexible tube was then connected from the tail of the funnel to the bottom outlet of the permeameter and the valves on the top of the permeameter were kept open.

Subsequently, the tubing was placed into a sink from the top outlet to collect any water that came out and the bottom valve of the tube was opened to allow water to flow into the permeameter. As soon as water began flowing out of the top control valve, the bottom valve was closed, allowing water to flow out of the outlet for some time. Following this, the bottom outlet valve was closed and the tubing disconnected at the bottom and the funnel tubing was connected to the top side port and the bottom outlet valve was opened and the funnel raised to a convenient height to get a reasonable steady flow of water and allowed adequate time for the flow pattern to stabilize.

The amount of water coming out from the outlet valve was measured at 30 seconds interval for all the soil samples together with the temperature of the soil and the vertical distance between the funnel head level and the chamber outflow level. The soil was then removed from the chamber and poured into a weighing pan and then weighed on a scale balance to determine the weight after the test and the permeability of the soil was determined using equation 3.6.

$$K_T = \frac{Ql}{Ath} \qquad (3.6)$$

Where: K_T= Coefficient of permeability at temperature T in cm/sec. L= Length of the sample in centimeters, T= time for discharge in second, Q= Volume of discharge in cm^3, A= cross sectional area of the permeameter, and h= hydraulic head difference across length L in centimeters.

3.4. Quality Assurance of Data

Quality data assurance involves adoption of strategies during data collection and processing to reduce error and biasness and to ensure the validity, reliability, precision and the integrity of the data. In order to achieve this, reliable, accurate and proven methods were adopted in order to obtain information on the physical and engineering properties of the soil. For example, methods such as CBR, cone penetration and compaction have been in use since the 1930 while laboratory methods such as sieve analysis and atterberg limits have been in use since the 1960. Equipment such as magnetometer, resistivity meter and cone penetrometer used to collect information about the physical and engineering properties of the soil were properly calibrated using standard known measurements units. Soil and rock samples were collected randomly within the study area to ensure that they were representative of the soil in the study area and were collected in duplicates and were analyzed at both UNIVEN and NWU and the results were compared and an average of all these results were recorded. In addition, the determination of the atterberg limits and the permeability of the soil involved performing at least three trials and an average was taken. The determination of the CBR, grading modulus as well as the atterberg limits were performed by an independent and experienced laboratory (Sim lab) in order to ensure objectiveness and reliability of the data. Furthermore, all the softwares such as Strata and IP2Win are reliable, accurate and have been in use to process and analyze data for years.

CHAPTER FOUR: DATA PRESENTATION AND ANALYSIS

The main aim of this chapter was to present and interpret the data obtained using the methods and procedures described in Chapter 3. The data presented includes the physical, engineering, behavioral and hydraulic properties of the soils from the study area where the proposed solar plant is to be constructed.

4.1. Description of Site for Construction of Solar Facility

The surface area of the proposed site is approximately 1032 ha and the site is 53.23 km south of Kakamas town which can be easily accessed by existing farm roads. The terrain is generally flat and is underlain by the granites and gneisses of the Namaqua-Natal metamorphic provinces as well as quartz, calcretes and dorbanks in certain areas of the site. The soil is mainly sandy/ sandy silt, reddish-brown and dry and changes from aeolian to alluvial towards the streams. The streams in the area are largely dendritic and non-perennial and the surface conditions are dry with sparse vegetation, consisting mainly of dwarf shrubs intermixed with grasses succulents, geophytes, annual forbs and small trees along the drainage lines. The climate of the region can be classified as dry with very hot summers and very cold dry winters (Weinert climatic No.>10). Presently the site is being used for grazing goats and it appears that the site has been used in the past for small scale agriculture such as subsistence farming and crop rotation, as indicated by evidence of tilling and windmills. Soil and rocks are abundant on site and can be easily used as construction material or as base material or fill for foundation.

4.2. Surface and Subsurface Geology

The surface geology was determined through geological survey in order to enable determination of different rock types present of the surface in the study areas while the subsurface geology was established through magnetic survey in order to outline different rock types and structures beneath the surface at the site that in turn would influence the soil type, stability and the chemistry of the soil. In essence, this section presents and analyze the results of the geological and the magnetic survey so as to fully describe the geology of the study area. However, it is worth noting that both the geological and magnetic survey were only performed as a criteria for selection of sampling point and to correlate resistivity survey results.

4.2.1. Geological setting of the study site

The purpose of the geological survey was to determine the surface geology by identifying and mapping different rock units because it is very cheap and easy to perform, and the apparatus were readily available. The results of the geological survey are presented in Figure 4.1 which shows the surface geology of the proposed site. From the results, the study area is comprised of different rock types such as granites, gneisses, quartz, quartzite, pegmatite and calcrete and dorbank. All these rocks are igneous and metamorphic with an exception of calcrete and dorbank which are chemical sedimentary rocks. This suggests that, in general, the study area is dominated by igneous and metamorphic rocks. These results agree with the literate of Almond (2012) that this study area is characterized by the granites and gneisses of the Namaqua-natal metamorphic province and these rocks are igneous and metamorphic.

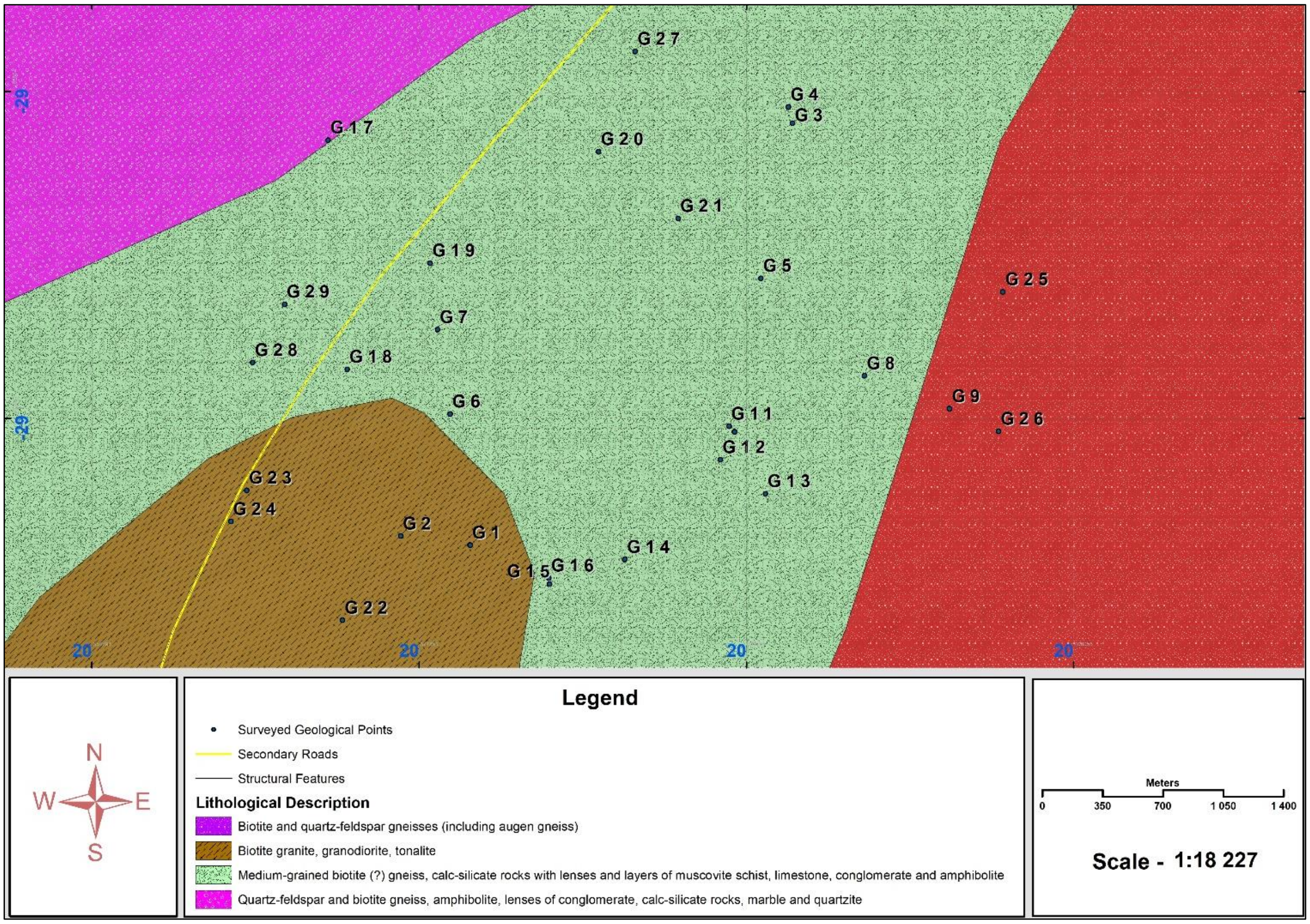


Figure 4.1: Geological setting of the study area.

4.2.2. Magnetic properties of subsurface formations in the study area

The aim of the magnetic survey was to investigate subsurface geology on the basis of the anomalies in the earth magnetic field because it is very accurate and the equipment was readily available in the laboratory. The results of the magnetic properties of the subsurface formations are presented in Figure 4.2. Figure 4.2a shows the traverses along which the readings of the magnetic survey were taken while Figure 4.2b shows the magnetic susceptibility values of different points in the study area.

The results showed that the area is characterized by different rock types with a magnetic value ranging from 26747.6 to 26575.5 nT which is typical of igneous and metamorphic rocks as stated by Murdock et al, (2013). Generally, sedimentary rocks have a very small magnetic susceptibility compared with igneous or metamorphic rocks, which tend to have a much higher magnetite (a common magnetic mineral) content. According to Hunt et al. (1995), sedimentary rocks tend to have very low magnetic susceptibility values of 0-25000 nT while igneous and metamorphic rocks are characterized by very high values which range from 0 to 270 000 nT. This agrees with the results of the study conducted by Kayode et al. (2013) in Nigeria which revealed that sedimentary rocks in the study area had a magnetic susceptibility of 80-330 nT which was very low as compared to that of the igneous and metamorphic rocks in the area which were 31490 nT and 34672 nT respectively. This suggest that the study area is comprised of igneous and metamorphic rocks and correlates accurately with the literature of Almond (2012) and the results of the geological survey that the study area is underlain by the granites and gneisses of the Namaqua-natal metamorphic province.

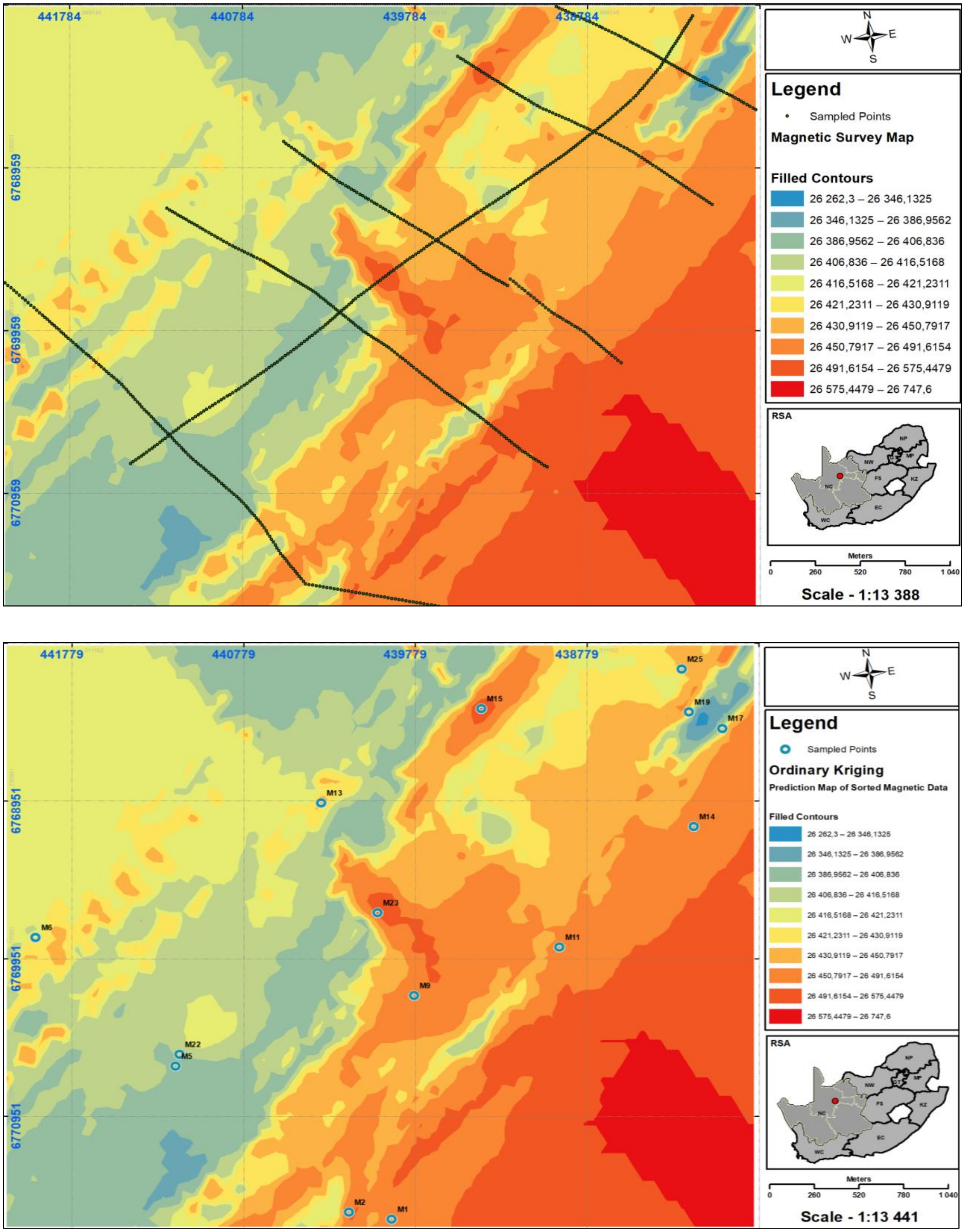


Figure 4.2: Magnetic properties of the subsurface formations within the study area.

4.3. Characteristics of the Subsurface Conditions

The main aim of establishing the characteristics of the subsurface condition was to allow for determination of structures and conditions that would have an impact on the suitability of the site for construction of the solar plant and to also assess their impact. This evaluation of the subsurface conditions was carried out through profile logging, resistivity surveys and cone penetration because they are cheap methods of investigation to shallow subsurface, allow visual inspection of the strata or soil layers and that the equipment was readily available in the laboratory. As such this section aims to present the results of the profile logs, resistivity survey and cone penetration.

4.3.1. Vertical profiles of the trial pits

The profile logs were described using the Moisture, Colour, Consistency, Structure, Soil type and Origin (MCCSSO) classification system because it is a cheap method of investigation to shallow depth and that it allows visual inspection of the strata or soil layers. The results of the soil profiling were then used to produce profile logs shown in Figure 4.3 which summarizes the properties of soil layers and classifies the soils in the profile logs in accordance with Fey's, (2010) South African Taxonomy. This classification system classifies soil into top soil and sub-soil based on horizons and diagnostic material, properties and genesis and the results are presented in Table 4.1. Based the profile logs, the soil in the study area had either two or three layers and all had an orthic top soil, as is typical of arid area soils as stated by Ahrens et al. (2003) in his study of the South African Arid areas and desert soils which revealed that arid area and desert soils mainly comprises of two or three layers and that they all have an Orthic A-horizon.

Based on the South African taxonomy results shown in Table 4.1, soils in the study area can be classified as either Glenrosa, Mispah, Coega, Dundee or Knersvlakte. In accordance with South African Taxonomy classification system, Fey (2010) stated that Glenrosa soils comprise of an orthic top soil and a Lithocutanic B subsoil and as can be seen from Table 4.1, their top soil is transported (alluvium/minor aeolian) and their subsoil is comprised of weathered and foliated bedrock of either gneiss, Quartz or Pegmatite.

Soils in this group included soils from profiles G5, G6, G8, G9, G11, G12, G16, G18, G19, G20, G21, G22, G23, G28, G29, M4, M10, M12, M13 and M23.

Table 4.1: Classification of the on-site soil according to Fey's South African Taxonomy.

Profile ID	Description of layers/horizons	Year of formation	SA Taxonomy
G14, G28, M5, M6, M14	Top soil comprises of either sandy, sandy silt or clay soil (Alluvium/minor Aeolian) and the subsoil consists of either pedogenic calcrete soft, calcrete powder or calcrete hardpan	Top soils are 7.5yr while the calcretes are 10yr	Coega/Bland vlei
G3	Topsoil consists of sandy silt transported soil (Alluvium/minor Aeolian) and the subsoil comprises of Dorbank (Silistic chemical sedimentary rock)	Both the top and subsoil are aged 7.5yr	Knersvlakte
G5, G6, G8, G9, G11, G12, G16, G18, G19, G20, G21, G22, G23, G28, G29, M4, M10, M12, M13, M23	Top soil comprises of sandy silt or silty clay or simply sandy or clay transported soils (mostly Alluvium with minor Aeolian or just alluvium and the subsoil compose of either moderately or highly and foliated weathered gneiss or moderately or slightly weathered pegmatite or quartz	Majority are 7.5yr with the exception of very few which are 10yr	Glenrosa
G1, G4, G7, G10, G13, G25, G26, G27, M1, M2, M3, M7, M8, M9, M11, M21, M24	Topsoil comprises of transported (Alluvium/minor Aeolian) sandy, sandy silt, silty clay or simply clay soil. The subsoil consists of either pedogenic calcrete powder, calcrete soft or calcrete hardpan while the bedrock comprises of either moderately or highly weathered and foliated gneiss, slightly weathered pegmatite, quartz or granite	Topsoil are 7.5yr while the calcretes are either 10 or 5yr	Mispah
G2	Topsoil comprises of transported (Alluvium) sandy soil while the subsoil comprises of silty clay colluvial soil	Both top and subsoil are 7.5yr	Dundee

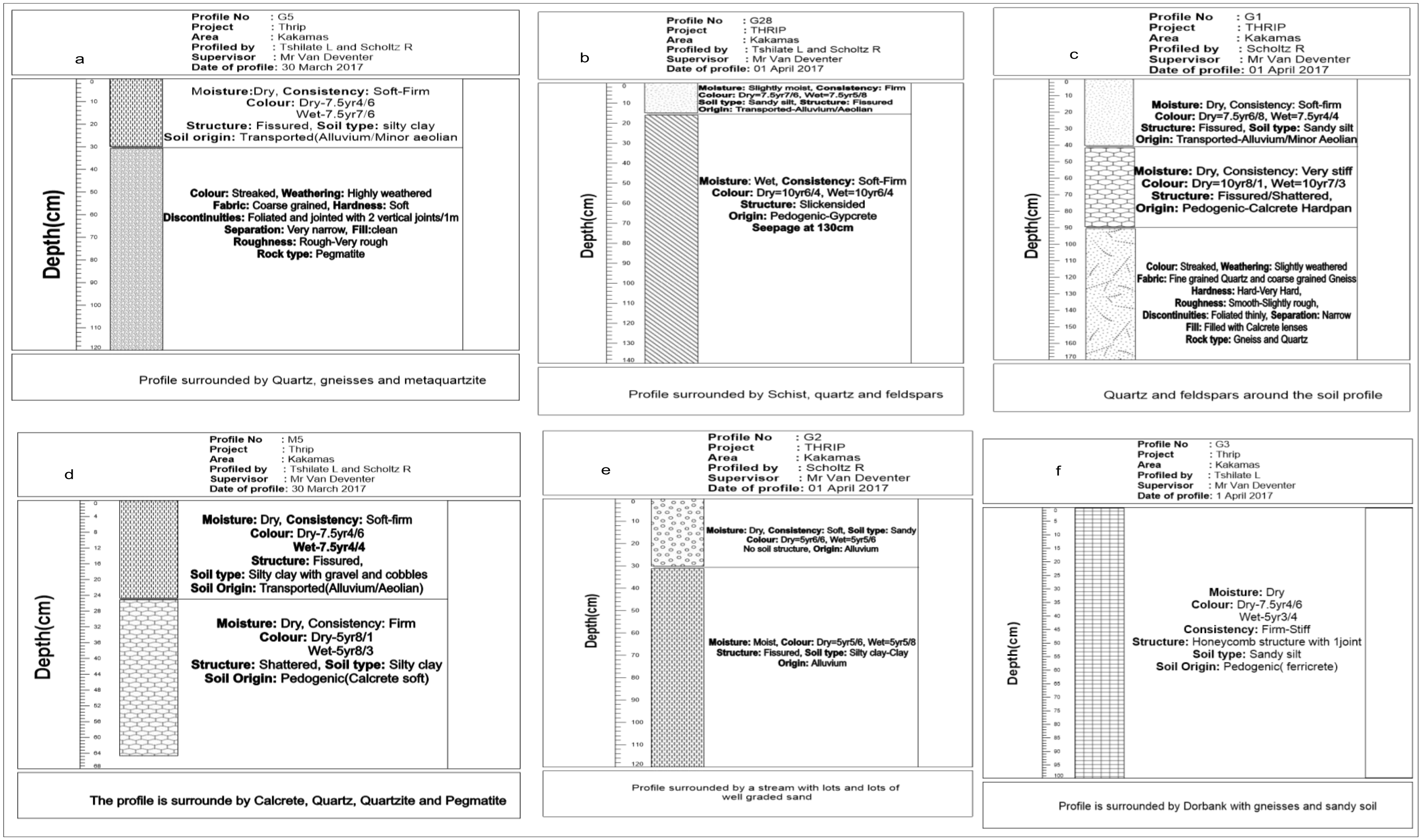


Figure 4.3: Soil Profile logs showing different soil types found in the study area a). Glenrosa b). Mispah c). Anomalous profile with water seepage d).Coega e).Dundee and F).Knersvlakte

Standing out from these profiles as can be seen in Figure 4.3a was profile G29, G22, G20, G18 and G5 with each having 15, 1, 7, 8 and 2 vertical joints/m respectively. Evidence of these joints is shown in Figure 4.4. Profile G28 had water seepage at 130 cm and is shown in Figure 4.5.

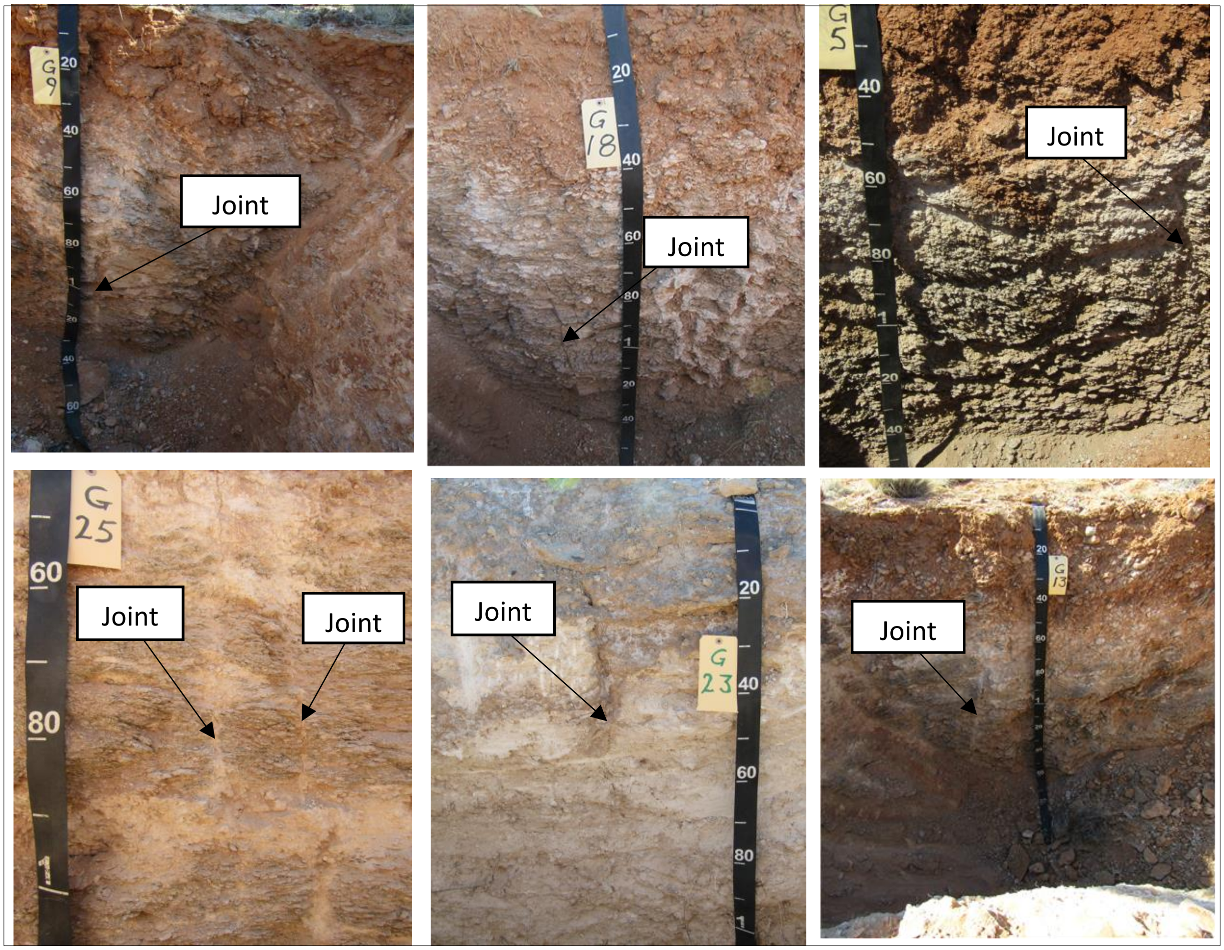


Figure 4.4: Evidence of joints in some of the profiles within the study area.

Figure 4.5: Profile G28 showing seepage of water/encounter with the water table at 1.3 m.

Mispah comprises of an orthic top soil, a calcrete/ferricrete/silcrete or dorbank subsoil and a hard rock bedrock. Soils in this group included soil from profiles G1, G4, G7, G10, G13, G25, G26, G27, M1, M2, M3, M7, M8, M9, M11, M21 and M24 and comprised of transported (Alluvium/minor Aeolian) topsoil, pedogenic sub soil and a bed rock made up of moderately or highly weathered and foliated gneiss or slightly weathered Quartz or pegmatite. Profiles M7, G27, G13 had 4, 1 and 13 vertical joints/m respectively which indicate area of potential failure/collapse in the soil. Coega consist of an orthic top soil and a hardpan carbonate bedrock. From the results, soils in this group comprised of transported(Alluvium/minor Aeolian) topsoil and pedogenic subsoil which could be of either calcrete soft, calcrete Powder, Honeycomb or calcrete hardpan and included soils from profiles G14, G28, M5, M6 and M14.

Dundee comprises of an orthic top soil and a stratified alluvium subsoil while Knersvlakte comprises of an orthic top soil and a chemical sedimentary bedrock (Dorbank). Belonging to Dundee group was soils from profile G2 which comprised of an alluvial top soil and a colluvial subsoil while profile G3 belonged to the Knersvlakte group and consisted of transported topsoil (alluvial/minor aeolian) and chemical sedimentary bed rock (Dorbank). In addition, this profile had 1 vertical joint/m indicating an area of potential failure in the soil.

Generally, the study area was dominated by soils from Glenrosa and Mispah group. These findings agrees with the literature of van der Waals (2011) that Kakamas area is dominated by Mispah and Glenrosa forms with minor occurrences of Coega, Dundee and Knersvlakte. Contrary to this, Dold et al., 2006 stated that the most dominating soil forms in the Northern Cape are Hutton and Mispah. According to Fey (2010), soils that belong to Glenrosa group are Lithic, while those from Mispah and Dundee are Lithic Cumulic. These soils are referred to as Young soils and consist of an orthic topsoil but weakly developed subsoil, as is typical of arid areas soil. Zhang et al. (2017) attributed this to the hot and dry climate which results in soils having a naturally loose structure, a shortage of aggregates and are susceptible to wind erosion, especially under tillage. As such these soils tend to be fairly weak and require reinforcement to be able to support high loads (Duo et al., 2018). In addition, the bedrock in majority of the profiles was highly weathered, foliated and jointed, therefore reducing the capacity of the top soil to support large loads (Turner, 2013).

Fey (2010) also classified soils from Coega and Knersvlakte group as calcic silicic, saying that these soils have special subsoil characteristics relating to pedogenic accumulation and having an orthic topsoil, the characteristics of these soft rocks are often obscured by their burial with detrital sediments. Furthermore, the carbonate material in these soils tends to be soluble, chemically reactive, and easily recrystallizable (Abubakar et al., 2014). Moreover, the formation of these materials has been reported to defy any satisfactory geologic, chemical or pedological definition (Al-Amoudi et al., 2002). Consequently, these soils are fairly weak (those underlain by calcrete soft or powder) with an exception of those that are underlain by Dorbank or calcrete hardpan which are extremely competent and can be able to support large loads without any reinforcement (Turner, 2013). However, it is worth noting that the strength of soils is also dependent on other properties of the soil as well.

4.3.2. The electrical resistivity properties of the site

The resistivity survey was conducted to map the subsurface features and to correlate the results of the profile logs largely because the method is non-disturbing, meaning that measurements were taken on undisturbed samples which are true representative of the on-site soil, field equipment is potable and inexpensive and detects both lateral and horizontal changes. In addition, the equipment was readily available and that it is relatively quiet simple and that it can also be used for ground water exploration.

Table 4.2 shows the electrical resistivity results of the soil. The Vertical Electrical Sounding (VES) graphs and pseudo sections produced from these are presented in Appendix-D. From the results, the area was generally characterized by two to three major geo-electric layers within the shallow subsurface depth of up to 43 m. These results correlate with those of profile logging which also showed that soil profiles consisted of either two or three layers. From these, fourteen soil profiles (G27, M1, M2, M22, M23, G2, G7, G23, G26, M11, M12, G12, G20 and G21) had two geo-electric layers whilst ten (G13, M14, M9, M8, M6, G25, G9, G5, G3 and M24) indicated three geo-electric layers.

Table 4.2: Electrical resistivity of the soils at the site.

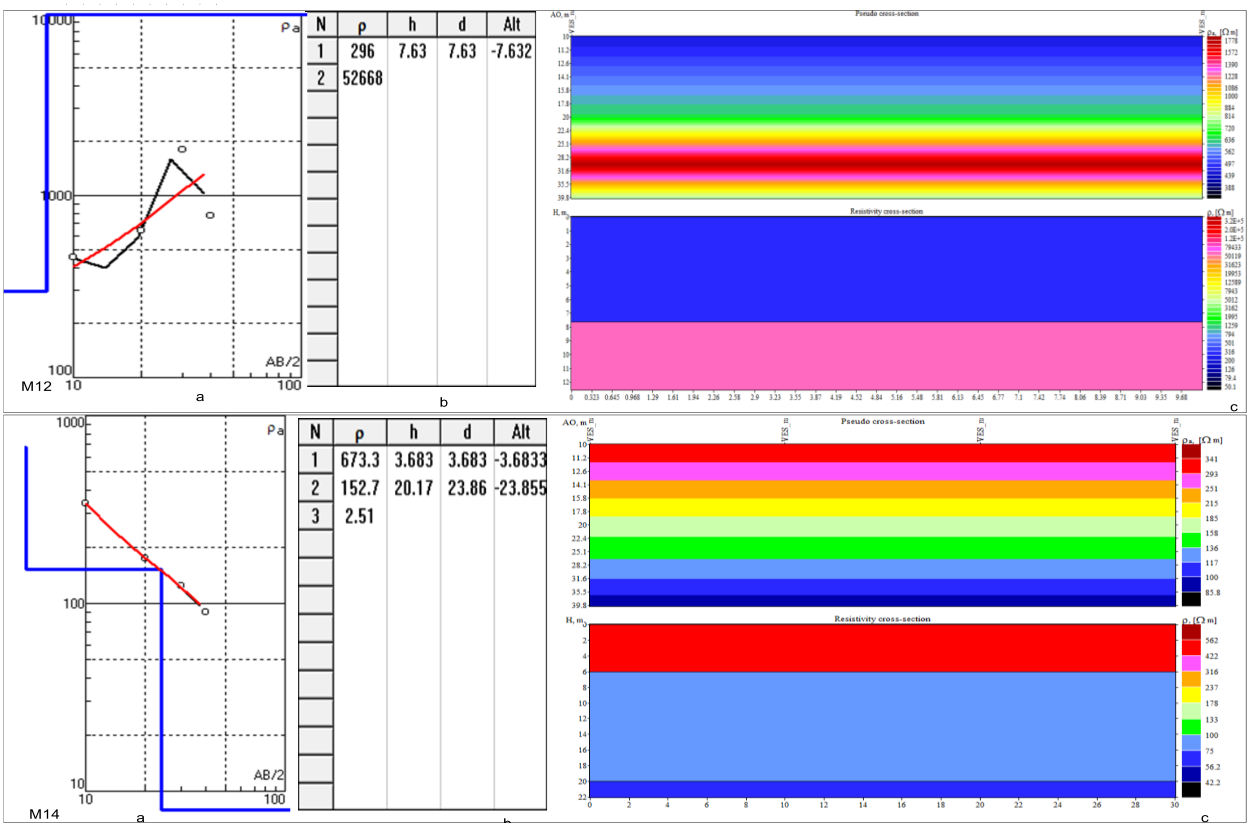
		Apparent Resistivity (Ωm)				Voltage (mV)/ Spacing				Electric Current (mA)/Spacing				Standard deviation/Spacing (%)			
	Pit ID	Spacing 10 = 5m depth	Spacing 20 = 10m depth	Spacing 30 = 15m depth	Spacing 40 = 20m depth	10	20	30	40	10	20	30	40	10	20	30	40
1	**G12**	1370.51	1173.03	1348.6	1188.86	2.02	13.45	9.01	8.05	2.02	5.52	6.93	12.20	0.0	0.0	0.0	0.0
2	**G13**	251.20	219.17	238.35	238.94	17.26	1.18	1.14	0.77	9.35	2.61	4.98	5.81	0.0	1.3	0.1	0.0
3	**G2**	10.37	16.98	27.85	46.73	3.50	1.32	0.69	0.32	46.02	37.56	25.73	12.61	0.0	0.1	0.5	0.3
4	**G20**	289.79	162.82	153.45	137.20	39.32	6.15	6.04	1.22	18.47	18.21	41.50	16.11	0.0	0.2	0.0	0.0
5	**G21**	668.87	514.60	485.37	427.45	20.64	10.13	5.29	4.57	4.20	9.49	11.37	15.63	0.0	0.0	0.0	0.0
6	**G23**	39.22	53.51	63.38	69.60	20.94	47.94	7.61	20.37	72.70	431.59	124.60	527.23	0.1	0.0	0.0	0.0
7	**G25**	607.88	622.60	687.60	754.36	24.80	39.82	8.52	10.01	5.55	30.81	12.84	23.92	0.0	0.0	0.0	0.0
8	**G27**	1261.80	2830.42	4354.61	6722.59	28.22	52.11	42.31	23.34	3.04	8.86	10.07	6.25	0.1	0.0	0.0	0.0
9	**G26**	49.26	81.01	133.12	165.07	16.32	5.61	2.92	4.16	45.12	33.39	22.79	45.46	0.0	0.0	0.0	0.0
10	**G3**	280.07	397.29	472.92	588.57	124.02	40.89	20.04	3.28	60.54	49.57	43.93	110.04	0.0	0.0	0.0	0.0
11	**G5**	102.79	125.34	202.77	224.31	16.09	1.29	3.13	0.91	21.32	7.39	16.03	7.33	0.0	0.0	0.0	0.0
12	**G6**	94.83	189.70	318.84	258.82	26.73	22.65	7.41	1.21	38.38	57.52	24.12	8.46	0.0	0.1	0.0	0.0
13	**G7**	295.74	469.67	696.01	887.40	30.74	16.49	22.23	17.74	14.15	16.92	33.11	36.01	0.0	0.0	0.0	0.0
14	**G9**	534.54	635.62	677.83	777.84	35.28	21.79	21.53	8.49	6.22	16.51	32.93	19.68	0.0	0.0	0.0	0.0
15	**M1**	6714.00	1131.59	1419.94	1507.95	60.11	28.69	19.36	23.11	12.18	12.21	14.13	27.61	0.0	0.0	0.0	0.0
16	**M11**	61.59	51.09	41.68	36.25	0.95	0.80	0.96	2.50	2.11	7.56	23.91	24.44	1.7	0.7	0.2	0.2
17	**M12**	454.77	641.94	1793.01	772.85	28.93	4.51	3.84	3.43	8.66	3.38	2.22	8.01	0.0	0.3	1.0	8.2
18	**M14**	342.48	173.18	123.71	90.14	14.57	2.07	0.88	0.41	5.79	5.74	7.37	8.29	0.0	0.0	0.0	0.2
19	**M2**	133.68	198.54	289.13	373.17	37.99	24.90	2.62	9.18	38.69	60.43	9.42	44.32	0.0	0.0	0.0	0.0
20	**M22**	526.60	611.62	728.60	758.21	24.18	24.80	5.97	17.66	6.25	19.53	8.50	41.96	0.0	0.0	0.0	0.0
21	**M23**	415.82	583.80	829.83	861.99	38.51	14.70	9.80	12.74	12.60	12.13	12.24	26.62	0.0	0.0	0.0	2.6
22	**M24**	137.78	121.60	135.23	188.17	5.53	0.83	2.72	1.05	5.46	3.29	20.87	10.05	0.0	0.0	0.0	0.1
23	**M6**	112.99	121.83	108.66	243.46	11.52	7.84	7.72	1.90	13.89	31.03	47.51	14.08	0.0	0.3	0.0	0.4
24	**M8**	362.81	349.57	307.02	380.05	21.63	9.08	5.45	2.72	8.11	12.52	18.43	12.91	0.1	0.0	0.0	0.1
25	**M9**	228.95	148.28	239.62	194.66	50.02	5.61	4.57	1.58	29.74	18.23	19.78	14.65	0.0	0.1	0.1	0.2

Generally, for three layered profiles, the resistivity was decreasing-increasing with depth. This means that the resistivity of the first layer was greater than that of the second layer which was less than that of the third layer (i.e. $\rho1>\rho2<\rho3$). These included the resistivity of profiles G13, M9, M8, G25, G5 and M24 and as a result these profiles manifested an H-type of curve as stated by Zohdy et al. (1990). According to Raju et al. (2012), this type of sounding curve are not normally in hard rock formations and it consists of dry top soil of high resistivity as the first layer, water saturated weathered layer of minimum resistivity as second layer and compact hard with very high resistivity as the third layer. On the other hand, the resistivity of profiles G9 and G3 was increasing with depth (i.e., p1<p2<p3) and according to Zohdy et al, (1990) manifested an A-type of curve. Raju et al. (2012) stated that these types of curves are obtained in hard rocks with conductive topsoils. These results are somewhat similar to those found by Balasubramanian et al. (2012) in Tamil Nadu, India in his study of Vertical Electrical Sounding for Ground water Prospecting In Uppodai Tambaraparani river which indicated that the soil consisted of three geo-electric layers and that they manifested A, K, AK and KH Curve types.

