sport, arts, culture & recreation
Department of
Sport, Arts, Culture and Recreation
FREE STATE PROVINCE

SMITHFIELD MULTI-PURPOSE INDOOR SPORT CENTRE

GEOTECHICAL INVESTIGATION REPORT

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Reg No. 2019 / 014304 / 07

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TECHNICAL REPORT

Smithfield Multi-Purpose Indoor Sport Centre

ERF 1117 Mofulatshepe- Farm Smithfield 277- Free State Province

Engineering Geological Site Investigation

Prepared for
BVi Consulting Engineers- Bloemfontein

Date
09/03/2022

Compiled by
Kevin Coertzen (Pri.Sci.Nat)

Technical Report

GC22/104

09 March 2022

Smithfield Multi-Purpose Indoor Sport Centre

ERF 1117 Mofulatshepe- Farm Smithfield 277- Free State Province

Project Details

Project Title	Smithfield Multi-Purpose Indoor Sport Centre
Client	Free State Department of Sports, Arts, Culture and Recreation
Consulting Engineers	BVi Consulting Engineers- Bloemfontein
Report Description	Engineering Geological Site Investigation
Report Number	GC22/104
Report Version	Final Report

Report Compilation and Signatory

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Report Distribution List

Date	Name	Institution	Version	Format
09/03/2022	Johnnie Haefele	BVi Consulting Engineers- Bloemfontein	Final Report	PDF.

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TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

LIST OF APPENDICES

1. Project Introduction

1.1. Background

This report describes the results of the detailed Engineering Geological Site Investigation conducted in support of the proposed **Smithfield Multi-Purpose Indoor Sport Centre.** This multi land use development is planned to span the north eastern extent of **ERF 1117** in the Mofulatshepe suburb of Smithfield.

The area in question falls in the boundaries of the Farm Smithfield 277, under the jurisdiction of the Mohokare Local Municipality- Xhariep District Municipality of the Free State Province in South Africa

The detailed investigation was undertaken in order to assess the engineering geological character of the site; focussing on the geotechnical properties which will affect the overall development potential of the parcel of land in question.

1.2. Terms of Appointment

GeoCalibre Geotechnical Consultancy was appointed by the *BVi Consulting Engineers* to undertake the required Engineering Geological Site Investigation.

The information presented in this document is based on the information supplied by the Client prior to the commencement of the investigation; therefore, GeoCalibre Geotechnical Consultancy (Pty) Ltd- shall not be held liable for, and is indemnified against all actions, claims, demands, losses, liabilities, costs, damages and expenses prompted by, or in connection with, inaccurately relayed information pertaining to the site and/or the development.

1.3. GeoCalibre- Company Background and Information

GeoCalibre is a specialist geotechnical consulting firm made up of a team of qualified professional geo-practitioners. The firm was established out of a love for the industry and an urge to define a new calibre of professional consulting.

GeoCalibre uses advanced scientific methods to create accurate and reproducible geotechnical models; successfully guiding the implementation of site-specific design precautionary measures/engineering solutions. The methodology followed throughout the investigative process accounts for the nature and location of the development as well as adhering to the standards of our practice (SANS and SAICE).

Investigations undertaken by GeoCalibre are overseen by suitably qualified Engineering Geologists *professionally registered* (Pr.Sci.Nat) with the South African Council of Natural Scientific Professions (SACNASP)- in accordance with all the relevant and required procedures and legislations.

GeoCalibres employees are also members of the South African Institute for Engineering and Environmental Geologists (SAIEG).

1.4. Codes of Practise and Investigative Standard

The investigation was carried according to the following standard practice codes and guidelines:

- Guidelines for Urban Engineering Geological Investigations (SAIEG & SAICE, 1997) for urban development.
- Site Investigation Code of Practice put forward by the Geotechnical Division of SAICE (2010).
- SANS 10400.
- The guideline and specification documents by the South African Institute of Engineering and Environmental Geologists (SAIEG) and South African Institution for Civil Engineers (1997), AEG/SAIEG/SAICE (2002) were consulted.

1.5. Limitations of the Geotechnical Assessment

The presented geotechnical model is based on point data; with our opinions based on what was visible at the time of the investigation. The investigation has therefore attempted, through interpolation and extrapolation of known testing locations, to identify problem issues of a geotechnical nature on which this report is based. Variances in soil and rock quality and quantity from those predicted may be encountered during construction and these should be recorded.

Foundation trenches and bulk excavations should be overseen by a competent person in order to identify and assess any variance in the geotechnical character along the route/across the footprint of the planned development.

1.6. Development within 1 : 100 year-flood lines

It must be noted that the National Water Act (Act 36 of 1998) states the following regarding development within the 1 : 100 year-flood line of any stream or river (Thompson, 2006):

- **Section 21(c):** Impeding or diverting the flow of water in watercourses (including alteration of the hydraulic characteristics of flood events) requires licensing according to the Act.
- **Section 21(i):** Any action that may alter the bed, banks, courses or characteristics of watercourses (including flood events) requires licensing according to the Act, including: widening or straightening of the bed or banks of a river to allow for the construction the housing development and altering the course of a river partially or completely (i.e.: river diversion) to be able to use or develop the area where the watercourse originally was.

The National Water Act does not prohibit development within 1 : 100 year-flood lines; however, the Act requires detailed analysis of the effects of the proposed development on the surrounding environment, with special reference to surface and sub-surface water flow.

The Act requires that suitable precautionary measures be implemented to limit the effect within and downstream from the proposed development.

1.7. Information Sources

- **Geological Map:**
  - Geological Series Map 3026 Aliwal North; scale 1:250 000 (digital format)
- **Hydrogeological Data:**
  - SADC Groundwater Information Portal (SADC GIP)
  - Hydrogeological Series Map 2924 Bloemfontein (2002); scale 1 : 500 000 (digital copy)
- **Topocadastral maps:**
  - 3026 Ba; scale 1:50 000 (digital format)
- **Remote Sensing Information:**
  - Google Earth Pro ™
  - Planet GIS
  - Free State Cadastral Land Layer (2019)
- **Provided by the Client:**
  - Concept site development plan (Smith SDP-000; Rev A)
  - Date- 2021-12-07
- **Available geotechnical reports:**
  - Numerous geotechnical reports from the region compiled by Kevin Coertzen (Pri.Sci.Nat and MSAIEG)

1.8. Scope of the Investigation

A geotechnical site investigation was be carried out across the site in question to assess the mechanical nature of the underlying strata as well as model the geomorphological nature of the area as a whole.

Following the detailed assessment, GeoCalibre provides recommendations on the implementation of **site-specific engineering solutions.**

The aim of the overall site investigation can be summarised as follows:

- Establishment of a **regional geological**, **geomorphological** and **geo-technical** model for the site.
- To delineate the **succession of strata** (soil and rock) underlying the site; with the identification of problematic **physical, chemical** and **mechanical** characteristics which may influence the development.
- To quantify the **in-situ mechanical properties** of the soft materials underlying the site; specifically, with regards to the proposed future developments.
- To compute the **excavatability properties** of the on-site materials.
- To assess shallow **groundwater** patterns.
- To evaluate the **re-usage potential** of the materials underlying the site.
- To aid the development moving forward through the formulation of an **accurate geotechnical model** for the site under investigation.

It must be noted that this investigation was conducted to assist with the design and construction phases of the development.

The investigation excludes the following aspects, where applicable:

- Detailed hydrogeology, flood lines and wetland delineation
- Detailed slope stability assessment
- Undermining investigations
- Geo-environmental assessments.

1.9. Investigative Methodology

The investigation is undertaken in several phases in order to achieve the aims discussed above. The investigative phases are as follows:

- **Phase 1: Introduction and Regional Assessment of the Site**
- **Phase 2: Geotechnical Analysis - Engineering Geological Investigation**
- **Phase 3: Data Assessment and Report Compilation**

1.9.1. Phase 1: Introduction and Regional Assessment of the Site

The collation and evaluation of all the available topographic, geo-morphological and geological data across the investigated site and its' surroundings. This assessment is done using available regional maps and remote sensing images.

This section of the report will include a description and summary of the site's nature, based on existing literature, and is supplemented with the compilation of a series of base maps.

1.9.2. Phase 2: Geotechnical Analysis- Engineering Geological Investigation

Trenching and Sampling

The field work phase of the investigation was conducted by GeoCalibre in January 2021. Test pits were distributed across the area in question; at locations identified in the desktop study and site walkover phases of the investigation.

Several challenges arose regarding trial pit placements (relict infrastructure/ fill, existing services, etc.); however, the project team managed the risk- with the placement of trial pits in such a way so as to create an accurate and reproducible site-specific geotechnical model.

A total of SIX (6) test pits (TP1 to TP6) were excavated using a TLB-type light mechanical excavator (JCB 3CX). The succession of soil layers exposed within the test pits and exposures were logged and a series of detailed photographs were taken of the different soil layers.

Undisturbed, disturbed and bulk samples were taken of the material deemed to be important to the proposed development. Laboratory testing commenced on the samples collected during the field work phase of the investigation.

In-situ Testing- DCP Testing

In-situ penetration tests will be conducted alongside the excavated test pits in order to determine the quantitative soil consistency conditions.

SIX (6) Dynamic Cone Penetrometer (DCP) tests were completed across the site.

Standardised DCP tests (8 kg hammer falling 575 mm with a tip/cone of 60°) were implemented as to give an indication of the consistency of the in-situ material of the layer house in question.

In the design of light structures, the DCP is a practical test used for onsite measuring of material strength. The test results can be correlated with the California Bearing Ratio (CBR) by applying a formula; results of which can be correlated to the laboratory tests conducted on the soils from the same location.

Laboratory Testing

Standard foundation indicator and soil compaction tests were conducted by Letaba Lab Bloemfontein (**SANAS Accredited**) on disturbed and bulk soil samples. These tests were undertaken in order to determine the composition of the underlying soils (i.e.: the relative percentages of gravel, sand, silt and clay) and to evaluate the suitability of the materials for the re-use in the proposed construction.

The following tests were conducted:

I. Atterberg limits (Liquid Limit, Plasticity Index and Linear Shrinkage) and Particle-size Distribution
II. Maximum Dry Density versus Optimum Moisture Content
III. Californian Bearing Ratio versus Compaction Effort (MOD AASHTO method)
IV. pH and EC analysis (chemical analysis)
V. Double hydrometer as an indication of material dispersivity.

Specialist **undisturbed sample testing** was conducted by Steyn-Wilson Geotechnical, to quantify the in-situ mechanical properties of the soft materials underlying the site (settlement/heave characteristics).

The following specialist tests were conducted:

I. Shear box
II. Single and Double Oedometer
III. Free Swell
IV. Bulk Density
V. Moisture Content

1.9.3. Phase 3: Data Assessment and Report Compilation

The investigation concluded with the compilation of a technical report detailing the methodology utilised during the study and the summarised results obtained.

This includes a potential geotechnical evaluation of the site based on the cumulative results of the investigation.

2. Description of the Environment

2.1. Site Location and Description

The **study area** for this investigation is seen to fall within the far southern portion of the Free State Province of South Africa. On a more localised scale, the study area is situated along the southern extent of the town of Smithfield (Smithfield 277), within the low-density **residential suburb** of **Mofulatshepe** (Figure 1).

The **site** encompasses the **north eastern extent** of **ERF 1117-** Mofulatshepe suburb. This predominantly undeveloped parcel of land exhibits a roughly rectangular shape, with a combined extent of approximately **2.3 Ha**. The boundaries of the site are defined by numerous existing gravel roads; namely Oettle Street (north west), Butler Street (north east) and an unnamed street to the south east. Access to the site can be obtained by either of these streets. Furthermore, the south western boundary of the site is defined by an existing fence orientated ~NW/SE, bisecting ERF 1117.

The planned development across the surface will encompass the subdivision of this land portion into various land-use zones i.e., infrastructural units, sports fields, roadways, and services etc. Each of these zones may require their own set of unique geotechnical recommendations determined by the properties of the in-situ material. The site for this investigation is located at the following coordinates:

Latitude: 30.222107° S **Longitude:** 26.534418° E

The sites surface was seen to display a **reworked nature** attributed to past and ongoing **human activities** in the area. This reworking was predominantly in the form of **relict infrastructure**, roadways/pathways, soil berms, **scattered heaps of dumped fill** and localised levelled sports fields. The combination of the items raised above have resulted in the formation of an anthropogenically induced undulating landscape, with the occurrence of small-scale anomalies.

The photo series below depicts the reworked surficial nature of the site.