The two layered geo-profiles manifested two types of curves namely Ascending /p2>p1(M1, M2, M23, G2, G7, G23, G26 and M12) or Descending/p2<p1 (G21, G20, G12, M11, M22 and G27) respectively where in the p2>p1 represent unconsolidated overburden laying on a bed rock and p2<p1 typified a poor conducting alluvium laying over a better conducting sand or a clay formation which is usually indicative of igneous or metamorphic rock of very high resistivity (>1000 Ohm-m) (Telford et al., 1990).

The anomalous stations M12 and M14 as presented in Figure 4.6 had very resistive second layer at depth >7.63 m and conductive third layer at depth >27.54 m respectively. The first zone had a depth ranging from 1.65 to 25.3 m surface layer with low to moderate resistivity values. The second zone had variable resistivity values ranging from 37.4 to 52668 Ωm and was fairly thick. The third zone for eight stations was highly resistive with resistivity values ranging from 280-32054 Ωm whilst, one station, M14 had a very conductive anomaly of 2.51 Ωm on the third zone to infinity depth>27.54 m. Electrical conductivity at this depth could be due to water saturation on a highly saline bedrock (Kaboli et al., 2011).

Six stations (G23, G26, G2, G6, G27 and M11) had resistivity <100 with depth ranging from 3.34 to 5.57 m and could be due to water saturated sandy soil and gravels. Geotechnical logging of these pits at these stations reported moist soils of up to 1.8 m depth. Twelve station (G3, G5, G7, G8, G13, M2, M6, M8, M12, M22, M23 and M24) had moderate resistivity of 100-500 Ωm with depth ranging from 2.72 to 25.3 m and typifying the presence of weathered bedrock material with variable moisture content as revealed by Akpila et al., (2012). Seven stations (G12, G20, G21, G25, G27, M9 and M14) had resistivity of 500-1000 Ωm with a maximum depth of 11 m and this high resistivity could be due to unweathered gneiss or quartzite bedrock. Figure 4.6 shows the Vertical Electrical Sounding of the soil profiles, the properties of the profile as well as the resistivity and pseudo-sections of the soil profiles.



***Note:** N=number of layers p= apparent resistivity h= thickness of the layer d= depth

Figure 4.6: Vertical Electrical Sounding of the anomalous stations with pseudo and resistivity sections.

Six stations (G23, G26, G2, G6, G27 and M11) had resistivity <100 Ωm with depth ranging from 3.34 to 5.57 m and could be due to water saturated sandy soil and gravels. Kaboli et al., (2011) showcased this in his study of Vertical Electrical Sounding Resistivity Survey technique to Explore Groundwater in an Arid Region, Southeast Iraq which indicated lowest resistivity values were observed in saturated zones and that all the saturated alluvium sandy layers had resistivity of less than 100 Ωm. Geotechnical logging of these pits at these stations reported moist soils of up to 1.8 m depth.

Twelve station (G3, G5, G7, G8, G13, M2, M6, M8, M12, M22, M23 and M24) had moderate resistivity of 100-500 Ωm with depth ranging from 2.72 to 25.3 m and typifying the presence of weathered bedrock material with variable moisture content as revealed by Kaboli et al. (2011). Seven stations (G12, G20, G21, G25, G27, M9 and M14) had resistivity of 500-1000 Ωm with a maximum depth of 11m and this high resistivity could be due to unweathered gneiss or quartzite bedrock.

The layers with infinite depth had resistivity values which ranged from 1035 to 52668 Ωm. This could be due to fresh basement of resistive quartzite and gneiss lenses as discussed in section 4.3.1. Clasts of these lithologies were observed in most of the pits excavated at the site during geotechnical studies. The lithology at the three stations was basically the same, consisting mainly of three types of soil namely sand, sandy silty and gravel. Variation in the layer resistivity is attributed to grain size variations (medium to coarse), and degree of water saturation.

4.3.3. Consistency and stiffness of the soils within the study area

The cone penetration was conducted in order to establish the stiffness and the consistency of the subsoil underlying the site at shallow to moderate depth because it is very quick, fairly non-destructive and that it is the only on-site method that can determine strength on undisturbed soil sample. Figure 4.7 and Table 4.3 show the soil's cone penetration results with depth. Generally, as it can be seen from Table 4.3, there was a constant increase in the number of blows with depth. This gave an indication that soils in shallow profiles are less stiff since they had small number of blows.

Table 4.3: The DCP results of the selected soil profiles in the study area.

	DPSH 13	**DPSH 14**	**DPSH 15**	**DPSH 16**	**DPSH 17**	**DPSH 18**	**DPSH 19**	**DPSH 20**	**DPSH 21**	**DPSH 22**	**DPSH 23**	**DPSH 24**
Depth	***M 11***	***M 8***	***G 6***	***M 12***	***M 24***	***G 13***	***G 12***	***M 23***	***M 9***	***G 2***	***G 23***	***G 24***
0.3	14	6	8	11	6	12	21	13	12	10	11	17
0.6	33	18	13	22	13	20	50	31	17	10	50	23
0.9	55	34	23	42	21	31	Solid	50	17	15	Solid	23
1.2	Solid	30	31	50	27	44		Solid	32	37		31
1.5		21	42	Solid	39	50			50	45		48
1.8		43	50		50	Solid			Solid	50		50
2.1		50	Solid		Solid					Solid		Solid
2.4		Solid										

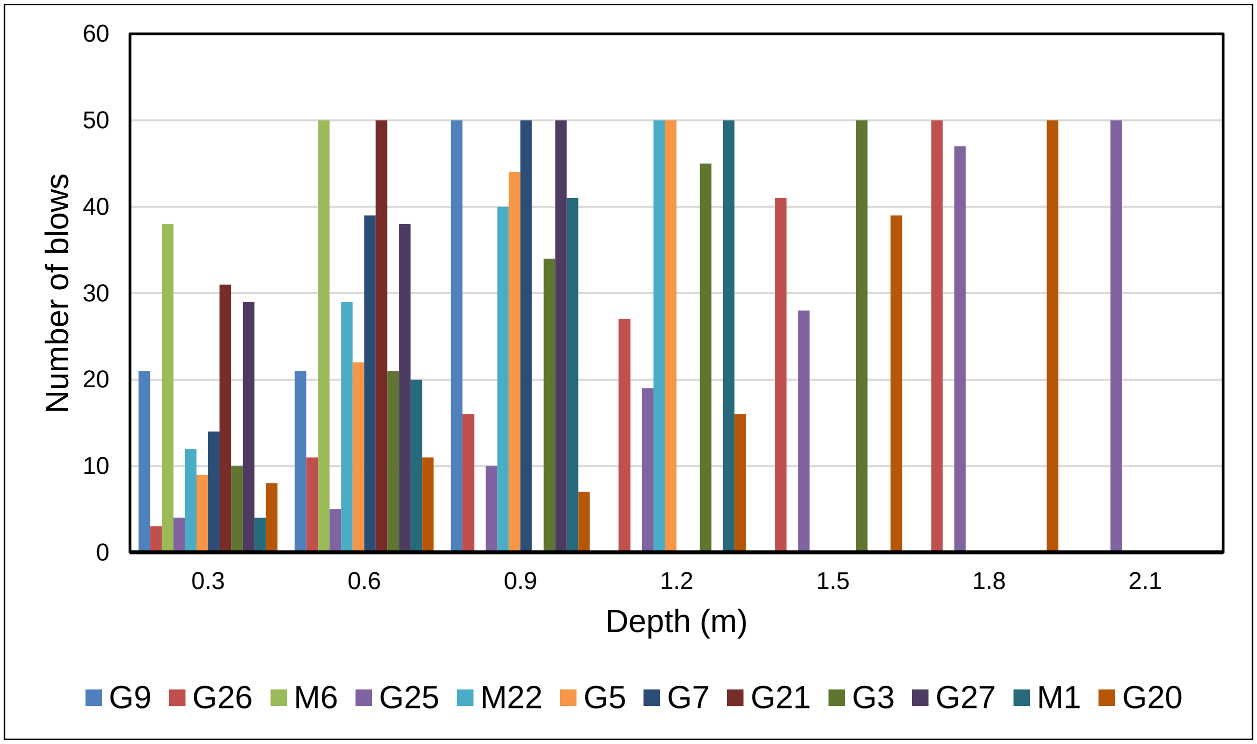


Figure 4.7: Consistency of the soils at the proposed site.

The results showed that the soil with the highest number of blows was soil from profile M8. This soil required 202 blows in order to reach the bed rock. As such it was considered to be having the greatest stiffness and this can be attributed to the fact that it had the greatest depth of 2,4m. Soils in profiles G24, G2, M9, M23, G13, M24, G6, and M12 were fairly stiff with 192, 167, 128, 157, 157, 167 and 125 number of blows respectively. This means that these materials had moderate resistance to penetration. However it is worth noting that although all these materials are considered to be of moderate stiffness, the individual number of blows were different due to the differences in depth. Soil in profiles M11, M23 and G12 had low number of blows of 102, 94 and 71 number of blows respectively. The least number of blows came from the soil in G23 which only required 23 number of blows to reach the solid, impenetrable rock. Consequently, the profile was very shallow, and the soil was considered to have very little resistance to penetration.

Generally the number of blow increase with the stiffness of the soil (Rogers, 2010).This means that soil layers with high number of blows were considered to be very stiff while those with the least number of blows were considered to be fairly loose. According to SMEC SA (2013), cohesive soils with between 5 and 12.5 number of blows are regarded as soft and those that have between 12.5 and 20 number of blows are considered as firm. Furthermore, those that have 20-38 number of blows are stiff while those with 38-75 number of blows are considered to be very stiff. Based on this, soil layers with 5-12.5 number of blows were regarded as very soft while those with 12.5-20 number of blows were regarded as firm. In addition, those with 20-38 number of blows were considered stiff while those with between 38 and 75 number of blows were regarded to be very stiff. In general, the results indicated that the soils in the study area were moderately firm to stiff, as is typical for DCP test (SMEC SA, 2016).

In essence, the in-situ cone penetration indicated highly variable soil consistency with no general pattern emerging from the results. These results are identical to those observed by SMEC SA, (2013) in Lime Acres Northern Cape for Olien PV Solar Plant which indicated highly variable soil consistency with no general pattern emerging from the results. Likewise, Outeniqua Geotechnical Services, (2015) revealed that their cone penetration results at Nieubethesda in the Eastern Cape for a proposed subsidy housing project also indicated a highly variable soil consistency with no general pattern emerging from the test. Contrary to this, the cone penetration carried out for Sibaya Bulk Services by SMEC SA, (2016) indicated medium dense soil with a coefficient of variation equal to 40% as is typical for DCP test.

In general, the greater the number of blows, the greater the resistance to penetration and the greater the stiffness and the strength of the soil. The greater the depth, the greater the resistance to penetration (stiffness), number of blows and the soil's strength. Consequently, soils with great depth have the highest resistance to penetration and number of blows and thus strength while shallow soils have least resistance to penetration, number of blows and strength, as was typical of the result. Therefore, they require some form of backfilling of support to be able to carry large weight.

4.4. Laboratory Test Results

The main purpose of this section is to present and analyze all the results that were obtained from the laboratory in order to determine the engineering and behavioral properties of the soil. The laboratory test conducted in order to determine the engineering and behavioral properties of the soil at the proposed site included sieve analysis and atterberg limits/foundation indicator, optimum moisture content and maximum dry density, CBR, pH and electrical conductivity.

4.4.1. Particle size distribution

The particle size distribution was determined through sieve analysis which involved pouring soil onto a stack of sieves which were then placed on the mechanical shaker and then weighed to determine the percentage retained on each sieve. The individual results of the sieve analysis results are presented in Appendix-E and were used to determine the various particle sizes present in the collected soil samples. Soil samples were collected from every soil horizon and laboratory analysis was conducted using mechanical sieving in order to determine the engineering and behavioral properties of the soil. This is because the particle size present, together with their shapes affect the swelling and linear shrinkage of the soil (Das, 2010). The results of the sieve analysis were then used to produce gradational curves for each soil layer as shown in Figure 4.8.

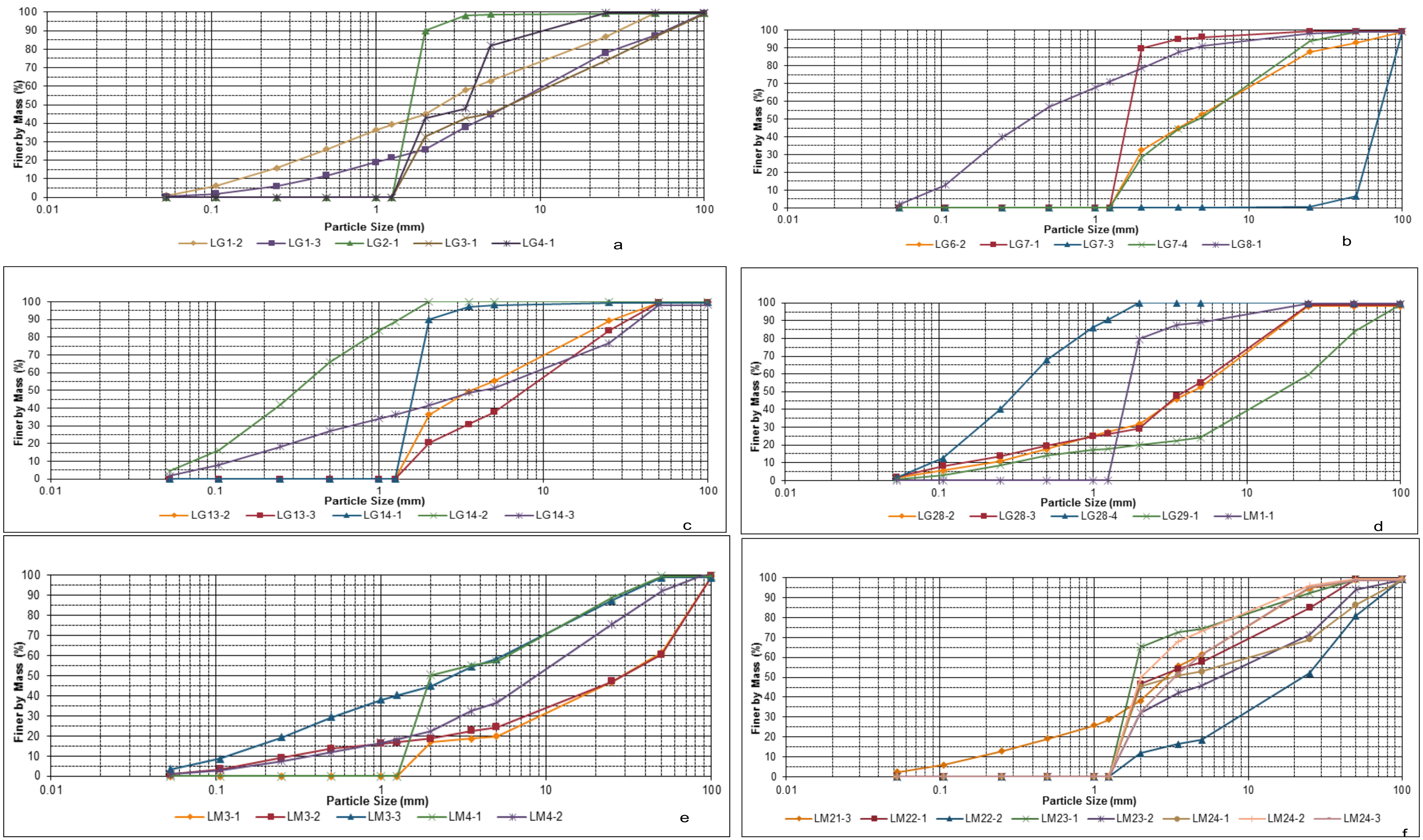


Figure 4.8: Particle distribution curves of selected soil horizon within the study area.

Based on the results, the majority of the soils in all layers contained a significant amount of sand and gravel sized material with no clay while very few contained only sand and clay sized particles. Consequently, the sieve sizes used ranged between 100- 0.053 mm. However, as can be seen in the gradational curves in Figure 4.8, the majority of the soils only had particles ranging from 100- 2 mm with an exception of soils from LG1-2, LG1-2, LG4-2, LG8-1, LG14-2, LG14-3, LG22-2, LG28-2, LG28-3, LG29-1, LM2-2, LM2-3, LM3-2, LM3-3, LM4-2, LM5-2, LM7-2, LM10-2, LM13-2, LM14-2, LM16-2, LM18-2, LM21-2 and LM21-3 which had particle sizes ranging from 100- 0.053mm. Contrary to these, were soils from LG14-2 and LG28-4 as shown in Figures 4.8c and 4.8d, which only had particles ranging from 1.25- 0.053 mm, typifying medium sized to fine graded particles as stated by Das (2010).

According to the USCS (2004), the soils in the study area can be generally classified as GP (poorly graded gravel-sand mixture with very little or no fine), SP (Poorly graded sands/sands with little or no fines) and SM-SC which is silty/clayey sands with very low plasticity. The foundation indicator test performed by Outeniqua Geotechnical Service (2015) in Eastern Cape for a proposed subsidy housing project in Nieubethesda revealed very similar results to these ones stating that the soils in this areas could be classified Unified Soil Classification System (USCS) as CL (Inorganic clays of low to medium plasticity), SM-SC (silty/ clayey sands with low plasticity) and SP which is poorly graded sands or sands with little or no fines. However, it is worth noting that the proposed study area only contains SM-SC (silty/ clayey sands with low plasticity) and SP (poorly graded sands or sands with little or no fines) and no CL (Inorganic clays of low to medium plasticity).

Contrary to this, the foundation indicator test performed by HC Petersen and Associates, (2002) at the Coves Hartebeespoort dam indicated that the aeolian sands classified mainly as SM soil type, these being silty sands and poorly graded sand-silt material. Similarly, SMEC (2016) used a particle size distribution/ foundation indicator test for the Sibaya Bulk Service Project which showed that the majority of the site soil consisted of clayey and silt sands of negligible low plasticity denoted as SP-SM and SM-SC in terms of the USCS.

Based on all these findings, it is evident that the soils in the study area are coarse-grained with very little or no fine-grained material in them. To complement this, the soil profile logs revealed that the soils in the study area were of alluvial with minor aeolian origin and this is typical of soils around Kakamas town as expressed by Almond, (2012). The gradational curves of all the soils from the study area manifested poorly-gap graded curves. This is because there was not much variation in their particle sizes as can be seen in their gradational curves. Hence, they only contained certain particle sizes and not all of them. Therefore, all the soils in the study area are likely to behave very similarly. The coefficient of curvature and uniformity as presented in Appendix-C ranged between 0-12.7 and 0-0.8 respectively. This is typical of coarse-grained sandy soil as stated by Das (2004).

The grading moduli of the majority of the profiles as presented in Table 4.4 was > 2 indicating the coarse-grained nature of material of relatively good quality as stipulated by the South African National Road Agency (2013). However, profiles G2-2, G3, G26-A and G28 had grading moduli of 1.35, 1.56, 1.30 and 1.33 respectively, indicating medium coarse-grained material of moderate quality as stated in the technical recommendations for highways (South African Road Agency, 2013). According to the TRH14 material specifications, the soils on site can be generally classified as G6 because 2.6 ≥ GM ≥ 1.2.

Table 4.4: The Grading Modulus (GM) of the selected soil profiles in the study area.

Pit ID	**G2-2**	**G3**	**G12**	**G20**	**G23**	**G26**	**G26-1**	**G27-1**	**G28**	**G29**	**M1**	**M9**	**M14**	**M23**
GM	1.35	1.56	2.28	2.26	1.67	2.27	1.30	2.37	1.33	2.12	2.14	2.06	2.15	2.10

These results are very different from the results obtained by HC Petersen and Associates, (2002) for the Coves Hartebeespoort dam which showcased a highly variable grading modulus which ranged from 0.3 to 1.16. Furthermore, the grading test revealed that the grading modulus of sand averaged 0.93 in the range of 0.89 and 0.99 which reflected its fairly fine nature. However, the grading modulus of calcretes ranged from 2.18 and 2.29 indicating its relatively coarse-grained nature, just like the majority of sands and calcretes in the proposed study area. Subsequently, the grading test performed by SMEC (2016) for Sibaya Bulk Service Project revealed a grading modulus of <1 for all the material on site. According to the South African Road Agency, these materials are fine grained and are of poor quality for construction purposes.

4.4.2. Atterberg limits of the soil

Representative samples of various soil horizons were collected for Foundation Indicator tests in order to estimate potential expansivity and evaluate their suitability as founding mediums and the results are presented in Table 4.5. Based on the results, the liquid limit of the soil was very low (25-50) which corresponded very well with the findings of Du et al. (2018) that the soils in arid areas are very dry. The correlation of these results with the profile logs and in-situ moisture content also revealed that the soils in this area are very dry.

The individual linear shrinkage of different soil layers as outlined in Appendix-G indicated that the average linear shrinkage in different layers ranged from 0-9.69% which is regarded as low as per van der Merwe method of assessing potential expansiveness of the soil. Figure 4.9 showed that the statistical variation in linear shrinkage in different soil layers was extremely significant ($p < 0.0001$) in accordance to ANOVA method. The results presented in Table 4.5 however indicated that the linear shrinkage of selected soil profiles ranged from 0.9-25% which is regarded as low to medium in accordance with the van der Merwe method (1964). Consequently, the soil in the study area has low potential expansiveness.

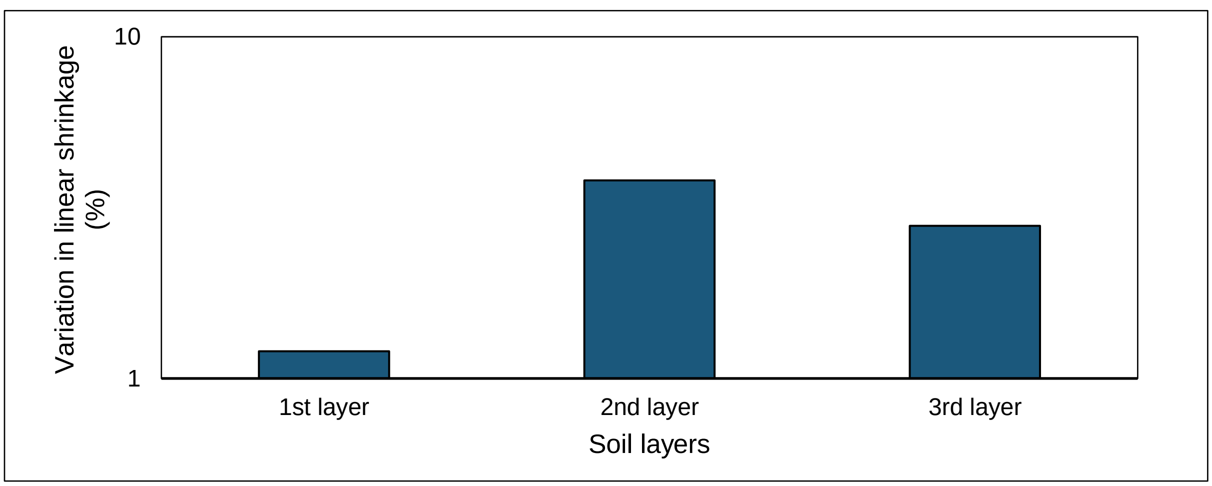


Figure 4.9: Statistical variation of linear shrinkage with soil horizons.

Table 4.5: Summary of the atterberg limits of selected soil profiles/horizons within the study area.

Pit ID	Material Type	Atterberg limits			PE[1]	USC[2]
		PI	LL	LS		
G2-2	Sand	9	29	4.5	Low	SP
G3	Sand	SP	-	1.8	Medium	SP
G12	Sand with quartzite and calcrete	4	26	2	Low	GP
G20	Sand with gneiss and calcrete	5	25	25	Low	GP
G23	Sand with calcrete and gneiss	SP	-	1.5	Low	GP
G26	Sand with gneiss	5	32	25	Low	GP
G26-A	Sand with gneiss	SP	-	0.9	Medium	GP
G27-1	Sand with granite gneiss	4	27	20	Low	GP
G28	Sand with calcrete and gneiss	23	50	11.5	Low	GP
G29	Sand with granite and gneiss	7	26	3.5	Low	GP
M1	Sand with calcrete	5	29	2.5	Low	GP
M9	Sand with calcrete	SP	-	1.5	Low	GP
M14	Sand with gneiss	10	30	5	Low	GP
M23	Sand with gneiss	6	25	3	Low	GP

***Note:** 1. Potential expansion, 2. Unified Soil Classification

The tests also confirmed that the soils are dominantly poorly graded sands with gravel grained sands and silts, although silty clay soils were also encountered. The plasticity index of the soil was low to medium, but the clay content is generally low. This can be seen in Table 4.5 which shows that the plasticity index of the soils was very low, and the majority of the soils were considered to be Non-Plastic (NP). However, some slightly plastic soils (G3, G23 and G26-A) were encountered due to the presence of clay in the soil.

The geotechnical investigation for Medupi Power Station also revealed non-plastic and slightly plastic silty sands (SM), poorly graded sands-silt mixes (SP), gravels or gravel-sand-clay mixtures and poorly graded silty gravels or gravel-sand mixtures (GP and GM), indicating that it had little plastic fines in it (Inroad Consulting, 2009). The Plasticity indices of calcretes ranged widely from slightly plastic up to 20, which is typical of calcretes. Similar results were obtained from the Foundation Indicator Test performed by Outeniqua Geotechnical Services (2015) at Nieubethesda which revealed that the soils had low liquid limit (0-39) and linear shrinkage (0.5-9.5). The plasticity index was also low to medium and the clay content was generally low. The tests however, confirmed that the soils in that area were dominantly fine-grained sands and silts, although some gravelly soils were also encountered.

Similarly, the Foundation Indicator Tests performed by SMEC, (2016) for Sibaya Bulk Service Project showed that the majority of the on-site soil consist of clayey and silty sands of negligible to low plasticity, denoted as SP – SM and SM – SC in terms of the Unified Soil Classification System (USCS). The potential expansiveness according to van der Merwe (1964) for such materials was negligible. However, test pits WM08 – WM10 indicated medium to high expansiveness potential for the silty clay residual soils of the Ecca Group and the corresponding USCS classifications ranged from CH to CL and SC with moderate to high plasticity index (PI) values of 14 – 24.

4.4.3. Moisture-density relationship and CBR

Thirteen representative soil samples were collected for a Mod/CBR/Indicator tests in order to determine the moisture-density relationship, compaction and CBR properties for road subgrade purposes. The results of the tests are summarized in Table 4.6. The determination of moisture/density relationship showed dense to very dense granular soils with the maximum dry density ranging from 1925 kg/m^3 to 2200 kg/m^3. Consequently, all these materials had no swell. The corresponding optimum moisture content indicated very dry to slightly moist soils (7-12.6) and the CBR showed that the soils in the study area is dominated by G6 material as per TRH14 material specifications.

Table 4.6: Summary of the results of the compaction and CBR test.

Pit ID	Opt Moist (%)	MDD (Kg/m³)	CBR (%)			AASHTO CBR at					TRH14 classification
			AASHTO	NRB	Proctor	90%	93%	95%	98%	100%	
G2	11.9	1940	45	15	5	5	10	15	29	45	**G8**
G3	9.6	2014	41	25	15	15	20	25	33	41	**G6**
G12	8.2	2002	66	20	6	6	12	20	41	66	**G8**
G20	8.8	2061	55	36	24	24	31	36	47	55	**G6**
G26	10.1	2026	47	37	29	29	33	37	43	47	**G6**
G26/A	7.6	2120	66	24	9	9	16	24	44	66	**G7**
G27	7.0	2064	67	53	42	42	48	53	61	67	**G6**
G28	12.6	1925	31	19	12	12	16	19	25	31	**No class**
G29	6.8	2121	84	41	20	20	31	41	63	84	**G6**
M1	8.1	2152	59	32	17	17	25	32	46	59	**G6**
M9	7.4	2200	53	44	36	36	40	44	49	53	**G6**
M14	7.0	2070	49	40	32	32	37	40	45	49	**G6**
M23	8.5	2067	68	41	25	25	34	41	56	68	**G6**

Based on the results, soils profiles M9 and M1 had the highest maximum dry densities of 2200 and 2152 kg/m^3 respectively at optimum moisture content of 7.4 and 8.1 and were regarded as very dense as per Jennings et al, (1973) classification of granular soils. Likewise, all these soils had no swell, meaning that the potential for expansion is very small/negligible as also shown by the atterberg limit results in Table 4.6. The California Bearing ratio of these soils were 53%AASHTO, 44%NBR and 36% Proctor and 59% AASHTO, 32%NBR and 17% Proctor which indicated that both these materials classify as G6 in accordance with the TRH14 materials specification. This coincides with the results of the particle size distribution which showed that the soils on-site classifies as G6 because of their grading Modulus (2.6≥GM≥1.2).

The soil in profile G28 had a maximum dry density of 1925 kg/m^3 at the highest optimum moisture content of 12.6 and was regarded as dense. The highest optimum moisture content of the soil in this profile could be attributed to the water seepage at 1.3 m as shown in Figure 4.5. The CBR of this profile was 31%AASHTO, 19%NBR and 12% Proctor indicating material with no class in accordance to the TRH 14 specifications. Soils in other profiles (G3, G20, G26, G27, G29, M14 and M23) had a maximum Dry Density greater than 1925 and were regarded as very dense granular soils and their CBR indicated that they classify as G6 material with the exception of soils from profiles G2, G12 and G26-A whose CBR classifies them as G8, G8 and G7 respectively. Their moisture content indicated dry to slightly moist soils with their optimum moisture content ranging from 7-11.9%.

The AASHTO CBR for the selected soil samples was determined at 90%, 93%, 95%, 98% and 100% and the results are presented in Figure 4.10. Figure 4.10a show different CBR readings for the selected soil samples and from these results, it can be seen that there is a constant decrease in the CBR reading from AASHTO to the Standard proctor mode. Figure 4.10b on the other hand shows the AASHTO CBR of the selected soils. From these results, it can be observed that there is a constant increase in AASHTO CBR with time (increase in percentage). Standing out with the highest CBR reading was profile G29 with AASHTO CBR reading of 84%, NRB CRB of 40%and Proctor CBR of 20%. This means that this profile has the highest load bearing capacity.

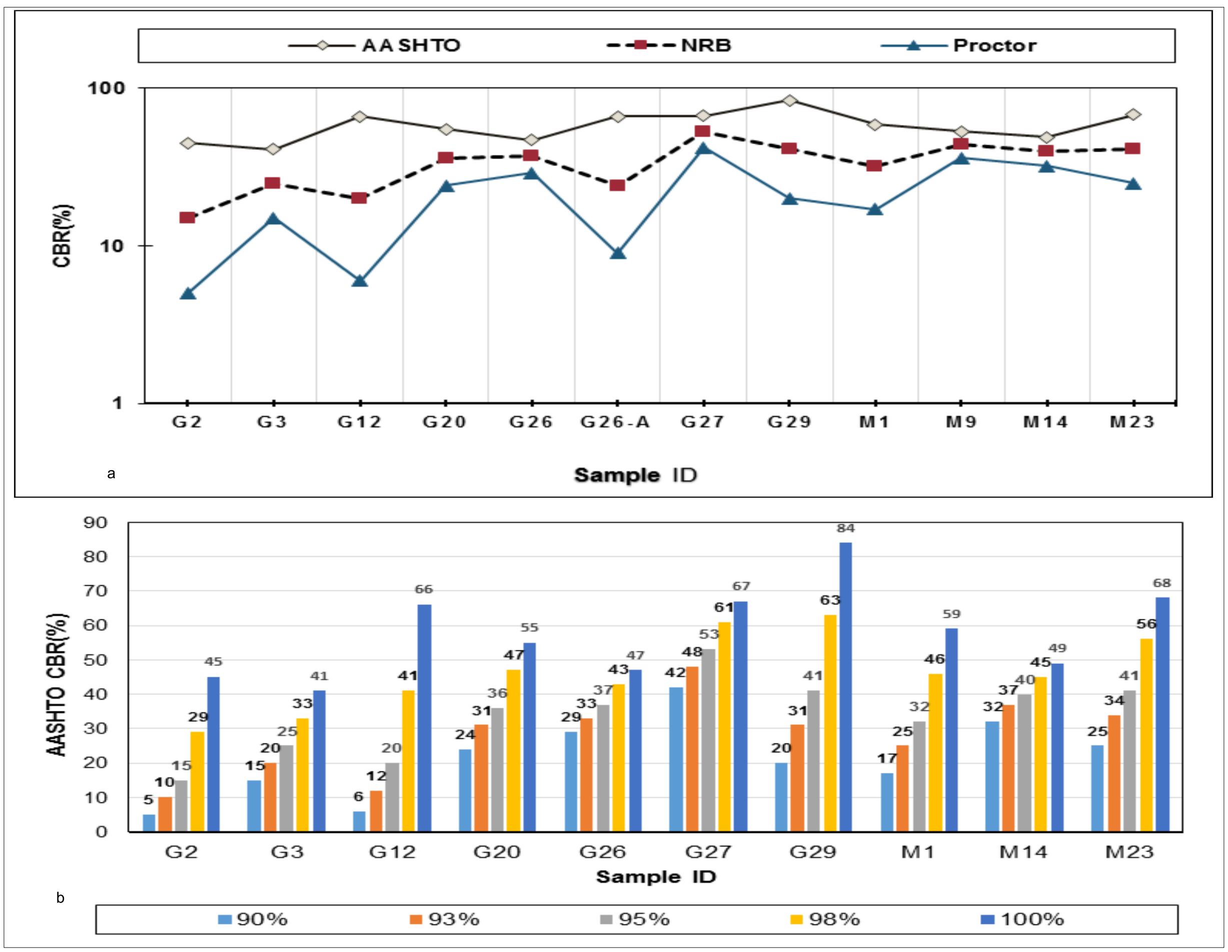


Figure 4.10: The California Bearing Ratio of selected soil samples on site a). Different CBR readings b). AASHTO CBR readings at different percentages.

Following this was soil in profile M23 with an AASHTO CBR of 68%, NRB CBR of 41% and a Proctor CBR of 25%. Profiles G27 and G12 also had high CBRs readings 67% and 66% for AASHTO CBR, 53%and 20% NRB CBR and 42% and 6% Proctor CBR respectively. Moderate CBR were obtained from soils in profiles M1and G20 with an AASHTO reading of 59%and 55%, NRB reading of 32%and 36%and a proctor reading of 17%and 24% respectively. Low CBR values came from M14, G26 and G2. These soils had 49%, 47%and 45% AASHTO CBR. The corresponding NRB CBRs were 40%, 37% and 15% respectively while the Proctor CBRs were 32%, 29% and 5% respectively. The least CBR reading came from G3 with 41%AASHTO CBR, 25%NRB CBR and 15% proctor CBR. This means that this is the profile with the least load bearing capacity.