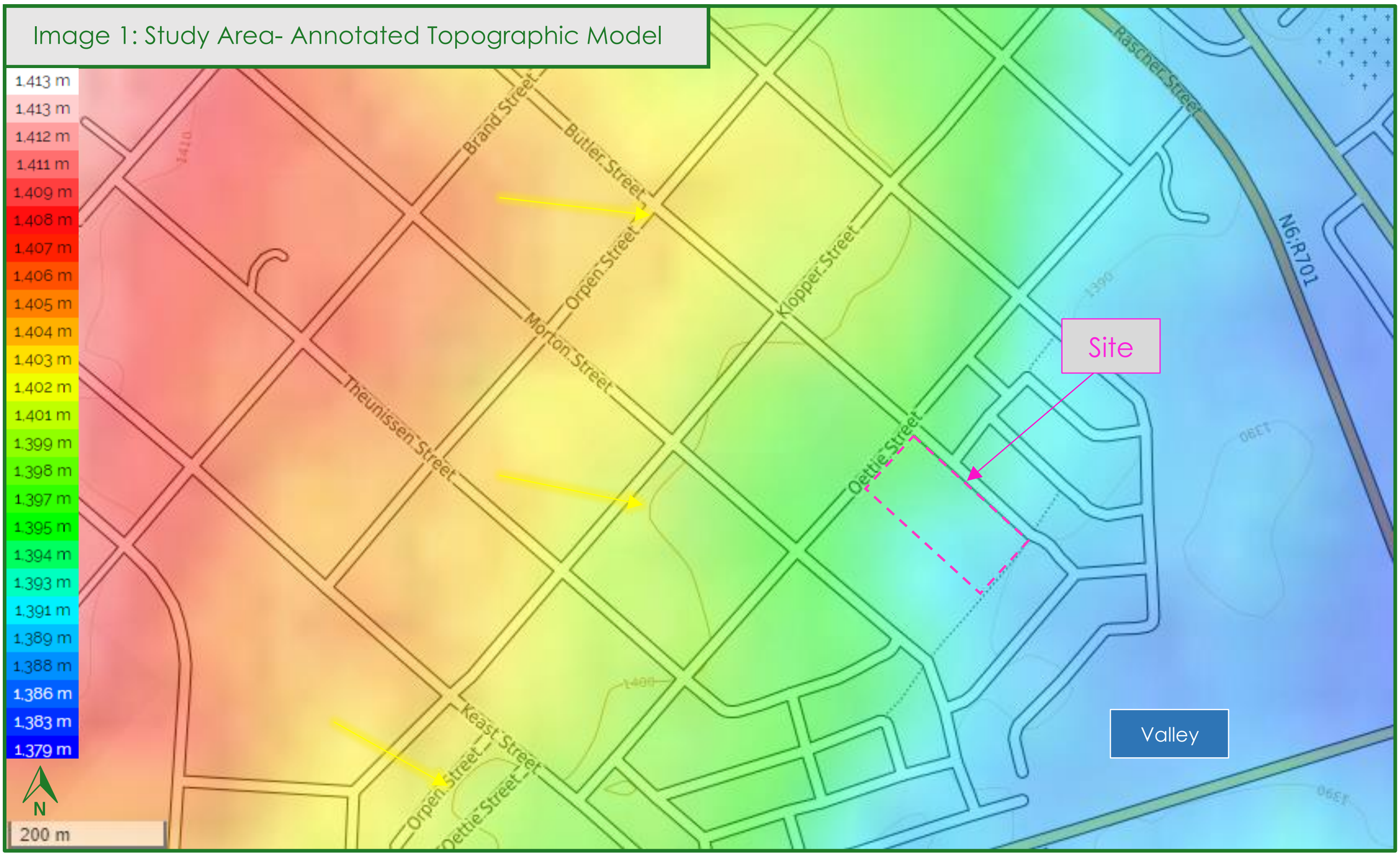



2.2. Site Topography

As seen in the annotated topographic map below (Image 1); the western and north western portions of the study area are seen to exhibit a high lying topographic nature (red, orange and yellow colours). Moreover, the **natural slopes** (yellow arrows) extend from this high lying terrain towards the valley/low-lying area situated to the east and south east.

Image 1 graphically depicts the topographic nature of the study area.

Image 1: Study Area- Annotated Topographic Model



Although the image depicts a fluctuating geomorphological environment (constant changes in colour), the range in elevation is notably low. This serves as evidence for a uniform and continuous surficial morphology.

With reference to Image 1; the site is seen to fall along the eastern to south eastern side slope of the above described high-lying terrain, immediately north west of the annotated valley landform. Consequently, the **natural** topographical nature of the site can be characterised by very gentle to gentle slopes extending from the higher lying areas to the west and north west, in a general south easterly to easterly direction.

As a whole, the site is seen to host a **very gentle sloping surface morphology**, with average measured slopes of approximately **2 degrees**. The north western portions of the site were seen to display the highest measured elevations of approximately 1394 meters above mean sea level (MAMSL); decreasing to an elevation of approximately 1390 MAMSL in the far eastern to south eastern portions (calculated using Google Earth PRO tm).

As discussed in Section 2.1 of this report; the surface of the site has been **remoulded** through ongoing **anthropogenic processes**. It is envisaged that this parcel of land has been primarily remoulded through both agricultural and recreational activities, followed by uncontrolled dumping.

The artificial reworking of the sites surface affects the continuity and degree of **natural slopes** traversing its' footprint, impacting its' natural drainage character and resulting in the formation of **small-scale topographic anomalies**. Small scale topographic anomalies will need to be addressed individually in the engineering design and in so doing eliminating their localised effects. Please note that these processes have not only artificially diversified the topographical nature of the investigated site, but also its geotechnical character.

Based on the available information, the planned development will entail the complete rehabilitation of the site's surface.

2.3. Drainage

The drainage nature of the site will mirror its' topographic nature as described in Section 2.2 of this report.

Due to the sites' **very gentle sloping nature**, it will drain mainly by means of **low energy surface run-off (sheet-flow);** with storm water flowing from the high lying western to north western portions of the site, in a general easterly to south easterly direction. The very gentle sloping portions of the site will be subjected to elevated degrees of **surface water infiltration** into the underlying soils, rather than rapid surface water flow, accentuating **surface water ponding** and **fluctuating moisture conditions** after prolonged precipitation events. Amplified surface water ponding is expected to occur across the site due to the **clayey nature** of the soils (low permeability).

Surface water ponding is exacerbated through the occurrence of compacted surfaces and localised depressions (artificial).

The land surface located upslope of the site is seen to exhibit a similar topographic nature. Should excessive infiltration take place across this area; it is predicted that elevated volumes of **shallow ground water throughflow** may occur across the investigated site.

According to the available information (Topocadastral Map 3026Ba - Figure 3), there are <u>no</u> **natural** drainage structures traversing the investigated site. The closest mapped natural structure is the meandering non-perennial river located approximately 200 m south east of the site (mapped to fall within the aforementioned valley landform- Image 1).

The surface runoff will be channelled away artificially due to the presence of surficial infrastructure between the site and the major natural drainage systems in the area.

The anthropogenic reworking of the sites surface will result in local variations of surface water flow- both rate and direction. The continuity and manipulation of the topography and associated drainage plays a pivotal role in the longevity and sustainability of the development.

2.4. Climate

Smithfield normally receives approximately **260 mm** of rain per year, with most rainfall occurring during **summer**. Smithfield experiences a semi-arid climate, with hot summer days. The average annual temperature for Smithfield is 22°C and it is dry for 258 days a year with an average humidity of 49% and an UV-index of 5. According to *Köppen and Geiger* climate classification, the climate is classified as **Arid Climate (BSk)** and **Cold Interior** (SANS 204-2).

Climate determines the mode and rate of weathering. The effect of climate on the weathering process (i.e., soil formation) is determined by the **climatic N value** defined by Weinert, 1980. The Climatic N-Value (Weinert, 1980) for the area is deemed to be **less than 5**; therefore, **chemical disintegration** of the parent rocks in the regional setting is deemed the principal mode of weathering.

This mode of weathering promotes the breakdown of the primary minerals within the parent rock results in the formation of secondary minerals such as clays and sesquioxides. Consequently, the climatic region favours the development of **fine-grained soils**, rather than an abundance of rocky fragments. Physical/ mechanical weathering of the parent rock will take place but on a lower scale.

2.5. Vegetation and Biotic Activity

The study area is located within the **Eastern Karoo Bio-Region** of the Nama-Karoo Biome (Mucina and Rutherford, 2006).

The **natural vegetation** was generally **denuded** during the initial activities undertaken across the site. At the time of this investigation, the vegetation across the site was comprised of a sparse grass cover with occasional/intermittent shrubs and small trees. Large trees were encountered intermittently along the boundary of the site. Extensive sub-surface vegetation is predicted to occur in areas large trees. The effects of the removal of trees on sites should also be considered, particularly where trees have depressed the water table over a period of time. The removal of large trees can result in the formation of highly compressible zones of voided soils.

The degree of organic material and biotic activity was seen to decrease with an increase in depth, with minor root systems reaching to an average depth of **approximately 0.18 m** below the existing ground level (organic rich topsoil).

Many structures are likely to be near planted or self-sown trees during their useful life. In some situations, trees can adversely affect structures and induce damage. All trees should be regarded as a potential source of damage. The greatest risk of direct damage occurs close to the tree from the growth of the main trunk and roots and diminishes rapidly with distance.

Where adequate distances are not observed, precautions, such as the reinforcement of foundations to resist lateral thrusts and the bridging over of roots to allow for future growth, should be adopted.

2.6. Cemetery Sites

No mounds, suggestive of semi-formal graves, or burial sites were observed during the site investigation.

3. Regional Geological and Hydrogeological Setting

3.1. Regional Stratigraphic Setting

According to the available geological information (geological series map: 3026 Aliwal North), the study area is primarily underlain by the following **two** geological units (Figure 2):

1. "⌒⌒"- **Alluvium** of the **Quaternary Period**.
2. "TRt"- **Sandstone** and **Mudrock** of the Tarkastad Formation of the Beaufort Group- Karoo Supergroup.

The site lies within the Great Karoo Basin. The Karoo basin is a retro-arc foreland basin with a present area of approximately 600,000 km^2 (Cole, 1992). The sediments of the Karoo basins were deposited in fluvial floodplains and shallow marine shelves over a period of more than one hundred million years extending from the late Carboniferous (300 million years ago) to the early Jurassic (190 million years ago) prior to the separation of southern Africa from Gondwanaland.

Rocks within the main Karoo basin have been subjected to several cycles of erosion, which resulted in widespread planation. The great depths of weathering typically associated with the African surface are seldom preserved unless a surface pedocrete armour ("duri-crust") has remained to protect the weathered residue from erosion.

The study area does <u>not</u> reflect any risk for the formation of sinkholes or subsidence's caused by the presence of water-soluble rocks (dolomite), and as such is **<u>not</u> deemed "*dolomitic land*".**

3.2. Alluvium- Quaternary Sediments

Quaternary sediment deposits are seen to be extensive throughout the study area- the prevalence of which can be linked to the region's geomorphology.

The predominant sediment deposits within the region are seen to comprise of transported material of an **alluvial origin**. Alluvium is a general name for sediments which have been deposited by river and stream systems during different depositional times. The properties of the alluvium can vary substantially and are dependent on three dominant factors, including- the geology within the catchment area, the depositional nature of the river system, as well the competence of the river (Chapter 8, Brink, 1979).

The site was seen to by display **alternating sequences** of transported sediment, exhibiting a heterogenous composition. These young deposits consist of **multiple cycles** of deposition resulting from varying transport mechanisms (a combination of both alluvial and aeolian deposits). The final product is a **layered sediment deposit** with frequent in-situ variations in predominantly composition and colour.

3.3. Sandstone Bedrock- Tarkastad Formation

Upon excavation it was established that the vast majority of the site is underlain by fine-grained sedimentary rocks deemed to be **Sandstone**.

Sandstone is a sedimentary rock that has formed from the lithification of sand dominated sediments.

Sandstones are among the most common sedimentary rocks, with the Karoo Sandstones and quartzitic Sandstones covering extensive portions of the interior of South Africa. Quartz grains are the dominant constituents of sandstones but are frequently accompanied by varying amounts of feldspars, clays, rock fragments and other materials (Davis et al, 1978).

The sandstone bedrock within the Tarkastad Formation is characterised as being fine to medium grained lithofeldspathic sandstone (rich in rock fragments and feldspar content). The weathering of sandstone bedrock may lead to the formation of residual soils ranging between sandy to gravel dominated soils, depending on the climatic nature of the area (physical, chemical or biological weathering), as well as the degree of exposure (jointing).

The sandstone is predicted to be underlain by the weathered constituents of mudrock bedrock (discussed below).

3.4. Mudrock Bedrock- Tarkastad Formation

Upon excavation it was established that the low-lying eastern portions of the site are underlain by very fine-grained sedimentary rocks deemed to be **mudrock**.

The weathering of mudrock bedrock may lead to the formation of residual soils ranging between silty/clay to clayey gravel soils, depending on the climatic nature of the area (chemical, physical or biochemical weathering), as well as the degree of exposure (jointing).

Furthermore, due to the fine-grained nature of these soils, groundwater has a detrimental effect on these soils and their mechanical properties. The shear strength of fine-grained soils is dependent on both the intergranular forces (cohesion) between the particles and the internal angle of friction. Clayey soils typically have as low internal angle of friction and upon saturation displays a reduction in shear strength as a result of a loss in cohesion.