These results are very different from those obtained from the CBR test performed by SMEC SA (2015) for Lime Acres Solar Plant in Northern Cape which indicated that the insitu roadbed was generally free drained and had poor quality in terms of the CBR. Subsequently SMEC SA (2016) carried out a CBR test on the Berea formation and recent aeolian materials of Sibaya Bulk Service Project and deduced that the materials may be classified as G7-G8 in terms of the COLTO classification while the grading moduli were determined to be less than 1 unlike the soils/ material in this study area.

4.4.4. Aggressiveness of the soil to objects

The pH of the soil and the electrical conductivity were measured in order to establish the aggressiveness of the soil which can affect buried objects such as cables, metal fittings and foundation structures. The pH of the soil was determined by weighing approximately 100g of soil and mixing it with at least 100ml of deionized water, then stirring intermittently for 30 mins and let to stand for about an hour and then pH electrode was then immersing a pH electrode into the soil sample to take measurements. The electrical conductivity on the other hand was determined in the laboratory by weighing 10g of air-dried soil into a sample bottle and mixing it with 50ml of deionized water and then mechanically shaking for an hour to dissolve soluble salts and then immersing a conductivity cell into the mixture to take measurements. The selection of these methods was entirely based on the fact that the equipment was readily available in the lab and that both these methods are very cheap and accurate.

The pH of the soil shows the acidity and alkalinity of the soil while the Electrical Conductivity shows the presence of ions in the soil to conduct electricity. For this research, two kinds of pH were determined, namely water pH and Potassium chloride pH and the results are presented in Table 4.7 which show the variation in pH with soil layers. Generally, the H_2O pH is greater than the KpH by 1 and from the results these soils are no exception. The results showed that the soils pH were quite high with pH values ranging from 7.77 to 9.59, showing very neutral to basic conditions. The corresponding KpH values were lesser than these by 1 with pH values ranging from 7.71 to 8.48.

Table 4.7: Variation of pH with soil layers and influence on the corrosiveness of the soil.

Pit ID and horizon		**pH(H_2O)**	**Potential corrosiveness**
1st horizon	**G2, G3, G4, G5, G6, G7, G8, G9, G10, G11, G12, G13, G14, G16, G18, G19, G20, G21, G22, G23, G25, G26, G27, G28, M1, M2, M3, M4, M5, M6, M7, M9, M10, M11, M12, M13, M16, M18, M21, M22, M23, M24**	7-9	Non-corrosive
2nd horizon	**G2, G5,G7, G9, G12, G13, G14, G18, G20, G21, G25, G26, G27, G28, M1, M6, M9, M11, M12, M22, M23, M24**		
3rd horizon	**G13, G25, G26, M1, M9, LM11, M24**		
G1-1, G20-1 and M14-1		9-10	Slightly corrosive

According to NSW Agriculture (2000) these values are regarded as fairly neutral with the exception of the H values >9 which are slightly corrosive and alkaline and may results in some plant nutrients being unavailable. Similarly, the pH test performed by SMEC (2013) and (2016) for Olien PV Solar Plant in Northern Cape and for Sibaya Bulk Service Project also indicated neutral soil conditions with a pH range of 6.30-6.83 and 6.8-7.3 respectively. However, the in-situ soil corrosivity testing conducted by EGA Consultants, (2011) for Proposed Mount Signal Solar Farm and Associated structures presented results which correlated with the field resistivity survey in showing that the soils were considered to be severely corrosive toward ferrous metals. In 2015, Outeniqua Geotechnical Services performed soil pH test which also indicated mildly soil conditions for the proposed subsidy housing project at Nieubethesda in the Eastern Cape with pH ranging from 8.66-9.38.

The electrical conductivity of the soils on the other hand was generally highly variable (16mS/m-4620mS/m) with no pattern emerging from the results as can be seen in Table 4.8. From the results, the most conductive layer was G2-2 with an EC of 4620 mS/m indicating highly corrosive conditions as indicated in Table 4.8. This means that ions are really abundant in this layer and that it is highly conductive and would have detrimental impacts of structures and buried objects and metal fittings. Also, with exceptionally high EC were layers M2-1, G2-2, G23-1 and G28-1 with EC values of 3640 mS/m, 2840 mS/m, 2041 mS/m and 1866 mS/m respectively. These results of the electrical conductivity typify highly conductive and corrosive conditions that would have severe impact on buried objects and structures as stated by Outeniqua Geotechnical Services, (2015). Layer G6-2 had an EC on 636 mS/m, indicating corrosive soil condition while thirteen layers (G3-1, G13-3, G18-2, G20-1, G20-2, G26-3, M3-1, M11-1, M11-2, M12-2, M21-1, M24-2, and G25-3) had moderate EC values ranging 129 mS/m- 492 mS/m and these show slightly corrosive conditions.

Table 4.8: Variation of soil's Electrical Conductivity (EC) with layers and influence on soil corrosiveness.

Pit ID	**Conductivity (mS/m)**	**Potential Corrosiveness**
G2-1, G4-1, G5-1, G6-1, G7-1, G8-1, G9-1, G10-1, G11-1, G12-1, G13-1, G14-1, G16-1, G18-1, G19-1, G21-1, G22-1, G25-1, G26-1, G27-1, M1-1, M4-1, M5-1, M6-1, M7-1, M9-1, M10-1, M13-1, M16-1, M18-1, M22-1, M23-1, M24-1, G5-2, G7-2, G9-2, G12-2, G13-2, G14-2, G21-2, G25-2, G26-2, G27-2, G28-2, M1-2, M6-2, M9-2, M22-2, M23-2, G26-3, M1-3, M9-3, M11-3, M24-3, G1-1, G20-1 AND M14-1	<100	Non-corrosive
G3-1, G13-3, G18-2, G20-1, G20-2, G26-3, M3-1, M11-1, M11-2, M12-2, M21-1, M24-2, and G25-3	100- 500	Slightly corrosive
G6-2	500- 1000	Corrosive
M2-1, G2-2, G23-1 and G28-1	>1000	Highly corrosive

The majority of layers had very low EC values. These values ranged from 20 mS/m-81 mS/m which indicate non- corrosive conditions that would have no or very little impact on structures and buried objects. The least EC values came from G2-1 AND M7-1 with EC values of 16 mS/m and 19 mS/m respectively. This means that the majority of the soils would have no impact on structures and foundation. However, special considerations and selection of material would have to be made for all the material with EC values > 500 mS/m.

The pH and electrical conductivity results for this study area are similar to those expressed by SMEC (2016) for Sibaya Bulk Serve Project which revealed neutral and non-corrosive conditions with a typical range of EC between 0.03-0.07 mS/m. Nevertheless, the conductivity test for Olien PV Solar Plant showed the soils to be corrosive towards steel (SMEC, 2013) while that of the proposed subsidy Housing project in Nieubethesda indicated mildly soil conditions with EC range of 216-568 mS/m indicating slightly corrosive to corrosive soils (Outeniqua Geotechnical Services, 2015). In addition, the in-situ soil corrosivity testing conducted by EGA Consultants (2011) for Proposed Mount Signal Solar Farm and Associated structures presented results which correlated with the field resistivity survey in showing that the soils were considered to be severely corrosive toward ferrous metals.

4.5. Determination of Hydraulic Properties of the Soil

Permeability refers to the ease with which water can flow through a soil. The hydraulic properties of the soil at the proposed site were determined through a constant head permeability test. Constant head permeability test involves flow of water through a column of cylindrilcal soil sample under a constant pressure difference and the reason for selection of this method was purely based on the fact the soil at the proposed site is coarse grained and the fact that the equipment was readily available in the laboratory. Consequently, a permeability test was performed on the five different types of material identified in the study area as shown in Table 4.9 which include sand, sand with calcrete, sand with granite, sand with calcrete and gneiss. Figure 4.11 shows the variation of the permeability of different soil types with time.

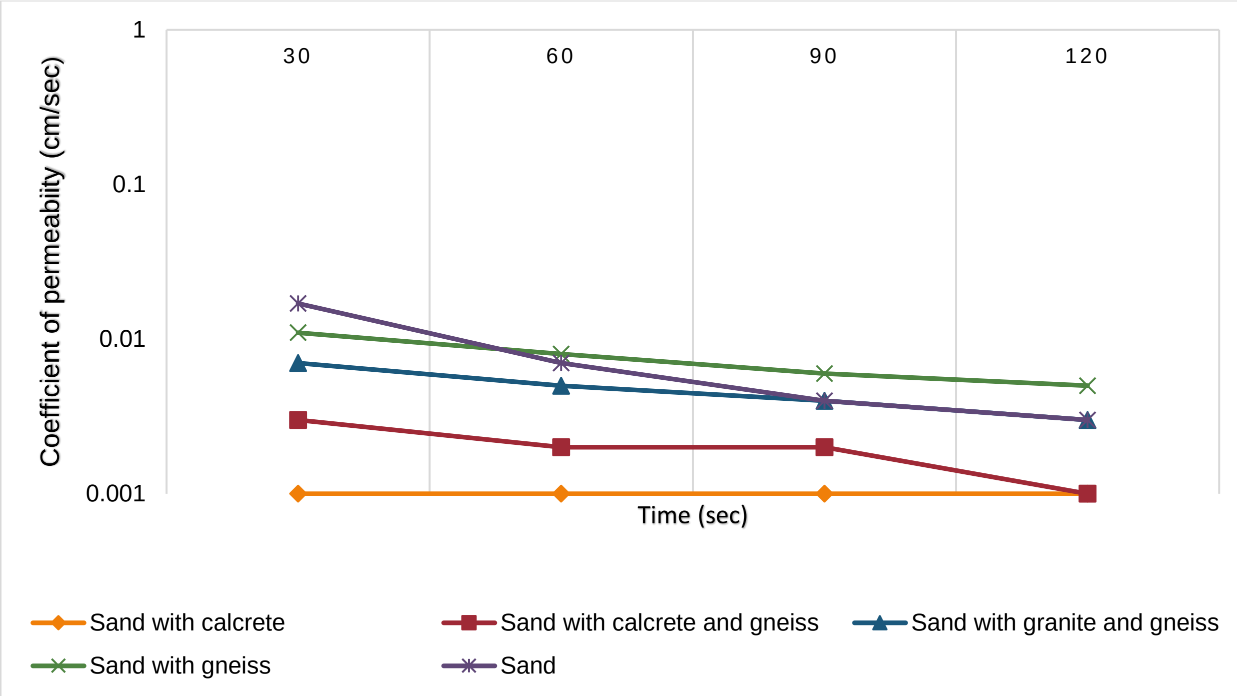


Figure 4.11: Variation of the permeability of different soil types within study area with time.

Based on these results, the permeability of sand with calcrete was constant with time. The other materials however showed no distinct patters because their permeability kept fluctuating with time. The coefficients of permeability of the different materials as indicated in Table 4.9 ranged from $1x10^{-3}$ to $8x10^{-3}$ cm/sec, typifying fine and silty sands as stated by Ishbashi and Hazarika, (2011).

Table 4.9: Average coefficients of permeability of different soil types in the study area.

Identified material	Coefficient of permeability (cm/sec)	Relative permeability
Sand	$8x10^{-3}$	Medium
Sand with calcrete	$1x10^{-3}$	Low
Sand with gneiss	$8x10^{-3}$	Medium
Sand with calcrete and gneiss	$2x10^{-3}$	Low
Sand with granite and gneiss	$5x10^{-3}$	Medium

According to Roy et al. (2017), these materials have medium permeability and can therefore be able to transmit water though at an intermediate rate. However, the permeability of sand with calcrete together with that of sand with calcrete and gneiss was quite low despite having coefficients of permeability which typify materials with medium permeability. This can be attributed to the calcrete within the soil which is a fine-grained carbonate material which has low permeability and tends to dissolve in water (Paige, 2002). These results are very different to those obtained by EGA (2011) for the proposed Mount Signal Solar Farm which indicated low permeable clayey soils and ground water at 3- 5 m depth and correlate with the results of the particle size distribution and grading modulus that the material at the site is sandy and sandy silty.

4.6. Limitations of Geotechnical Site Investigation

This research has been conducted in accordance with generally accepted engineering practice, and the opinions and conclusions expressed are made in good faith based on the information at hand at the time of the investigation. Therefore, the contents of the research are valid as of the date of preparation. However, changes in the condition of the site can occur over time as a result of either natural processes or human activity. In addition, advancements in the practice of geotechnical engineering and changes in applicable practice codes may affect the validity of this research. Consequently, the findings and recommendations of this research should not be relied upon after an eclipsed period of one year without a review by specialist or a competent person.

Unless otherwise stated, the investigation did not include any specialist studies, including but not limited to the evaluation or assessment of any potential environmental hazards or permeability test or groundwater contamination that may be present. The investigation was conducted within the constraints of the budget and time and therefore limited information was available and although the confidence in the information is reasonably high, some variation in the geotechnical conditions should be expected during and after construction. The investigation comprised a limited number of trial pits and is not likely to reveal the detail of the conditions that will become evident during construction.

The nature and extent of variations across the site may not become evident until construction. As such, if variations then become apparent this could affect the proposed project, and it may be necessary to re-evaluate recommendations in this research. It is thus imperative that a Competent Person inspects all excavations to ensure that conditions at variance with those predicted do not occur and to undertake an interpretation of the facts supplied in this research. Therefore, it is advised that the same people who performed this site investigation be retained to provide specialist geotechnical engineering services during construction in order to observe compliance with the design concepts, specifications and recommendations and to allow design changes in the event that subsurface conditions differ from those anticipated prior to the start of construction. Any significant deviation from the expected geotechnical conditions would be further investigated and corrected.

The assessment and interpretation of the geotechnical information were made with reasonably high confidence, but no specific calculations were made for foundation loads and design structures due time and budget constraints, therefore all the foundation recommendations were provided in good faith based on theoretical knowledge. Hence, a suitable study on foundations and foundation loads specifically suited for this site should be carried out.

CHAPTER FIVE: FOUNDATION ANALYSIS AND DESIGN

The main aim of this chapter is to provide insight about foundation types and geometries that should be considered for the proposed site as well as assess the suitability of the onsite material to be used as construction or fill material. In essence it provides information about foundation types, foundation design and loads, material excavatability and material utilization potential.

5.1. Types of Foundation for PV Solar Facility

There are several types of foundation available for PV solar plants. According to Brearly et al (2015), foundation options for ground-mounted PV systems include driven piles, earth screws, pre-cast concrete ballasts, concrete piers cast in-situ and bolted steel baseplates. The selection of these foundation types is influenced by the topographic conditions of the site, the geology, soil properties and the cost of the foundation (IEA, 2015).

Driven piles are support beams, usually made of steel, that are driven into the ground at pre-determined depth (Smalley, 2014). The superstructure of the rack and panels is then attached to those beams and the size and the length of the beams are determined by site conditions and array configuration (ARUP, 2013). Driven piles are suitable where soils are firm and compacted, with enough fine-grained material such as silt or clay to offer high skin friction and provide good lateral and vertical bearing (SMEC SA, 2013). These however, are problematic in soils that resist installation. These include soils with very coarse gravel or rock fragments, very hard soils or bedrock. They are typically constrained to slopes less than15° (Brearly et al., 2015). If a geotechnical survey proves suitable, a structural steel profile driven into the ground can result in low-cost, large-scale installations that can be quickly implemented. (IFC, 2015).

An earth screw is steel post shape with threads welded onto or machine into it to create a large screw and can adapt to a wider range of soil and site conditions (Smiley, 2014). Earth screws are typically hot dip galvanized with corrosion resistant zinc and are usually installed after predrilling holes into a rock or ledge and then screwing them into the holes with bobcats, excavators or other equipment using auger attachments. They can be installed in rocky soils and even bedrock if pilot holes are predrilled and are feasible on slopes up to roughly 30° (Brearly et al., 2015). Helical anchors are ideal for sites with poor soil cohesion such as clean sand or weak saturated soil, but they are not well suited to hard soils and soils with very coarse gravel or rock fragments. Helical earth screws typically made of steel have good economics for large-scale installations and are tolerant to uneven or sloping terrain (IFC, 2015). These require specialist skills and machinery to install.

A ballast system uses man made foundation to hold the rock and panel in place and usually has two vertical posts connected to a single concrete block approximately1.5m x1.5m x 6m (Smiley, 2014). Pre-cast or pour-in-place concrete ballast foundations best suit sites where soil penetration is undesirable or impractical due to rocky outcrops or subsurface obstacles and has low tolerance to uneven or sloping terrain, but requires no specialist skills for installation (IFC, 2015). This is a common choice for manufacturers with large economies of scale because it is suitable even at places where the ground is difficult to penetrate due to rocky outcrops or subsurface obstacles (Brearly et al., 2015). Consideration must be given to the risk of soil movement or erosion (IFC, 2015).

Concrete piers cast in-situ foundations are most suited to small systems and have high tolerance to uneven and sloping terrain (IFC, 2015). They do not have large economies of scale, but are best suited for shallow profiles where the ground is difficult to penetrate due to rocky outcrops (Brearly et al., 2015). Bolted steel baseplates are applicable in situations where the solar plant is located over suitable existing concrete ground slabs, such as disused airfield runway strips, a steel baseplate solution bolted directly to the existing ground slabs may be appropriate (IFC, 2015). Figure 5.1 shows the different foundation options for PV solar facilities. Figure 5.1 shows different foundation options for PV solar facility while Table 5.1 compares and contrast these different foundation types based on different selection criteria.

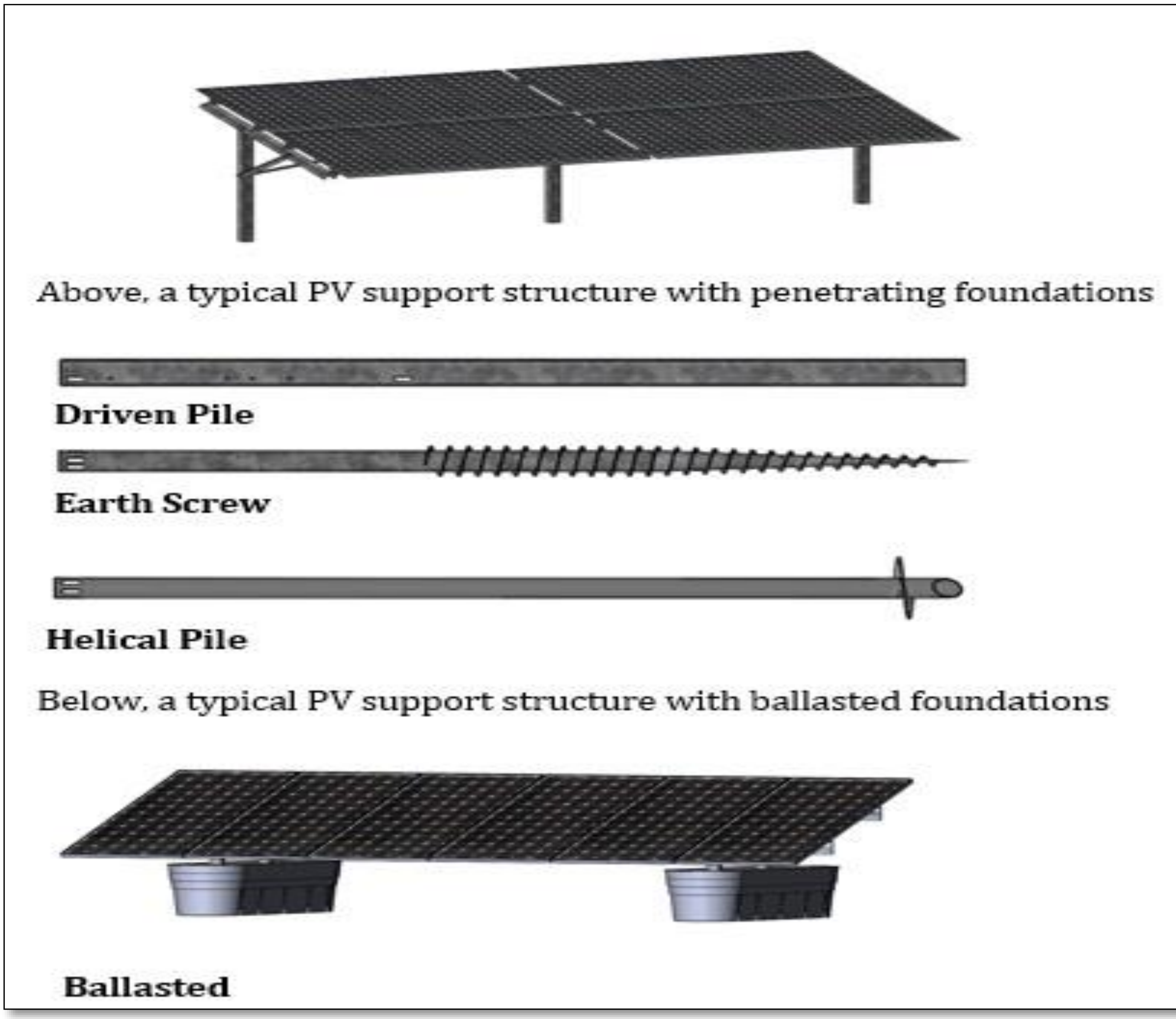


Figure 5.1: Different foundation types for PV solar facility (Smalley, 2014).

Table 5.1: Comparison of different foundation types for PV solar facility (Smalley, 2014).

Criteria		**Precast concrete**	**Driven piles**	**Poured concrete**	**Earth screws**	**Concrete piers**	**Ballast**
Project management	Geological study	Not necessary, but recommended	Necessary	Not necessary, but recommended	Necessary	Necessary	Not necessary, but recommended
	On site test	Not necessary	Necessary	Not necessary	Not necessary	Necessary	Not necessary
	Complexity	Low	Medium	High	Medium	Medium	Low
Installation	Site preparation	Yes	No site prep	Yes	No site prep	No site prep	yes
	Process	Pre-fabricates simple setup	Easy installation using a pile driver	Extensive site prep and casting	Easy installation using small equipment	Extensive site prep and casting	Foundation built onsite
	Equipment	no special equipment necessary onsite	Pile driver	Concrete pump	Auger and special attachment	Auger	No special equipment necessary onsite
	Speed	fast setup	1MW/week	Time consuming	Easy and fast	Time consuming	Medium speed
Physics	Uplift	Possible if footing design is wrong	High uplift resistance	High uplift resistance	High uplift resistance	High uplift resistance	Possible if footing design is wrong
	Sliding	Subgrade very important to avoid sliding	Not possible	Not possible	Not possible	Not possible	Subgrade very important to avoid sliding

‘Table 5.1, continued’

	Bearing pressure	Needs to be observed-setting possible	N/A	Needs to be observed	N/A	N/A	Needs to be observed-setting possible
	overturning	Possible if footing design is wrong	Only if system totally fails	Only if system totally fails	Only if system totally fails	Only if system totally fails	Possible if footing design is wrong
Environment	Reneutralization	Compacted soil	Perfect for environmentally sensitive areas	Excavation of foundation and remediation	Perfect for environmentally sensitive areas	Excavation of foundation and remediation	Compacted soil
	Recyclable	Maybe	Yes	No	Yes	Maybe	Yes
	Soil penetration	No	Medium	No	Yes	Yes	No
	Carbon footprint	High	Medium	High	Medium	High	Low
Applications		Brownfields, high water tables, over bedrock, on top of landfills	Solar farms, any application where soil penetration is possible		Solar farms, any application where soil penetration is possible	Solar farms, any application where soil penetration is possible	Brownfields, high water tables, over bedrock, on top of landfills
Overall cost performance		Low	Low	High	Low	Medium	Medium
Summary		Good solution for specific applications	Very economical for solar farms	For special applications only	Works well for solar installation	Special application solution	Not state of art for solar farms

5.2. Selection of Foundation for Solar Facility

Selection of an appropriate foundation for a solar facility is quite a tedious task. This is because it involves adequate characterization of the site within which the solar facility is to be constructed. According to Smalley (2015), there are four main factors that influences selection of an appropriate foundation for a specific solar facility. These are geotechnical attributes of site, module size, array tilt and the cost of the foundation.

The geotechnical attributes of the site are of paramount importance in selection of an ideal foundation for a specific solar facility and include mainly the topography, soil composition and climate (Brearly et al., 2015). Different foundation options are suited to areas with different attributes. For example, driven piles are suited to relatively flat areas where soils are firm and compacted, with enough fine grained material such as silt or clay whereas earth screws are suited to relatively steep areas where soils have poor cohesion such as clean sand or weak saturated. According to IFC, (2015) the topography of an area influences the tilt angle of the foundations. This is the angle at which the foundation will be laid or driven into the soil, therefore careful consideration should be given to the topography of an area prior the selection of an ideal site for a specific site (Smalley, 2014).

The soil composition is also of paramount importance because it influences the way the soil would react with buried metal objects, fittings as well as the foundation. For example, basic and alkaline soils tend to be unreactive and have little or no impact on metal objects, fittings and foundations whereas acidic soils tend to be very corrosive and highly reactive and results in the corrosion of metal objects, metal fittings and foundations (SMEC SA, 2013). The climate of the study area is also very crucial because it influences not only the soil composition and the hydrology of the area, but also influences the performance of the solar facility. According to Solek (2012) solar facility are best suited in dry hot climate than in wet cold climate because it rains a lot in this areas which negatively influences the performance of the solar facility as a whole.

Module size is the size of individual solar panels that would be grouped together to make a solar facility (IFC, 2015). This module size is important because it outlines the load carrying capacity of the foundation. The carrying capacity of the foundation is determined through foundation load tests such as pile uplift, pile compression and pile strength. While the design loads associated with ground-mounted PV systems may be small compared to those for other structures, the foundation still needs to support considerable dynamic loads (Brearly et al., 2015). The tilt angle is the angle at which foundations are laid down or driven into the soil and primarily influenced by the topography of the area. The load carrying capacity of the foundation may also influence the tilt angle. According to Fellenius (2009) certain foundation types carry more load when they have a great tilt angle than when they have a small tilt angle.

Cost is of paramount importance while selecting foundation for PV solar systems. According to Brearly et al. (2015), a well-designed solar foundation needs to be cost effective. Quality geotechnical data are key to designing a reliable and cost-effective foundation (Smalley, 2015). Without the proper geotechnical information, we have to make conservative foundation design assumptions. Foundation design must be based on adequate site characterization. The better you understand these conditions, the more effectively you can work with your engineer to optimize the foundation.

5.3. Cases of Applications of PV Solar Facility Foundations

In 2002, Solek recommended driven piles and earth screws for Scuitdrift Solar Project located approximately 100 km Northwest of Kakamas town in the Northern Cape. This was done in order to minimize the environmental impacts of the facility and to also accommodate the relatively flat topography which is entirely dominated by reddish alluvial/Aeolian sandy soil with minor gravel and cobbles. These foundation types were also chosen because they imposed a lower cost over the lifetime of the project and offered high equity returns for investors and the operational risks were limited due to proven performance and track record.

The preliminary geotechnical investigation for proposed Solar Plant, Lost Hills, Kern County, California by URS (2010) recommended deep foundation type of concrete piers supported by steel beams or shallow cast-in-drilled hole or Driven steel sections because of the presence of potentially expansive, collapsible and corrosive clay at the site with carbonate cementation. Geotechnical investigation for Olien PV Solar Plant in Lime Acres, Northern Cape suggested cast in situ concrete piles and concrete block foundations due to the presence of strongly cemented calcrete occurring over a flat terrain on site (SMEC SA, 2013).

Preliminary geotechnical investigations for proposed Mount Signal Solar farm, Seeley, California recommended driven steel sections or H-piles instead of cast-in–place caissons over alluvium, lacustrine and fill/crop soils because of less soil disturbance, less concrete, limited access, shorter construction time and target load capacity of representative samples (EGA, 2011). They also recommended underpinning of the solar module array using galvanized steel H-pile or Helical anchors. Furthermore, they recommended Mat slab foundation for the inverter and associated structures due to the presence of heavy loads and expansive soils.

5.4. Recommended Foundation Type for the Proposed site

The site of the proposed solar facility is situated in a slightly undulating rocky terrain which comprises of calcretes, gneisses, granites, quartz, quartzite and dorbanks. The soil is dry, alluvial/aeolian sandy soil with gravel boulders and cobbles and cohesionless. Based on these attributes of the site, the ideal foundation types for this site are driven piles or earth screws. These foundation types are very economical for solar plants because they impose a lower cost over the lifetime of the project and offer high equity returns for investors and the operational risks are limited due to proven performance and track record (Solek, 2012). They are also very environmentally friendly, have medium carbon foot print and their installation is very easy. In addition, no site preparation is required and do not require any special equipment for installation (Smalley, 2015).

Driven piles and earth screws offer very high uplift resistance and do not slide once erected. They can be deployed over any soil type that allows penetration at any tilt angle and the bearing pressure does not need to be observed since they can be able to carry almost any pressure they are subjected to (Brearly et al., 2015). However, these foundation types cannot be recommended to all the trial pits in the study area due to the presence of strongly cemented calcrete in some of the profiles such as G8, G2, and M24. This will prevent installation at very shallow depth because the bearing capacities on the strongly cemented calcrete will be high and the shallow depth of penetration will mean that resistance required to the moment and uplift forces will not be achieved from the overlying soils alone.

These recommendations are similar to those given by Solek (2012) for Scuitdrift Solar plant located 100 km Northwest of Kakamas which suggested that driven piles and earth screws be used as foundation types of the solar facility due to their economic and environmental benefits. The soil is also alluvial/aeolian sandy soil, dry and cohesionless and the topography is relatively flat just like that of Scuitdrift Solar Plant.

Two other foundation types may be considered. These include cast in situ concrete piles and concrete block foundation. The cast in situ concrete will be ideal because it will be possible to adopt as the holes at the pile positions would be formed by drilling using conventional shot hole rigs which in turn would overcome the problem of installation where strongly cemented calcrete is present and concrete block foundations would be ideal because it ca be able bear the calcrete at shallow depth (SMEC, 2013). According to SMEC (2013), a bearing capacity of 300 kPa may be taken for the strongly cemented calcrete and normal spread foundations may be taken for any proposed structures at a nominal depth of 0.5 m, bearing on the calcrete. However, these foundation types are very expensive, require extensive side preparation and take a long time to build. They also have high carbon foot print.

5.5. Foundation Ground

Based on the findings of soil profiling and resistivity survey, the study area comprises of two- or three-layered soil whose top soil comprises of alluvial sandy silty soil, a subsoil of calcrete for three-layered soils and the bed rock which comprises of weathered and foliated gneiss mainly or slightly weathered pegmatite or quartz or calcrete or very rarely dorbank. In addition, the soil cover is very thin, and the study area is dominated by alluvial/aeolian cohesionless sand with gravel and calcrete boulders and cobles. Based on these findings, it is recommended that the foundations of the proposed solar plant be laid on the bedrock a layer of very pale brown to pale yellow fractured weak gneisses and granites or pegmatite or on the very strong layer of calcrete, dorbank or quartz where they occur. In all cases, all the friable and loose materials should be removed before laying the foundations. This means that the foundation depth will vary depending on the quantity of these undesirable materials. Differences in foundation depth for adjacent footings can be filled with lean concrete. Reinforced wall beams are also recommended where big differences in cut levels occurs.

5.6. Foundation Depth

The foundation depth varies due to different levels in the site (0.5 m-1.9 m) and differences in the depth of the bearing layer. The shallowest pit was 0.5 m while the deepest profile was 1.9 m. In this case, the site is covered by a thin layer of fill material, thus, the footing depth will vary and should be laid on the natural bearing layer at a depth of 0.5m- 2m and excavated at least 0.5 m in the natural rock bedding in order to meet the TRH 14 specifications for earthworks.

5.7. Excavatability, Stability and Material Use Potential

Table 5.2 presents a detailed geotechnical appraisal for all the trial pits within the study area. This was carried out in accordance with the requirements of SANS 1200 with particular reference to sections D (Earthworks).

Table 5.2: Geotechnical appraisal of all trial pits within the study area.

Profile ID	Description of material	Ground water conditions	Excavatability
G14, G28, M5, M6, M14 (Coega)	Top soil comprises of either sandy, sandy silt or clay soil (Alluvium/minor Aeolian) and the subsoil consists of either pedogenic calcrete soft, calcrete powder or calcrete hardpan	Ground water encountered in profile G28 at 1.3m depth	No specific difficulties anticipated to excavate the top soil and subsoil of calcrete soft or powder (Soft Excavation). Excavation of Clacrete hardpan perceived to be problematic due to its competent nature (Hard Excavation)
G3 (Knersvlakte)	Topsoil consists of sandy silt transported soil (Alluvium/minor Aeolian) and the subsoil comprises of Dorbank (Silistic chemical sedimentary rock)	No ground water encountered	Soft excavation for the top soil and intermediate excavation for the subsoil (Dorbank)
G5, G6, G8, G9, G11, G12, G16, G18, G19, G20, G21, G22, G23, G28, G29, M4, M10,M12, M13, M23 (Glenrosa)	Top soil comprises of sandy silt or silty clay or simply sandy or clay transported soils (mostly Alluvium with minor Aeolian or just alluvium and the subsoil composes of either moderately or highly and foliated weathered gneiss or moderately or slightly weathered pegmatite or quartz	No ground water encountered	Soft excavation of the cohesionless top sandy soil and soft to intermediate excavation of the gneiss given the weathered nature of the rock, fine horizontal laminations and high degree of fracturing. Pegmatite and quartz however would require hard excavation because of their resistance

‘**Table 5.2**, continued’

G1, G4, G7, G10, G13, G25, G26, G27, M1, M2, M3, M7, M8, M9, M11, M21, M24 (Mispah)	Topsoil comprises of transported (Alluvium/minor Aeolian) sandy, sandy silt, silty clay or simply clay soil. The subsoil consists of either pedogenic calcrete powder, calcrete soft or calcrete hardpan while the bedrock comprises of either moderately or highly weathered and foliated gneiss, slightly weathered pegmatite, quartz or granite	No ground water encountered	
G2 (Dundee)	Topsoil comprises of transported (Alluvium) sandy soil while the subsoil comprises of silty clay colluvial soil	No ground water encountered	No specific difficulties anticipated to excavate the top soil and subsoil of calcrete soft or powder (Soft Excavation).

5.7.1. Material excavatability

The excavatability of material is a measure of the material to be excavated or dug with conventional equipment such as bulldozer, backhoe, scraper and other grading equipment. As discussed in Chapter 4, the study area comprises of two- or three-layered soil whose top soil comprises of alluvial sandy silty soil, a subsoil of calcrete for three-layered soils and the bed rock which comprises of weathered and foliated gneiss mainly or slightly weathered pegmatite or quartz or calcrete or very rarely dorbank. Generally, the site is characterized by zero to thin alluvial cohesionless sandy soil cover, observed as 0- 0.5 m in the trial pits, overlying foliated gneiss or cemented to strongly cemented calcrete.

The sandy soil and soft rocks such as calcrete soft and calcrete powder were excavated with ease using a mere shovel during sample collection. However, due to the presence of strongly cemented calcrete, very resistant quartz and strongly cemented dorbanks at very shallow depth across the site, blasting or breaking out of the calcrete will have to be undertaken to allow for installation of the solar plant infrastructure such as cables, metal fittings and foundation because these are very difficult to excavate. Collection of these materials in the field was very difficult and required the use of heavy-duty equipment such as geological and sledge hammers to break off these materials. The weathered gneiss and granites were excavated with intermediate difficulties without any blasting using only a sledge hammer and shovel since they were not that difficult to break.

5.7.2. Side wall stability

Side wall stability refers to the ability of the pit wall to remain intact after being excavated. The side walls of the trial pit remained stable during the investigations as can be seen in Figure 5.2. All these trial pits were dry, excavated vertically and their depth ranged from 0.5 m- 2 m, with an exception of trial pit G28, which had water seepage at 1.3 m depth. Therefore, where such trial pits are dry and not below the water table, excavations to 1.3 m depth can be excavated vertically. Generally, the groundwater table is not anticipated to cause pit instability difficulties such as collapse and erosion, except in trial pit G28. This notwithstanding, the investigation was carried out during a notably dry year, and thus shallower water tables may be encountered in the future Excavations deeper than 1.3 m or excavations with water seepage will need to be shored or battered.