One of the biggest issues with the Karoo Mudrocks is the **tendency of rapid and spontaneous breakdown upon exposure**, with the Mudrocks exhibiting three main responses to exposure; very little breakdown, disintegrating into hard fragments of various sizes and slaking Mudrock which breaks down into clay and silt sized particles (Brink, 1983).

The **slaking process** is not yet fully understood, but the dissolution of cementing material as well as air-breakage (process of building up internal pressure due to water absorption) play a definite role. The disintegration process is not significantly affected by the abovementioned processes; but is deemed to be controlled by moisture loss and/or stress relief which results in micro-cracks. When these micro-cracked Mudrocks encounter water, they expand and break along these planes of weakness. Previous experiments by Olivier (1979a) and Venter (1980) also concluded that the Karoo Mudrocks were seen to be very sensitive to changes in humidity.

Residual soils of the Karoo Mudrock have been known to exhibit **expansive behaviour**, but it is now known that fresh Karoo Mudrocks also display **dimensional changes** following **changes** in **moisture content**. Olivier (1976) documented free swells ranging from 0.01 to 7% for samples from the Beaufort Mudrocks of the Tarkastad Formation.

3.5. Mineral Deposits

According to the geological maps and accompanied explanation no specific mineral deposits are present on the site.

3.6. Prominent Geological Structures

According to the available information, there are no prominent geological structures traversing the investigated site.

Geological mapping is based on surficial outcrops and aeromagnetic data (regional geophysical data), either of which are not feasible in a geological setting of this nature. The thick sediment deposits blanket geological structures, with their extensive nature and thickness obscuring traceable geophysical patterns and evidence.

3.7. Seismic Risk

According to **Kijko et al (2003)** the regional seismic hazard in the project area can be defined as **LOW**, exhibiting a 10% probability of a seismic event with a peak ground acceleration of between approximately **100 and 125 cm/s^2** within a period of 50 years.

3.8. Hydrogeological Setting

It is envisaged that the future development across the site will be serviced by local municipal services, for this reason, no site-specific hydraulic conductivity tests or borehole searches were undertaken.

According to the available hydrogeological information (hydrogeological series map: 2924 Bloemfontein (2002) and SADC GIP); the study area is mainly underlain by **Inter-granular and Fractured Aquifers** of medium yield with **limited potential (D3).** The bedrock underlying the study area at depth represents a weathered- and fractured aquifer where groundwater rest level occurs within fractures of the bedrock at depth.

Borehole yields are typically between 0.5 and 2.0 l/s.

The groundwater quality is deemed to be between 0 and 70 mS/m (electrical conductivity range).

According to the available information, large scale groundwater abstraction does not take place within close proximity to the site.

Based on the SADC Groundwater Information Portal; there are no recent boreholes drilled across or within close proximity to the study area in question. For this reason, the static rest level and chemistry of the ground water cannot be discussed. A hydro census can be conducted across the site to determine the ground water table depth, location and quality/category underlying the site.

The site falls within the Quaternary Catchment Area D24H, which forms part of the Caledon Local River Catchment Area of the greater Orange River Catchment Area.

4. Geotechnical Evaluation

4.1. Slope Stability and Erosion

Emphasis should be placed on surface drainage and storm water control measures to avoid both surface water ponding and concentrated water flow (erosion) across the development area. Structures constructed perpendicular to the natural slopes will result in the ponding of surface water. Furthermore, the development will influence natural infiltration and run-off rates and appropriate precautions against concentrated flow must therefore be implemented.

No natural slope instabilities were visible in these areas at the time of the investigation (basic visual inspection).

Specialised methods for the stabilisation of cuts into the existing slopes are not deemed necessary. Due to the site gradient- cut to fill site preparation is also not expected (unless structures exhibit a large footprint parallel to the natural slope).

The very gentle sloping nature of the site will aid surface water infiltration into the underlying soils, rather than rapid surface water flow, accentuating **surface water ponding** and fluctuating moisture conditions after prolonged precipitation events. Surface water ponding is exacerbated through the anthropogenically reworked nature of the sites surface and the **low permeability subsoils.**

Surface water ponding will be more prolific in areas hosting anthropogenic depression and where bulk earthworks are employed to create level platforms. Attention must be given to site contouring to ensure an effective gradient is achieved so that standing water does not occur, and the draining of water is efficient to minimise erosion and damage to the construction.

Due to the sites' <u>natural slope</u>, erosion will not be a concern. The compaction of the topsoil through vehicle and/or foot traffic will result in poor drainage characteristics and the possibility of channelized surface water flow- elevating the risk of erosion. Erosion is predicted to be more prolific in areas where the natural vegetation is stripped.

The anthropogenic reworking of the sites surface will result in local variations of surface water flow- both rate and direction.

The continuity and manipulation of the topography and associated drainage plays a pivotal role in the longevity and sustainability of the development. Topographic anomalies identified/measured during the professional survey can be addressed individually in the design.

Adequate drainage measures can be discussed with the project team once the design/layout of the development has been formulated.

4.2. Rock- and/or Pedocrete Outcrops

<u>No</u> bedrock or pedocrete outcrops were encountered across the site.

4.3. Trenching

A total of SIX test pits, namely TP1 to TP6, were excavated in January 2022, by means of a TLB-type light mechanical excavator (JCB 3CX) to a depth of approximately 2.5 meters below the existing ground level, or refusal. Thereafter, the exposed soil and rock layers were profiled and sampled. Several challenges arose regarding trial pit placements (relict infrastructure/fill, etc.); however, the project team managed the risk- with the placement of trial pits in such a way so as to create an accurate and reproducible site-specific geotechnical model.

Test pits were distributed across the designated development area (Figure 5), in accessible locations deemed safe for excavations and free of subsurface infrastructure. The succession of soil layers exposed within the test pits were logged and a series of detailed photographs were taken.

Undisturbed, disturbed and bulk samples were taken of the material deemed to be important to the proposed development.

An Engineering Geologist supervised the excavation of the trial pit as well as the recording the soil profiles using the standard procedures as prescribed by AEG/SAIEG/SAICE (2002).

4.4. Generalized Ground Profile

Note: this description is based on field observations and does **not** reflect the results of any laboratory tests.

4.4.1. Introduction

The results of the trenching phase indicate that most of the site is blanketed by a fine-grained transported sediment deemed to be of an **alluvial origin.** This topsoil was reworked and/or contaminated to a degree through the past human activities undertaken across the site. The uppermost extent of these exposures was seen to be organic rich, with the degree of organic material seen to decrease with an increase in depth.

Across the **central to north western portions**, the clayey transported materials were typically underlain by granular materials deemed to be **residual sandstone-** with these excavations transitioning into **weathered sandstone bedrock** at depth. The quality and continuity of the rock mass was seen to increase with an increase in depth. Only thin exposures of the rock mass were possible before undergoing refusal.

Furthermore, across the **low-lying south eastern portions**, the alluvium was typically underlain by a fine-grained **residual mudrock**, transitioning into **weathered mudrock bedrock** at depth. These portions were generally seen to host thin beds of decomposed sandstone bedrock.

Uncontrolled fill was not exposed in any of the excavated test pits (surficial anomalies avoided). This attribute is **expected** to occur numerous times across the site due to the extent of previous and ongoing human activities in the area (i.e., between known testing locations). Surficial fill materials were encountered sporadically across the site- with the most prolific occurrences along the various boundaries of the site.

Detailed soil profile logs are included in **Appendix A.** Refer to Table 1 overleaf which depicts the summarised ground profile of the site.

Table 1: Summarised Ground Profile for the Site

Summarised Ground Profile																					
Test Pit	Topsoil			Alluvium			Residual Sandstone			Weathered Sandstone Bedrock			Residual Mudrock			Weathered Mudrock Bedrock			Excavation Character		
	From	To	Thickness (m)	From	To	Thickness (m)	From	To	Thickness (m)	From	To	Thickness (m)	From	To	Thickness (m)	From	To	Thickness (m)	Test pit depth (m)	Termination Conditions	Termination Material
TP1	0,00	0,15	0,15	0,15	0,45	0,30	0,45	0,70	0,25	0,70	1,10	0,40	-	-	-	-	-	-	1,10	R	WSB
TP2	0,00	0,15	0,15	0,15	1,00	0,85	-	-	-	1,00	1,30	0,30	-	-	-	-	-	-	1,30	R	WSB
TP3	0,00	0,15	0,15	0,15	0,60	0,45	-	-	-	0,60	1,23	0,60	-	-	-	-	-	-	1,23	R	WSB
TP4	0,00	0,30	0,30	-	-	-	-	-	-	0,30	0,90	0,50	-	-	-	-	-	-	0,90	R	WSB
TP5	0,00	0,20	0,20	-	-	-	-	-	-	0,20	0,43	0,23	0,43	0,90	0,47	0,90	1,30	0,40	1,30	R	WMB
TP6	0,00	0,15	0,15	-	-	-	-	-	-	-	-	-	0,15	0,80	0,65	0,80	1,60	0,80	1,65	DE	WMB
Data Summary	**Minimum**		0,15	0,15	0,45	0,30	0,45	0,70	0,25	0,20	0,43	0,23	0,15	0,80	0,47	0,80	1,30	0,40	0,90		
	Max		0,30	0,15	1,00	0,85	0,45	0,70	0,25	1,00	1,30	0,60	0,43	0,90	0,65	0,90	1,60	0,80	1,65		
	Average		**0,18**	0,15	0,68	**0,53**	0,45	0,70	**0,25**	0,56	0,99	**0,41**	0,29	0,85	**0,56**	0,85	1,45	**0,60**	**1,25**		
Notes:	Thickness refers to the exposed thickness of each soil horizon (meters)																				
	All depths displayed in meters (m)																				
	Horizon depths displayed in the table are average measured values																				
	Excavation conditions: R-Refusal; DE - Difficult Excavation; ES- Excavation Stopped																				
	Materials: WSB - Weathered Sandstone Bedrock; WMB - Weathered Mudrock Bedrock																				

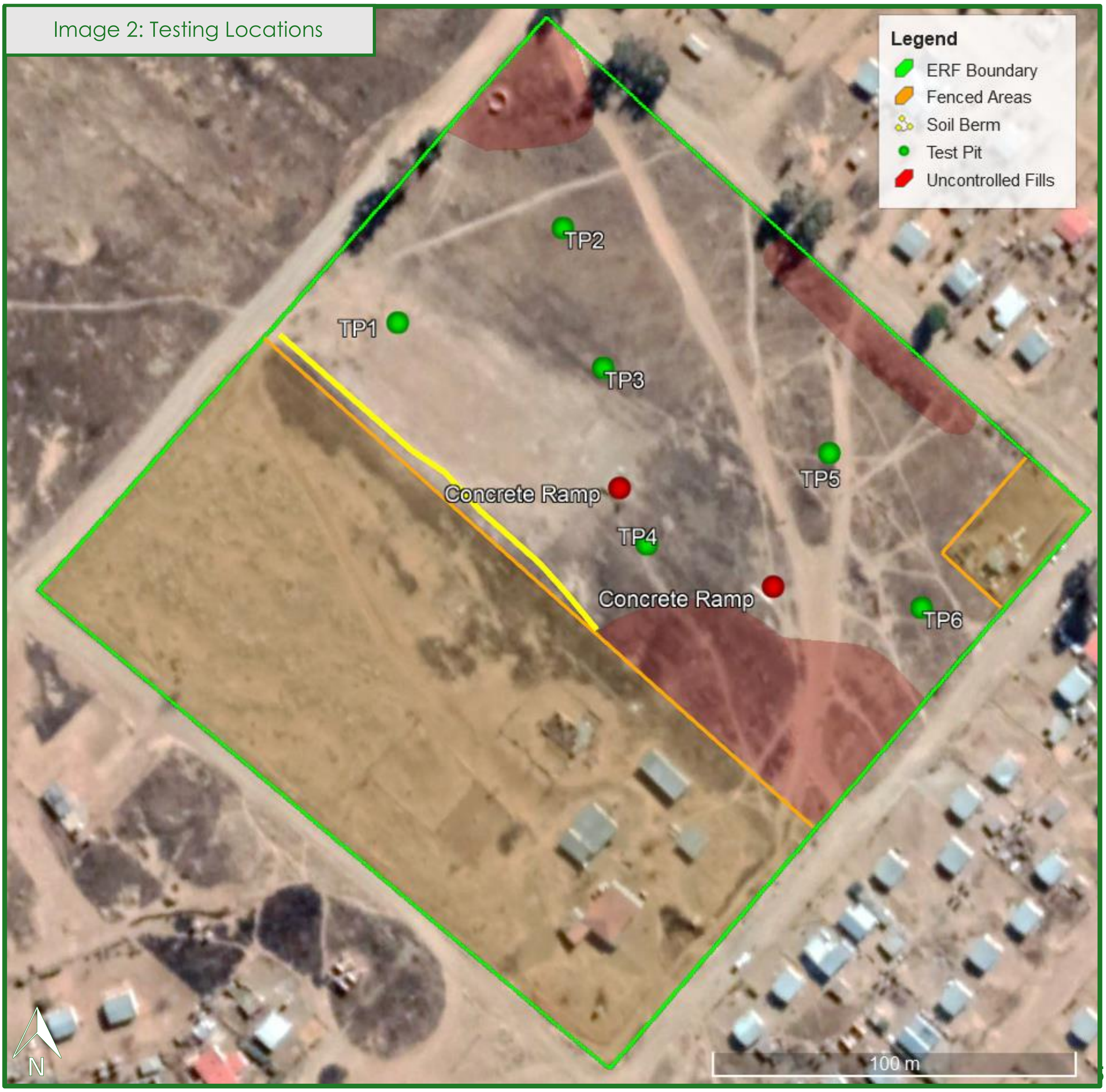


Image 2: Testing Locations

The information presented below is based on point data. Every effort was made during the site investigation to ensure that generally accepted practices of our profession were used in the sub-surface evaluation of the site, and that the sampling and testing was representative of the soil/rock conditions observed on-site. Variances in soil quality and quantity from those predicted may be encountered during construction and these should be recorded.