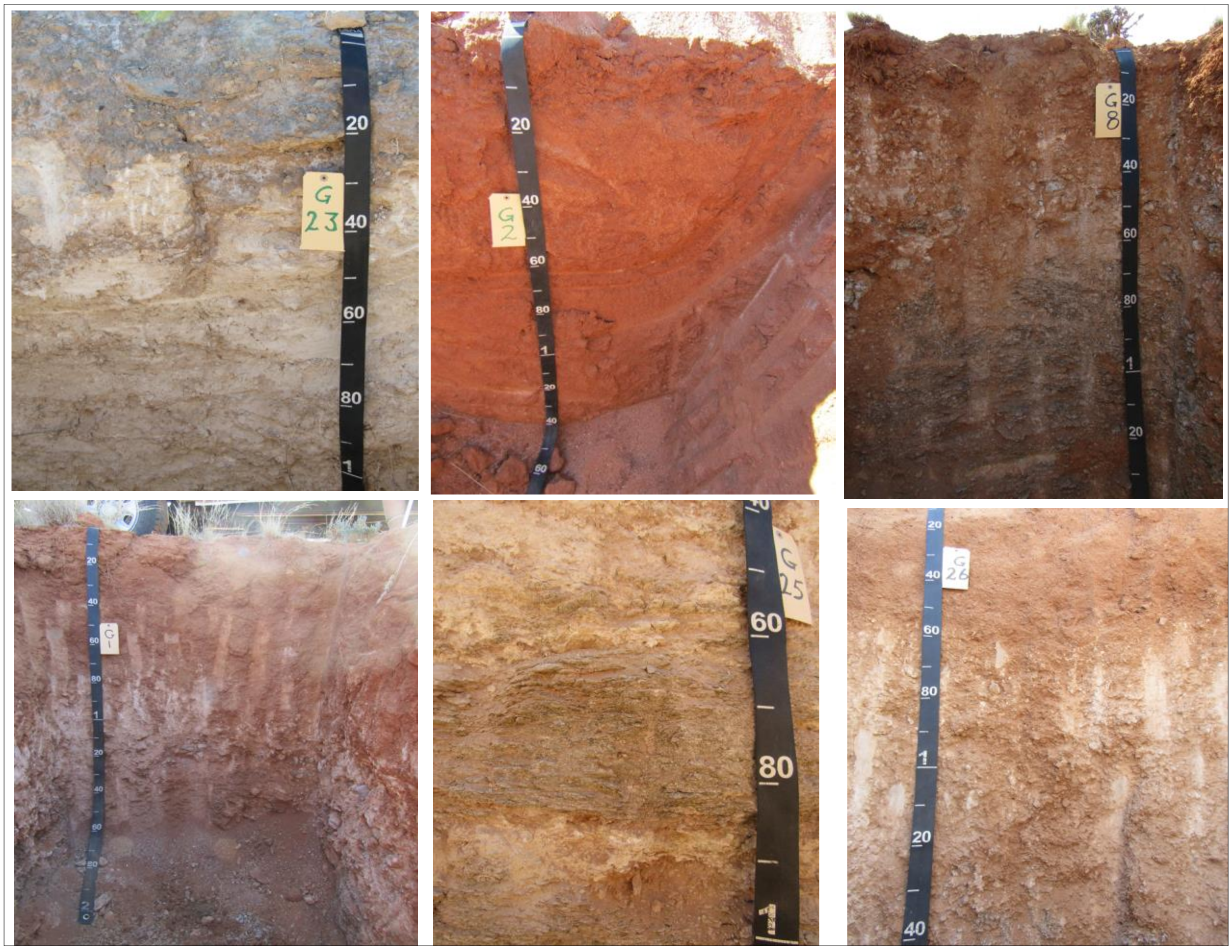


Figure 5.2: Test pits showing evidence of stable pit walls during investigation.

Shoring is a process of temporarily supporting a building, trench, pit or structure with props when in danger of collapsing or during alterations or repairs and battering refers to repeated pounding of something using a hammer until it becomes stable. This is very important for all trial pits with water seepage because water creates instability on the pit walls by filling in the void spaces of the soil in the pit wall causing it to collapse over time. It must however be noted that although the trial pits excavated during the geotechnical investigation gave an optimistic indication of the stability of long pit excavations, it is very crucial to ensure that excavations are safe by either shoring or battering them and the soils should be stable enough through compaction for the predrilled holes at the pile positions to remain open prior to filling with concrete. Overall, shoring is mandatory for trial pit G28 and instability of sidewalls for pits deeper than 1.5 m should be anticipated and a batter of 1:1 for the upper 2/3 of a 1.5 – 3.0 m pit is recommended according to SANS 1200 specifications.

5.7.3. Material utilization

Material utilization refers to the potential use of the material for construction purpose. Generally, the study area is characterized by cohesionless alluvial/aeolian sandy soil mixed with gravel and calcrete cobbles. This soil is dry, compactible and has no or very little plasticity. Rocks are very variable and range from calcretes, gneiss, granite, quartzite, quartz, pegmatite, gypsum and dorbank. According to the specific guidelines for material use laid out by the South African National Agency (2014), granular medium to coarse grained soils such as free drained and poorly graded sandy and sandy silt soils are ideal for use as backfilling and structural fill material. In addition, these soils should be non-plastic or slightly plastic and should be compactible and are classified according to the Unified Soil Classification as non-plastic and slightly plastic silty sands (SM), poorly graded sands-silt mixes(SP), gravels or gravel-sand-clay mixtures and poorly graded silty gravels or gravel-sand mixtures (GP and GM). However, these materials are too free drained to be used as bedding material.

The study area is also dominated by very strong and resistant igneous and metamorphic rocks such as granite, pegmatite, quartz and quartzite which are very suitable for use as backfilling and construction material as stipulated by the South African National Road Agency, (2014). Lastly, but not least, study area also comprises abundant carbonate rocks mainly calcrete and very minimal gypcrete and as stated in the guidelines for material utilization, these rocks are ideal for construction of access roads and tracks.

Bases above mentioned properties and specifications, it is safe to say that the material encountered on site are suitable for use as backfilling, fill and construction material as SANS 1200LB and LD section for construction. None of these materials however are suitable as selected granular material for bedding purposes. Consequently, selected suitable granular material stipulated in the SANS 1200 would have to be imported from elsewhere.

5.8. Design of On-Site Road Ways

Any organic material, topsoil, softened or disturbed soils should be removed from the road sub grade to a minimum depth of 45 cm. The exposed subgrade layer should be rolled with a loaded tandem axle truck or heavy roller. Any soft or distressed areas such as faulted or unstable areas identified during the investigation should be sub-excavated and backfilled with selected suitable materials that meet requirements of TRH 14/ AASHTO specifications for roads construction.

Backfills placed within the upper 1m of the roadway should be compacted to 95% Modified Proctor maximum dry density and the top layer of the sub-grade surface should be compacted to 98% of modified proctor (South African National Road Agency, 2014). In this study, the encountered material such as sand, gravel, rocks and soft rocks except debris and fill that encountered at the top 0- 0.5 m, if crushed or compacted can be used as subgrade and aggregate base course for multiple construction purposes such as backfilling, use as fill material or construction of structures and access roads.

CHAPTER SIX: CONCLUSION AND RECOMMEDATIONS

The purpose of the site investigation conducted was to determine the suitability of the proposed site for construction of a solar facility. In essence this concluding chapter provides a summary of the study, research-based conclusions, the main contributions of the research and recommendations for future work.

6.1. Summary of the Research

The proposed site was evaluated to determine its suitability for construction of a solar plant. The site is located in the arid area approximately 53.23 km away of Kakamas town. Although arid areas are ideal for solar plant construction because they have a lot of heat that could be harnessed into producing solar energy, they often tend to be very unstable due to presence of carbonate rocks suck as calcrete and gypsum which tend to dissolve in water. These areas are also characterized by dust storms which results in accumulation of dust particles on the surface of solar panels. In addition, the soils in these areas are highly variable and are characterized by different structures which could have serious impacts on the stability and suitability of these areas for construction.

The main objective of this research was to assess the suitability of a proposed site for construction of a PV solar facility and the specific objectives of this research were to assess and establish all the geotechnical aspects that may have an impact of the development of the site, to explore the surface conditions at the proposed site, to establish the soil properties and comment on the use of the on-site soils in the construction of the solar facility, to determine the variability of ground conditions and effects of such variability on the proposed development and to provide foundation recommendations for the design and construction of the solar facility.

Consequently, geological survey and magnetic survey were conducted as part of reconnaissance survey and revealed the area is dominated by igneous and metamorphic rocks. Field soil tests such as soil profiling, resistivity survey and cone penetration were performed in order to determine the soil type, subsurface structures, soils stiffness and strength and these indicated that the study area is characterized by alluvial/aeolian sand and soil profiles had either two or three layers comprised of either soil, soft rock or rocks of variable consistency and stiffness. The results also showed that the soil comprised of several vertical joints and that the parent rock is highly weathered.

Foundation indicator tests (sieve analysis and atterberg limits) of the soil were performed and these indicated that the soil is poorly graded, non-plastic to slightly plastic, sandy and has no swell. In addition, a CBR test was conducted on collected soil samples which showed dense to very dense granular soils which predominantly classify as G6 in accordance to the TRH14 classification of material. Furthermore, soil chemistry tests were conducted and revealed fairly neutral non-corrosive conditions that would have no impact on buried objects to highly corrosive conditions that would have detrimental impacts on buried objects. Lastly, but not least, the permeability test was performed of the onsite soil and revealed medium permeable silty sands.

Although the study area is characterized by a lot of cohesionless sandy soil and soft rocks such as calcrete and gypcrete, the results show that these materials have good potential for use as backfilling material and road bed if compacted and for construction of access roads. The study also showed that the study area comprises a lot of competent rocks such as quartz and quartzite and gravel which are good for base and sub base material and construction of structures. The compaction test also indicated dense to very dense soils which are ideal for construction purposes if compacted. Lastly, the side wall stability indicated staple pit walls which remained stable during the investigation giving an optimistic indication of the stability of long pit excavations.

6.2. Conclusion

The following conclusions were made based on the findings of the analysis and interpretation of various results that were generated during the study. The conclusions drawn from the study are presented in the following paragraphs.

The study area is located in an arid or semi-arid environment and is underlain by igneous and metamorphic rocks such as granite, pegmatite, gneiss and quartzite/meta-quartzite. This was confirmed by the low moisture content of the soil during soil profiling and CBR test as well as the results of the geological survey, magnetic survey and soil profiling. Dry arid and semi-arid areas are ideal for solar facility construction because they have less precipitation and strong and competent rocks such as granite and quartzite are ideal for use as base and sub-base material during construction.

The subsurface consisted of several soil horizons which had several vertical joints in them. This was revealed by the correlation of the soil profiling and resistivity surveys which indicated that the soil profiles/structures constituted of either two or three layers which were made up of either sandy soil and soft rocks (calcrete/gypcrete) or sandy soil with rocks or sandy soil with soft rocks and rocks and had several vertical joints in them. Sandy soil and soft rocks are very weak material and tend to collapse very easily as compared to rocks which are very strong and durable material while joints, foliations and weathered material are regarded as weak zones or potential zones of failure. This means where these structures are present, failure is most likely to occur.

The study area is characterized by dry, poorly graded, dense to very dense, non-cohesive alluvial/aeolian sandy or sandy silt soil which classifies mainly as G6 according to TRH 16 specifications. This soil has medium permeability, low or slight plasticity, low potential for expansion and is good for use as base and sub base material and backfilling if compacted. Dry, poorly graded non-plastic and slightly plastic sandy soil is excellent for construction purposes because it exhibits no expansion-contraction properties under different moisture content which normally results development of cracks and collapse of buildings overtime. The consistency and the stiffness of this on-site soil was highly variable and ranged from very soft to stiff soil in accordance with the results of the soil profiling and the cone penetration results.

The study proved that the excavatability of material was variable ranging from soft material which require soft excavation to very competent material which require hard excavation while the side wall stability indicated dry stable pit walls during the investigation. This gave an indication of stability of long pit excavations except in profile G28 due to the presence of water in the pit at approximately 1.3 m. Water causes instability in the pit walls by forcing the individual soil particles apart which results in collapse of the pit wall.

The foundation analysis of the site showed that the ideal foundation types for this site are driven piles and earth screws because of their wide area of applicability and adaptability. However due to the presence of calcretes at shallow depth, cast in-situ concrete piles and concrete block foundations should also be considered. Generally, all the materials on-site have potential and can be used for different activities such as backfilling, roadbed, foundation base or sub-base and construction of access roads and structures in accordance with the SANS 1200LB and LD section for construction and the site is suitable for the construction of the solar facility provided all the recommendations are implemented.

Based on the above conclusions, it can be said with confidence that the study area is suitable for the construction of the solar facility provided all the recommendations are implemented.

6.3. Recommendations for Practice

In this section, recommendations for practice based on the findings are presented for consideration and implementation. The recommendations are:

- All the on-site soil should be compacted to at least 95% Mod AASHTO prior to erection of any structure. This is because the study area is dominated by sandy material and sandy soil is cohesionless and could results in collapse of structures if built on uncompacted sandy soil. Compaction to at least 95% Mod AASHTO would help increase its strength by bringing individual grain particles together thereby increasing the intramolecular forces between them.

- Excavations deeper than 1.5 m should be shored by steel posts or battered with a hammer or any heavy pounding equipment in accordance to section D of SANS 1200.
- The pit side walls of profile G28, should be supported in over 1.0- 1.3 m depth by steel post or beams due to the shallow groundwater table and sandy materials and the sidewalls should be battered at a slope of at least 1V:3H, or shored.
- The study area is a relatively flat terrain and is underlain by igneous and metamorphic rocks with calcretes. It is also characterized by dry, cohesionless alluvial/aeolian sandy silty soil mixed with calcrete boulders and gravel. Based on these, it is recommended that driven piles and earth screws should be the ideal foundation types for this area because of the economic, environmental and technical benefits associated with them. Alternatively, two other foundation types such as cast in situ concrete piles and concrete block foundations should be considered where strongly cemented calcrete (calcrete hardpan and calcrete honeycomb) occurs.
- All the material at the site should be used in accordance with the specifications of potential use outlined in section LB and LD of SANS 1200. Failure to adhere to these standards could results in improper use of material which could results in collapse or failure of buildings and structures.
- During the placement of any structural fill, it is recommended that a sufficient amount of field tests and observation be performed and that any areas of fill or sub-grade instability encountered during the construction be identified and appropriate measures taken to ensure stability.

6.4. Recommendations for Future Studies

These recommendations describe topics that require closer examination and generate new questions for further study. In essence, the following recommendations were made:

- Future studies should focus on groundwater studies and monitoring to determine the water table and its fluctuations in different seasons as well as the hydraulic properties of the soil. This can be done through pumping tests such as constant-rate test, step-drawdown tests and recovery test to determine aquifer properties.
- Studies on foundation tests, foundation loads and analysis should also be conducted in order to come up with more detailed foundation loads, carrying capacity, tilt angle and module array as well as specific and detailed foundation types that are suited for this area. This could be done by performing load test and determining the consolidation and the stresses of the soil as well as its compressive strength.

REFERENCES

Abdelrahman, M. (2017). Groundwater Exploration using Vertical Electrical Sounding: A Case of al-Amar region, Southwest Riyadh, Saudi Arabia. International Conference on Engineering Geophysics, United Arab Emirates, pp. 9-12.

Akpila, S. B and Omunguye I.W. (2013). Soil Resistivity on PMS Tank Foundation in Granular Soil Lithology: A case Study in Lekki, Nigeria. Scholars Research Library, European Journal of Applied Engineering and Scientific Research Vol. 2, No. 4, pp. 28-36.

Al-Khafaji, A.N., Al-Mosawi, M.J and Al-Saoudi, N. K. S. (2013). Challenging Problems of Gypseous Soils in Iraq. Proceedings of the 18th International Conference on Soil Mechanics and Geotechnical Engineering, Paris, 4 pp.

Ahrens, R., Eswaren H., Rice, T. and Stewart, A.B. (2003). Soil Classification: A Global Desk Reference. CRC press, 23 pp.

Almond, E.J. (2012). Proposed Keren Solar Plant on ERF Kakamas Kai Garib Municipality, Northern Cape: Recommended Exemption for further Paleontological studies and Mitigation. Cape Town, Naturaviva cc, 3pp.

Almond, E.J. (2009). SAHRA Paloentechnical Report: Paleontological Heritage of Northern Cape. Cape Town, Naturaviva cc, 88pp.

Apparao, K.V.S and Rao, V.C.S. (1995). Soil Testing Laboratory Manual and Question Bank, Universal Science Press, New Delhi, 7 pp.

Ariffin, K.S. (2001). Geophysical Surveying Using Magnetic Methods. EBS 309: Geofizkic Carigali, 34 pp.

ARUP. (2013). Grootvlei PV Facility Mounting Structure Construction Methodology, 14 pp.

Balasubramanian, A. (2017). Engineering Properties of Soils. Unpublished technical report of the University of Mysore. Mysore, 8 pp.

Balasubramanian, T and Kumar, J.R. (2012). Vertical Electrical Sounding for Groundwater Prospecting in Uppodai of Tambaraparani River, Tirunelveli and Thoothukudi Districts, Tami Nadu, India. Research India Publications, International Journal of Environmental Engineering and Management Vol. 3, No. 3, pp. 147-157.

Bailey, M. (2007). Cone Penetration Testing (CPT). NCHRP, Miami 3 pp.

Beaurez, N., Bievre, G., Jongmans, D., Orengo, Y., Renalier, F. and Schwartz, S. (2008). Geophysical Investigation of a Large Landslide in Glaciolacustrine in the Trieves Area (French Alps). Engineering Geology, Elsevier, pp. 45-47.

Bernard, J. (2003). Short Notes on the Principles of Geophysical Methods for Groundwater Investigations. [Online] Available from: http://www.Terraplus.com. [Accessed: 12 June 2018].

Breytenbach, I.J. (2009). The Relationship between Index Testing and California Bearing Ratio Values for Natural Road Construction Material in South Africa. Unpublished Dissertation at the University of Pretoria, pp. 1-305.

Breusse, J. J., and Mathiez, J. P. (1956). Application of electrical prospecting to tectonics in the search for natural steam at Larderello (Italy): Geophysical Case Histories, Vol.2, pp. 623-630.

Boscia Environmental Solutions. (2017). Scoping Report for the Brypaal Solar Project. Bryanston, pp.1-93.

Bowles, J.E. (2012). Engineering Properties of Soils and their Measurements, 4th edition, McGraw Hill Education (India) Private Limited, New Delhi, pp. 1-476.

Brearley, D. and Donaldson, B. (2015). Geotechnical Analysis and PV Foundation design. Courtesy Advanced energy pp. 1-13.

Brink, A.B.A. and Bruin, R.M.H. (2002). Guidelines for soils and rock logging in South Africa. AEG, SAICE and SAIEG, Rivonia pp. 8-45.

Brown, L.L, Murdock, K.J and Wilkie, K. (2013). Rock Magnetic Properties, Magnetic Susceptibility and Organic Geochemistry Comparison in core LZ1029-7 Lake El'gygytgyn, Russia Far East. Toronto 1(9) pp. 467-479.

Chidley, C., Le Roux, R., and Holton, E. 2011. Environmental Management Framework and Strategic Environmental Management Plan for the Namakwa District Municipality. Prepared by Nemani Consulting CC. Prepared for the Department of Environment and Nature Conservation. Available online: http://nc.spisys.gov.za/docs.html, accessed May 2013.

City Development Planning and Environmental Directorate. (2016). Geotechnical Assessment Guidelines. Gold Coast, 13 pp.

Cornell, D. H., Thomas, R. J., Moen, H. F. G., Reid, D. L., Moore, J. M. & Gibson, R L , 2006. The Namaqua-Natal Province. In: The Geology of South Africa. Johannesburg: Geological Society of South Africa, pp. 325-379.

Dafalla, M.A. (2013). Effects of Clay and Moisture Content on Direct Shear tests for clay-sand mixtures, Adv. Mater. Sci. Eng., Vol.3, pp. 365-380.

Das, B.M. (1997) Advanced Soil Mechanics, 2nd Edition, Taylor & Frances Ltd, London, 457 pp.

Das, B.M. (2004). Fundamentals of Geotechnical Engineering. 2nd ed. Chris Carson: Madrid, Spain, pp. 53-60.

Das, B.M. (2006). Fundamentals of Geotechnical Engineering 4th ed. B.H.C.SUTTON. Solving Problems in: Soil Mechanics 2nd ed., 753 pp.

Das M.B. (2010): Principles of Geotechnical Engineering. Cengage learning, Stamford, USA, pp. 73-102.

Department of Environmental Affairs (DEA). (2018). Renewable Energy- Hope to Meet the Increasing Energy Demand in South Africa. Published article.

Dobrin, M. B. (1976). Introduction to Geophysical Prospecting. New York, McGraw-Hill.

Dold, P.A and Milton, S.J. (2006). Nama-Karoo Biome. Nelspruit, Trelitzia 19, 4 pp.

Du, C., Wang, P., Yu, J and Zhang, y. (2018). Analysing the Mechanisms of Soil Water and Vapour Transportation in the Desert Vadose Zone of the Extremely Arid Region of Northern China. Elservier, Journal of Hydrology Vol. 588, pp. 592-606.

Duarte, I.M.R, Pinho, A.B and Ladeira, F.L. (2014). Penetrometer Testing in Residual Soils from Granitic Rocks in the South of Portugal. Elservier, Vol. 2, No.3, pp. 3-6.

Durgunoglu, H.T., Varaksin, S., Briet, S. and Karadayilar, T. (2003) A case study on soil improvement with heavy dynamic compaction, Proc. XIII ECSMGE, vol. 1, pp. 651-656.

Emenike, E.A. (2001). Geophysical exploration for groundwater in a Sedimentary Environment. A case study from Nanka over Nanka Formation in Anambra Basin, South eastern Nigeria. Global Journal of Pure and Applied Sciences. Vol.7, No. 1, pp. 1-11.

EGA Consultants. (2011). Preliminary Geotechnical Investigation: Proposed Mount Signal Solar Farm and Associated Structures West of Drew Road and South of Interstates Imperial Country, California. US Solar Holdings, California, 90 pp.

Englington B.M. (2006). Evolution of the Namaqua-Natal Belt, Southern Africa: A geochronological and Isotope Geochemical Review.Elservier, Journal of African Earth Sciences Vol. 46, No.1, pp. 93-111.

Fellenius, B.H. (2009). Basic Foundation Designs. Electronic Edition, accessed in July 2018, 13 pp.

Fernandes J, Flyn N, Gibbes S, Griffis M, and Issihiki T. (2010). Renewable Energy in the California Dessert: Mechanism for Evaluating Solar Development on Public Lands. Michigan, University of Michigan, 343 pp.

Fey M.V and Mills A.J. (2004). Declining Soil Quality in South Africa: Effects of Land Use on Soil Organic Matter and Surface Crusting. Stellenbosch. South African Journal of Plant and Soil Vol. 21, No. 5, pp. 388-398.

Fey M.V. (2010). A Short Guide to the soils of South Africa, their Distribution and Correlation with the World Reference base soil groups. Stellenbosch. South African Journal of Plants and Soil Vol. 22, No. 7, pp. 1-25.

Gogoi, J.C. and Laskar, A.A., 2015, Mechanical compaction - a simple ground improvement technique: a case study. Discovery, Vol.40, No.185, pp. 377-383.

Greenwood R.J, Norris E.J and Jo W. (2006). Site Investigation for the Effects of Vegetation on Ground Stability. England, Geotechnical and Geological Engineering Vol. 6, No. 24, pp. 467-481.

Heritage Management Consultants. (2012). Heritage Impact Assessment Report: Basic Assessment Phase: Proposed Establishment of the Southern Cross Solar Energy Facility Located West of Upington on the Farm Southern Farms 425, Northern Cape Provinces. Upington, Savannah Environmental, 158 pp.

Humes J.S, Holland and Knight L.L.P. (2012). Solar Energy Project Development Issues: Preliminary Considerations. New York, Practical Law Company, 10 pp.

HC Pietersen and Associates. (2002). Geotechnical Investigation for Roads at the Coves Hartebeespoortdam. Hartebeest, 13 pp.

Hunt, C.P, Moskowitz, B.M and Banerjee, S.K. (1995). Magnetic Properties of Rocks and Minerals. University of Minnesota, pp. 189-203.

IAEA. (2004). Geotechnical Aspects of Site Evaluation and Foundations for Nuclear Power Plants: A Safety Standard Guide, Vienna, 66 pp.

Indian Institute of Technology Gandhinagar. (2002). Laboratory Determination of California Bearing Ratio (CBR). IS: 2720-1987, pp. 1-6.

International Finance Corporation. (2015). Utility-Scale Solar Photovoltaic Power Plants: A Project Development Guide. Washington DC, IFC, 173 pp.

Inroads Consulting. (2009). Eskom Landfill and Hazardous Waste Storage Facility: Draft Final Report. Polokwane, Inroad Consulting, 11 pp.

Kaboli, R., Mumpour, M., Najib, O.A and Nejad, H.T. (2011). Vertical Electrical Sounding (VES) Resistivity Survey Technique to Explore Groundwater in an Arid Region, Southeast Iran. Journal of Applied Sciences Vol. 11, pp. 3765-3774.

Jain, V.K., Dixit, M. and Chitra, R. (2015). Correlation of Plasticity Index and Compression Index of Soil. IJIET Vol. 5, No. 3, June, pp. 263-270.

Jatau, B.S., Patrick, N.O., Baba, A., and Fadele, S.I. (2013). The use of Vertical Electrical Sounding (VES) for Subsurface Geophysical Investigations around Bomo Area, Kaduna State, Nigeria. IOSR Journal of Engineering. Vol. 3, No.1, pp. 10-15.

Kayode, J.S, Adelusi, A.O and Nyabeze, P.K. (2013). Interpretation of Ground Magnetic data of Ilesa, Southwestern Nigeria for Potential Mineral Targets. Advances in Applied Sciences Research, Vol. 4, No. 1, pp. 163-172.

Kong, C., Xu, K., Wu, C., Li, J and Zhang, L. (2011). Suitability Evaluation of Urban Construction Land based of Geo-Environmental factors of Hangzhou, China. Elservier, Computer and Geosciences Vol. 37, pp. 992-1002.

Laskar, A. and Pal, S.K. (2012). Geotechnical Characteristics of two Different Soils and their Mixture and Relationships between Parameters. EJGE 17, pp. 2821-2832.

Lambrechts, J.N. (2013). Soil Genesis and Classification and Soil Resources Database. South African Journal of Plants and Soil Vol. 2, No. 5, pp. 1-25.

Lange, M. E. (2006). Women Reading the Gariep River, Upington: Structured Conclusion. Unpublished mastered of the University of KwaZulu-Natal. Durban, 266 pp.

Le Roux, P. A. L, Turner, D. P and Van Huyssteen, C.W. (2013). Nature and Distribution of South African Plinthic Soils: Conditions for Occurance of Soft and Hard Plinthic Soils. Pretoria. South African Journal of plant and soils Vol. 30, No.1, pp. 23-32.

Li, B., Li, L., Kaseke, K.F., Seely, M.K., Vogt, R and Wang, L. (2018). The Impact of Fog on Soil Moisture Dynamics in the Namib Desert. Elservier, Advances in Water resources Vol. 113, pp. 23-29.

Li, D., Zhang, S., Zhang, H., Nie, K., Tian, K and Wu, Y. (2018). Experimental Investigation of Solidifying Desert Aeolian Sand using Microbially Induced Calcite Precipitate. Elservier, Construction and Building Material Vol. 172, pp. 251-262.

Lin, C and Maryol, E. (2015). Geochemical Caracteristics of Soils in Fezzan, Sahara Desert: Implications for Environment and Agriculture. Elservier, Journal of Geochemical Exploration Vol. 158, pp. 122-131.

Liu, Y., Xing, Z and Yang, H. (2017). Effects of Biological Soil Crusts on Microbial Activity in Soils of the Tegger Desert (China). Elservier, Journal of Arid Environments Vol. 144, pp. 201-211.

Lund P.D. (2009). Effects of Energy Policies on Industry Expansion in Renewable Energy. Renewable Energy, Vol. 34, pp. 53-64.

Lucian C. (2006). Geotechnical Aspects of buildings on expansive soils in Kibaha, Tanzania: Preliminary study. KTH Architecture and the Build Environment, Stockholm 110 pp.

Mariita N.O. (2007). The Magnetic Method. United Nations University, KenGen1 (1) 17 pp.

Mahmoud, I.I., Mohamaden, A., Shagar, S., and Gamal, A.A. (2009). Geoelectrical Survey for Groundwater Exploration at the Asyuit Governorate, Nile Valley, Egypt. JKAU: Mar. Sci. 20. pp. 91-108.

Mayne W.P. (2007). Cone penetration Testing State-of Practice: Lecture Notes. Georgia institute of Technology, Georgia pp. 4-17.

McCung R.C. (2008). South Africa's Newest Mineral Oasis: the Salt River Zn-Cu-Pb-Ag-Au VMS Deposits. Salt River Resources Ltd, Upington pp. 5-30.

Megger. (2010). A practical Guide to Earth Resistance Testing, 41 pp.

Meglioli, P.A., Riveros, C.V., Aranibar, J.N and Villagra P.E. (2017). Spatial Patterns of Soil Resources under Different Land Use in Prosopis Woodlands of the Monte Desert. Elservier, Catena Vol. 149, pp. 86-97.

Middleton, N. J. (2017). Desert Dust Hazards: A Global Review. Elservier, Aeolian Research Vol. 24, pp. 53-63.

Mikes, H, Duncan, J.M, Wringt, S.G and Thomas, L. (2012). Soil Strength and Slope stability. Journal for Soil Science and Engineering. Elservier, Vol. 5, No.7, pp. 207-227.

Minaar H. (2006). The Exploitability of Pegmatites Deposit in the Lower Orange River Area (Vloolsdrif-Henkries-Steinkopf). Unpublished masters of the University of Pretoria, Pretoria, 90 pp.

Milsom, J., and Erikson, A. (2011). Field Geophysics, 5th Edition. United Kingdom, John Wiley and sons, pp. 1-278.

Moore, J.E. (2012). Field Hydrogeology: A Guide for Site Investigations and Report Preparation. United States of America. Taylor & Francis Group, 29 pp.

Mossom R.J and Resources M. (2006). Resource Estimation of the Pb-Ag-Zn-Cu Deposits Located on the Farm Rozynenbosch 104, Kenhardt District, Northern Cape, SA. Miranda Minerals, pp. 5-33.

Murdock, K. j, Brown, L. L and Wilkie, K. (2013). Rock Magnetic Properties, Magnetic Susceptibility and Organic Geochemistry Comparison in Core LZ1029-7 Lake El'gygytgy, Russia Far East. Toronto. Climate of the past, Vol. 9, pp. 461-479.

Murthy, V. N. S. (2002). Principles of Soil Mechanics and Foundation Engineering, UBS Publishers' Distributors Ltd., New Delhi, pp. 1-386.

Nwankwo, L.I. (2011). 2D Resistivity Survey for Groundwater Exploration in a Hard Rock Terrain: A Case Study of MAGDAS Observatory, Unilorin, Nigeria. *Asian Journal of Earth Sciences*. 4 (1). pp. 46-53.

Nwankwoala, H.O. and Amadi, A.N., 2013, Geotechnical investigation of sub-soil and rock characteristics in parts of Shiroro-Muya-Chanchaga area of Niger State, Nigeria., IJEE, Vol. 6, No. 1, pp. 8 – 17.

Outeniqua Geotechnical Services. (2015). Proposed Subsidy Housing Project Nieubethesda, Eastern Cape Province: Phase 1 Geotechnical Investigation Report. Nieubethesda, 70 pp.

Nwankwoala, H.O. and Warmate, T. (2014). Geotechnical assessment of foundation conditions of a site in Ubima, Ikwerre Local Government Area, and Rivers State, Nigeria. IJERD, Vol. 9, No. 8, pp. 50 – 63.

NSW Agriculture. (2000). Understanding Soil pH. A published Leaflet. Acid Soil Action, New South Wales, pp. 1-4.

Osburn C.K. (2011). The Nature and Origin of the Polymetallic Salt River Massive Sulfide Deposit, Northern Cape Province, South Africa. University of Johannesburg, 99 pp.

Paige-Green, P. (2003). The Geology and Petrology of Road Construction Materials Revisite. Pretoria, CSIR Transportek, 7 pp.

Paige-Green, P and Du Plessis, L. (2009). The Use and Interpretation of the Dynamic Cone Penetrometer (DCP) Test. Pretoria, CSIR Built Environment, 82 pp.

Parasnis, D.S. (1997). Principles of Applied Geophysics. London. Chapman & Hall.

Pease, P and Tchakerian, V. (2015). The Critical Zone in Desert environments. Lowa, Elservier, Vol. 19, pp. 449-469

Pulmit Mining and Exploration (Pty). (2003). Environmental Management Plan. Department of Mineral Resources SA, Upington 40 pp.

Prakash, S and Jain, J.K. (2002). Engineering Soil Testing, Nem Chand & Bros, Roorkee, 28 pp.

Raj, P.P. (2012). Soil Mechanics and Foundation Engineering, Dorling Kindersley (India) Pvt. Ltd., New Delhi, pp. 1-43.

Reynolds, P. (2013). Engineering Correlations for the Characterization of Reactive Soil Behavior for Use in Road Design. Unpublished dissertation of the University of Southern Queensland. Queensland, 100 pp.

Rogers J.D. (2010). Fundamentals of Cone Penetrometer Test (CPT) Soundings: Lecture Notes. UMR, S&T University of Science and Technology 14 pp.

Roy, S and Bhalla, K.S. (2017). Role of Geotechnical Properties of Soil on Civil Engineering Structures. Uttar Pradesh. Resources and environment Vol. 7, No. 4, pp.103-109.

Skempton, A.W. (1953). The Colloidal activity of clays; Proc. 3rd Int. Conf. London. Soil Mechanics and Foundation Engineering, Vol. 1, pp. 47–61,

SAICE. (2010). Site Investigation Codes of Practice. SAICE, 24pp.

South African National Road Agency. (2014). Pavement Engineering Manual. 2nd edition. Road Agency Ltd, 89 pp.

Shunqukela T. (2014). A study of the Structural Geology of an Area between the Neusspruit Shear Zone and the Brakfontein Shear Zone near Kakamas, Northern Cape. University of Western Cape, Cape Town pp. 8-42

Sithole N. (2013). A study into the Main Structural Features of the Namaqua Region and their Relation to the Intrusion of the Keimoes Suite. University of Western Cape, Cape Town 5 pp.

Sindhu, S, Luthra, S and Nehra, V. (2017). Investigation of Feasibility Study of Solar Farms Development using Hybrid AHP-TOPSIS Analysis: A Case study of India. Elservier, Haryana, Renewable and Sustainable Energy Reviews, Vol. 73, pp. 496-511.

Sivasubramanian, P and Selvam, S. (2012). Groundwater potential zone identification using geoelectrical survey: A case study from Medak District, Andhra Pradesh, India. *International Journal of Geomatics and Geosciences* Vol. 3, No. 1. p. 55-62.

Smalley, J. (2015). What is the Best Foundation for Ground-mount Solar Array? DCE Solar, pp. 1-5.

SMEC. (2016). Geotechnical Investigation for the Sibaya Bulk Services Project. Ferndale, SMEC, 106 pp.

SMEC SA. (2013). Geotechnical Investigation for Olien PV Solar Plant Lime Acres, Northern Cape AMD Renewable Energy, Upington pp 4-98in the Pofadder Area, North Western Cape, South Africa. Toens and Partners, Springbok 82 pp.

Solek. (2012). Scuitdrift Solar Project: EIA Engineering Report. Upington, pp. 1-32.

Dr Talukdar, D. K. (2014). A Study of Correlation between California Bearing Ratio (CBR) Values with other Soil Properties. International Journal of Emerging Technology and Advanced Engineering, Vol. 4, No. 1, pp. 1- 4.

Telford, W. M., Geldart, L. P., and Sheriff, R. E. (1990). Applied Geophysics, 2nd Edition. New York, Cambridge University Press, pp. 1-217.

Teymur, B. (2015). Site Investigation. Unpublished Lecture notes form the University of Cambridge, pp. 1-13.

Turner P David. (2013). a Revised Perspective on the Principles and Structure of the Soil Classification System in South Africa: Discussion and Practical Examples. Pretoria. South African Journal of Plant and Soil Vol. 30, No. 2, pp. 61-68.

URS Corporation. 2010. Preliminary Geotechnical Investigation for Proposed Solar Plant, Lost Hills, Kern Country, California. URS, 52 pp.

Van Der Waals J.H. (2013). Soil Colour Variation between Topsoil and Subsoil Horizons in a Plinthic Cetena on the Mpumalanga Highveld, South Africa. South African Journal of Plant and soils Vol. 5, No. 2, pp. 14-27.

Wicander and Monroe. (2002). Essentials of Geology.3rd Edition. 528 pp.

Youssef, Y.G. (2015). Site Investigation. Unpublished lecture notes from Fayoum University. Fayoum, 56 pp.

Zhang, Y., Zhao, W. and Fu, W. (2017). Soil Macropore Characteristics Following Conversion on Native Desert Soil to Irrigated Croplands in a Deser-oasis Ecotone, Northwest China. Elservier, Soil &Tillage Research Vol. 168, pp. 176-186.

Zhao, C and Yi, Z. (2017). Desert “Soilization”: an Eco-Mechanical solution to Desertification. Elservier, Engineering Vol. 2, pp. 270-273.

Zohdy, A.A.R., Eaton, G.P., and Mabey, D.R. (1990). Application of Surface Geophysics to Groundwater Investigations: Techniques of Water-Resource Investigations of the United States Geological Survey, pp. 12-20.

Van der Waals J.H. (2011). Proposed INCA Kakamas Photovoltaic Solar Energy Facility: Basic Assessment Report. Terra soil science, 4-7 pp.

APPENDICES

APPENDIX A: MAGNETIC AND GEOLOGIC DATA OF THE STUDY AREA

Profile data												
Magnetic Data								Geology Data				
Line	Point on data sheet	Point on the map	UTM Northing	UTM Easting	UTM Zone	Latitude	Longitude	Point on the map	Latitude	Longitude	Rock description
7	260	**M1**	6771604.87	439917.89	34J	29°10'58.04"S	20°22'55.38"E	**G1**	29°12'27.37"S	20°22'28.30"E	Alluvial sandy soil
	500	**M2**	6771560.43	440167.95	34J	29°10'59.52"S	20°23'04.63"E	**G2**	29°12'25.60"S	20°22'15.28"E	Alluvial sandy soil
	900	**M3**	6771391.33	440514	34J	29°11'05.06"S	20°23'17.45"E	**G3**	29°11'07.81"S	20°23'28.95"E	Dorbank
	1200	**M4**	6771139.3	440680	34J	29°11'13.31"S	20°23'23.53"E	**G4**	29°11'14.78"S	20°23'28.19"E	Granite
	1920	**M5**	6770635.1	441177.1	34J	29°11'29.76"S	20°23'41.82"E	**G5**	29°11'37.02"S	20°23'23.01"E	Quartzite
	3080	**M6**	6769819.08	441992.0769	34J	29°11'56.4"S	20°24'11.84"E	**G6**	29°12'02.60"S	20°22'24.57"E	Pegmatite
6	17	**M7**	6770609	439266.58	34J	29°11'30.26"S	20°22'31.08"E	**G7**	29°11'46.72"S	20°23'22.24"E	Pegmatite
	27	**M8**	6770480	439419	34J	29°11'34.51"S	20°22'36.07"E	**G8**	29°11'55.44"S	20°23'42.51"E	Gneiss
	51	**M9**	6770186.95	439788.65	34J	29°11'44.12"S	20°22'50.03"E	**G9**	29°12'01.59"S	20°23'58.47"E	Calcrete
	119	**M10**	6769963	440083	34J	29°11'51.4"S	20°23'01.21"E	**G10**	29°12'05.95"S	20°23'18.07"E	Meta-quartzite
5	440	**M11**	6769879.18	438941.29	34J	29°11'53.95"S	20°22'18.88"E	**G11**	29°12'04.89"S	20°23'17.04"E	Granite
	1000	**M12**	6769590.38	439401	34J	29°12'03.42"S	20°22'35.87"E	**G12**	29°12'11.18"S	20°23'15.40"E	Quartz
	2120	**M13**	6768966.3	440328.7	34J	29°12'23.83"S	20°23'10.07"E	**G13**	29°12'17.61"S	20°23'23.88"E	Calcrete
4	29816	**M14**	6769116.67	438159.33	34J	29°12'18.58"S	20°21'49.78"E	**G14**	29°12'29.97"S	20°22'57.44"E	Calcrete
	31276	**M15**	6768368.7	439396.4	34J	29°12'43.13"S	20°22'35.44"E	**G15**	29°12'34.60"S	20°22'43.29"E	Meta-quartzite
3	100	**M16**	6768551.13	437893	34J	29°12'36.09"S	20°21'39.82"E	**G16**	29°12'33.49"S	20°22'43.15"E	Granite
	220	**M17**	6768494.25	437993.5	34J	29°12'38.77"S	20°21'43.52"E	**G17**	29°12'11.01"S	20°22'01.63"E	Meta-quartzite
	360	**M18**	6768434.17	438109	34J	29°12'40.75"S	20°21'47.81"E	**G18**	29°11'54.16"S	20°22'05.24"E	Alluvial Sandy soil
	460	**M19**	6768389.67	438187.67	34J	29°12'42.23"S	20°21'50.69"E	**G19**	29°11'34.17"S	20°22'20.82"E	Gneiss
	580	**M20**	6768335	438279	34J	29°12'43.99"S	20°21'54.07"E	**G20**	29°11'13.19"S	20°22'52.51"E	Alluvial sandy soil
	1140	**M21**	6768073.7	438750	34J	29°12'52.06"S	20°22'11.46"E	**G21**	29°11'25.75"S	20°23'07.45"E	Pegmatite
1	-360	**M22**	6770559.4	441154.3	34J	29°12'32.21"S	20°23'40.96"E	**G22**	29°12'41.44"S	20°22'04.32"E	Structure (Dyke)
	-1840	**M23**	6769661.91	440000.45	34J	29°12'01.19"S	20°22'58.08"E	**G23**	29°12'16.94"S	20°21'46.33"E	

	-2320	**M24**	6769372	439615.8	34J	29°12'10.51"S	20°22'43.75"E	**G24**	29°12'22.83"S	20°21'43.34"E	Small stones mixed sand and clay
	-4280	**M25**	6768118.67	438229.33	34J	29°12'51.05"S	20°21'52.16"E	**G25**	29°11'39.60"S	20°24'08.52"E	Structure (Dyke)
								G26	29°12'05.84"S	20°24'07.71"E	
								G27	29°10'54.29"S	20°22'59.39"E	Structure (Dyke)
								G28	29°11'52.92"S	20°21'47.43"E	Gypsum
								G29	29°11'41.92"S	20°21'53.42"E	Rock formation

APPENDIX B: THE MSSCO CLASSIFICATION SHEET TO BE USED FOR SOIL PROFILING

PROFILE No		**SUPERVISOR**		**PEDOGENIC CLASSIFICATION: FORM**
PROJECT:		**CONTRACTOR:**		
AREA:		**DATE EXCAVATED:**		**REMARKS:**
CLIENT:		**DATE PROFILED:**		
PROFILED BY:		**REVIEWED**		

	MOISTURE	**CONSISTENCY**		**STRUCTURE**	**SOIL TYPE**	**SOIL ORIGIN**			
		Cohesionless	**Cohesive**			**Transported**	**Pedogenic**	**Residual**	**Other**
SOIL	Dry	Very loose	Very soft	Intact	Boulders > 200	Hillwash	Gypcrete	Gneiss	Anthropogenic
	Slightly moist	Loose	Soft	Fissured	Cobbles 60-200	Colluvium	Ferricrete	Granite	Backfill
	Moist	Mod dense	Firm	Slickensided	Gravel 2-6-20-60	Alluvium	Silcrete	Pegmatite	
	Very moist	Dense	Stiff	Shattered	Sand	Littoral	Calcrete Soft	Quartz	
	Wet	Very dense	Very stiff	Micro-shattered	Sandy silt	Aeolian	Calcrete Powdered	Meta Quartzite	
				Stratified	Silty clay	Talus	Calcrete Nodular	Shale/schist	
				Inherent	Clay		Calcrete Honeycomb	Dyke	
							Calcrete Hardpan		
ROCK	**COLOUR**	**WEATHERING**	**FABRIC**	**DISCONTINUETIES Banded, foliated, jointed**	**HARDNESS**	**DISCONTINUITY DETAILS**			**ROCK TYPE**
						Separation	**Fill material**	**Roughness**	
	Banded	Fresh	Very fine grained	Very thinly	Very soft	Closed	Clean	Smooth	Granite/gneiss
	Streaked	Unweathered	Fine grained	Thinly	Soft	Very narrow	Stained	Slightly rough	Pegmatite

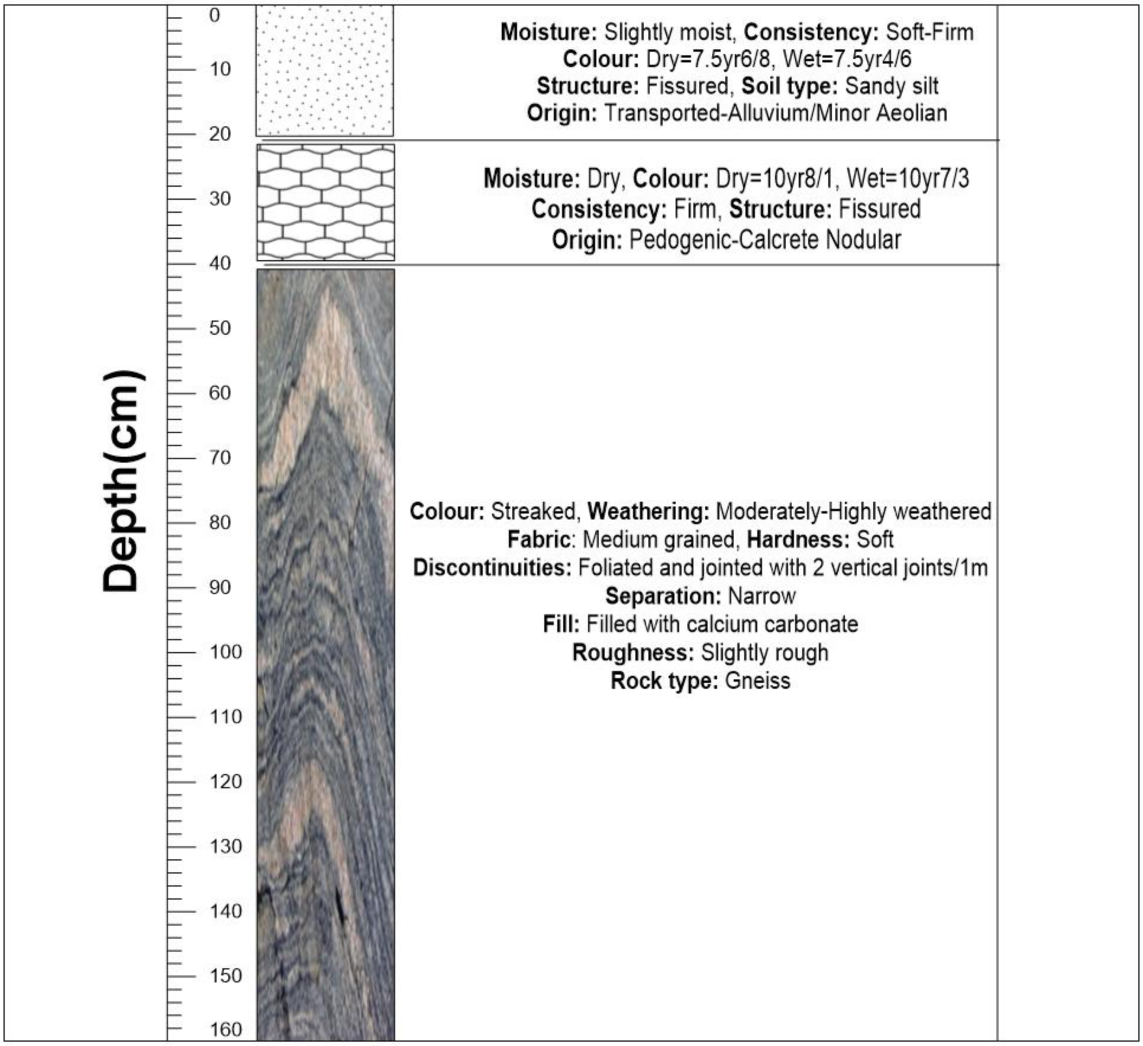
	Blotched	Slightly weathered	Medium grained	Medium	Moderately hard	Narrow	Partial	Medium rough	Quartz
	Mottled	Mod weathered	Coarse grained	Thickly	Hard	Wide	Filled	Rough	M Quartzite
	Speckled	Highly weathered	Very coarse	Very thickly	Very hard	Very wide		Very rough	Shale/schist
	Stained								Dyke

TEST PIT M or G		**Excavation**	**Seepage**		**Sidewall Stability**	**Backfilling**	**Latitude**	**Longitude**	**DCP Test**
		Refused	Slight	Strong	Good	Good			Surface
		Stopped	Moderate	Very strong	Moderate	Moderate	**Elevation**	**GPS waypoint**	Bottom of pit
		mln	Depth	mbgl	Poor	Poor			Mbgl

DEPTH (m)	**DESCRIPTION**	**SAMPLE NR**	**SAMPLE TYPE**	

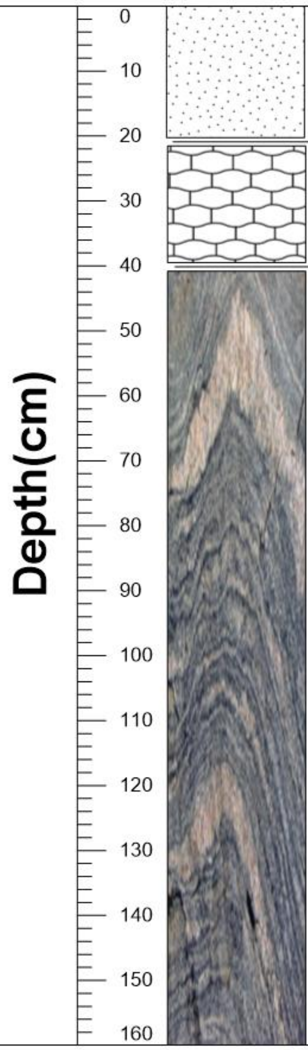
APPENDIX C: SOIL PROFILE LOGS

Profile No : M24
Project : THRIP
Area : Kakamas
Profiled by : Tshilate I and Scholtz R
Supervisor : Mr Van Deventer
Date of profile : 01 April 2017



Profile surrounded by calcretes, quartz, feldspars and gneiss outcrops

Profile No : M24
Project : THRIP
Area : Kakamas
Profiled by : Tshilate I and Scholtz R
Supervisor : Mr Van Deventer
Date of profile : 01 April 2017



Moisture: Slightly moist, **Consistency:** Soft-Firm
Colour: Dry=7.5yr6/8, Wet=7.5yr4/6
Structure: Fissured, **Soil type:** Sandy silt
Origin: Transported-Alluvium/Minor Aeolian

Moisture: Dry, **Colour:** Dry=10yr8/1, Wet=10yr7/3
Consistency: Firm, **Structure:** Fissured
Origin: Pedogenic-Calcrete Nodular

Colour: Streaked, **Weathering:** Moderately-Highly weathered
Fabric: Medium grained, **Hardness:** Soft
Discontinuities: Foliated and jointed with 2 vertical joints/1m
Separation: Narrow
Fill: Filled with calcium carbonate
Roughness: Slightly rough
Rock type: Gneiss

Profile surrounded by calcretes, quartz, feldspars and gneiss outcrops

Profile No : M23
Project : Thrip
Area : Kakamas
Profiled by : Tshilate L and Scholtz R
Supervisor : Mr Van Deventer
DAte of profile: 31 March 2017

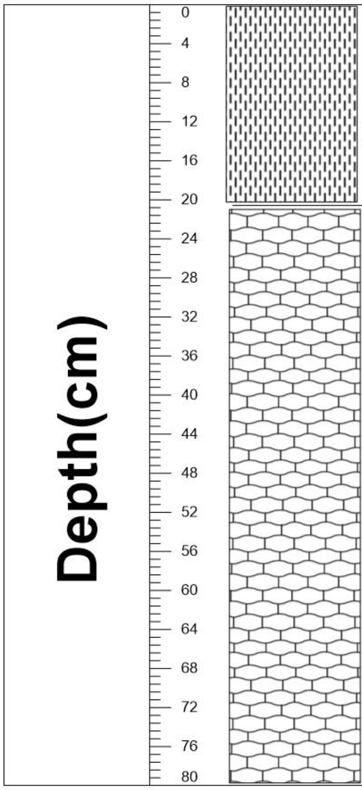
Depth(cm)

Moisture: Dry, **Consistency:** Soft-Firm
Colour: Dry-7.5yr5/6
Wet-7.5yr4/6
Structure: Fissured/Clast supported
Soil type: Silty clay
Soil Origin: Transported(Alluvium/Minor Aeolian)

Colour: Streaked, **Weathering:** Moderately weathered
Fabric: Coarse grained, **Hardness:** Moderately hard
Discontinuties: Thinly-Medium foliated
Separation: Narrow, **Fill:** Filled with Calcrete
Roughness: Rough
Rock type: Gneiss

Profile surrounded by Quartz, Green schist, Metaquartzites and gneisses

Profile No : M22
Project : Thrip
Area : Kakamas
Profiled by : Tshilate L and Scholtz R
Supervisor : Mr Van Deventer
DAte of profile: 30 March 2017

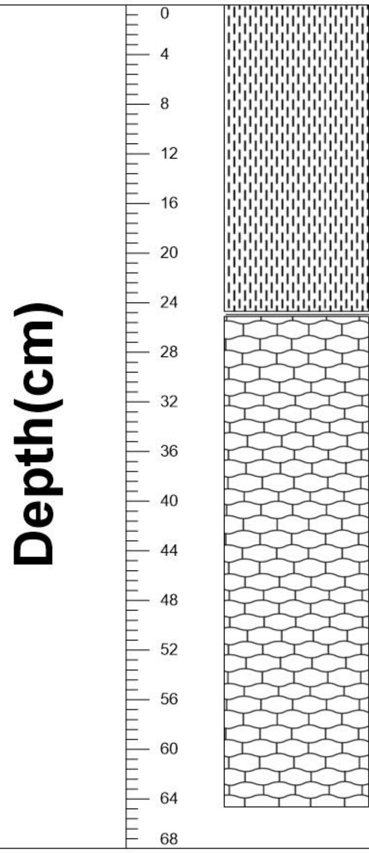


Moisture: Dry, **Consistency:** Soft-firm
Colour: Dry-7.5yr4/6
Wet-7.5yr4/4
Structure: Fissured, **Soil type:** Silty clay with boulders
Soil Origin: Transported(Alluvium/Minor Aeolian)

Moisture: Dry**, Consistency:** Stiff
Colour: Dry-5yr8/1
Wet-5yr8/3
Structure: Fissured
Origin: Pedogenic(Calcrete soft)

Profile surrounded by Quartz, Calcrete, Script granites and feldspars

Profile No : M5
Project : Thrip
Area : Kakamas
Profiled by : Tshilate L and Scholtz R
Supervisor : Mr Van Deventer
Date of profile: 30 March 2017

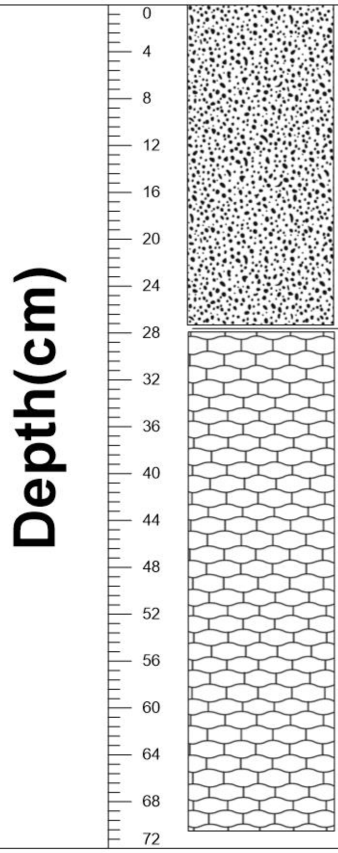


Moisture: Dry, **Consistency:** Soft-firm
Colour: Dry-7.5yr4/6
Wet-7.5yr4/4
Structure: Fissured,
Soil type: Silty clay with gravel and cobbles
Soil Origin: Transported(Alluvium/Aeolian)

Moisture: Dry, Consistency: Firm
Colour: Dry-5yr8/1
Wet-5yr8/3
Structure: Shattered, **Soil type:** Silty clay
Soil Origin: Pedogenic(Calcrete soft)

The profile is surrounde by Calcrete, Quartz, Quartzite and Pegmatite

Profile No : M6
Project : Thrip
Area :Kakamas
Profiled by : Tshilate L and Scholtz R
Supervisor : Mr Van Deventer
Date of Profile: 30 March 2017

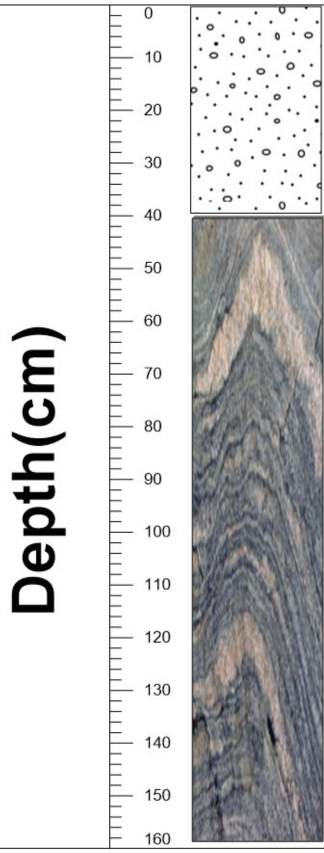


Moisture: Moist, **Consistency:** Soft-Firm
Colour: Dry-7.5yr5/4
Wet-7.5yr4/6
Structure: Fissured, **Soil type:** Sandy silt
Soil Origin: Transported(Alluvium/Monir Aeolian)

Moisture: Slightly moist, **Consistency:** Stiff-very stiff
Colour: Dry-5yr8/1
Wet7yr8/3
Structure: Shattered
Origin: Pedogenic(Calcrete Hardpan)

The profile is surrounded by Calcrete, Quartz, Gneisses and Quartzite

Profile No : M7
Project : Thrip
Area : Kakamas
Profiled by : Thsilate L and Scholtz R
Supervisor : Mr Van Deventer
Date of profile: 31March 2017

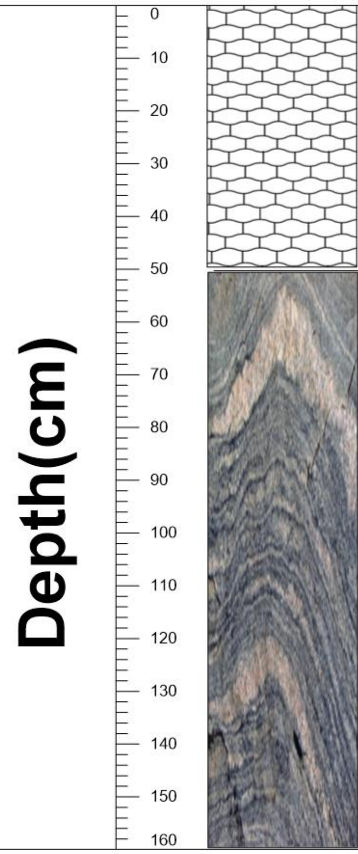


Moisture: Dry, **Consistency:** Soft-Firm
Colour: Dry-7.5yr5/6
Wet-7.5yr3/4
Structure: Fissured, **Soil type:** sandy silt with boulders
Soil origin: Transported(Alluvium/Minor Aeolian)

Colour: Streaked, **Weathering:** Moderate-Highly weathered
Fabric: Coarse grained, **Hardness:** soft
Discontinuities: foliated and jointed with 4 vertical joints/1m
Separation: Ver narrow, **Fill:** Partial fill of Calcrete
Roughness: Medium rough-rough
Rock type: Gneiss

Profile surrounded by Quartz, Gneiss and Calcrete

Profile No : M8
Project : Thrip
Area : Kakamas
Profiled by : Tshilate L and Scholtz R
Supervisor :Mr Van Deventer
Date of profile: 31 March 2017

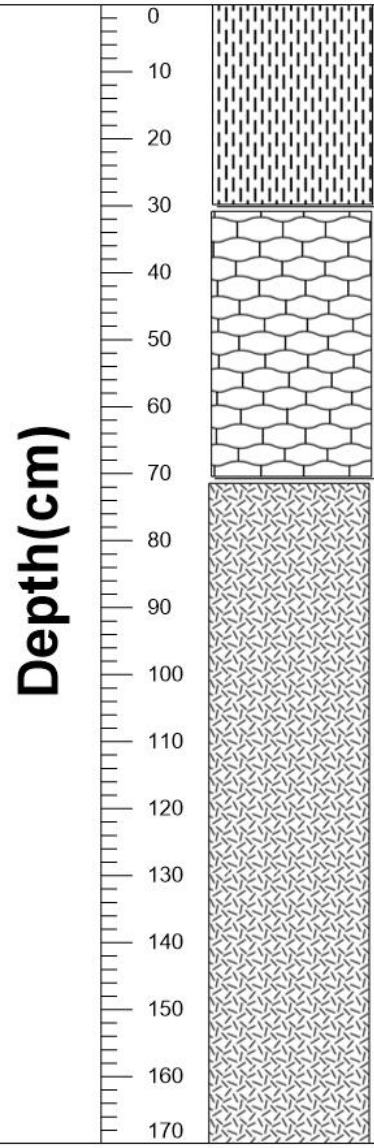


Moisture: Dry, **Consistency:** Stiff-very stiff
Colour: Dry-7.5yr8/1
Wet-7.5yr5/4
Structure: Shattered/Clast supported
Origin: Pedogenic(Calcrete hardpan)

Colour: Streaked, **Weathered:** Highly weathered
Fabric: Coarse grained, Hardness: Soft
Discontinuities: Foliated thinly
Separation: Narrow, **Fill:** Completely filled with calcrete
Roughness: Rough
Rock type: Gneiss

The profile is surrounded by Calcrete, Quartz, Gneisses and Feldspars

Profile No : M2
Project : THRIP
Area : Kakamas
Profiled by : Tshilate I and Scholtz R
Supervisor : Mr Van Deventer
Date of profile: 01 April 2017



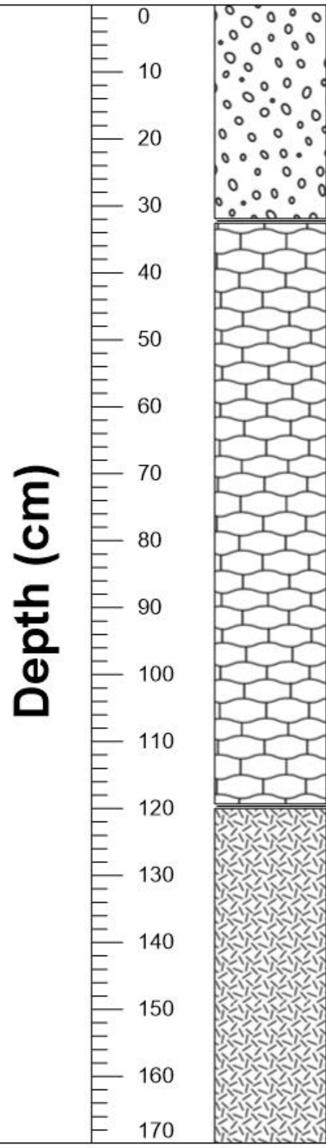
Moisture: Dry, **Consistency:** Firm-Stiff
Colour: Dry=7.5yr4/5, Wet=5yr4/8
Structure: Fissured, **Soil type:** Silty clay
Origin: Transported-Aeolian

Moisture: Dry, **Consistency:** stiff
Colour: Dry=10yr3/4, Wet=10yr4/4
Structure: Shattered
Origin: Pedogenic-Calcrete Hardpan

Colour: Blotched, **Weathering:** Moderately weathered
Fabric: Coarse grained, **Hardness:** Very Hard
Discontinuties: Thinly-Medium Foliated
Separation: Closed
Fill: Clean, **Roughness:** Rough
Rock type: Pegmatite

Profile surrounded by Feldspars, quartz and micas(potential pegmatite).
There was also some calcrete and gneiss present

Profile No : M1
Project : THRIP
Area : Kakamas
Profiled by : Tshilate L and Scholtz R
Supervisor : Mr van Deventer
Date of profile : 01 April 2017



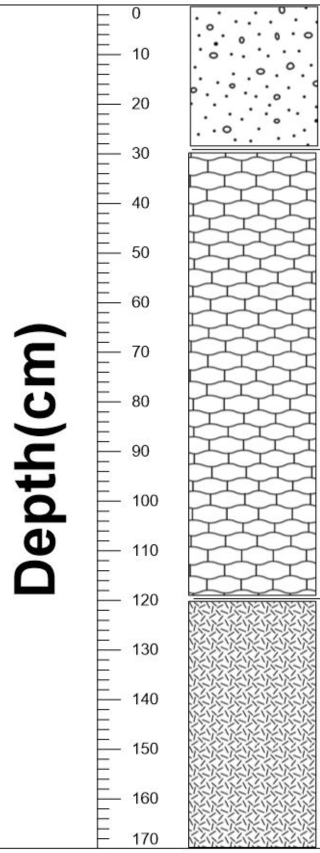
Moisture: Moist, **Soil type:** Sandy silt soil with gravel, **Consistency:**Very soft, **Structure:** Fissured, **Origin:** Transported-Alluvium/Aeolian, **Colour:** Dry=7.5yr6/6, Wet=7.5yr4/4

Moisture: Slightly moist, **Consistency:** Very stiff **Origin:** Pedogenoc- Calcrete Nodular, **Structure:** Clast and matrix supported, **Colour:** Dry=7.5yr8/4, Wet=7.5yr7/8

Colour: streaked, **Weathering:** Highly weathered, **Fabric:** Coarse grained, **Hardness:** Hard, **Discontinuity:** Foliated thickly, **Separation:** Narrow, **Fill:** partial fill of calcium carbonate **Roughness:** medium rough-rough **Rock type:** Pegmatite

Profile surrounding dominated by calcrete, quartz, feldsparsand pegmatie. There was also some nice present and a small stream with sandy material

Profile No : M1
Project : Thrip
Area : Kakamas
Profiled by : Tshilate L and Scholtz
Supervisor : Mr Van Deventer
Date of profile: 29 March 2017



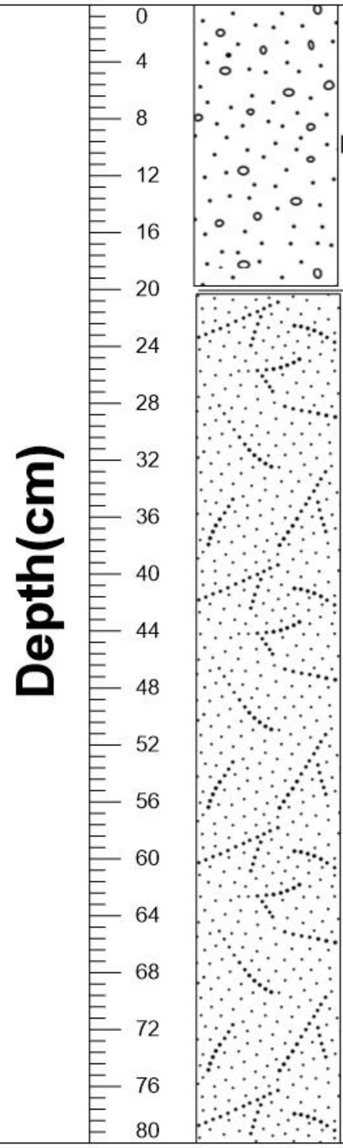
Moisture: Moist, **Consistency:** Very soft-soft
Colour: Dry-7.5yr6/6
Wet-7.5yr4/4
Structure: Fissured, **Soil type:** Sandy silt with gravel
Soil Origin: Transported(Alluvium/Minor Aeolian)

Moisture: Slightly moist, Consistency: Very stiff
Colour: Dry-7.5yr8/4
Wet-7.5yr7/8
Structure:50/50 Clast and Matrix supported
Soil type: Silty clay with gravel
Soil origin: Pedogenic(Calcrete Nodular)

Colour: streaked, **Weathering:** Moderately -highly weathered
Fabric: Coarse grained, **Hardness:** soft
Discontinuities: Medium-thickly foliated
Separation: Very narrow-Narrow
Fill: partial fill of Calcium carbonate
Roughness: Medium rough

Profile is surrounded by a small stream with sandy material, Calcretes, Quartz,Feldspars and pegmatites

Profile No : G29
Project : THRIP
Area : Kakamas
Profiled by : Tshilate L
Supervisor : Mr Van Deventer
Date of profile: 01 April 2017

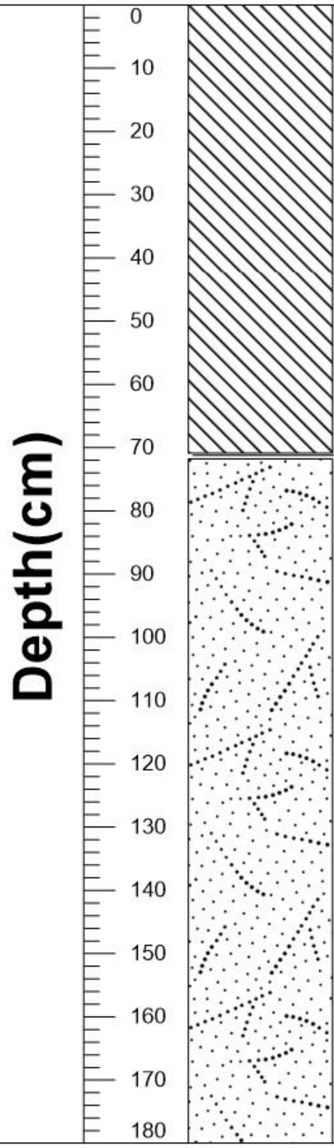


Moisture: Dry, **Consistency:** Soft, **Structure:** Fissured
Colour: Dry=10yr6/6, Wet=10yr6/8
Soil type: Sandy silt with gravel
Origin: Transported-Alluvium/Minor Aeolian

Colour: Streaked, **Weathering:** Moderately weathered
Fabric: Fine grained, **Hardness:** Hard-Very hard
Discontinuities: Foliated and jointed with 15 vertical joints/meter
Separation: Narrow, **Fill:** Clean, **Roughness:** Smooth
Rock type: Meta-Quartzite

Abundant quartzite and pegmatite outcroups around the profile

Profile No : G23
Project : THRIP
Area : Kakamas
Profiled by : Tshilate I and Scholtz
Supervisor : Mr Van Deventer
Date of Profile : 01 April 2017

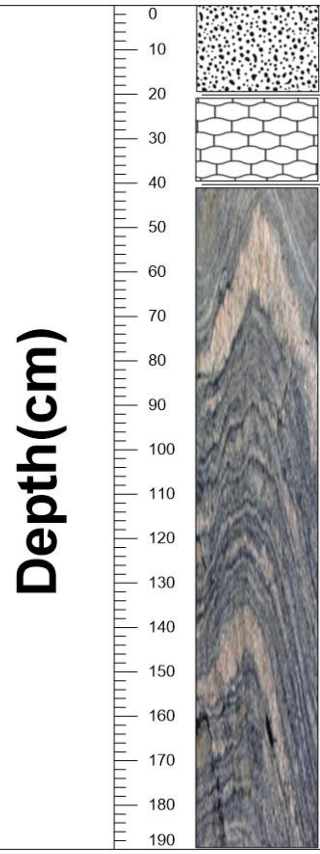


Moisture: Moist, **Consistency:** Firm
Colour: Dry=10yr8/4, Wet=10yr8/6
Structure: Fissured, **Soil type:** Clay
Origin: Pedogenic-Gypcrete

Colour: Mottled, **Weathering:** Highly wethered
Fabric: Fine grained, **Hardness:** Moderately hard
Discontinuities: Foliated thinly, **Separation:** Narrow
Fill: Filled with gypcrete, **Roughness:** Smooth
Rock type: Quartzite

Profile surrounded by Quartz, calcrete, feldspars and quartzite

Profile No : G25
Project : Thrip
Area : Kakamas
Profiled by : Tshilate L and Scholtz R
Supervisor : Mr Van Deventer
Date of profile: 30 March 2017



Moisture: Moist, **Consistency:** Soft
Colour: Dry-7.5yr5/6
Wet-7.5yr3/3
Structure: Fissured, **Soil Type:** Sandy silt
Soil Origin: Transported(Alluvium/Minor Aeolian)

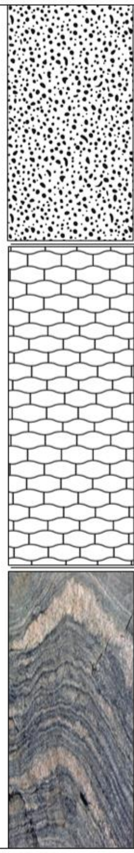
Moisture: Moist, **Consistency:** Firm
Colour: Dry-7.5yr8/1
Wet-7.5yr8/3
Structure: Shattered, **Origin:** Pedogenic(Calcrete Powder)

Colour: Streaked, **Weathering:** Highly weathered
Fabric: Medium-coarse grained
Discontinuities: Foliated thinly, **Separation:** Narrow
Fill: Clean, **Hardness:** Very soft-soft
Roughness: Rough-very rough
Rock type: Gneiss

The profile is surrounded by Calcretes, Quartz, Feldspars, Pegmatites and gneisses

Profile No : G26
Project : Thrip
Area : Kakamas
Profiled by : Tshilate L and Scholtz R
Supervisor : Mr Van Deventer
Date of profile: 30 March 2017