The ground profiles across the site can be **generally summarised** as follows:

4.4.2. Uncontrolled Fill- Human Origin

Surficial uncontrolled fill (landfill) material was present across scattered portions of the site. The red areas presented on **Image 2** above are annotated zones where **prolific extents** of surficial dumped uncontrolled fill materials can be expected. Similar localised phenomenon can be assumed across most of the site due to the extent of previous and ongoing human activities in the area.

The fills were prominent surrounding relict infrastructure and adjacent to the boundary roads/fences. These successions of fill material were dumped across the site in an **uncontrolled** manner during past anthropogenic activities- with the material **not** placed in controlled layers.

Based on surficial exposures, this material displayed a highly variable composition (heterogenous range of particles), thickness and consistency, with the occurrence of anthropogenic contamination in the form of building rubble and waste. Surficial boulders and boulder-sized concrete blocks were also seen to litter the surface. These parameters have resulted in subsoils which display an **inconsistent/discontinuous geotechnical nature** (soil mechanics).

The **mechanics** of these deposits are predicted to be **highly variable** as a result of the both the variable nature of the particles (rock or waste) and the variable interaction between individual particles. Based on the available information, uncontrolled fills are limited to **surficial occurrences**- excluding notable thick deposits at depth. Surficial heaps of fill ranged in shape and size sporadically.

Uncontrolled fill was not exposed in any of the excavated test pits (surficial anomalies avoided). The extent, occurrence, consistency, soil texture and depth of the fill horizons are expected to be highly variable across the site.

It is strongly recommended that this material be **selectively mined** and **removed** from within the footprint of the proposed development (to an extent deemed suitable by the competent design engineer).

4.4.3. Alluvium

In areas not seen to host fills, the site was seen to be blanketed by a topsoil comprised of transported material deemed to be of alluvial origin- the uppermost extent of which was reworked to varying degrees through anthropogenic processes.

This layer of transported material has been transported by a natural agent during relatively recent geological times and has not undergone lithification into a sedimentary rock.

As a whole, the exposed successions of alluvium were seen to be slightly variable, with fluctuations in moisture content, composition and colour across the site. Furthermore, due to its age and shallow occurrence- these alluvial deposits may **lack** essential pre-consolidation characteristics (normally consolidated state). This may result in additional consolidation settlement upon saturation and loading.

The alluvium was typically present in **two distinct horizons**; namely, an **organic rich topsoil** followed by a **clayey/silty alluvium** with depth. Moreover, both the topsoil and underlying alluvium displayed a low in-situ density, fine-grained and structured nature; attributes typically associated with **highly compressible and potentially active soils**. Furthermore, these soils are predicted to have a low in-situ permeability.

The **topsoil** was generally described as a: *slightly moist; light and dark brown;* ***loose*** *and medium dense;* ***pinholed****;* ***silty sand*** *with traces/minor gravel and traces of* ***surficial cobbles****; alluvium/topsoil;* ***organic rich topsoil****; minor* ***anthropogenic contamination****; minor/abundant fine roots.*

The topsoil was seen to extend from the surface to depths of between **0.15 and 0.30 m-** displaying an **average exposed thickness of 0.18 m.** The thickness and moisture content of the topsoil is expected fluctuate across the site.

The **clayey/silty alluvium** was generally described as a: *moist; dark brown and dark yellowish/orangey brown, mottled black and white, blotched white and grey;* ***firm and stiff****;* ***pinholed****,* ***fissured*** *and* ***slickensided****;* ***clayey silt*** *and* ***sandy clay*** *with traces/minor fine sandstone gravels; alluvium; traces of fine roots; traces of* ***ferricrete nodules****.*

The clayey/silty alluvium was seen to extend from below the topsoil to depths of between **0.45 and 1.00 m-** displaying an **average thickness of 0.53 m.**

4.4.4. Weathered Constituents of the Sandstone Bedrock- Tarkastad Formation

Residual Sandstone

In the far north eastern portion of the site (test pit TP1) **residual sandstone** was encountered below the above defined transported materials. Residual material has formed from the weathering of the in-situ bedrock to the point where the material will behave/respond as a **soil**.

Thin exposures of residuum were present in surrounding test pits but due to the rapid transition in weathered rock they could not accurately be defined. The residuum was seen to host **pedogenic materials** in the form of ferricrete nodules and staining. In this case, **pedogenic materials** are predicted to have formed because of fluctuating moisture conditions- with the perching of water upon the dense weathered rock mass at depth.

The residual sandstone was generally profiled as a: *slightly moist; light brown, blotched white, stained red;* ***dense****;* ***intact****;* ***silty sand*** *with minor angular* ***sandstone gravels****; residual sandstone; traces of fine roots; traces of* ***ferricrete*** *nodules and* ***ferruginisation on gravels.***

The residuum was encountered from below the alluvium, extending to a depth of **0.7 m** below the E.G.L.- displaying an **exposed thickness of 0.25 m.**

Weathered Sandstone Bedrock

Within all the test pits excavated across the **high-lying central** to **north eastern portions** of the site, thin exposures of **weathered sandstone bedrock** were encountered below the alluvium/residuum. In general, the residual soils have undergone **a larger degree** of chemical and physical weathering, resulting in less resemblance to the protolith.

The degree of weathering within the protolith/ parent rock was seen to fluctuate, however, the rock mass was intact at depth- displaying a favourable geotechnical nature. The exposed constituent of the weathered protolith was deemed to be **highly weathered bedrock.**

The **weathered sandstone bedrock** was generally described as a: *slightly moist; light greyish olive cream, blotched white and brown, banded white, stained red, brown and grey;* ***dense to very dense at the base****;* ***fine grained****; thinly bedded;* ***highly weathered****;* ***medium hard rock****;* ***sandstone*** *of the* ***Tarkastad Formation****;* ***ferruginisation and calcification along open joints/bedding planes****; rapid transition into less decomposed sandstone bedrock; open joints with clay infilling; minor decomposed organic materials.*

The weathered bedrock was generally encountered from below the alluvium/ residuum, extending to the **final excavation depth** of between **0.90 and 1.30 m** below the E.G.L. Thereafter, the TLB-type light mechanical excavator underwent **refusal** on less weathered bedrock.

Thin exposures of the rock mass were present in the test pits- with the nature of the rock mass at depth unknown.

Within test pit TP5- a thin bed of completely weathered sandstone bedrock was encountered between the blanketing topsoil and the underlaying decomposed mudrock. This thin bed proves the predicted interbedded nature of the on-site geological units and further proves the thinning out of the sandstone bed with a decrease in elevation.

4.4.5. Weathered Constituents of the Mudrock Bedrock- Tarkastad Formation

Residual Mudrock

Within test pits **TP5 and TP6-** excavated across the low-lying south eastern portions of the site- **residual mudrock** was encountered from below the previously described alluvium.

As for the blanketing alluvium, the **residual mudrock** displayed a low in-situ density, clayey texture and fine-grained nature; attributes typically associated with **compressible and potentially active soils**. Furthermore, these soils are predicted to have a low in-situ permeability.

Preferential flow paths (infiltration zones) with bands of reworked residuum were encountered within the residuum. These attributes are predicted to be because of elevated degrees of **eluviation** (mobilisation of clays from the reworked residuum/topsoil) and **perching of ground water** upon the underlying rock mass.

The **residual mudrock** encountered across the site was generally described as a: *moist; dark olive grey; mottled white, banded purple and red;* ***firm*** *and* ***medium dense*** *to dense at the base;* ***relict bedding****;* ***sandy clay*** *with minor angular mudstone gravels OR angular* ***clayey silty gravel****; residual mudrock; traces of* ***calcification along relict bedding****; traces of fine roots; traces of decomposed roots;* ***minor bands of less decomposed protolith***.

The residual mudrock was encountered from below the alluvium- extending to depths of between 0.80 and 0.90 m below the existing ground level: exhibiting an **average thickness of 0.56 m**. Furthermore, the residual mudrock was seen to share a gradual contact with the underlying less decomposed bedrock.

Weathered Mudrock Bedrock

Within all the excavations conducted across this portion of the site, **weathered mudrock bedrock** was encountered below the residua.

The **weathered mudrock bedrock** was generally described as a: *slightly moist; dark greyish olive, banded cream, brown and purple;* ***dense****;* ***very fine grained****;* ***thinly bedded*** *and closely jointed;* ***completely weathered; soft rock****;* ***mudrock of the Tarkastad Formation****; traces of decomposed roots.*

The weathered bedrock was typically encountered from below the residuum, extending to the **final excavation depth** of between **1.30 and 1.65 m** below the existing ground level- displaying an exposed **average thickness** of **0.60 m.** Thereafter, excavations were terminated due to **refusal or gradual slow advance/difficult excavation conditions** though the use of a TLB-type light mechanical excavator. Difficult excavation conditions were encountered within this rock mass due to the hard nature of the mudrock bedrock coupled with its horizontally orientated bedding.

4.5. Groundwater and Shallow Seepage

No ground water seepage or permanent water table was encountered during the fieldwork phase of the investigation. Without knowledge on the depth to permanent groundwater in the region- it is not possible to comment on the occurrence of permanent groundwater flow within/below the weathered bedrock. It is therefore assumed that the natural/permanent groundwater level on site is below the zone of foundation influence (light structures).

All the excavations in natural materials were seen to host pedogenic materials at depth (both calcrete and ferricrete). These pedogenic inclusions indicate the periodic occurrence of fluctuating moisture conditions after prolonged precipitation events. The evidence for fluctuating moisture conditions was encountered within the upper 1.0 meter of the soil profile.

The clayey nature of the uppermost soil horizons will result in prolonged saturation of the sites subsoils, with poor infiltration characteristics.

The predominant runoff will occur as sheet wash following the topography. Waterlogged conditions and/or surface water ponding following prolonged and intense precipitation events are anticipated across the site due to its near flat and anthropogenically induced undulating nature.

Ground water is predicted to flow/perch along the rock-soil interface, with its flow dynamics dependant on the topography of the rock mass at that point. Bulk excavations down to hard rock may need to be dewatered during construction. The implementation of a sub-surface drainage system is only deemed necessary should foundation configurations result in the damming up of ground water flow (i.e., implementation of deep strip foundations and/or retaining structures).

Significant changes in moisture content may contribute to the anticipated consolidation settlement and/or expansive behaviour of the site soils. During construction and after development, shallow perched water systems may develop yet further due to stormwater management practices, localised infiltration and site modification practices.

Altering the soil profile in areas underlain by shallow bedrock commonly affects the subsurface seepage.

The weight of the structures on the surface may result in an increased ground water rest level. Adequate damp-proofing measures should be implemented beneath individual structures.

Drainage precautions are required to minimise infiltration that may lead to perching on the underlying bedrock.

Precautions should be implemented for deep structures such as subsoil tanks, deep pipelines etc.

If the site or a portion thereof is situated within the 1:100-year flood lines, or have been delineated as a wetland, it is the prerogative of the Civil Engineer or other suitably experienced specialist to overwrite the geotechnical recommendations for such portions.

4.6. Site Excavatability

The **average** excavation depth across the site was approximately **1.25 m.** Profiles were described in trenches excavated by means of TLB- type light mechanical excavator. End of hole conditions were typically due to refusal or gradual slow advance in weathered bedrock at depth.

No significant problems are foreseen during the excavation of **<u>shallow foundation</u>** trenches to a depth of approximately **0.90 m** below E.G.L. Thereafter, localised significant **problems** are foreseen in the excavation of **<u>deep service trenches</u>** though the use of light excavation methods (i.e., hand excavation, TLBs etc.)- due to the shallow occurrence of weathered bedrock.

The excavation type to an **average** depth of **1.25 m** below the existing ground level is deemed to be **SOFT Excavation** (SANS 10400G/SANS1200D). **Category 1 hard excavation** estimated to be between **approximately 10 and 40%** of material to 1.5 meters below ground level (mbgl).

With regards to both the sandstone and mudrock bedrock, due to the hard nature of the rock masses- the machine could only partially rip through the uppermost crust before undergoing refusal. Thin exposures of the rock mass were present in the test pits- with the nature of the rock mass at depth unknown (i.e., excavatability relating to frequency and spacing of joints).

Excavations within the weathered rock mass were notably challenging; with exposed material displaying a very dense nature at depth. Its' inherent excavatability predicted to fluctuate over short distances as a result of varying degrees of weathering and spacing of discontinuities. The rock mass at depth will require **hard rock** excavation methods to unearth.

The surface of the site was seen to host uncontrolled fill, scattered boulders and relict infrastructure- with these anomalies hampering terrain mobility and the continuity of site excavatability.

4.7. Sidewall Stability

Across most of the site, excavations generally remained **stable** for a period of at least 1 hour with little or no over break or collapse occurring. Following prolonged saturation, instabilities may occur.

Localised sidewall instabilities are predicted in areas hosting thick fills and/or relict infrastructure. Should bulk excavations be conducted in these portions they will need to be shored or adequately sloped/modified to limit failure.

Any excavation deeper than 1,5 mbgl must be stabilised as prescribed in the relevant act. Trenches may need to be dewatered following prolonged precipitation events because of water temporarily perching upon the underlaying less permeable materials and/or rock masses.

Although it is predicted that fresh exposures in bedrock exhibit stable conditions in near vertical cuts (depending on orientation), it is recommended that excavations be adequately sloped (approximately 60 to 75 degrees) or stabilised to reduce the risk of instabilities. Instability is predicted to be in the form of localised rock block failure, necessitating the removal or stabilisation of unstable rock blocks.

4.8. Engineering and Material Characteristics

4.8.1. Sampling

The engineering material properties of the various sampled soil horizons were measured in laboratory conditions as per accredited testing procedures.

Standard foundation indicator, compaction tests and soil chemistry tests were conducted by Letaba Lab Bloemfontein (**SANAS Accredited**) on disturbed and bulk soil samples. These tests were undertaken in order to determine the composition of the underlying soils (i.e.: the relative percentages of gravel, sand, silt and clay) and to evaluate the suitability of the materials for the re-use in the proposed construction.

Full lab test results are presented in **Appendix B**.

The sampling which took place during this investigation was based on both the in-situ geotechnical properties of the exposed soil horizons as well as the nature of the development. Problem soil horizons were accurately sampled where encountered (i.e., expansive soils).

This section focuses on the identification and assessment of the soil properties which will influence the proposed construction.

The uncontrolled fill materials seen to blanket the site were omitted from the sampling process. These materials are predicted to display variable material characteristics.

The sampling process **excluded** the following inclusions:

- Oversized particles
- Organic materials
- Anthropogenic contamination

4.8.2. Bulk and Disturbed Samples- Laboratory Test Results

The soil testing which was conducted across the site can be subdivided into three broad categories, as follows:

- **Foundation Indictor Tests**

  Atterberg limits (Liquid Limit, Plasticity Index and Linear Shrinkage) and Particle-size Distribution

- **Compaction Tests**

  Maximum Dry Density versus Optimum Moisture Content and Californian Bearing Ratio versus Compaction Effort (MOD AASHTO method)

- **Soil Chemistry Tests**

  pH and EC analysis (corrosivity) as well as double hydrometers as an indication of material dispersivity.

The results presented in the summaries to follow are as received from the accredited testing facility. Although the summaries have been annotated no amendments have been made to the results themselves.

The tables to follow summarise the results of the soil tests conducted on the various sampled materials.

Table 2: Foundation Indictor Test Results

Foundation Indictor Test Results																		
				Particle Size Distribution								Atterberg Limits				Material Classification		
				Standard Sieves Cumulative percentage passing				Hydrometer Result Soil Mortar Analysis % of material <0,425 mm										
Sample Number	Test Pit	Depth (m)	Sampled Material	37,5 mm	20 mm	2,00 mm	0,425 mm	% Coarse Sand	% Fine Sand	% Silt	% Clay	LL	PI	LS	Grading Modulus	TRB	USC	Heave
8885-1	TP1	0,10-0,55	Alluvium	100	100	96	94	2,1	40,9	14,6	23,3	24	5	2,4	0,6	A-4	CL	LOW
8885-2	TP2	0,20-1,10	Alluvium	100	100	97	95	1,1	48,1	12,4	24,0	24	5	2,2	0,6	A-4	SM/SC	LOW
8885-3	TP3	0,15-0,65	Alluvium	100	100	89	86	3,0	39,3	13,0	31,8	29	9	4,2	0,7	A-4	CL	LOW
8885-4	TP4	0,30-0,90	Weathered Sandstone Bedrock	81	65	37	34	9,2	35,8	17,0	14,0	26	6	3,0	2,1	A-1-b	GM/GC	LOW
8885-5	TP5	0,50-0,90	Residual Mudrock	100	100	85	81	4,9	13,1	34,4	22,6	28	14	6,6	0,6	A-6	CL	LOW
8885-6	TP6	0,20-0,80	Residual Mudrock	100	100	86	80	7,1	40,8	19,5	10,6	24	6	3,0	0,9	A-4	SM/SC	LOW

Notes: 1. Hydrometer test and Atterberg Limits undertaken on material passing the <0,425 mm sieve
2. Atterberg Limits: Liquid Limit (LL), Plasticity Index weighted (PI), Linear Shrinkage (LS).
3. Heave: Potential expansiveness (acc. Van Der Merwe, 1964).

Table 3: Compaction Test Results

Compaction Test Results										
Sample Number	Test Pit	Depth (m)	Sampled Material	Compaction Test Results					Material Classification	
				MDD (kg/m³)	OMC (%)	Measured CBR Values			COLTO	Material re-usage potential
						90%	93%	100%		
8885-2	TP2	0,20-1,10	Alluvium	1807	14,3	1	1	2	<G9	POOR
8885-4	TP4	0,30-0,90	Weathered Sandstone Bedrock	2054	8,6	3	3	4	<G9	POOR
8885-5	TP5	0,50-0,90	Residual Mudrock	1987	10,2	2	3	6	<G9	POOR

Notes: 1. Compaction Test Results; percentage compaction of MDD (MOD AASTHO Method); CBR of 13.344 kN
2. MDD- Maximum Dry Density; OMC- Optimum Moisture Content
3. Material classification and re-usage potential based on the COLTO classification system

Table 4: Soil Chemistry Test Results

Soil Chemistry Test Results								
Sample Number	Test Pit	Depth (m)	Sampled Material	Corrosivity Indicators				Dispersivity Indicators
				Profiled In-Situ Moisture	% Clay	pH	EC (S/m)	Double Hydrometer Results (%)
8885-2	TP2	0,20-1,10	Alluvium	Moist	24,0	7,6	0,040	41
8885-5	TP5	0,50-0,90	Residual Mudrock	Moist	22,6	7,6	0,036	34

Notes: 1. Material as per soil profile description.
2. Moisture as per soil profile description or natural moisture content.
3. Percentage clay, pH, EC (electrical Conductivity) and double hydrometer as per laboratory results.

Alluvium

The results from the samples extracted from the alluvium across the site indicate the following:

- These soils grade as **silt/fine sand** with a grading modulus of between 0.6 and 0.7.
- In excess of 85 % of the sample materials were seen to pass the **0.425mm sieve**. This indicates the on-site materials have a **notably high fines content**.
- The soils exhibit a **low to MEDIUM plasticity**, medium linear shrinkage values and an overall LOW potential for heave (acc. Van Der Merwe, 1964). The measured PI values range between **5 and 9**.
- Typical Unified Soil Classes are **SM/SC and CL**.
- Typical TRB Class are **A-4.**
- Drainage will be poor, and a medium compressibility can be expected once compacted.
- The sampled materials typically displayed a **poor reaction** to **compaction**; classifying as a **worse than G9-** type material according to the COLTO classification system; and are therefore **not deemed suitable** for the reuse in the construction (suitability based on the engineer's design).

- The sample displayed a calculated **remoulded bearing capacity** of approximately **10 kPa** @ 93 % MOD AASHTO with a Factor of Safety of 1.5.
- In these fine materials- a rapid reduction in shear strength is anticipated upon saturation.
- Some of the soil samples were tested for pH and electrical conductivity. These results and other indicators of aggressiveness to steel and concrete (corrosivity) are shown in Table 4. Based on the measured **EC results**; the alluvium blanketing the site is deemed to be **potentially corrosive.** It is advisable not to use steel pipes.
- Based on the available double hydrometer readings the sampled alluvium is deemed to be **slightly dispersive.**

Residual Mudrock

The results from the samples extracted from the residual mudrock across the site indicate the following:

- These soils grade as **silt/fine sand** with a grading modulus of between 0.6 and 0.9.
- In excess of 80 % of the sample materials were seen to pass the **0.425mm sieve**. This indicates the on-site materials have a **notably high fines content**.
- The soils exhibit a **low to MEDIUM plasticity**, medium to high linear shrinkage values and an overall LOW potential for heave (acc. Van Der Merwe, 1964). The measured PI values range between **6 and 14**.
- Typical Unified Soil Classes are **SM/SC and CL**.
- Typical TRB Class are **A-4** and **A-6.**
- Drainage will be poor, and a medium compressibility can be expected once compacted.
- The sampled materials typically displayed a **poor reaction** to **compaction**; classifying as a **worse than G9-** type material according to the COLTO classification system; and are therefore **not deemed suitable** for the reuse in the construction (suitability based on the engineer's design).
- The sample displayed a calculated **remoulded bearing capacity** of approximately **25 kPa** @ 93 % MOD AASHTO with a Factor of Safety of 1.5.
- In these fine materials- a rapid reduction in shear strength is anticipated upon saturation.
- Some of the soil samples were tested for pH and electrical conductivity. These results and other indicators of aggressiveness to steel and concrete (corrosivity) are shown in Table 4. Based on the measured **EC results**; the residual mudrock is deemed to be **potentially corrosive.** It is advisable not to use steel pipes.
- Based on the available double hydrometer readings the sampled alluvium is deemed to be **slightly dispersive.**

Weathered Sandstone Bedrock

The results from the samples extracted from the weathered sandstone bedrock across the site indicate the following:

- These soils grade as fine GRAVEL with a grading modulus of **2.1.**

- The material exhibits a **LOW plasticity**, low linear shrinkage values and an overall LOW potential for heave (acc. Van Der Merwe, 1964).
- The measured PI value is **6**.
- Typical Unified Soil Classes are **GM/GC**.
- Typical TRB Class are **A-1-b.**
- The sampled materials typically displayed a **poor reaction** to **compaction**; classifying as a **worse than G9-** type material according to the COLTO classification system; and are therefore **not deemed suitable** for the reuse in the construction (suitability based on the engineer's design).
- The sample displayed a calculated **remoulded bearing capacity** of approximately **25 kPa** @ 93 % MOD AASHTO with a Factor of Safety of 1.5.

As a whole, the soft materials sampled across the site are **not deemed suitable** for the re-use in the proposed construction. These materials can be over-excavated and **spoiled**. Material will need to be imported for the use in layer works (roads, parking areas and selected fills (surface beds etc)).

4.9. In-Situ Mechanical Assessment

4.9.1. Introduction

The soils underlying the site have been examined and tested to determine their suitability as founding horizons for the proposed development (light structures) according to the following criteria:

- **Bearing capacities** of the founding materials determined from estimated field consistencies, Shearbox tests, DCP tests and inferred from tabulated strength values.
- **Compressibility** of the founding materials measured from laboratory test results.
- **Collapse potential**, where applicable.
- Potential **heave**, where applicable.
- Predicted **displacements** (settlement/collapse/heave) from the above factors.

Undisturbed sample extraction was not possible/feasible in the granular residua and exposed weathered bedrock. For this reason, the mechanical assessment of these horizons will be based on the available disturbed soil test results and field observations (i.e., soil structure, composition, and consistency).

Calculations are based on the geometry of the foundation. GeoCalibre is open to ongoing discussions regarding these calculations using data from exact depths and known structural configurations.

The **uncontrolled fill materials** displayed a highly variable composition (heterogenous range of particles), thickness and consistency, with the occurrence of anthropogenic contamination in the form of building rubble and waste. Surficial boulders and boulder-sized concrete blocks were also seen to litter the surface. These parameters have resulted in subsoils which display a **variable geotechnical nature** (soil mechanics), with indeterminate mechanical characteristics. For this reason, these materials have been omitted from the analysis to follow.

4.9.2. Sampling

A total of three (3) **undisturbed soil blocks** were extracted from the **alluvium** blanketing the site; in order to determine the in-situ mechanical properties of this horizon. The samples were extracted in areas and at a depth deemed suitable for the analysis of the geotechnical conditions to assist with foundation design.

It should be noted that the extraction of block samples changes the samples natural state (unloading of in-situ stresses); and as such, the test is only an indication of the in-situ material properties.