Depth(cm)	
0–50	**Moisture:** Moist, **Consistency:** Soft **Colour:** Dry-7.5yr5/6 Wet-7.5yr4/4 **Structure:**Fissured, **Soil type:** Sandy silt **Soil Origin:** Transported(Alluvium/Minor Aeolian)
50–120	**Moisture:** Moist, **Consistency:** Firm **Colour:** Dry-10yr8/3 Wet-10yr6/4 **Structure:** Intact **Soil Origin:** Pedogenic(Calcrete powder)
120–180	**Colour:** Streaked, **Weathering:** Highly weathered **Fabric:** Medium-coarse grained **Discontinuities:** Foliated very thinly, **Hardness:** Soft **Separation:** Closed, **Fill:** Clean, **Roughness:** Rough **Rock type:** Gneiss



The profile is surrounded by Gneisses, Quartz and Calcretes

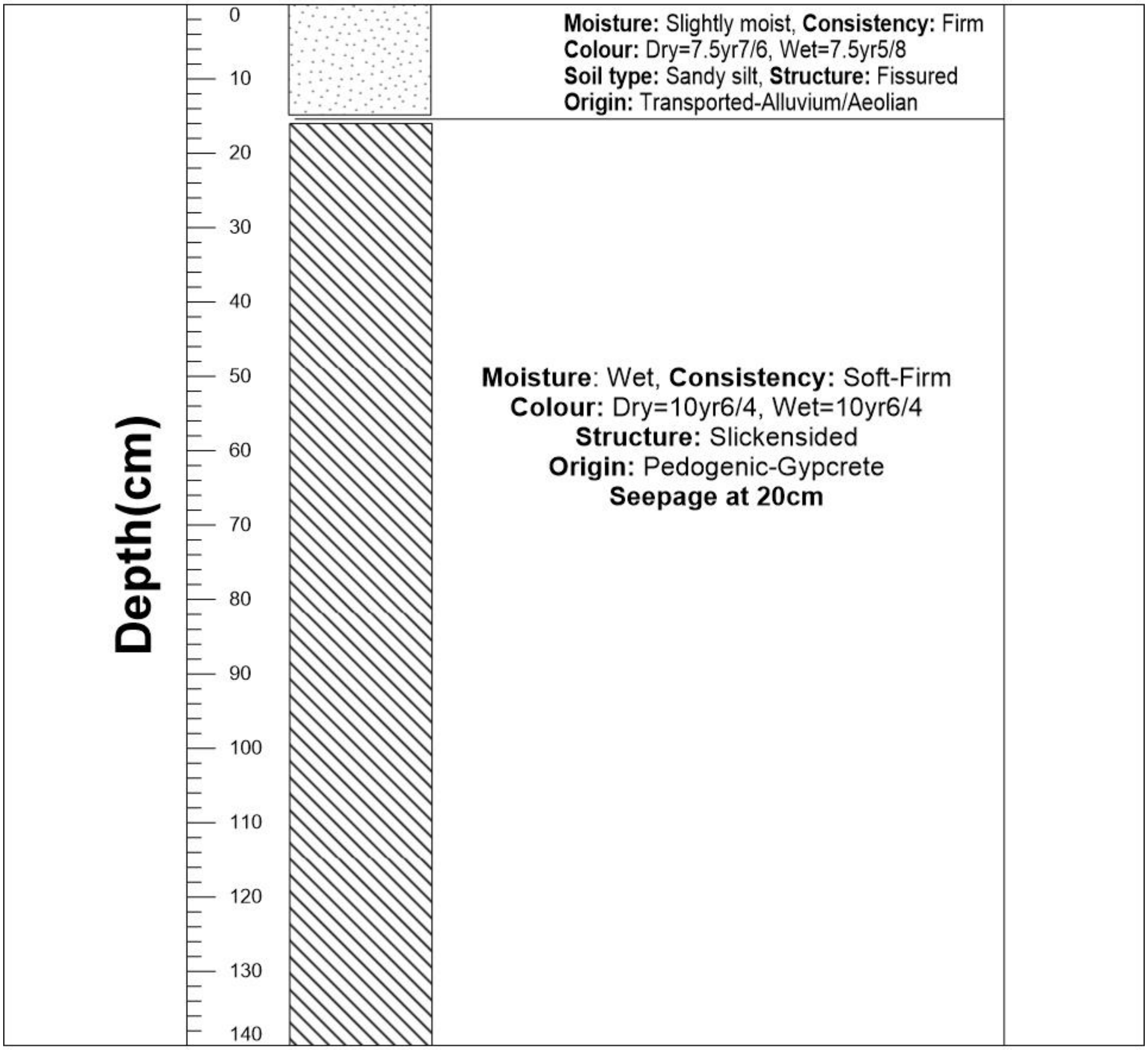
Profile No : G27
Project : THRIP
Area : Kakamas
Profiled by : Tshilate L and Scholtz R
Supervisor : Mr Van Deventer
Date of profile : 30 March 2017

Depth (cm)	Description
0–20	**Moisture:** Dry, **Consistency:** Soft-Firm **Colour:** Dry=7.5yr5/6, Wet=7.5yr4/6 **Structure:** Fissured, **Soil type:** Sandy silt with gravel **Origin:** Transported-Alluvium/Minor Aeolian
20–40	**Moisture:** Slightly moist, Colour: Reddish white **Consistency:** Firm-Stiff, **Structure:** Transition zone **Soil type**: Sandy silt with cobbles **Origin**: Transported-Alluvium(Elluviated into residual)
40–120	**Colour:** Streaked, **Weathering:** Moderately weathered **Fabric:** Coarse grained, **Hardness**: Hard **Discontinuities:** Foliated and jointed with 1 vartical joint **Separation:** Narrow, **Fill:** filled with calcium carbonate **Roughness:** Rough **Rock type:** Gneiss

Depth(cm)

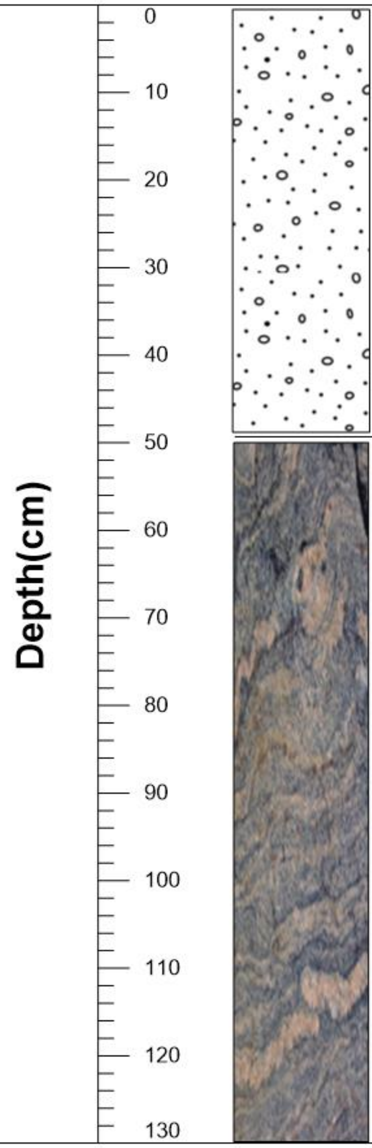
Abundant calcrete, gneiss and quartz around the profile

Profile No : G28
Project : THRIP
Area : Kakamas
Profiled by : Tshilate L and Scholtz R
Supervisor : Mr Van Deventer
Date of profile: 01 April 2017



Profile surrounded by Schist, quartz and feldspars

Profile No : G22
Project :THRIP
Area :Kakamas
Profiled by :Tshilate L
Supervisor :Mr Van Deventer
Date of profile :01 April 2017



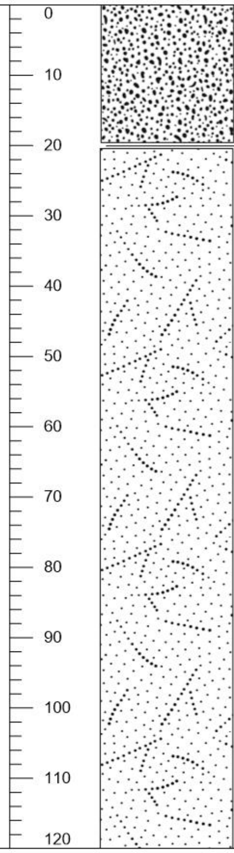
Moisture: Dry, **Consistency:** Soft-firm,
Structure: Shattered
Soil type: Sandy silt with gravel and cobbles
Colour: Dry=7.5yr7/8, Wet=7.5yr5/8
Origin: Transported- Alluvium/minor Aeolian
(There is calcerete soft with quartz pebbles on this layer)

Colour: Streaked, **Weathering:** Highly weathered
Fabric: Coarse grained, **Hardness:** Hard
Discontinities: Foliated and jointed with 1 vertical joint
Separation: Closed, **Fill:** Clean, **Roughness:** Rough
Rock type: Gneiss

Profile is surrounded by Quarts, calcrete, feldspars, granites and micas

Profile No : G21
Project : Thrip
Area : Kakamas
Profiled by : Tshilate L and Scholtz R
Supervisor : Mr Van Deveter
Date of profile: 30 March 2017

Depth(cm)

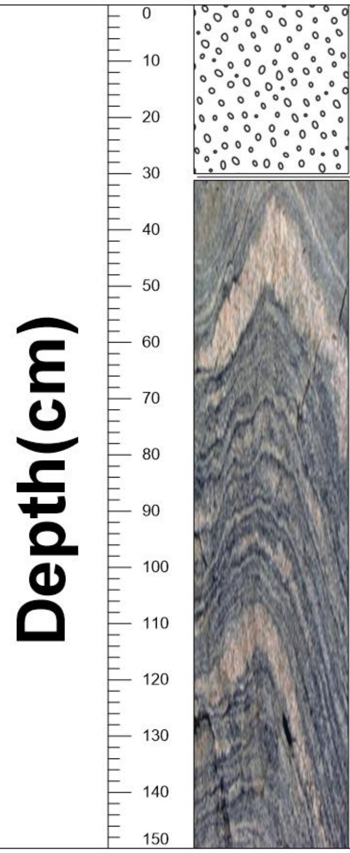


Moisture: Dry, **Consistency:** Soft-firm
Colour: Dry-7.5yr5/8
Wet-7.5yr3/4
Structure: Fissured, **Soil type:** Sandy silt
Soil Origin: Transported(Alluvium/Minor Aeolian)

Colour: Blotched, **Weathering:** Slightly weathered
Fabric: Fine-Medium grained
Discontinuities: Medium foliated
Hardness: Hard-very hard, **Separation:** Wide
Fill: Completely filled with calcium carbonate
Roughness: Medium rough-rough
Rock type: Pegmatite and quartzite

The profile is surrounded by Gneisses, Quartz and Pegmatites

Profile No : G20
Project : Thrip
Area : Kakamas
Profiled by : TshilateL and Scholtz R
Supervisor : Mr Van Deventer
Date of profile: 29 March 2017

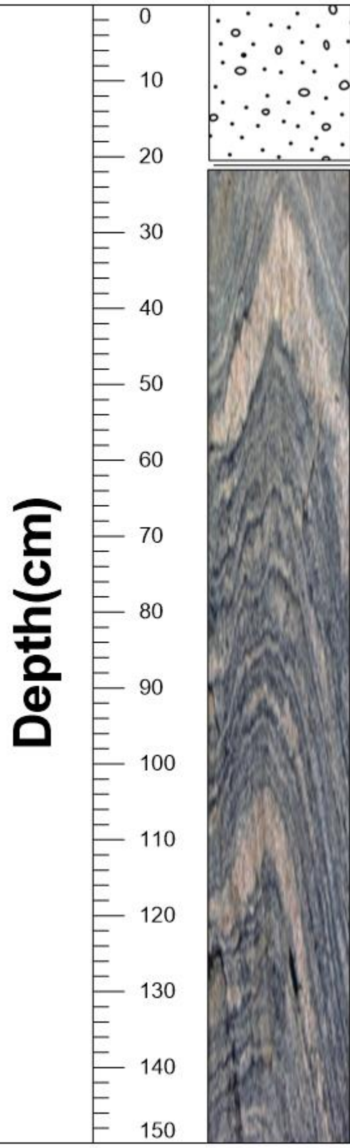


Moisture: Dry, **Consistency:** Soft
Colour: Dry-7.5yr5/8
Wte-7.5yr4/4
Structure: Clast supported, **Soil type:** Silty clay with boulders and cobbles
Soil Origin: Transported(Alluvium/Minor Aeolian)

Colour: streaked, **Weathering:** Highly weathered
Fabric: Medium-coarse grained
Discontinuities: foliated and jointed with 7 vertical joints/2m
Hardness: Soft, **Separation:** Closed
Fill: Clean, **Roughness:** Rough
Rock type: Gneiss

There are conglomerates, Calcrete,Feldspars, Quartz, Quartzite and Gneisses around the profile

Profile No : G18
Project : THRIP
Area : Kakamas
Profiled by : Tshilate L and Scholtz R
Supervisor : Mr Van Deventer
Date of profile: 10 April 2017

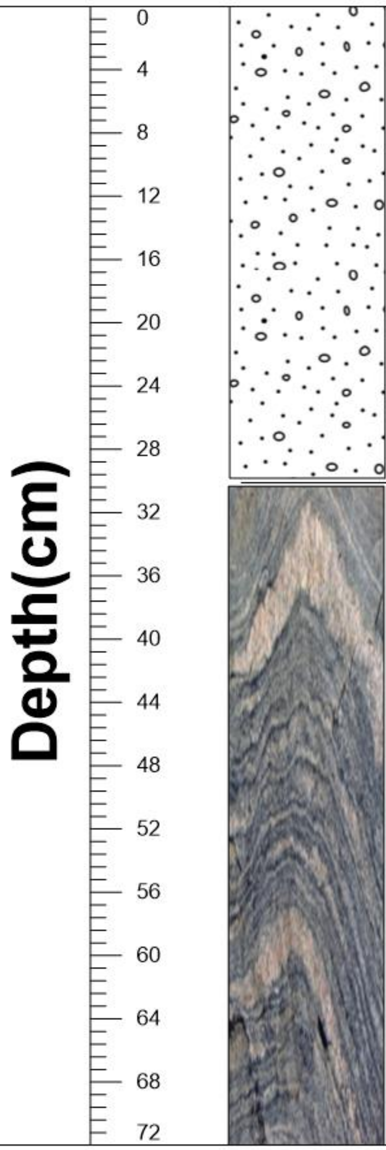


Moisture: Dry-Slightly moist, **Structure:** Fissured
Colour: Dry=10yr5/4, Wet=10yr4/4
Consistency: Soft-Firm, **Soil type:** Sandy silt with gravel
Origin: Transported-Alluvium/Minor Aeolian

Colour: Streaked, **Weathering:** Moderately weathered
Fabric: Medium-Coarse grained, **Hardness:** Soft
Discontinuities: Foliated and jointed with 8 joint/meter
Separation: Narrow, **Fill:** Carbonate fill
Roughness: Rough, **Rock type:** Gneiss

Abundant quartz, feldspars, gneiss and calcretes around the profile

Profile No : G16
Project : THRIP
Area : Kakamas
Profiled by : Tshilate L and Scholtz R
Supervisor : Mr Van Deventer
Date of Profile : 30 March 2017

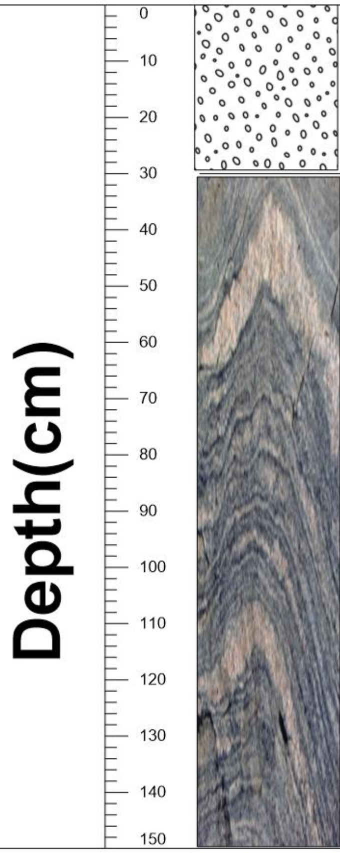


Moisture: Dry, **Consistency:** Soft
Colour: Dry=7.5yr5/6, Wet=7.5yr4/6
Structure: Fissured, **Soil type:** Sandy silt and gravel
Origin: Transported-Alluvium/Minor Aeolian

Colour: Streaked, **Weathering:** Slightly weathered
Fabric: Coarse grained, **Hardness:** Soft
Discontinuities: Thinly banded/Foliated
Separation: Narrow, **Fill:** Clean
Roughness: Slightly Rough
Rock type: Gneiss

Abundant quartz, gneiss and feldspars around the profile

Profile No : G9
Project : Thrip
Area : Kakamas
Profiled by : Tshilate L and Scholtz R
Supervisor : Mr Van Deventer
Date of profile: 29 March 2016

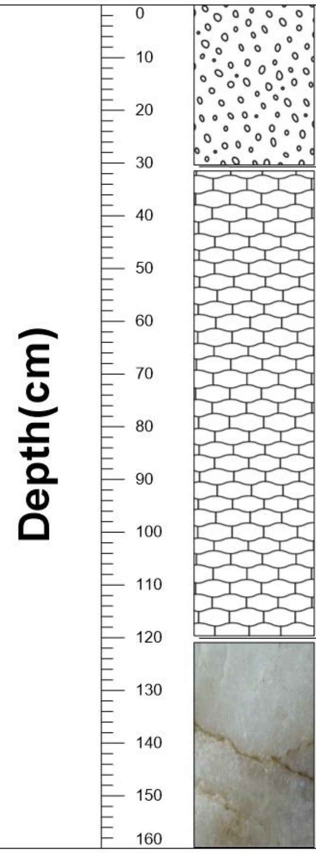


Moisture: Dry, **Consistency:** Firm
Colour: Dry-7.5yr4/6
Wet-7.5yr3/4
Structure: Fissured, **Soil type:** Sandy silt
Soil Origin: Transported(Alluvium/Minor Aeolian)

Colour: Streaked, **Weathering:** Highly weathered
Fabric: Coarse grained, **Hardness:** Very soft
Discontinuities: Foliated thinly
Separation: Narrow, **Fill:** Filled with Calcium Carbonate
Roughness: Rough
Rock type: Gneiss

Profile is surrounded by Quartz, Gneisses and Feldspars

Profile No : G10
Project : Thrip
Area : Kakamas
Profiled by : Tshilate L and Scholtz R
Supervieor : Mr Van Deventer
Date of Profile: 30 March 2017



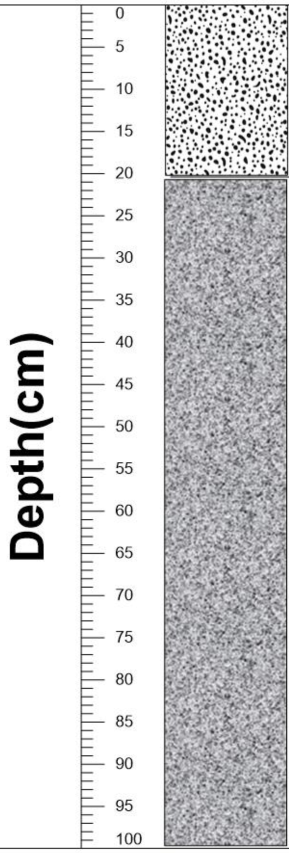
Moisture: Dry, **Consistency:** Soft
Colour: Dry-7.5yr5/6
Wet-7.5yr4/6
Structure: Fissured, **Soil type:** Sandy soil
Soil Origin: Transported(Alluvium)

Moisture: Dry, **Consistency:** stiff
Colour: Dry-7.5yr8/2
Wet-10yr8/2
Structure: Intact
Origin: Pedogenic(Calcrete soft)

Colour: Blotched, **Weathering:** Slightly weathered
Fabric: FIne-medium grained
Discontinuities: Foliated thinly, **Fill:** Clean
Separation: Closed, **Hardness:** Very hard
Roughness: Smooth
Rock type: Quartzite

Quartz, Calcrete and Quartzite around the profile

Profile No : G11
Project : Thrip
Area : Kakamas
Profiled by : Tshilate L and Scholtz R
Supervisor : Mr Van Deventer
Date of profile: 30 March 2017

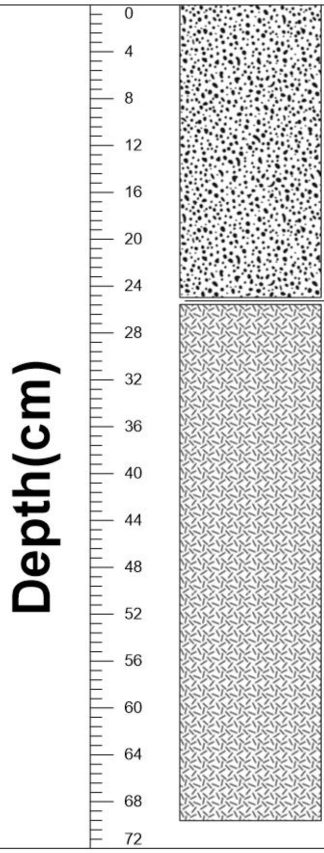


Moisture: Moist, **Consistency:**Soft
Colour: Dry-7.5yr5/6
Wet-7.5yr3/4
Structure: Fissured, **Soil type:** Sandy silt
Soil Origin: Transported(Alluvium/Minor Aeolian)

Colour: Streaked, **Weathering:**Moderately-highly weathered
Fabric: Medium-Coarse grained
Discontinuities: Foliated thinly
Separation: Very narrow, **Fill:** Filled with Calcium carbonate
Hardness: Moderately Hard
Roughness: Rough
Rock type: Script Granite

Profile is surrounded by Quartz, Feldspars , Calcrete, Gneisses and script granites

Profile No : G12
Project : Thrip
Area : Kakamas
Profiled by : Tshilate L and Scholtz R
Supervisor : Mr Van Deventer
Date of profile: 31 March 2017



Moisture: Dry, **Consistency:** Soft-firm
Colour: Dry-7.5yr5/6
Wet-7.5yr3/4
Structure: Fissured, **Soil type:** Sandy silt
Soil Origin: Transported(Alluvium/Minor Aeolian)

Colour: Blotched, **Weathering**: Moderately weathered
Fabric: Coarse grained
Discontinuities: Foliated thickly
Hardness: Hard-very hard, **Fill:** Filled with Calcrete
Separation: Wide, **Roughness:** Rough-very rough
Rock type: Pegmatite

There is Quartz, Calcrete, Metaquartzite and gneisses aroung the profile(Potential pegmatite)

Profile No : G13
Project : Thrip
Area : Kakamas
Profiled by : Tshilate L and Scholtz R
Supervisor : Mr Van Deventer
Date of profile: 31 March 2017

Depth(cm)

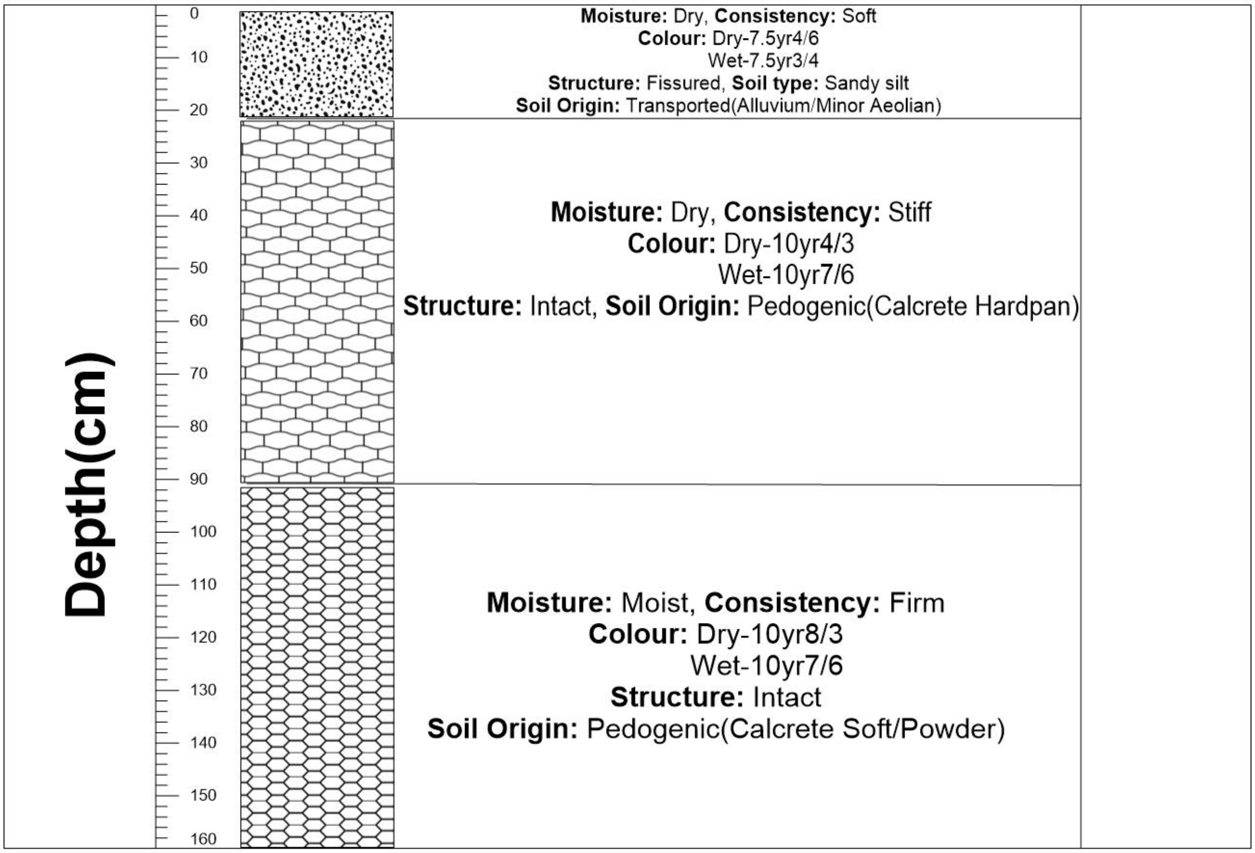
Moisture: Dry, **Consistency:** Firm, **Structure:** Fissured
Colour: Dry-7.5yr6/6, **Soil type:** Sandy silt
Wet-7.5yr4/6
Soil Origin: Transported(Alluvium/Minor Aeolian)

Colour: Streaked, **Weathering:**Moderately-highly weathered
Fabric: Coarse-very coarse grained
Discontinuities: Foliated thickly
Separation: Wide, **Fill:** Filled with calcrete lenses
Hardness: Soft, **Roughness:** Rough-very rough
Rock type: Calcrete and gneiss

Colour: Streaked, **Weathering:** Moderately weathered
Fabric: Medium-coarse grained
Discontinuities: Foliated and jointed with 13 vertical joints/1m
Separation: Very narrow-narrow
Fill: Filled with calcrete, **Hardness:** Soft
Roughness: Rough
Rock type: Gneiss

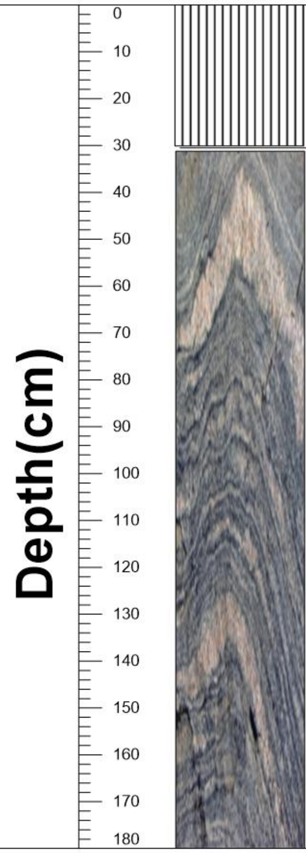
Profile surrounded by Calcrete, Quartz, Feldspars and weathered gneiss

Profile No : G14
Project : Thrip
Area : Kakamas
Profiled by : Tshilate L and Scholtz R
Supervisor : Mr Van Deventer
Date of profile: 31 March 2017



The profile is surrounded by Quartz and Calcrete

Profile No : G8
Project : Thrip
Area :Kakamas
Profiled by : Tshilate I and Scholtz R
Supervisor : Mr Van Deventer
Date of profile: 30 March 2017

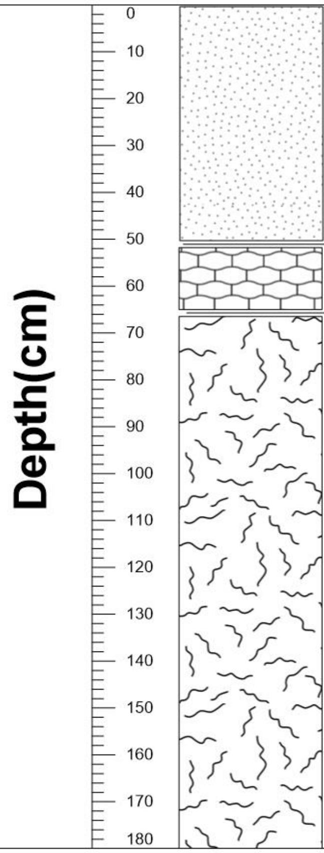


Moisture: Dry, **Consistency:** Soft-firm
Colour: Dry-7.5yr8/6
Wet-7.5yr3/3
Structure: Clast supported/Fissured
Soil type: Silty
Soil Origin: Transported(Alluvium/Aeolian)

Colour: Streaked, **Weathering:** Medium-Highly weathered
Fabric: Coarse grained, **Hardness:** Soft
Discontinuities: Banded and foliated thinly
Separation: Narrow, **Fill:** Clean
Roughness: Rough-very rough
Rock type: Gneiss

There is Quartz, Feldspars, Calcrete, Gneisses and micas surrounding the profile

Profile No : G7
Project : Thrip
Area : Kakamas
Profiled by : Tshilate L and Scholtz R
Supervisor : Mr Van Deventer
Date of profile: 30 March 2017



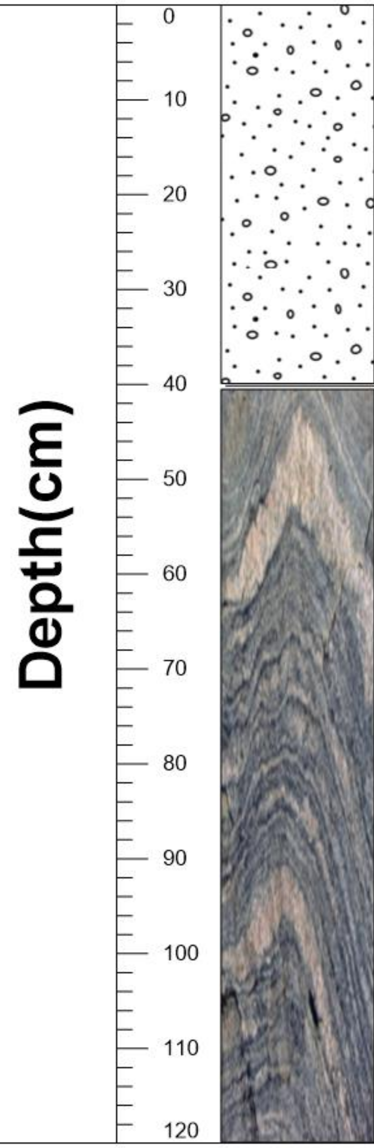
Moisture: Moist, **Consistency:**Soft
Colour: Dry-7.5yr5/6
Wet-7.5yr4/4
Structure: Fissured, **Soil type:** Sandy silt
Soil Origin: Transported(Alluvium/Minor Aeolian)

Moisture: Moist, **consistency:** Stiff, **Structure:** Shattered
Colour: Dry-7.5yr8/1
Wet-7.5yr8/2
Soil Origin: Pedogenic(Calcrete hardpan)

Colour: Streaked, **Weathering:** Slightly weathered
Fabric: Fine grained
Discontinuities: Foliated thinly
Hardness: Hard, **Separation:** Narrow
Fill: Filled with micas, **Roughness:**Smooth
Rock type: Quartz

The soil profile is surrounded by Quartz, Feldspars, Quartzite and calcrete

Profile No : G6
Project : THRIP
Area : Kakamas
Profiled by : Tshilate L and Scholtz R
Supervisor : Mr Van Deventer
Date of profile: 01 April 2017



Moisture: Slightly moist, **Consistency:** Soft-Firm
Colour: Dry=7.5yr6/4, Wet=7.5yr4/4
Structure: Fissured, **Soil type:** Sandy silt with gravel
Origin: Transporetd-Alluvium/Minor Aeolian

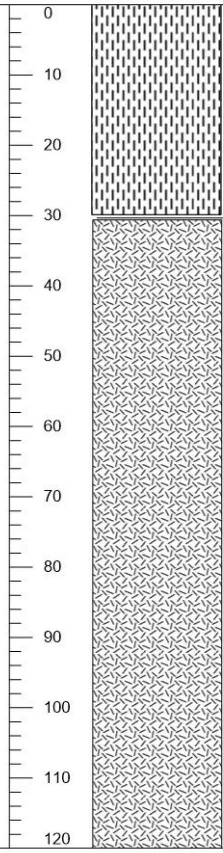
Colour: Streaked, **Weathering:** Slightly-Moderately weathered
Fabric: Coarse grained, **Hardness:** Soft
Discontinuities: Foliated and Banded thickly
Separation: wide, **Fill:** Calcrete fill, **Roughness:** Rough
Rock type: Gneiss

Abundant quartz and calcrete around the profile

University of Venda
Creating Future Leaders

Profile No : G5
Project : Thrip
Area : Kakamas
Profiled by : Tshilate L and Scholtz R
Supervisor : Mr Van Deventer
Date of profile: 30 March 2017

Depth(cm)

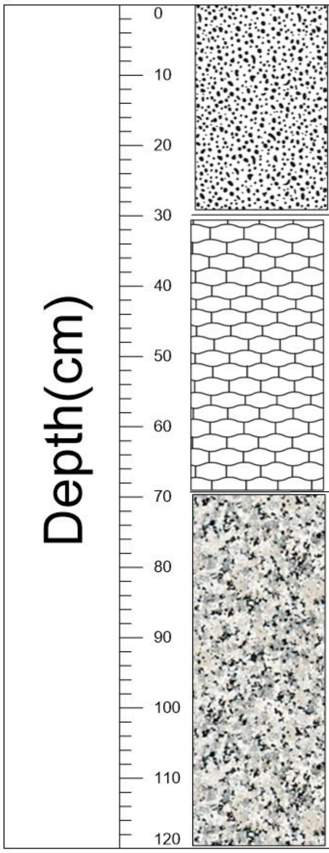


Moisture: Dry, **Consistency:** Soft-Firm
Colour: Dry-7.5yr4/6
Wet-7.5yr7/6
Structure: Fissured, **Soil type:** silty clay
Soil origin: Transported(Alluvium/Minor aeolian

Colour: Streaked, **Weathering:** Highly weathered
Fabric: Coarse grained, **Hardness:** Soft
Discontinuities: Foliated and jointed with 2 vertical joints/1m
Separation: Very narrow, **Fill:**clean
Roughness: Rough-Very rough
Rock type: Pegmatite

Profile surrounded by Quartz, gneisses and metaquartzite

Profile no : G4
Project : Thrip
Area : Kakamas
Profiled by : Tshilate L and Scholtz R
Supervisor : Mr Van Deventer
Date of profile: 30 March 2017



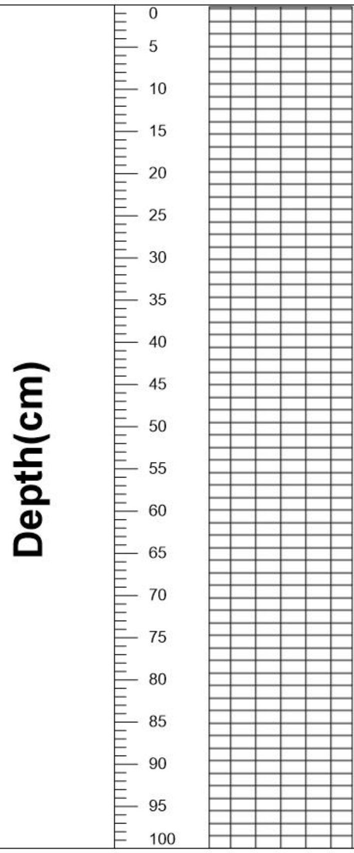
Moisture: Dry, **Consistency:** Soft-Firm
Colour: Dry-7.5yr5/6
Wet-7.5yr3/4
Structure: Fissured
Soil type: Silty clay with gravel adn cobbles
Soil origin: Transported(Alluvium)

Moisture: Dry, **Consistency:** Stiff
Structure: Shattered
Colour: Dry-5yr8/3
Wet-5yr8/1
Soil origin: Pedogenic(Calcrete Nodular)

Colour: Streaked, **Weathering:** Slightly weathered
Fabric: Medium grained
Discontinuities: foliated thinly
Fill: Filled with calcium carbonate
Separation: Wide
Hardness: Hard
Roughness: Slightly rough