Specialist **undisturbed sample testing** was conducted by Steyn-Wilson Geotechnical. The following specialist tests were conducted:

I. Shear Box

II. Single and Double Oedometer

III. Free Swell

IV. Bulk Density

V. Moisture Content

The calculated values presented in the sections to follow are based on estimated foundation configurations. Once figurations have been decided upon the calculations can be modified.

Detailed soil test results for the undisturbed sample are included in **Appendix C.**

The table below summarises the results from the undisturbed testing.

Table 5: Undisturbed Samples- Test Results

Summarised Undisturbed Sample Results												
Sample Details						Free Swell Results		Calculated Settlement (mm) Consolidation Test Results			Shear Box Test Results	
Sample Number	Test Pit	Depth (m)	Sampled Material	Average Dry Density (kN/m^3)	Particle Density (Mg/m^3)	Swell Pressure (kPa)	Swell Percentage (%)	25 kPa	50 kPa	100 kPa	Cohesion (kPa)	Internal Angle of Friction (degrees)
SWG00278-1	TP2	0,8 to 1,0	Alluvium	15,37	2,65	-	-	-	-	-	7,21	21,9
SWG00278-2	TP3	0,4 to 0,6	Alluvium	16,40	2,62	19	0,9	5-8	10-14	17-24	-	-
SWG00278-3	TP2	0,8 to 1,0	Alluvium	15,35	2,65	-	-	1-2	2-6	5-14		
Notes:	1. Results presented as received from Steyn-Wilson Geotechnical 2. A 1.5 Factor of Safety has been applied to all of the settlement ranges presented above, so as to account for sample bias and any disturbances induced during sampling and testing. 3. A strip footing with a width of 0.6 m was used during settlement calculations. 4. The settlement range presented above assumes a depth of influence factor of 1.5 times the presented foundation width.											

4.9.3. Inferred Material Properties Unified Soil Classification System

As part of this assessment a number of engineering material properties can be **inferred**- to be confirmed/measured in follow-up studies where required. Inferred figures are extracted from the data base available on geotechdata.info.

The Unified Soil Classification System (USCS) is a soil classification system used in engineering and geology to describe the texture and grain size of a soil. The classification system can be applied to most unconsolidated materials and is represented by a two-letter symbol. The demarcated result can be used to infer a wide range of soil properties and characteristics.

The **soils** sampled across the site classified as follows:

- The **fine clayey soils** sample across the site classified as **CL** and **SM/SC.**
- The **granular materials** sampled across the site classified as **GM/GC.**

The following inferred paraments/attributes can be used in **preliminary designs**:

1. Typical values of **soil friction angle (degrees)**
   a. CL Soils: 18 to 28
   b. SM Soils: 27 to 34
   c. SC Soils: 30 to 40
   d. GM Soils: 30 to 40
   e. GC Soils: 28 to 35
2. Typical values of **soil cohesion (kPa)**
   a. CL Soils: 4 to 86
   b. SM Soil: 20 to 50
   c. SC Soils: 5
   d. GP/GW/GM Soils: 0
   e. GC Soils: 20
3. Typical values of **soil permeability (m/s)**
   a. CL Soils: $5x10^{-10}$ to $5x10^{-8}$
   b. SC Soils: $5.5x10^{-9}$ to $5.5x10^{-6}$
   c. SM Soils: $1x10^{-8}$ to $5x10^{-6}$
   d. GM Soils: $5.0x10^{-8}$ to $5.0x10^{-6}$
   e. GC Soils: $5.0x10^{-9}$ to $5.0x10^{-6}$

Numerous additional inferences can be made based on the available laboratory test results and associated data bank of information. Additional properties can be presented on the request of the project team.

4.9.4. Bearing Capacity and Soil Shear Strength

Bearing capacity is defined as the pressure which would cause **shear failure** of the supporting soil immediately below and adjacent to a foundation. The estimated allowable bearing capacity of a soil material is a function of the angle of internal friction (reflected in the grading), consistency/cohesion (reflected in the soil grading and density) and degree of saturation (moisture content).

The estimated presumed bearing values of the foundation materials are only an empirical guide to the maximum load that can be placed on the soil/weathered rock particular to this site without shear failure, and as such are not an indicator of the possible settlement/collapse that may occur at foundation pressures up to the bearing capacity of the soil.

The allowable bearing pressures imposed on the material is a function of both the soils shear strength (ultimate limit state) and its' settlement characteristics (serviceability limit state).

Taking the additional movements due to soil consolidation/heave into account will imply that foundation improvements will be necessary for structures.

The presumed bearing values above are based on the materials exposed in situ in the test pits and ignore any improvement, which may be obtained by compacting, or treating the site soils.

Shearbox Testing- Alluvium

The **alluvium** exposed in test pit TP2 was seen to exhibit a measured in-situ dry density of approximately **1537 kg/m^3** with a moisture content of **21 %.**

This undisturbed sample was submitted for **shearbox testing**. The results of the shear box test are as follows:

- Measured internal angle of friction: **21.9°**
- Measured cohesion: **7.21 kPa**

Based on the measured parameters- for a **STRIP foundation** with a **width of 0.6 m at a depth of 0.8 m**; the alluvium yielded an **allowable bearing pressure** of approximately **86 kPa**. In these clayey/fine materials- a rapid reduction in shear strength is anticipated upon saturation.

Firm consistencies in these fine-grained materials may be roughly correlated to allowable strength of **12 - 50 kPa** (Look, 2014). Other sources list presumptive bearing capacity of sandy soils (SC/SM) to 95 – 150 kPa (Alemdag et al, 2017; BS8004-1986; Builders Engineer 27 October 2012).

This may not be adequate for the typical pressure from a light masonry structure and also disregards the effect of soil moisture changes that may induce **settlement** or **heave**.

Similar poor strength characteristics can be expected in the fine-grained residual mudrock. These problem soils were seen to extend to depths of between 0.45 and 1.00 m- average depth of 0.66 m.

Furthermore, the sampled fine-grained materials typically displayed a POOR reaction to compaction, classifying as a worse the G9- type material according to the COLTO classification system.

The samples displayed a calculated **remoulded bearing capacity** of between approximately 10 and 25 kPa @ 93 % MOD AASHTO with a Factor of Safety of 1.5.

Residual Sandstone and Weathered Mudstone/Sandstone Bedrock

The consistency increases to **medium dense** in the **residual sandstone**- becoming dense in the weathered rock mass at depth. Medium dense consistencies may be roughly correlated to allowable bearing pressure of **50 - 150 kPa**, dense with allowable 150 - 300 kPa and very dense >350 kPa.

Furthermore, the sampled weathered sandstone bedrock typically displayed a POOR reaction to compaction, classifying as a worse the G9- type material according to the COLTO classification system. The sample displayed a calculated **remoulded bearing capacity** of approximately 25 kPa @ 93 % MOD AASHTO with a Factor of Safety of 1.5. **Please note laboratory testing is undertaken on disturbed samples in saturated conditions.**

The applied load from a single storey masonry structure may be assumed to be between 30 kPa and 50 kPa, which will fall well within the bearing capacity limit of the deeper residual soil horizons, as well as the intact bedrock layers at depth.

The mechanical properties of a rock mass are dependant not only on the properties of the intact rock (Uniaxial Compressive Strength etc.) but also on the properties of the discontinuities (spacing, orientation, continuity, etc.). The various intact rock and discontinuity properties will each have their own impact on the overall geotechnical properties of the rock mass, such as the permeability and strength.

Based on the available theoretical data (see section 4.10); the fresh rock mass underlying the site **displays favourable strength characteristics** with regards to the construction of both small and large structures.

4.9.5. DCP Testing- Bearing Capacity (In-situ)

Introduction

A total of SIX (6) Dynamic Cone Penetrometer (DCP) tests were conducted across the site. DCP tests were performed across the site in order to give an indication of the consistency of the in-situ material.

The test results can be correlated with the **California Bearing Ratio (CBR)** by applying a formula. These CBR values can then be converted to an estimated **soil bearing capacity** (inferred).

DCP tests were performed at natural moisture conditions and thus the values vary greatly from the CBR results measured under controlled conditions within the laboratory. This variation is mainly due to the lab specimens being analysed under saturated conditions. It should be noted that the strength of the in-situ soil changes greatly with change in moisture conditions. DCP test should therefore only be used as indicative statistics for the site.

It must be noted that the presence of gravel sized rock fragments, in conjunction with fluctuations of the in-situ moisture content within the exposed soil horizons may have had a detrimental effect on the accuracy of the DCP results.

Refer to Figure 6 to view the DCP testing locations. DCP test results are presented in the table and graph to follow; with full full results included in **Appendix D.**

Results discussion

With reference to the CBR readings with depth (graphic overleaf);

- The CBR values acquired from DCP testing are generally higher than that of values measured in laboratory tests under controlled conditions. It is predicted that the clayey nature and low moisture content of these soils has increased the resistance on the probe.
- Both one- and two- meter DCP probes were used in the study.
- All the DCP tests conducted across the site reach final depths of between **395 and 1384 mm**; with an **average penetration depth** of **839 mm**.
- On average, (yellow dashed line on the graphic overleaf) the soils underlying the site display a gradual **increase in consistency** with an increase in depth, with individual tests exhibiting spikes intermittently.
- The variability portrayed in the graphic is indicative of the sites subsoils with fluctuations in composition and/or moisture content taking place across short distances.
- Due to the initial rapid penetration rates, **poor inferred CBR values** were measured to depths of approximately ~700 mm below E.G.L. (typically <15 with localised spikes in individual tests).
- An inferred in-situ CBR value of between 5 and 10 can be converted to a **soil bearing capacity** of between **approximately 40 and 75 kPa** with a factor of safety of 1.5.
- As presented in the table- average inferred bearing capacities exceeding 90 kPa are inferred at shallow depths. Upon saturation a rapid reduction in shear strength is anticipated (as seen in lab conditions)- especially in the fine-grained alluvium and residual mudrock.
- A **rapid increase in inferred CBR** values was recorded towards the **base** of the various DCP tests conducted across the site- envisaged to be the **rock/ soil interface.**
- Final penetration depths correlate well with the bedrock levels measured in exposed test pits.
- The variability portrayed in the graphic is indicative of the sites subsoils with fluctuations in composition and/or moisture content taking place across short distances.
- The probe itself is sensitive to moisture changes and contamination in the soil profile including attributes such as roots, calcrete, gravels etc.

The graph and table overleaf depict the combined DCP test results for the site.