The profile is surrounded by Quartz, Feldspars, Script granite and pegmitites

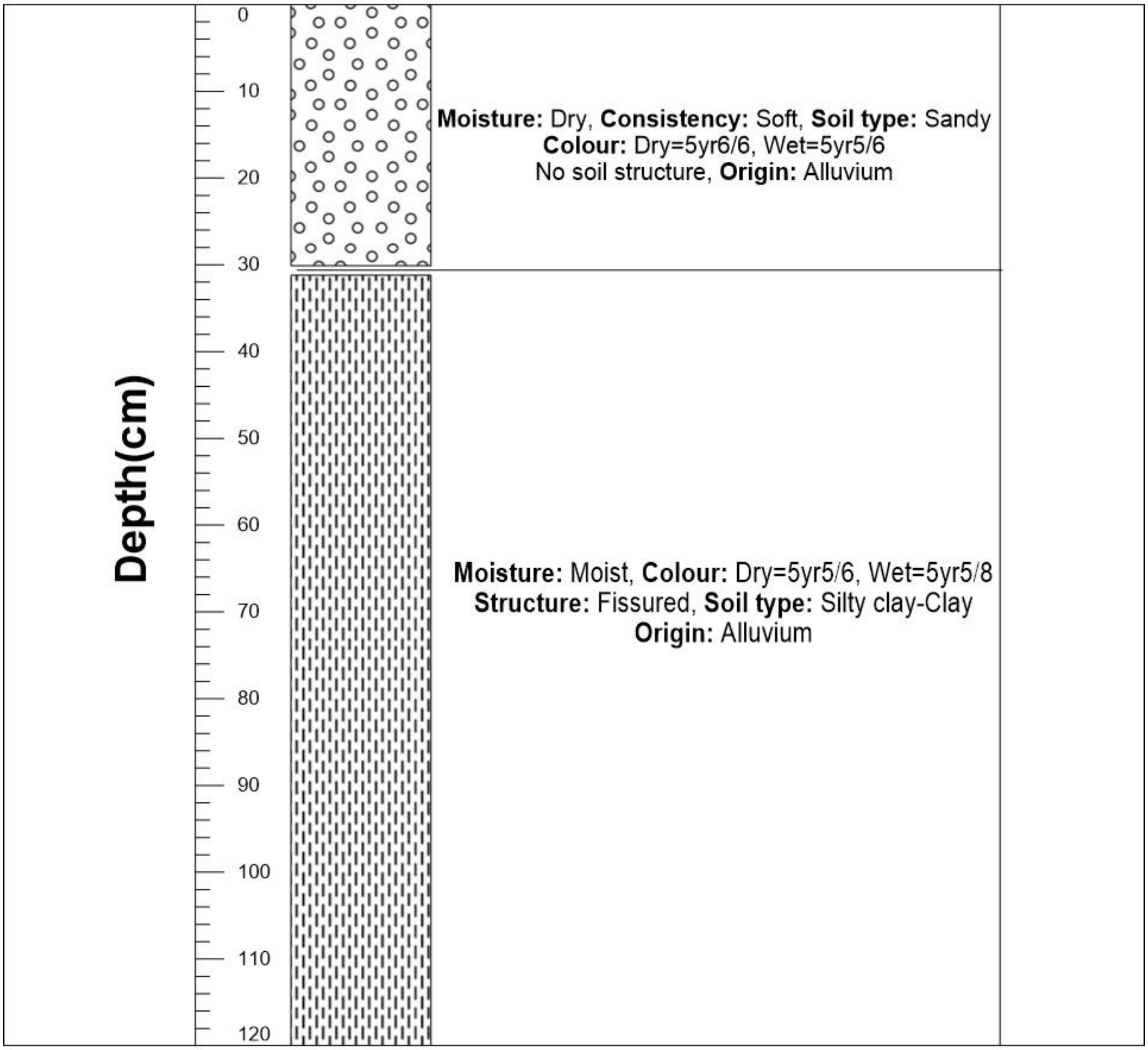
Profile No : G3
Project : Thrip
Area : Kakamas
Profiled by : Tshilate L
Supervisor : Mr Van Deventer
Date of profile: 1 April 2017



Moisture: Dry
Colour: Dry-7.5yr4/6
Wet-5yr3/4
Consistency: Firm-Stiff
Structure: Honeycomb structure with 1joint
Soil type: Sandy silt
Soil Origin: Pedogenic(ferricrete)

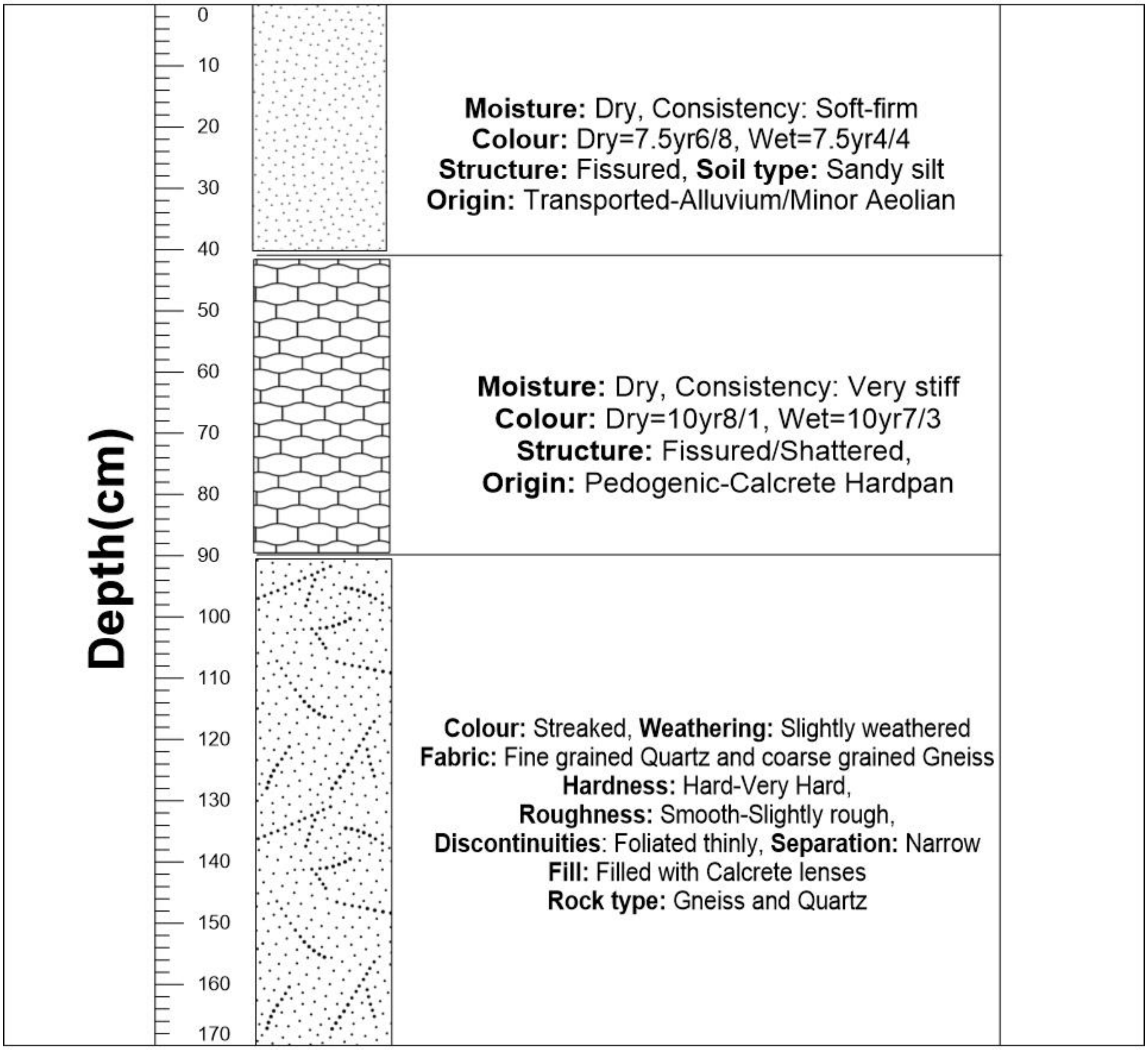
Profile is surrounded by Dorbank with gneisses and sandy soil

Profile No : G2
Project : THRIP
Area : Kakamas
Profiled by : Scholtz R
Supervisor : Mr Van Deventer
Date of profile: 01 April 2017



Profile surrounded by a stream with lots and lots of well graded sand

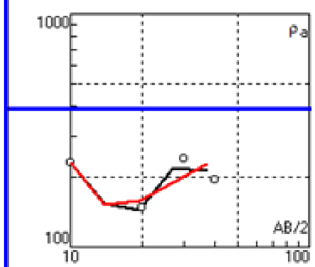
Profile No : G1
Project : THRIP
Area : Kakamas
Profiled by : Scholtz R
Supervisor : Mr Van Deventer
Date of profile: 01 April 2017



Quartz and feldspars around the soil profile

APPENDIX D: VERTICAL ELECTRICAL SOUNDING GRAPHS OF SELECTED SOIL PROFILES

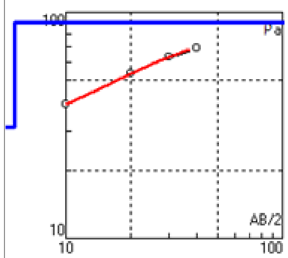
M9



M9

N	ρ	h	d	Alt
1	1766	2.34	2.34	-2.343
2	37.4	2.83	5.17	-5.174
3	394			

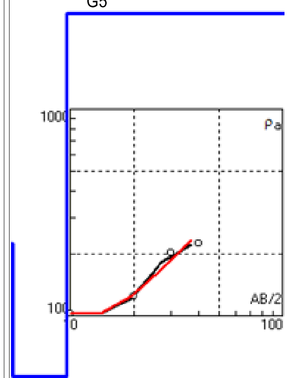
G23



G23

N	ρ	h	d	Alt
1	30.9	5.753	5.753	-5.7529
2	89.98			

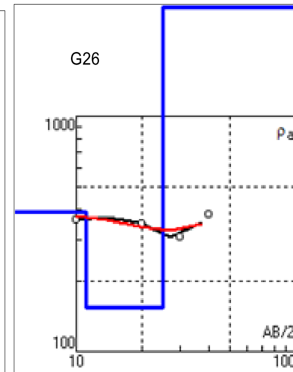
G5



G5

N	ρ	h	d	Alt
1	223	2.72	2.72	-2.717
2	50.9	6.86	9.57	-9.573
3	32054			

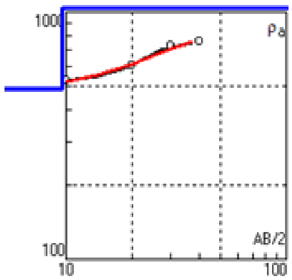
G26



G26

N	ρ	h	d	Alt
1	392	11	11	-11.04
2	154	13.7	24.8	-24.77
3	23411			

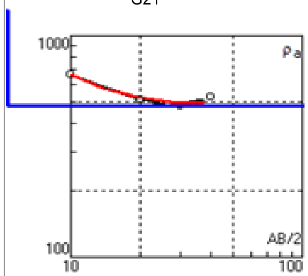
M22



M22

N	ρ	h	d	Alt
1	490.8	9.625	9.625	-9.6245
2	1035			

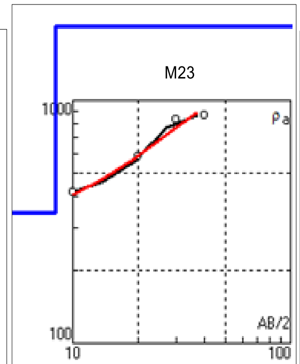
G21



G21

N	ρ	h	d	Alt
1	1295	2.66	2.66	-2.659
2	485			

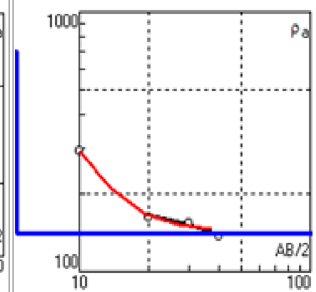
M23



M23

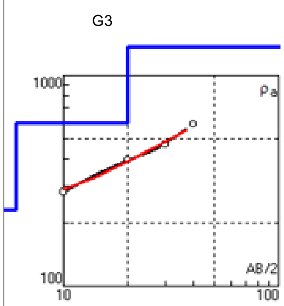
N	ρ	h	d	Alt
1	343	8.36	8.36	-8.362
2	2010			

G20



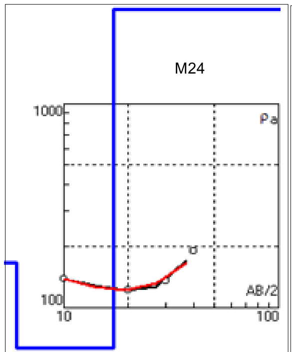
G20

N	ρ	h	d	Alt
1	705.5	3.128	3.128	-3.1281
2	141			



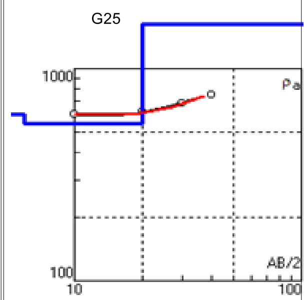
G3

N	ρ	h	d	Alt
1	230.3	6	6	-6
2	596.8	14	20	-20
3	1362			

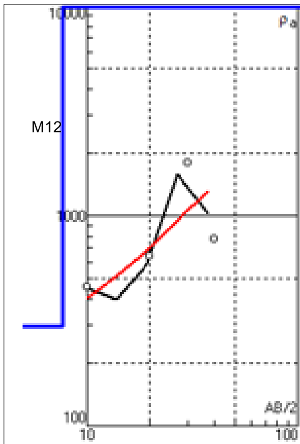


M24

N	ρ	h	d	Alt
1	165	6	6	-6
2	63.5	11	17	-16.95
3	10371			

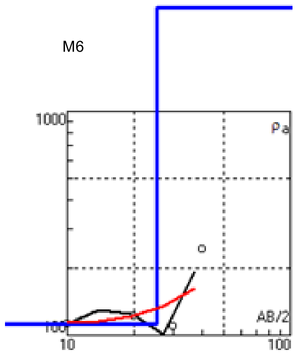


N	ρ	h	d	Alt
1	613.6	6	6	-6
2	549.3	14	20	-20
3	1622			



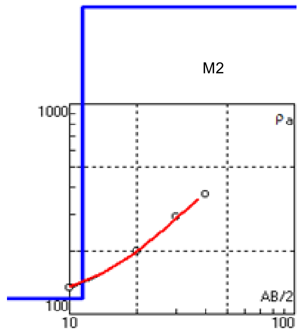
M12

N	ρ	h	d	Alt
1	296	7.63	7.63	-7.632
2	52668			



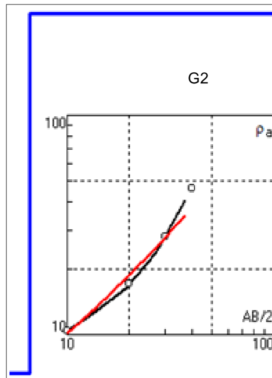
M6

N	ρ	h	d	Alt
1	112	25.3	25.3	-25.33
2	4586	87.1	112	-112.4
3	10172			



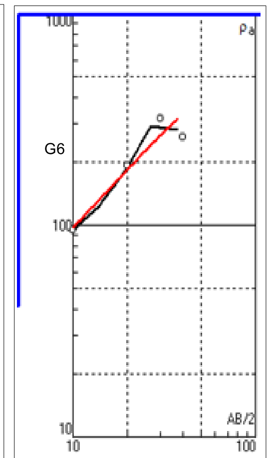
M2

N	ρ	h	d	Alt
1	117.9	11.37	11.37	-11.373
2	34366			



G2

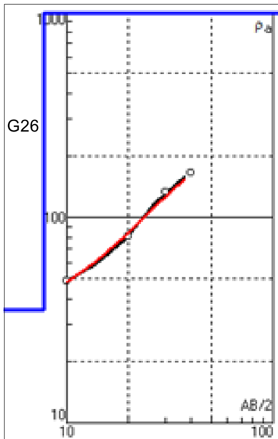
N	ρ	h	d	Alt
1	6.72	6.61	6.61	-6.61
2	4196			



G6

N	ρ	h	d	Alt
1	41.7	3.81	3.81	-3.812
2	1984			

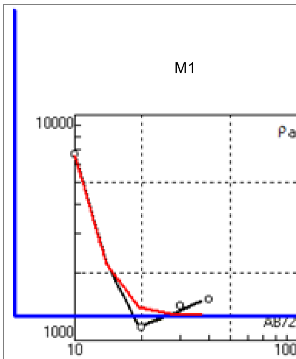
G26



G26

N	ρ	h	d	Alt
1	35.6	7.73	7.73	-7.728
2	6182			

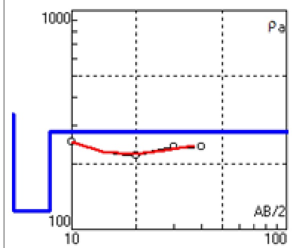
M1



M1

N	ρ	h	d	Alt
1	1.4E+5	1.98	1.98	-1.975
2	1280			

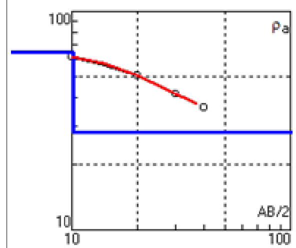
G13



G13

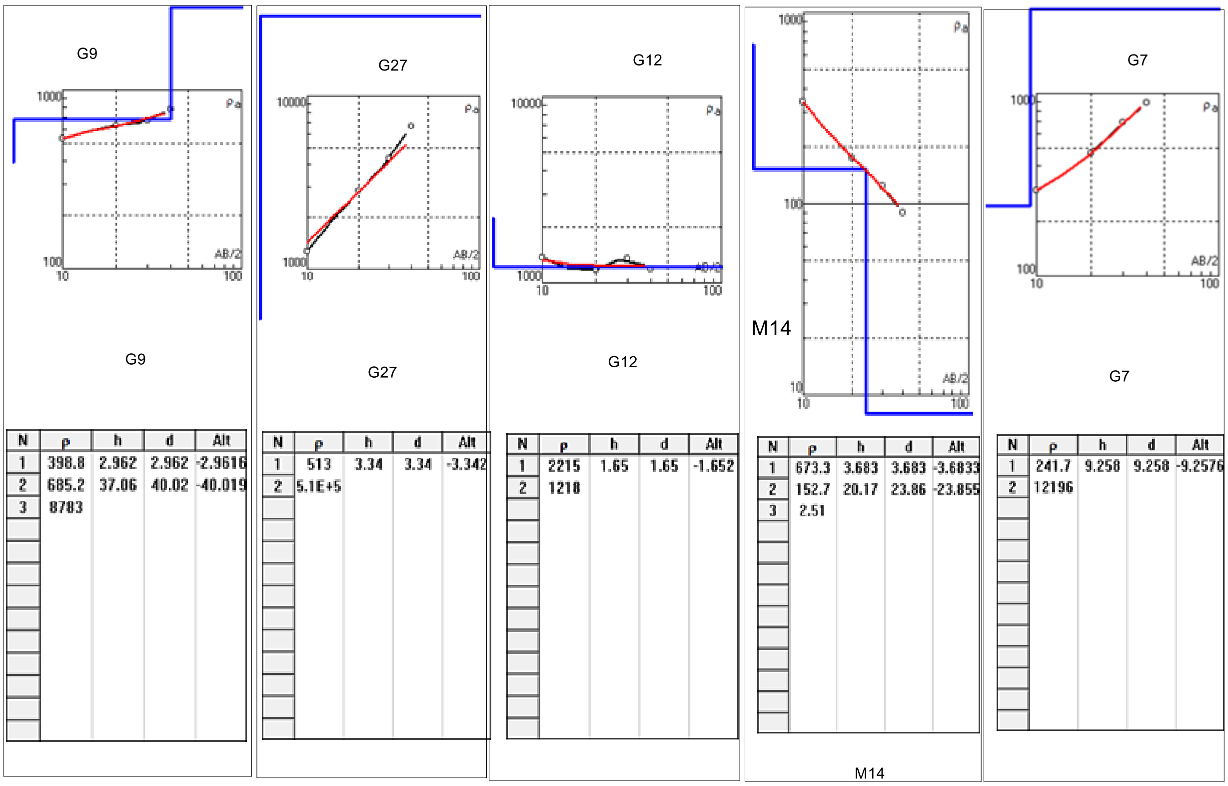
N	ρ	h	d	Alt
1	336.9	4.26	4.26	-4.2597
2	121.1	3.699	7.958	-7.9585
3	280.2			

M11



M11

N	ρ	h	d	Alt
1	65.23	10.18	10.18	-10.18
2	27.89			



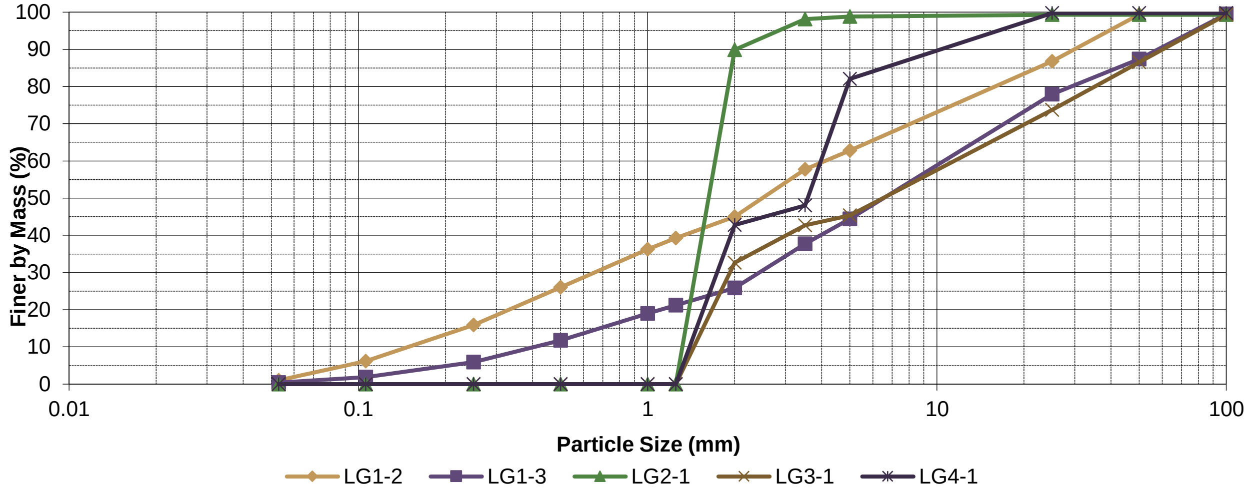
APPENDIX E: PARTICLE SIZE DISTRIBUTION

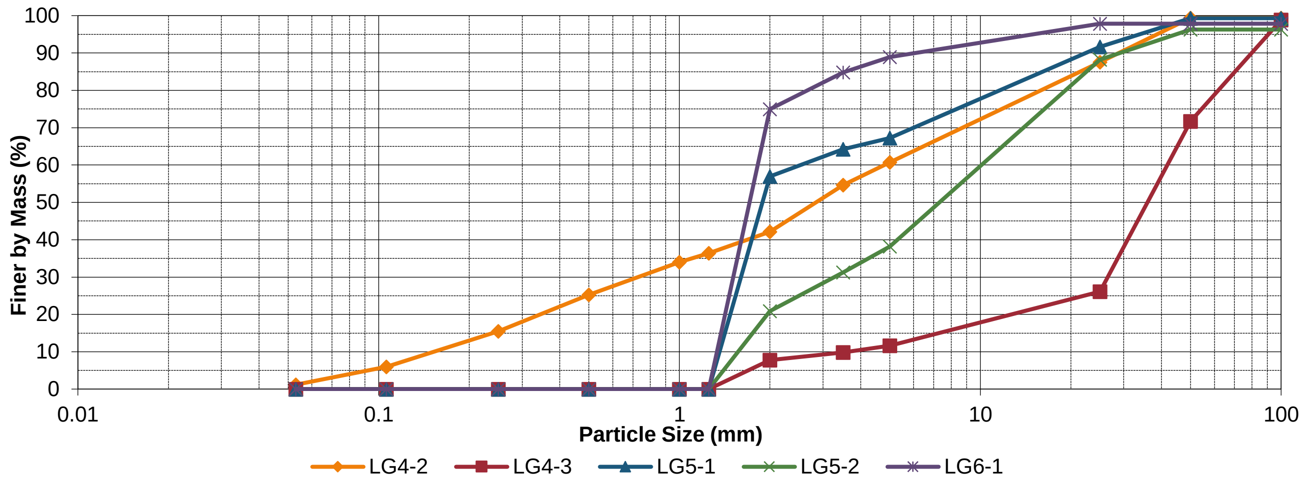
Sample no.	> 2mm (%)	Sand (%<2mm)	Silt (%)	Clay (%)
LG1-1	0.0	95.0	2.9	2.1
LG2-1	2.0	97.4	0.7	1.9
LG3-1	0.4	92.6	5.2	2.2
LG4-1	0.8	88.0	7.6	4.4
LG5-1	0.9	90.2	7.6	2.2
LG5-2	1.1	97.2	2.7	0.1
LG6-1	0.7	90.2	7.6	2.2
LG7-1	0.6	92.8	5.1	2.1
LG7-4	3.1	92.5	5.3	2.2
LG8-1	0.4	88.0	9.8	2.2
LG9-1	1.5	92.5	5.3	2.2
LG9-2	1.4	87.7	7.8	4.6

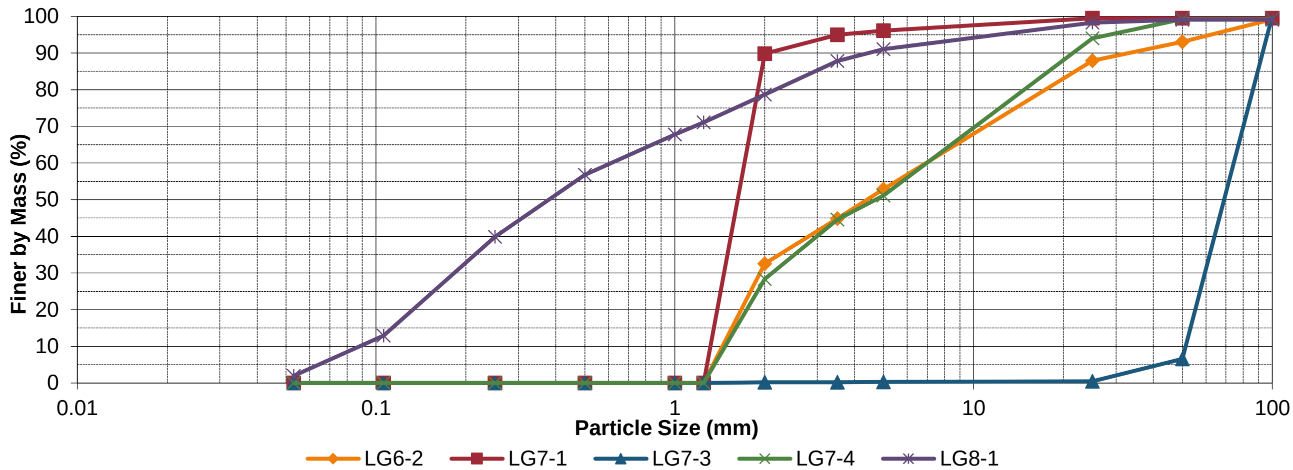
LG10-1	0.4	90.7	7.2	2.1
LG11-1	0.3	90.5	7.3	2.1
LG13-1	0.7	90.7	7.2	2.1
LG13-2	1.3	83.1	10.0	6.9
LG13-3	1.3	95.1	2.9	2.1
LG14-1	1.3	95.1	4.8	0.1
LG16-1	0.0	85.4	12.4	2.2
LG18-1	0.6	90.3	7.6	2.2
LG18-2	2.6	85.4	10.0	4.5
LG19-1	1.3	88.0	9.8	2.2
LG20-1	0.9	87.7	10.1	2.2
LG20-2	2.4	95.0	2.9	2.1
LG21-1	0.5	85.1	9.6	5.4
LG21-2	1.9	85.3	9.4	5.3
LG22-1	0.6	87.9	6.9	5.2
LG23-1	0.4	71.0	23.3	5.7
LG25-1	0.7	95.4	2.0	2.6
LG26-1	0.7	95.6	4.1	0.4
LG26-2	2.2	76.6	15.2	8.2
LG26-3	1.9	93.0	4.3	2.7
LG27-1	0.5	90.7	6.6	2.7
LG27-2	3.5	93.2	6.4	0.4
LG28-1	0.3	90.9	6.4	2.6
LG28-2	0.2	75.1	22.0	2.9
LM1-1	0.2	93.4	4.6	2.1
LM1-2	0.5	72.0	20.5	7.5
LM1-3	1.4	86.0	9.5	4.5
LM2-1	0.5	72.3	21.8	5.9
LM3-1	1.1	86.4	10.6	3.0
LM4-1	1.1	81.3	13.3	5.5
LM5-1	0.9	86.0	10.9	3.1
LM6-1	0.9	91.5	7.8	0.7
LM6-2	2.2	83.5	11.0	5.5
LM7-1	1.2	93.5	5.8	0.7
LM9-1	1.5	93.7	5.7	0.7
LM9-2	1.1	93.5	5.8	0.7
LM9-3	2.1	86.1	10.8	3.1
LM10-1	2.1	83.5	11.0	5.5
LM11-1	0.9	95.9	3.4	0.6

LM11-2	2.3	71.0	18.6	10.4
LM11-3	1.2	85.5	12.2	2.3
LM12-1	0.9	87.9	9.8	2.3
LM12-2	3.6	90.5	7.2	2.2
LM13-1	1.3	88.2	9.6	2.2
LM14-1	0.7	93.1	4.8	2.2
LM16-1	0.3	95.6	2.4	2.1
LM18-1	1.1	85.7	12.0	2.3
LM21-1	0.8	90.8	7.0	2.2
LM22-1	0.5	83.4	12.0	4.6
LM22-2	3.4	85.6	9.8	4.7
LM23-1	1.2	88.1	9.7	2.3
LM23-2	3.3	93.0	2.5	4.5
LM24-1	2.0	86.9	10.3	2.8
LM24-2	3.3	96.7	2.9	0.4
LM24-3	3.4	74.5	20.2	5.3
LG6-2	2.1	87.3	7.7	5.0
LG12-1	1.5	89.3	7.9	2.8
LG12-2	1.6	82.0	12.7	5.2
LG25-2	2.7	79.2	15.4	5.3
LG25-3	2.8	96.6	3.0	0.4
LG2-2	0.3	82.2	12.7	5.2
LG7-2	0.6	87.4	9.9	2.7
LG14-2	0.5	84.3	10.4	5.2

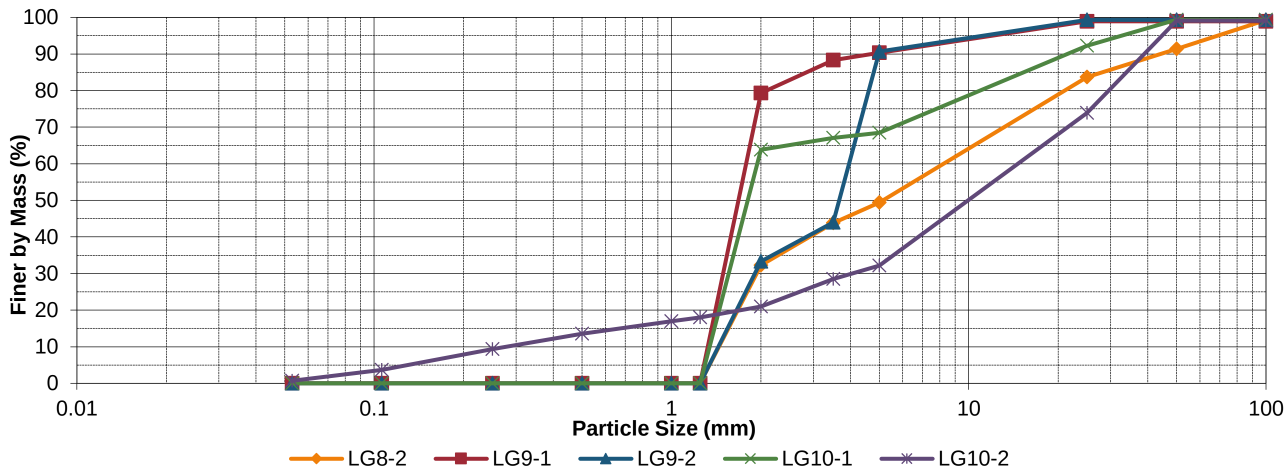
APPENDIX F: GRADATIONAL CURVES OF THE SOIL SAMPLES



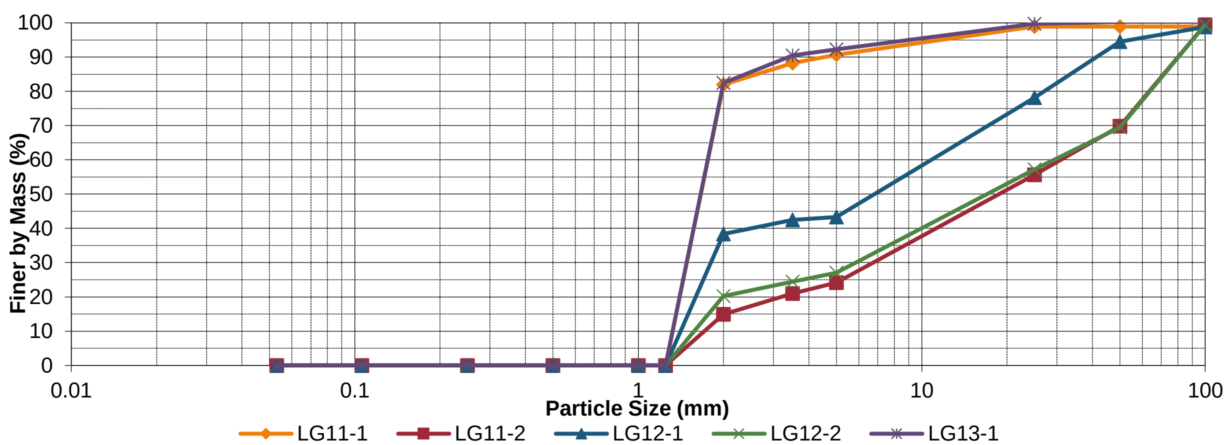




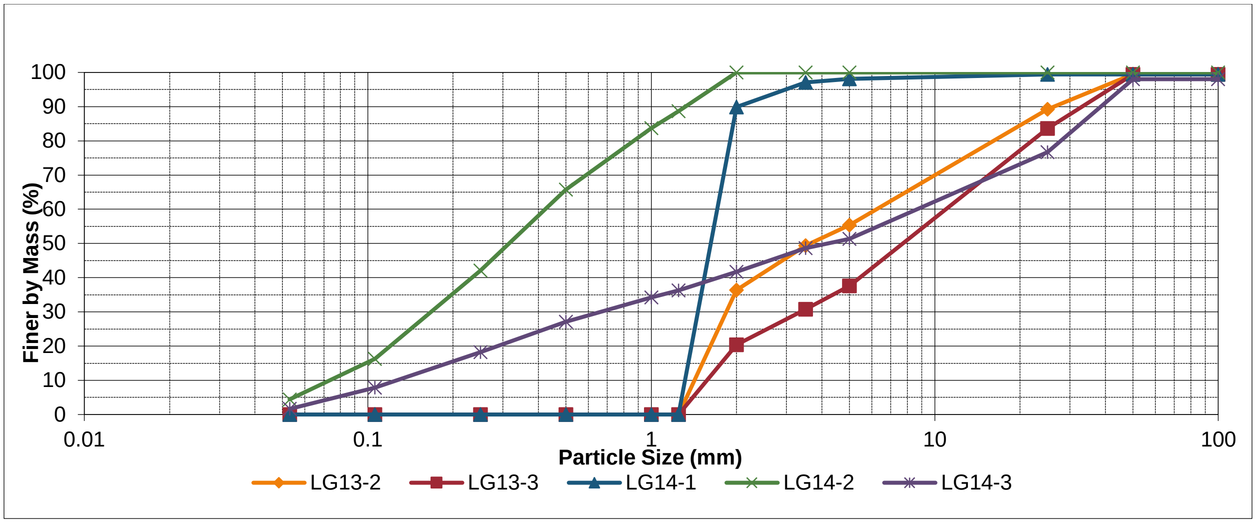
Finer by Mass (%)
Particle Size (mm)
0.01
0.1
1
10
100
0
10
20
30
40
50
60
70
80
90
100
LG6-2
LG7-1
LG7-3
LG7-4
LG8-1