Graph 1: Combined DCP Test Results- Inferred In-Situ CBR

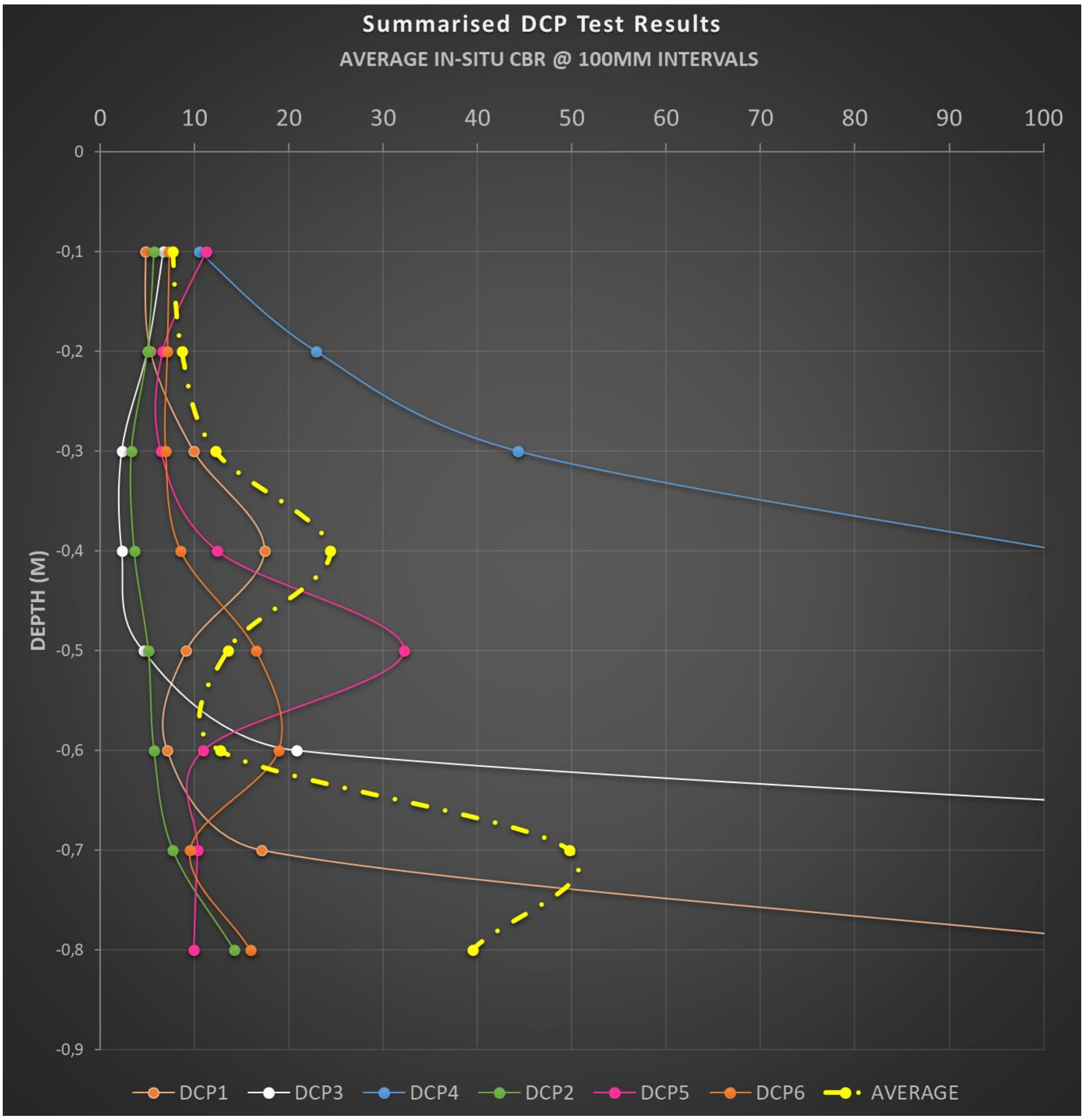


Table 6: Combined Bearing Capacity Values with Depth- Inferred

Summarised DCP Test Results								
Depth (m)	Average In-Situ **CBR** Values							*Inferred Bearing Capacity
	DCP1	DCP2	DCP3	DCP4	DCP5	DCP6	AVERAGE	
0,00	-	-	-	-	-	-	-	-
-0,10	5	6	7	11	11	7	8	60
-0,20	5	5	5	23	7	7	9	67
-0,30	10	3	2	44	6	7	12	90
-0,40	17	4	2	102	12	9	24	166
-0,50	9	5	5	-	32	17	14	99
-0,60	7	6	21	-	11	19	13	93
-0,70	17	8	204	-	10	10	50	311
-0,80	118	14	-	-	10	16	39	254
Termination Type	Refusal	Refusal	Refusal	Refusal	Refusal	Refusal (2m)	-	
Termination Depth (mm)	776	934	617	395	928	1384	839	

* A FOS of 1.5 is applied to inferred bearing capacity calculations

4.9.6. Heave Characteristics

Expansive soils are soils that undergo changes in volume due to changes in moisture content, swelling when the moisture content increases and shrinking when the moisture content decreases. The natural wetting up of the soil profile below the central portions of a structure typically leads to the development of a domed profile under the building in the long term, known as the "central doming" mode of deformation. In the short term, ingress of water into the soil around the perimeter of the structure can lead to heave around the perimeter of the building resulting in the "edge heave" mode of deformation.

According to the **free swell test,** the alluvium exhibits a **percentage swell** of **0.9%,** with a **measured swell pressure of 19 kPa.** The materials exhibited a potentially expansive nature; however, the *generalised heave equation* can only be used for materials which exhibit a swell pressure greater than that of the predicted foundation load.

It is envisaged that due to the high in-situ moisture content of the undisturbed sample (21.8%); the soil itself was not able to absorb additional moisture-inhibiting swell. Furthermore, the moisture content after the consolidation was only partially higher than the pre-test moisture (22.4%). For this reason, it is anticipated that most voids would have been filled with water prior to the swell test and the water would have been leached out as the void ratio decreases with the applied load.

Based on the disturbed sample results from the clayey materials (alluvium and reworked residual mudrock) it was noted that these soils exhibit a low to **MEDIUM** plasticity, moderate linear shrinkage values and an overall LOW potential for heave (**between 0 and 2 % swell-** acc. Van Der Merwe, 1964).

Using the conservative Van der Merwe (1964) method **total heave on surface**, if the materials **wet up from dry**, is calculated to be between **approximately 8 and 16 mm** at a swell percentage of **2%** and an exposed thickness of between 0.45 and 1.00 m- at an **average thickness of 0.66 m** heave of approximately **11 mm** can be expected.

Due to the measured low swell pressure; the degree of movement/ heave will increase with a decrease in the induced overall load of the proposed structures. Heave across the site should become of greater concern for light structures such as sports stands, guard houses, roadways, pathways, surface beds etc.

4.9.7. Settlement Characteristics of the In-Situ Soils

Compressible soils are soils of low stiffness that settle significantly when loaded. In free-draining soils (e.g., sands), this settlement occurs during and shortly after loading. In low permeability soils (e.g., clays), this settlement occurs over a period as the pore pressures set up during loading dissipate.

According to the **single oedometer test**, the consolidation settlement within the **alluvium** across the site will range between **2 and 14 mm** assuming a foundation pressure of **50 kPa** with a foundation width of **0.60 m** (normally consolidated material). Thereafter, at a foundation pressure of **100 kPa**, a settlement range of between **5 and 24 mm** can be expected. The drastic range in presented values is due to the low consolidation measured in the double oedometer test.

A factor of safety (FOS) of 1.5 is applied to the calculated settlement ranges so as to account for sample bias. The settlement measured in the fine-grained soils was seen to **exponentially increase** with an increased foundation load.

Taking into account the material characteristics measured in lab conditions and observed within the soil profiles (voided fine-grained nature); the soft materials underlying the site are deemed to display a **compressible nature** at foundation loads of 50 kPa.

The degree of expected soil consolidation upon loading is expected to decrease with depth due to the medium dense state of the underlying sandstone residuum (western portions) and dense state of the weathered sandstone and/or mudstone bedrock at depth.

4.9.8. Collapse Settlement Characteristics of the In-Situ Soils

Collapsible soils are open-textured (high void ratio) soils that are stiff when dry but lose their stiffness when they become wet. This can lead to sudden, large settlements taking place when the moisture content of the soils below a foundation increase, even many years after construction.

The fine materials (alluvium and residual mudrock) within the prominent foundation zone underwent minor swell upon saturation; these attributes typically **negate** collapse settlement. Furthermore, the difference between the natural moisture content and saturated curves in the double oedometer tests was insignificant at low loads.

The voided and open root channel presence in the upper soil horizon (organic rich topsoil) may in addition to the settlement also cause larger than normal settlements due to collapse under loading and saturation of these voids.

The topsoil immediately below roadways/surface beds will need to be removed or modified. The use of impact rolling, dynamic compaction or over excavation, sorting and re-compaction (in controlled layers) of these soils will result in the **destruction** of their non-favourable in-situ soil properties (i.e., collapsible fabric). The extent of this modification is based on the engineer's design at that point.

4.10. Geotechnical Properties of Mudstone Bedrock

4.10.1. Introduction

As discussed in the preceding sections of this report, the site is predicted to be underlain by shallow interbedded sandstone and mudrock lithologies.

According to the available exposures, the sandstone was notably more resilient to weathering, attributed to its internal hardness and associated rock mass strength. The mudrock was notable more decomposed and based on theoretical data display lower in-situ strength properties. For this reason, the data to follow focuses on this portion of the interbedded rock mass as a type of worst-case scenario for the site.

Ideally the assessment of the underlying rock mass would be undertaken through the implementation of a rotary core drilling regime; with associated rock core logging, photography, sampling and lab testing on the cores (physical and chemical attributes).

The section below summarises the theoretical mechanical and physical properties of the underlying mudrock rock mass.

4.10.2. Typical Physical and Mechanical Rock Properties

The mechanical properties of a rock mass are dependant not only on the properties of the intact rock (Uniaxial Compressive Strength etc.) but also on the properties of the discontinuities (spacing, orientation, continuity, etc.). The various intact rock and discontinuity properties will each have their own impact on the overall geotechnical properties of the rock mass, such as the permeability and strength.

The data to follow presents information for fresh mudstone and not the weathered extent of the rock mass. Based on the available theoretical data (WRC report snippet below); the **fresh** rock mass underlying the site **displays favourable strength characteristics** with regards to the planned construction (warehouse structures). **Report extract:**

ROCK TYPE: Mudstone (3C)

ORIGIN: Karoo Supergroup, Location: Kliprivier, Ladysmith

MACROSCOPIC DESCRIPTION:

Fine grained, massive, dark brown, unweathered mudstone

PETROGRAPHIC DESCRIPTION: The mudstone consists of 20 % of angular quartz fragments, which are < ± 0.08 mm in diameter in a matrix (80 %) of kaolinite, mica, chlorite and/or smectite plagioclase and possibly quartz.

COMMENTS:

The composition of the matrix has been determined by XRD.

Mudstone is usually a variable rock type in its different stages of weathering and its engineering characteristics are therefore not very constant.

MATERIAL CHARACTERISTIC	PROPERTY VALUE			AVERAGE VALUE
	Min	Max	n	
STRENGTH CHARACTERISTICS				
Uniaxial compressive strength (UCS), MPa	71	166	2	119
Point load strength index (PLSI), MPa	2,36	3,7	12	2.79
Triaxial strength parameters, deg; MPa	-	-	14	ϕ = 25, c = 47
Indirect tensile strength, MPa	17,32	20,04	4	18.9
Basic shear strength, deg; kPa	-	-	1	ϕ = 32.7, c = 9.9
Punch shear strength, MPa	32,91	36,89	5	36.3
DEFORMATION CHARACTERISTICS				
E-modulus, GPa	18,1	33,8	2	25.9
Poisson's ratio	0,137	0,2444	2	0.1912
GENERAL CHARACTERISTICS				
Hardness (Schmidt hammer)	34	41	20	38
Abrasiveness, %	-	-	1	11.4
Seismic wave velocity (P-wave),m/s	3423	3920	5	3675
Water absorption, %	2,56	3,92	3	3.19
Porosity, %	0,265	0,327	2	0.29
Density, kg/m^3	2388	2505	14	2473
Swelling (Free swell), %	0,006	0,206	2	0.106
Slake durability (7 cycles), %	-	-	1	99.69

Table 4.13 Characteristics of selected specimens of Mudstone (3C) of the Karoo Supergroup.

Chapter4a-WRC2000-F.doc

4.10.3. Possible Follow-up Actions- Rock Mass Modelling

In mudrock rock masses, due to the moderately high strength and low permeability of the intact rock, the rock mass properties are predominantly a function of the **discontinuity properties.**

As such, the properties of a jointed rock mass are generally best determined through the use of rock mass classification systems such as the Rock Mass Rating by Bieniawski (1989), and the Norwegian Q-Index, presented by Barton et al (1974). These rock mass properties are generally determined from exposures allowing for the classification of the foundation rock.

Based on the on-site observations and available theoretical data; the rock mass underlying the site displays **favourable strength characteristics** to serve as the founding median of a light structure. However, should more detailed information be required- one or more of the following actions can be implemented to model the rock mass as a whole:

1. Joint line surveys and/or window mapping of the available rock exposures- specifically with regards to slope/trench failure.
2. Geophysical surveys.
3. Borehole drilling with associated core logging, sampling and rock core laboratory testing. The rock core samples should be submitted for testing in order to determine the physical and chemical properties of the rock material. The drilling results will shed light on the extent and quality of the dolerite bedrock underlying the site- which will further aid the design of the proposed infrastructure.

5. Development Recommendations

5.1. Introduction

This report describes the results of the detailed Engineering Geological Site Investigation conducted in support of the proposed Smithfield Multi-Purpose Indoor Sport Centre. This multi land use development is planned to span the north eastern extent of ERF 1117 in the Mofulatshepe suburb of Smithfield.

The site encompasses the north eastern extent of ERF 1117 within the suburb. This predominantly undeveloped parcel of land exhibits a roughly rectangular shape, with a combined extent of approximately 2.3 Ha.

The boundaries of the site are defined by numerous existing gravel roads; namely Oettle Street (north west), Butler Street (north east) and an unnamed street to the south east. Access to the site can be obtained by either of these streets. Furthermore, the south western boundary of the site is defined by an existing fence orientated ~NW/SE, bisecting ERF 1117.

The planned development across the surface will encompass the subdivision of this land portion into various land-use zones i.e., infrastructural units, sports fields, roadways, and services etc. Each of these zones may require their own set of geotechnical recommendations determined by the properties of the material.

The sites surface was seen to display a reworked nature attributed to past and ongoing human activities in the area. This reworking was predominantly in the form of relict infrastructure, roadways/pathways, soil berms, scattered heaps of dumped fill and localised levelled sports fields. The combination of the items raised above have resulted in the formation of an anthropogenically induced undulating landscape, with the occurrence of small-scale anomalies.

The presented geotechnical model is based on a data base of available information and available on-site exposures. Parcels of land within the developmental area which are free of excavations are modelled using on-site observations and surrounding exposures.

Based on the results of the investigation, the in-situ soils display a poor shear strength (ultimate limit state) and display a **compressible** and **potentially expansive nature** (serviceability limit state).

Uniform heave, shrinkage, collapse settlements or consolidation settlements generally do not cause damage to structures but might detrimentally affect service (water and sewer) pipe entries at the perimeter of structures. Non-uniform or differential movements can cause structural distress, deformations and overstressing of structural components, resulting in damage to the building.