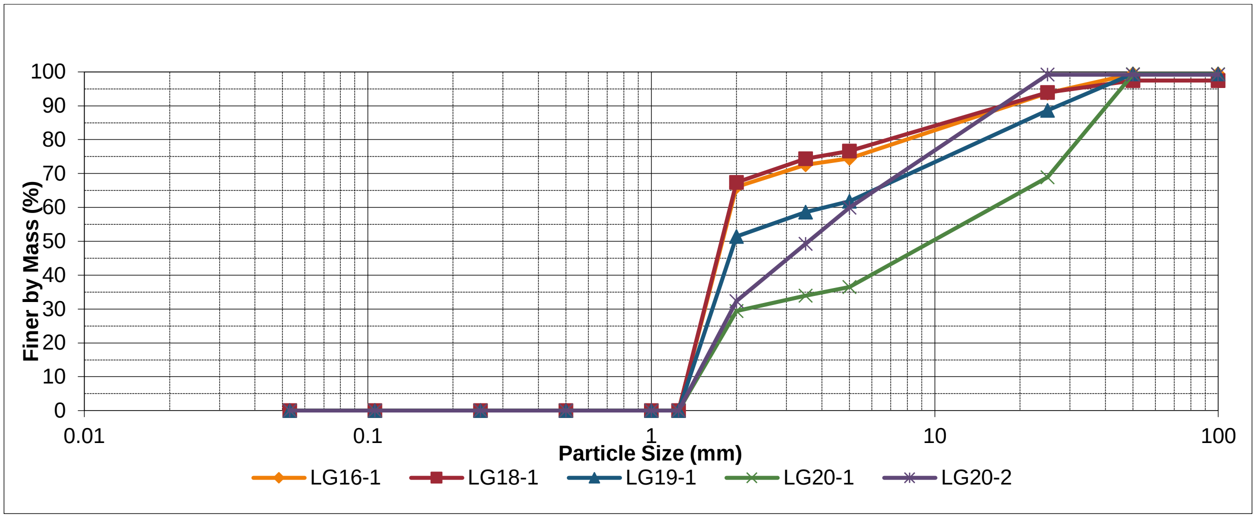
Finer by Mass (%)
Particle Size (mm)
0.01
0.1
1
10
100
0
10
20
30
40
50
60
70
80
90
100
LG8-2
LG9-1
LG9-2
LG10-1
LG10-2



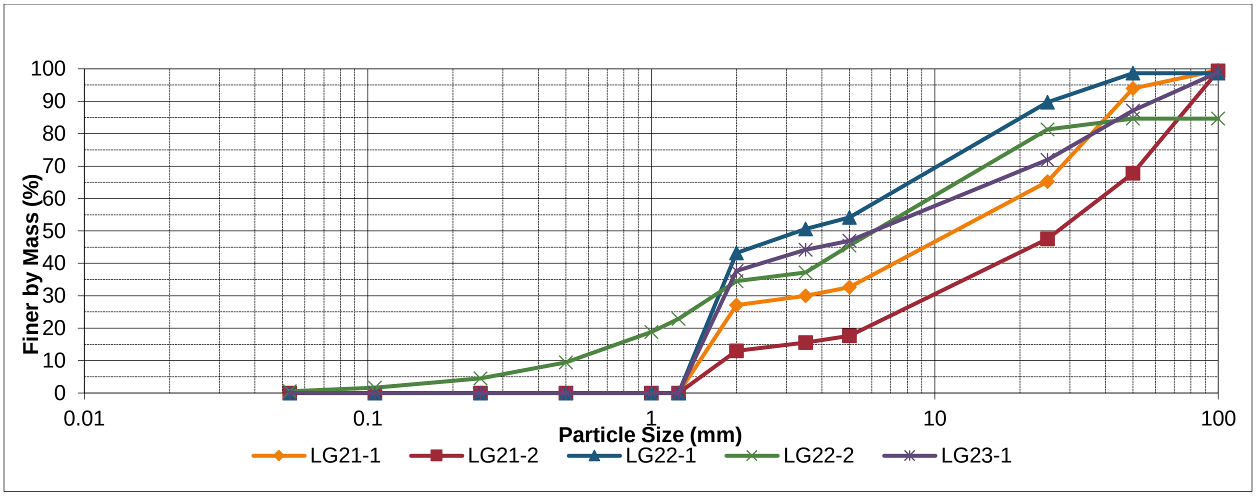
Finer by Mass (%)
Particle Size (mm)
0.01
0.1
1
10
100
0
10
20
30
40
50
60
70
80
90
100
LG11-1
LG11-2
LG12-1
LG12-2
LG13-1



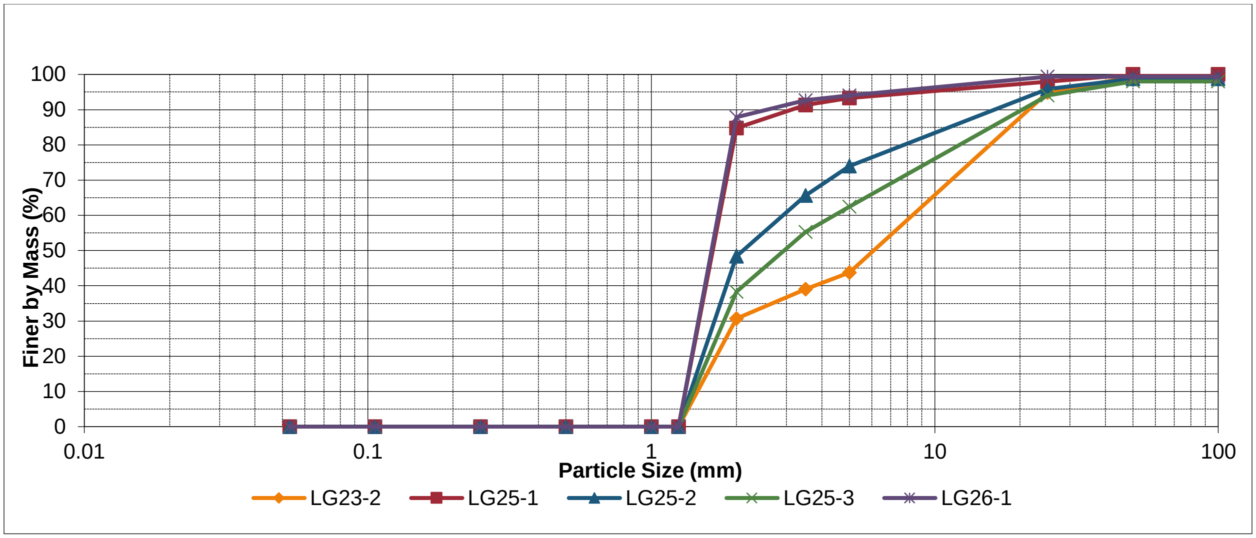
Finer by Mass (%)
Particle Size (mm)
LG13-2
LG13-3
LG14-1
LG14-2
LG14-3

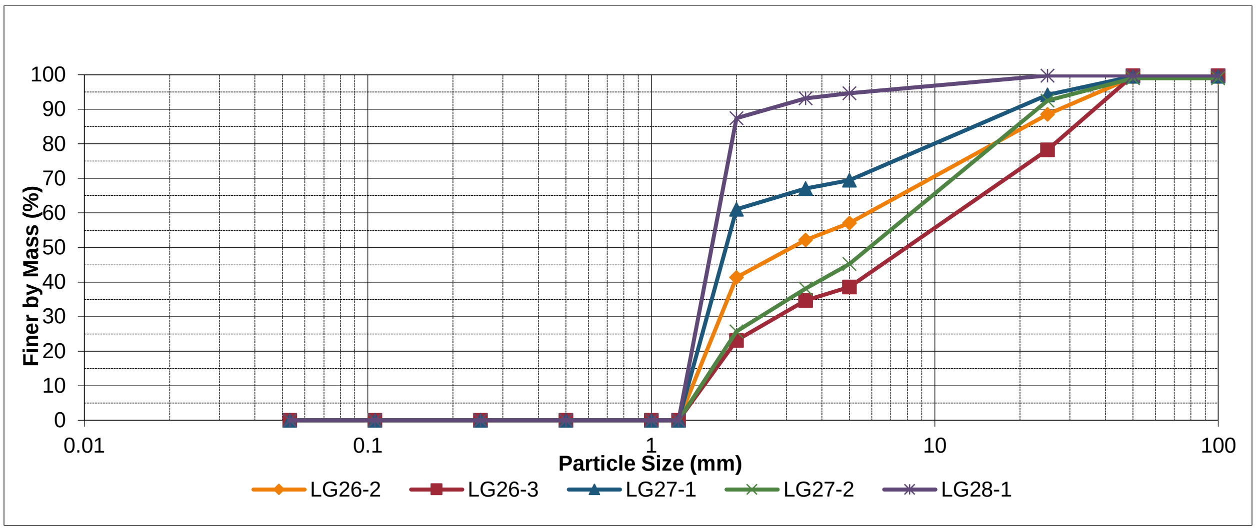


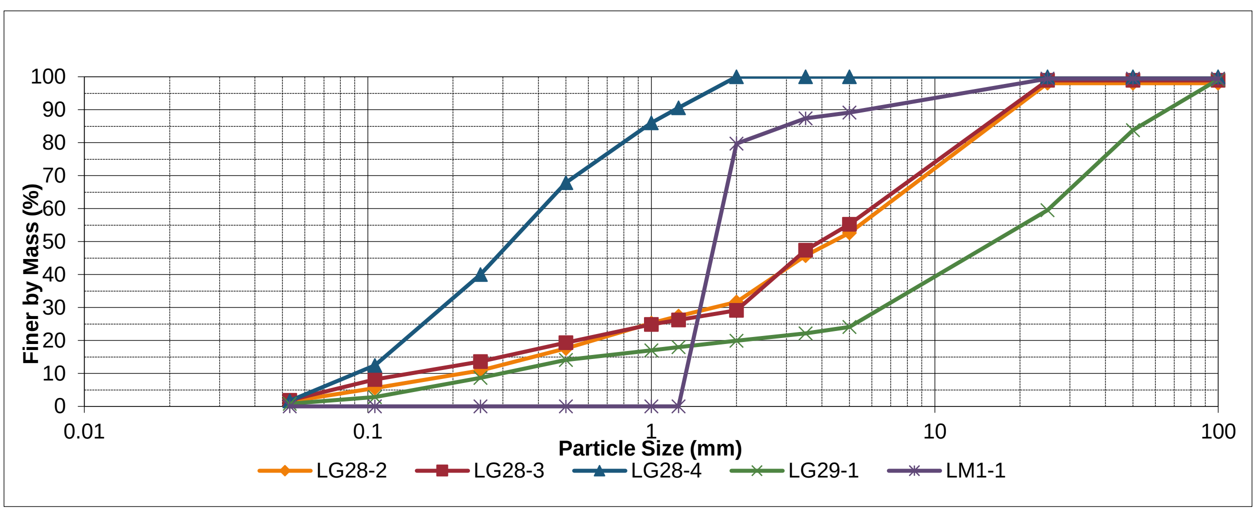
Finer by Mass (%)
Particle Size (mm)
LG16-1
LG18-1
LG19-1
LG20-1
LG20-2

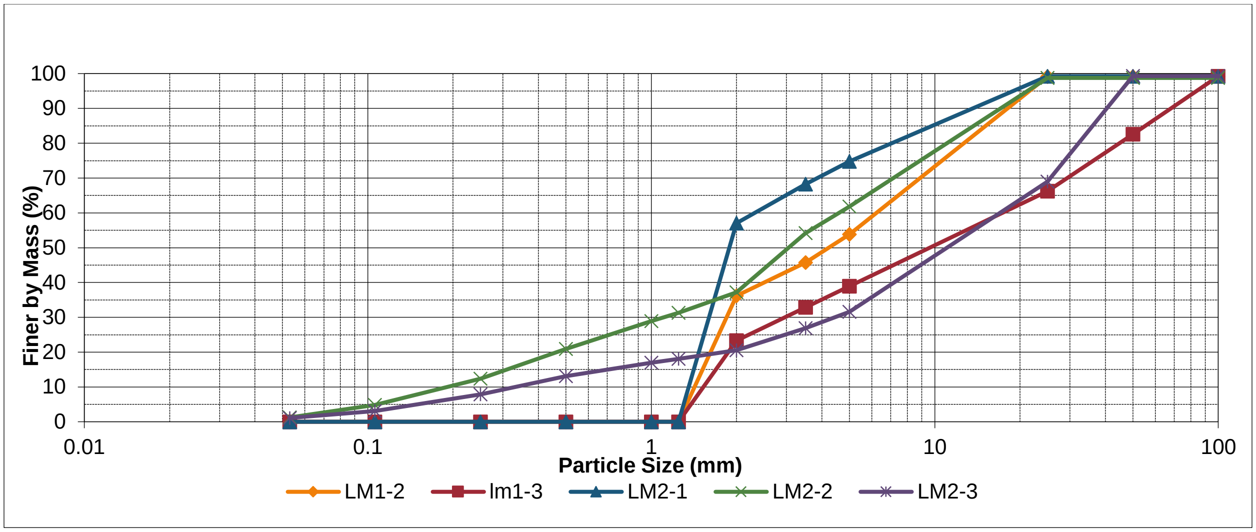


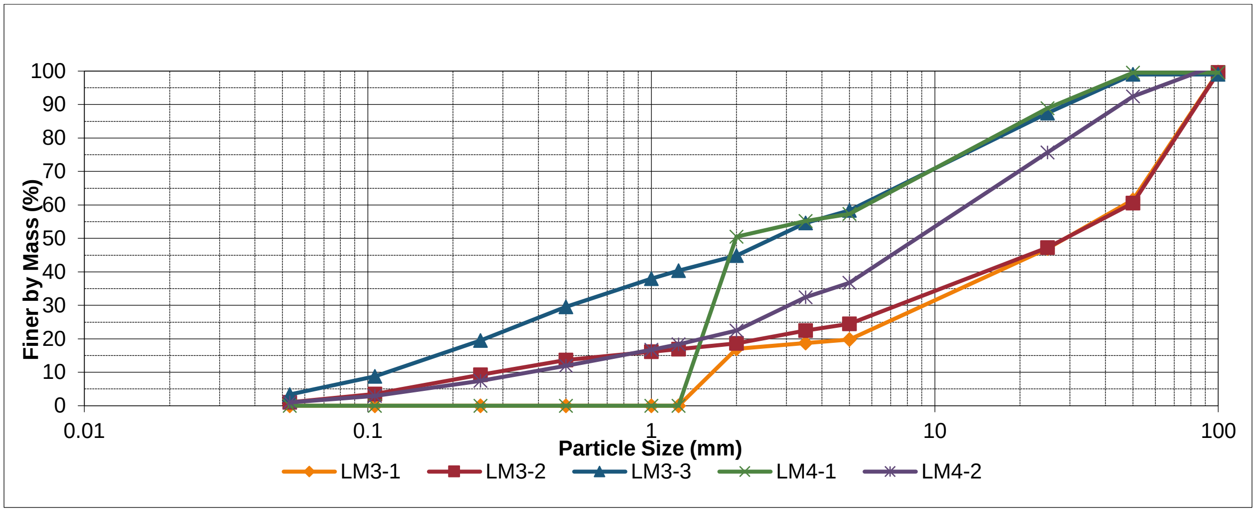
Finer by Mass (%)
Particle Size (mm)
LG21-1
LG21-2
LG22-1
LG22-2
LG23-1

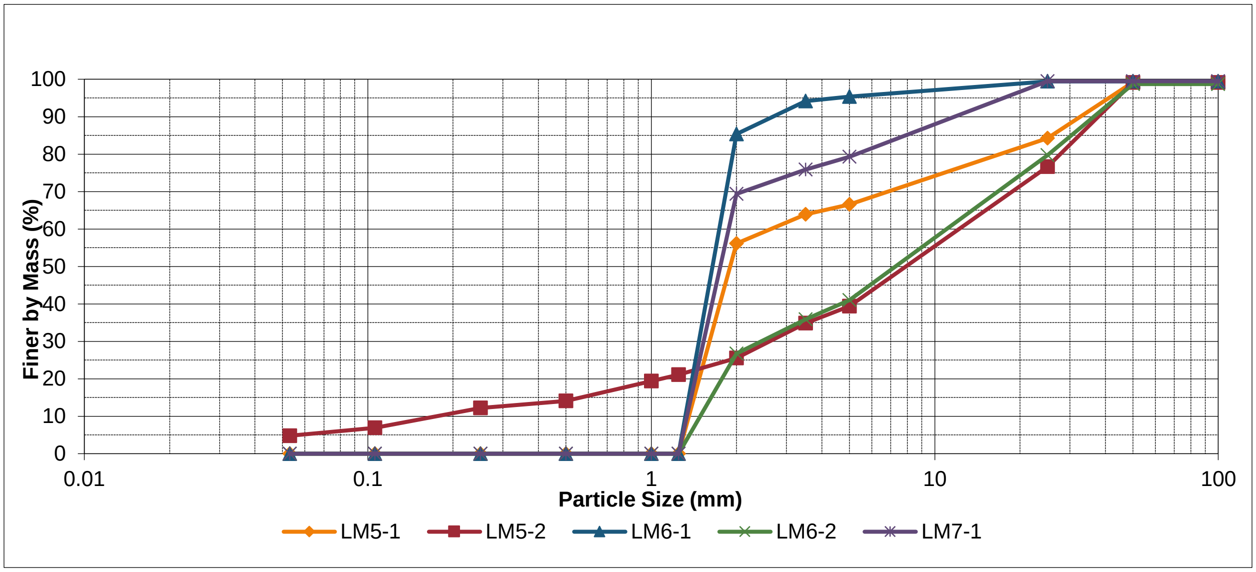

Finer by Mass (%)
Particle Size (mm)
LG23-2
LG25-1
LG25-2
LG25-3
LG26-1

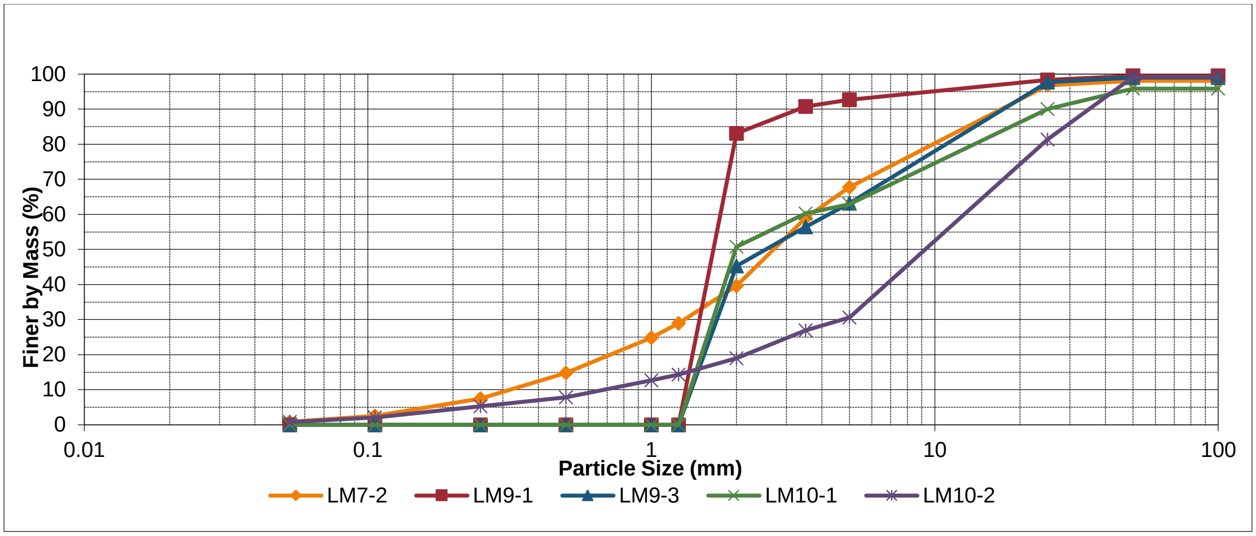

Finer by Mass (%)
Particle Size (mm)
LG26-2
LG26-3
LG27-1
LG27-2
LG28-1


Finer by Mass (%)
Particle Size (mm)
LG28-2
LG28-3
LG28-4
LG29-1
LM1-1

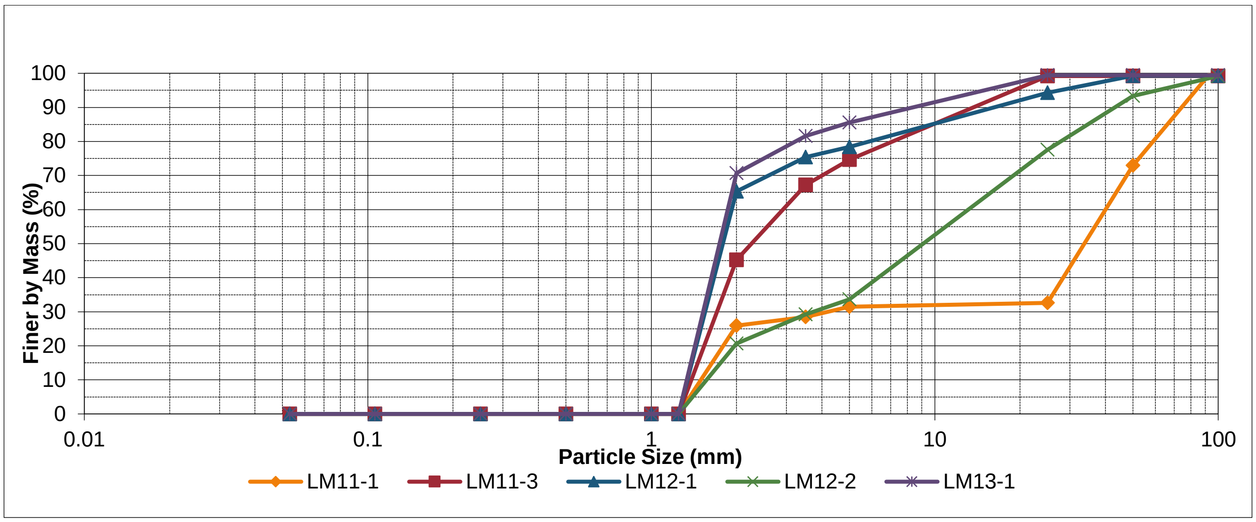

Finer by Mass (%)
Particle Size (mm)
0.01
0.1
1
10
100
LM1-2
lm1-3
LM2-1
LM2-2
LM2-3


Finer by Mass (%)
Particle Size (mm)
0.01
0.1
1
10
100
LM3-1
LM3-2
LM3-3
LM4-1
LM4-2

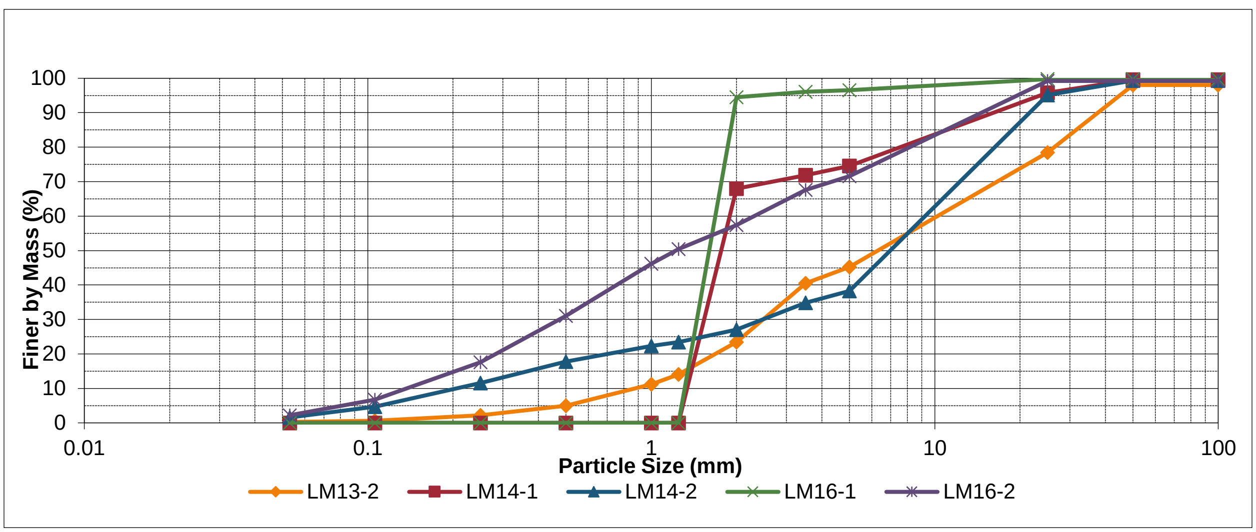

Finer by Mass (%)
Particle Size (mm)
0.01
0.1
1
10
100
LM5-1
LM5-2
LM6-1
LM6-2
LM7-1



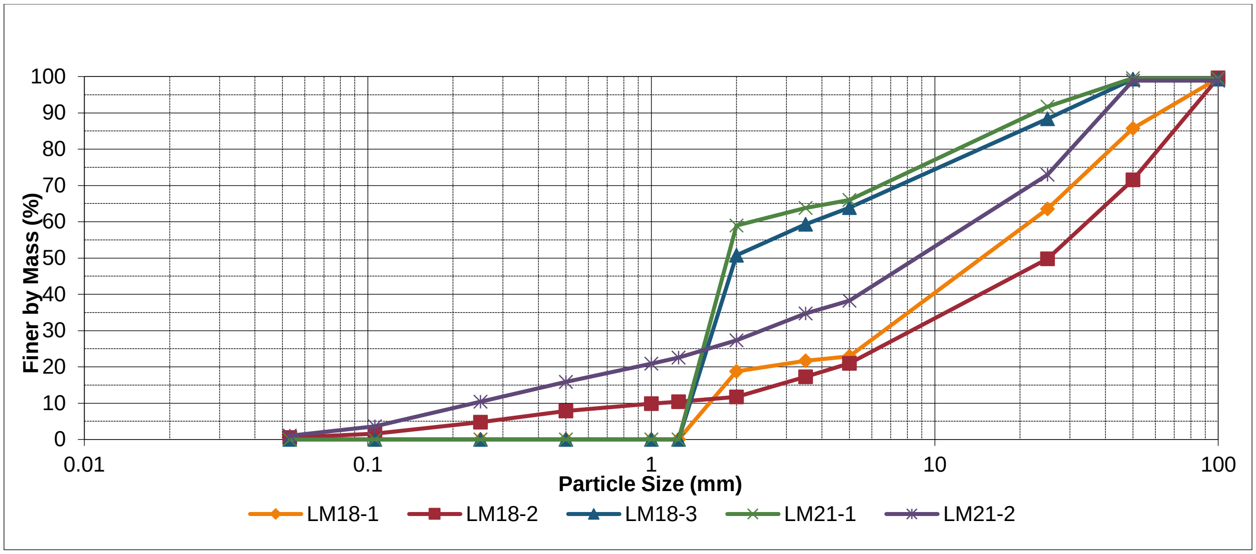
100
90
80
70
60
50
40
30
20
10
0
0.01
0.1
1
10
100
Finer by Mass (%)
Particle Size (mm)
LM7-2
LM9-1
LM9-3
LM10-1
LM10-2

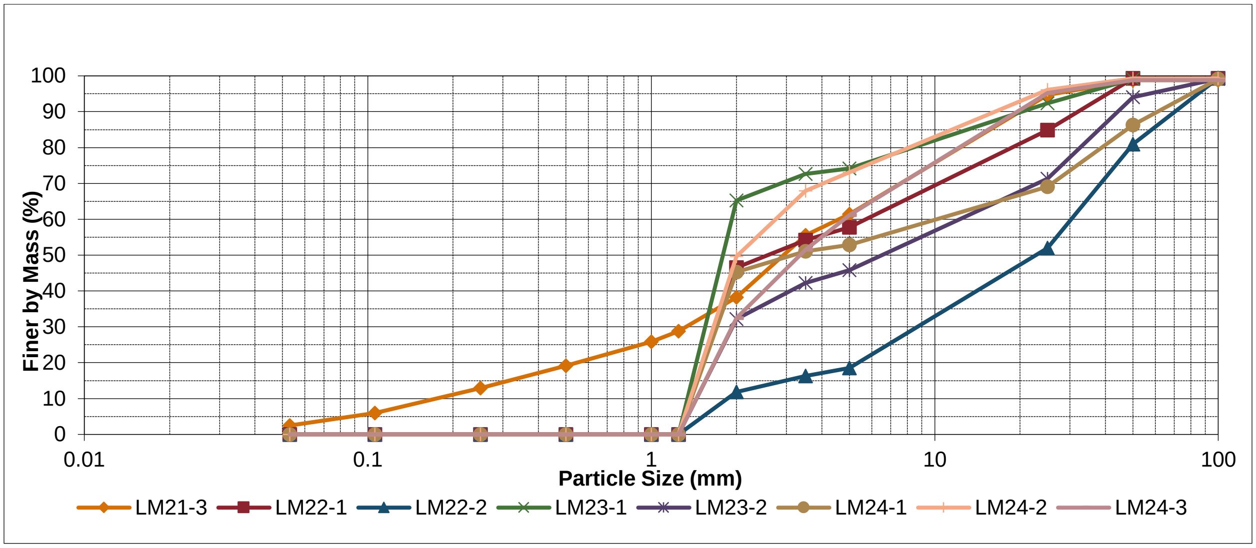


100
90
80
70
60
50
40
30
20
10
0
0.01
0.1
1
10
100
Finer by Mass (%)
Particle Size (mm)
LM11-1
LM11-3
LM12-1
LM12-2
LM13-1



100
90
80
70
60
50
40
30
20
10
0
0.01
0.1
1
10
100
Finer by Mass (%)
Particle Size (mm)
LM13-2
LM14-1
LM14-2
LM16-1
LM16-2


100
90
80
70
60
50
40
30
20
10
0
Finer by Mass (%)
0.01
0.1
1
10
100
Particle Size (mm)
LM18-1
LM18-2
LM18-3
LM21-1
LM21-2


100
90
80
70
60
50
40
30
20
10
0
Finer by Mass (%)
0.01
0.1
1
10
100
Particle Size (mm)
LM21-3
LM22-1
LM22-2
LM23-1
LM23-2
LM24-1
LM24-2
LM24-3

APPENDIX G: AVERAGE LINEAR SHRINKAGE OF THE SOIL

Sample no	Average linear shrinkage	Sample no	Average linear shrinkage
G1(1)	1.9	M10(1)	1.1
G2(1)	0	M12(1)	0.9
G3(1)	2.9	M13(1)	1.2
G4(1)	2.4	M14(1)	0.4
G5(1)	3.0	M16(1)	0
G6(1)	1.5	M18(1)	1.4
G7(1)	1.0	M21(1)	0.4
G8(1)	1.1	M22(1)	1.5
G9(1)	1.1	M23(1)	0.5
G10(1)	0.9	M24(1)	0.8
G11(1)	1.1	G1(2)	9.7
G12(1)	3.4	G2(2)	12
G13(1)	0.6	G4(2)	4.8
G14(1)	0.3	G5(2)	3.8
G16(1)	1.7	G8(2)	1.6
G18(1)	0.8	G9(2)	2.8
G19(1)	1.4	G11(2)	1.8
G20(1)	1.4	G13(2)	2.2
G21(1)	1.3	G14(2)	4.4
G22(1)	1	G18(2)	3.5
G23(1)	0.8	G20(2)	1.7
G25(1)	0.4	G22(2)	0.6
G26(1)	0	G23(2)	7
G27(1)	0.6	G25(2)	1.9
G28(1)	0	G26(2)	3.8
G29(1)	2.3	G27(2)	3.2
M1(1)	0.4	G28(2)	3
M2(1)	4.3	M3(2)	2.2
M3(1)	1.5	M18(2)	1.9
M4(1)	1.8	G6(3)	4.5
M5(1)	1	G7(3)	2.5
M6(1)	1.5	G13(3)	0
M7(1)	0.6	G26(3)	2.9
M9(1)	0.8	G27(3)	1
M10(1)	1.1	G28(3)	9.8
M2(3)	3.9	M3(3)	0.9
		M11(3)	0

APPENDIX H: PH AND ELECTRICAL CONDUCTIVITY

ID	pH (H_2O)	pH (KCL)	EC (mS/m)	ID	pH (H_2O)	pH (KCL)	EC (mS/m)
LG1-1	9.22	7.94	44	**LG5-2**	8.52	8.30	85
LG2-1	7.80	8.16	16	**LG9-2**	8.59	8.50	45
LG3-1	7.77	7.63	492	**LG13-3**	8.60	8.16	76
LG4-1	8.12	7.93	41	**LG18-2**	8.41	8.16	132
LG5-1	8.11	7.87	26	**LG20-2**	9.59	8.43	239
LG6-1	8.34	8.04	40	**LG21-2**	8.36	7.76	52
LG7-1	8.64	8.37	35	**LG26-1**	8.84	8.52	36
LG8-1	8.06	7.69	35	**LG27-2**	8.36	7.93	42
LG91	8.27	7.94	32	**LG28-2**	8.63	8.28	4620
LG10-1	8.39	7.16	30	**LM1-2**	7.89	7.90	25
LG11-1	8.43	7.04	27	**LM22-2**	8.19	8.09	81
LG13-1	8.70	7.68	30	**LM23-2**	8.43	8.34	57
LG14-1	8.72	8.48	33	**LM24-2**	8.24	8.20	246
LG16-1	7.99	7.50	43	**LM6-2**	8.36	8.07	40
LG18-1	8.17	8	40	**LM9-2**	8.63	7.85	21
LG19-1	8.28	7.92	38	**LM11-2**	8.40	8.27	163
LG20-1	8.24	8.03	147	**LG6-2**	8.36	8.36	636
LG21-1	8.27	7.29	54	**LG12-2**	8.41	7.92	32
LG22-1	8.47	7.95	34	**LG25-2**	8.49	8.32	34
LG23-1	8.07	8.10	2041	**LM12-2**	8.20	8.35	225
LG25-1	8.78	8.03	27	**LG2-2**	8.70	8.10	2840
LG26-1	8.75	8.32	26	**LG7-2**	8.96	8.11	47
LG27-1	8.41	8.02	41	**LG14-2**	8.62	8.25	55
LG28-1	8.77	8.52	1866	**LG13-3**	8.92	8.36	277
LM1-1	7.77	7.36	26	**LG26-3**	8.10	8.38	284
LM2-1	7.58	7.47	3640	**LM1-3**	8.36	8.10	71
LM3-1	8.87	8.02	170	**LM9-3**	8.51	8.20	20
LM4-1	8.34	7.54	41	**LM11-3**	8.50	7.80	76
LM5-1	8.38	7.30	49	**LM24-3**	8.51	8.39	73
LM6-1	8.07	7.61	29	**LG25-3**	8.30	7.96	388
LM7-1	8.35	7.70	19				
LM9-1	8.60	7.79	20				
LM10-1	8.24	7.71	34				
LM11-1	8.99	8.01	129				
LM12-1	8.07	7.59	65				
LM13-1	8.25	7.49	46				
LM14-1	9.46	8.30	81				
LM16-1	8.60	8.48	23				
LM18-1	8.33	8.04	46				
LM21-1	8.34	8.10	150				
LM22-1	8.01	7.38	34				
LM23-1	8.38	7.83	33				
LM24-1	8.31	7.88	31				
LG12-1	8.30	7.68	54				

APPENDIX I: PERMEABILITY RESLUTS OF THE SOIL

Trial No (Sand with calcrete)	Constant head, **h** (cm)	Elapsed time, **t** (Seconds)	Outflow volume, **Q** (cm^3)	Water temp, **T** (^oC)
1	30	30	4.46	32
2	40	60	8.9	32
3	50	90	12.9	32
4	60	120	16.85	32
Average K_T= $2x10^{-3}$				

Trial No (**Sand with calcrete and gneiss)**	Constant head, **h** (cm)	Elapsed time, **t** (Seconds)	Outflow volume, **Q** (cm^3)	Water temp, **T** (^oC)
1	30	30	8.4	32
2	40	60	13.52	32
3	50	90	19.82	32
4	60	120	20.67	32
Average K_T= $2x10^{-3}$				

Trial No (**Sand with granite and gneiss)**	Constant head, **h** (cm)	Elapsed time, **t** (Seconds)	Outflow volume, **Q** (cm^3)	Water temp, **T** (^oC)
1	30	30	19.05	32
2	40	60	32.22	32
3	50	90	47.57	32
4	60	120	58.3	32
Average K_T= $5x10^{-3}$				

Trial No (**Sand)**	Constant head, **h** (cm)	Elapsed time, **t** (Seconds)	Outflow volume, **Q** (cm^3)	Water temp, **T** (ºC)
1	30	30	43.61	32
2	40	60	49.49	32
3	50	90	50.59	32
4	60	120	61.78	32
Average K_T= 8x10^{-3}				

Trial No (**Sand with gneiss)**	Constant head, **h** (cm)	Elapsed time, **t** (Seconds)	Outflow volume, **Q** (cm^3)	Water temp, **T** (ºC)
1	30	30	29.46	32
2	40	60	51.7	32
3	50	90	72.98	32
4	60	120	94.51	32
Average K_T= 8x10^{-3}				