Due to the natural slopes in relation to the planned large footprints of the structures, variable founding conditions and materials are expected to be encountered. This variability includes soils of different ages, composition and associated pre-consolidation pressures.

Structural solutions shall improve the flexibility and strength of the structure to enable the building to tolerate potential soil movements so that the resulting response to actions is within the limits specified in SANS 10400-B.

5.2. Geotechnical Site Classification

The results of this geotechnical analysis models that the whole site exhibits geotechnical characteristics that may require the implementation of design and/or precautionary measures to reduce the risk of structural damage due to adverse geotechnical characteristics. However, these characteristics do **not** disqualify the site from being used for the development, but rather require the implementation of site-specific precautionary **engineering measures.**

The site is underlain by a variable sequence of materials; with the majority of the material deemed to be **compressible** and **potentially expansive.** The degree of heave and compressibility is predicted to decrease with an increase in depth.

The impact of the geotechnical constraints on development (applicable to single and double storey masonry structures) may be evaluated according to Table 7 in Appendix E), which is a summary of the general geotechnical constraints relevant to urban development (Partridge, Wood and Brink, 1993). The Class column indicates the severity of the specific constraints for this site.

The main expected geotechnical constraints for this site are:

- Expected shallow temporary perched groundwater tables and/or surface ponding during high intensity precipitation events: **2B**
- Low to moderate soil heave potential predicted: **1C/2C** (NHBRC Site Class H1/H2)
- Moderately compressible soil horizons with expected larger than acceptable differential movements: **2D**
- Difficult excavation conditions with between 10 and 40% of the material to a depth of 1.5 mbgl deemed to be Category 1 excavation: **2F**
- Very gentle slopes of less than 2 degrees: **2I**
- Mining induced seismic activity more than 100 cm/s^2 : **2K**
- Localised occurrence of surficial uncontrolled fill materials, relict infrastructure and anthropogenic reworking: **P ^{fill}**
- Materials predicted to be potentially **corrosive** and **slightly dispersive.**
- Poor terrain mobility following heavy/prolonged precipitation events.

The site has been classified as a **SINGLE Site Class Designation zone**, based on the above constraints and the criteria as set out in the National Home Building Manual (2015) guideline document.

Site Classification: H1/H2 and P $^{fill\ localised}$ with 2BCDFIK.

Please note the following regarding these site class designations:

- The site class designation is specific as suggested in the Home Builders Manual (2015), Part 4, 4.2 and derived from an estimation of the expected range of soil volume change in single- and double-storey structures constructed of masonry walls with soil pressures not exceeding 50 kPa.
- The classification and foundation recommendations are based on results from this and proximate investigations.
- The mechanical properties of the sites' subsoils are inferred based on the exposed soil profiles (soil consistency, composition and structure) and associated laboratory test results.

- Site class designations are based on the existing ground level.
- The primary **problem soils** include the **alluvium** and **residual mudrock**- extending from the surface to depths of between **0.45 and 1.00 m** below the E.G.L (average extent- 0.66 m). These problem materials are generally underlain by weathered sandstone/mudrock bedrock.

5.3. Foundation Design Options

The general site conditions with regards the geotechnical considerations are such that any light structure placed on the compressible and potentially expansive materials occurring on site will need special precautionary measures to prevent serious damage to the structure. Additional foundation modifications to prevent damage to structures due to differential settlements may be necessary.

Due to the variable founding conditions across the site, it is recommended that the structure be adequately jointed and/or strengthened to allow for the predicted differential settlement. Please consult a qualified/competent engineer for additional options and final designs.

It is recommended that the structural engineers calculate the best economical foundation option for the proposed development based on the type of structure and the different available construction methods/materials to remedy the negative effects of the measured geotechnical constraints.

GeoCalibre is open to ongoing discussions with consulting civil engineers surrounding suitable foundation configurations for the planned structures, taking into account the variable geotechnical nature of the site.

Considering the mechanical properties of the in-situ soils underlying the site, in conjunction with the nature of the development, there are **three main options** for the mitigation of the deleterious effects of the compressible/expansive soils:

- The **first option** entails shallow foundations with the **reinforcement** of the foundations to the point at which they can withstand the expected total and differential movements (**reinforced concrete foundations**). Due to the predicted extent of soil movement- a **rigid/stiffened/cellular concrete raft** will be required if this option is selected.
- The **second** option entails **deep foundations**- either strip footing, piles or deep pads. Founding on **weathered bedrock** below problematic materials. Refusal to TLB. Foundation not to span from rock to soil or engineered fill. Normal construction with lightly reinforced strip footings and light reinforcement in masonry. Removal of the problem soils below surface beds and drainage requirements (surface and sub-surface).
- The **third option** entails the destruction of unfavourable soil characteristics underlying the structure, through **over-excavation, replacement and compaction** of the in-situ material directly below the footings and surface beds in controlled layers; and in so doing creating a uniform engineered **fill/earth mattress.** The engineered fill should be constructed/designed in such a way as to **dissipate** the load of the structure- ensuring that excessive loads are not transferred into the underlying compressible/expansive natural soils.

Discussions surrounding suitable founding depths and methods can be discussed at length with the project team once final structural configurations are known.

5.4. Design Considerations and Summarised Site Constraints

The following geotechnical constraints should be considered during the final design of the development:

- The site will need to be extensively rehabilitated to create a stable and level working platform. This includes the removal of the vegetation, fill materials, relict infrastructure and organic materials form within the footprint of the planned development.
- The natural vegetation has been denuded across the site due to on-going human activity; with the sites surface being exposed to the elements. The degree of organic material and biotic activity was seen to decrease with an increase in depth, with minor root systems reaching to a depth of approximately 0.18 m below the existing ground level.
- The very gentle sloping nature of the site will aid surface water infiltration into the underlying soils, rather than rapid surface water flow, accentuating surface water ponding and fluctuating moisture conditions after heavy/prolonged rain.
- Surface water ponding is exacerbated through the anthropogenically reworked nature of the sites surface coupled with the clayey nature of the subsoils (low permeability).
- The anthropogenic reworking of the sites surface will result in local variations of surface water flow- both rate and direction.
- The continuity and manipulation of the topography and associated drainage plays a pivotal role in the longevity and sustainability of the development as a whole.
- Emphasis should be placed on surface drainage and storm water control measures to avoid both surface water ponding and concentrated water flow (erosion) across the development area. Structures constructed perpendicular to the natural slopes will result in the ponding of surface water.
- Attention must be given to site contouring to ensure an effective gradient is achieved so that standing water does not occur, and the draining of water is efficient to minimise erosion and damage to the construction.
- The results of the trenching phase indicate that most of the site is blanketed by a fine-grained transported sediment deemed to be of an alluvial origin. This topsoil was reworked and/or contaminated to a degree through the past human activities undertaken across the site.
    - The alluvium was typically present in two distinct horizons; namely, an organic rich topsoil followed by a clayey/silty alluvium with depth. Moreover, both the topsoil and underlying alluvium displayed a low in-situ density, fine-grained and structured nature; attributes typically associated with highly compressible and potentially active soils. Furthermore, these soils are predicted to have a low in-situ permeability.
    - The topsoil was seen to extend from the surface to depths of between 0.15 and 0.30 m- displaying an average exposed thickness of 0.18 m.
    - The clayey/silty alluvium was seen to extend from below the topsoil to depths of between 0.45 and 1.00 m- displaying an average thickness of 0.53 m.

- Across the central to north western portions, the clayey transported materials were typically underlain by granular materials deemed to be residual sandstone- with these excavations transitioning into weathered sandstone bedrock at depth. The quality and continuity of the rock mass was seen to increase with an increase in depth.
- Furthermore, across the low-lying south eastern portions, the alluvium was typically underlain by a fine-grained residual mudrock, transitioning into weathered mudrock bedrock at depth.
- Within test pit TP5- a thin bed of completely weathered sandstone bedrock was encountered between the blanketing topsoil and the underlaying decomposed mudrock. This thin bed proves the predicted interbedded nature of the on-site geological units and further proves the thinning out of the sandstone bed with a decrease in elevation.
- The primary problem soils include the alluvium and residual mudrock- extending from the surface to depths of between 0.45 and 1.00 m below the E.G.L (average extent- 0.66 m). These problem materials are generally underlain by weathered sandstone/mudrock bedrock.
- Surficial uncontrolled fill material was present in scattered portions of the site.
    - These successions of fill material were dumped across the site in an uncontrolled manner during past anthropogenic activities- with the material not placed in controlled layers.
    - The mechanics of these deposits are predicted to be highly variable as a result of the both the variable nature of the particles (rock or waste) and the variable interaction between individual particles.
    - Based on the available information, uncontrolled fills are limited to surficial occurrences- excluding notable thick deposits at depth. Surficial heaps of fill ranged in shape and size sporadically.
    - It is strongly recommended that this material be selectively mined and removed from within the footprint of the proposed development (to an extent deemed suitable by the competent design engineer).
- The clayey nature of the uppermost soil horizons will result in prolonged saturation of the sites subsoils, with poor infiltration characteristics. The predominant runoff will occur as sheet wash following the topography.
- Ground water is predicted to flow/perch along the rock-soil interface, with its flow dynamics dependant on the topography of the rock mass at that point.
    - Bulk excavations down to hard rock may need to be dewatered during construction.
    - The implementation of a sub-surface drainage system is only deemed necessary should foundation configurations result in the damming up of ground water flow (i.e., implementation of deep strip foundations and/or retaining structures).
- Significant changes in moisture content may contribute to the anticipated consolidation settlement and/or expansive behaviour of the site soils. During construction and after development, shallow perched water systems may develop yet further due to stormwater management practices, localised infiltration, and site modification practices.

- Adequate damp-proofing measures should be implemented beneath individual structures.
- The average excavation depth across the site was approximately 1.25 m.
    - Profiles were described in trenches excavated by means of TLB- type light mechanical excavator. End of hole conditions were typically due to refusal or gradual slow advance in weathered bedrock at depth.
    - The excavation type to an average depth of 1.25 m below the existing ground level is deemed to be SOFT Excavation (SANS 10400G/SANS1200D).
    - Category 1 hard excavation estimated to be between approximately 10 and 40% of material to 1.5 meters below ground level (mbgl).
    - Excavations within the weathered rock mass were notably challenging; with exposed material displaying a very dense nature at depth. Its' inherent excavatability predicted to fluctuate over short distances as a result of varying degrees of weathering and spacing of discontinuities. The rock mass at depth will require hard rock excavation methods to unearth.
- It should be noted that the allowable bearing pressures imposed on the material is a function of both the soils shear strength (ultimate limit state) and its' settlement characteristics (serviceability limit state). Taking the additional movements due to soil consolidation/heave into account will imply that foundation improvements will be necessary for structures.
- Based on the measured parameters- for a STRIP foundation with a width of 0.6 m at a depth of 0.8 m; the alluvium yielded an allowable bearing pressure of approximately 86 kPa. In these clayey/fine materials- a rapid reduction in shear strength is anticipated upon saturation.
    - This may not be adequate for the typical pressure from a light masonry structure and also disregards the effect of soil moisture changes that may induce settlement or heave.
    - Similar poor strength characteristics can be expected in the fine-grained residual mudrock. These problem soils were seen to extend to depths of between 0.45 and 1.00 m- average depth of 0.66 m.
    - The sampled fine-grained materials typically displayed a POOR reaction to compaction, classifying as a worse the G9- type material according to the COLTO classification system.
    - The samples displayed a calculated remoulded bearing capacity of between approximately 10 and 25 kPa @ 93 % MOD AASHTO with a Factor of Safety of 1.5.
- The consistency increases to medium dense in the residual sandstone- becoming dense in the weathered rock mass at depth. Medium dense consistencies may be roughly correlated to allowable bearing pressure of 50 - 150 kPa, dense with allowable 150 - 300 kPa and very dense >350 kPa.
- Based on the available theoretical data; the fresh rock mass underlying the site displays favourable strength characteristics with regards to the construction of both small and large structures.
- The CBR values acquired from DCP testing are generally higher than that of values measured in laboratory tests under controlled conditions.

- According to the free swell test, the alluvium exhibits a percentage swell of 0.9%, with a measured swell pressure of 19 kPa.
    - The materials exhibited a potentially expansive nature; however, the generalised heave equation can only be used for materials which exhibit a swell pressure greater than that of the predicted foundation load.
    - Using the conservative Van der Merwe (1964) method total heave on surface, if the materials wet up from dry, is calculated to be between approximately 8 and 16 mm at a swell percentage of 2% and an exposed thickness of between 0.45 and 1.00 m- at an average thickness of 0.66 m heave of approximately 11 mm can be expected.
- According to the single oedometer test, the consolidation settlement within the alluvium across the site will range between 2 and 14 mm assuming a foundation pressure of 50 kPa with a foundation width of 0.60 m (normally consolidated material). Thereafter, at a foundation pressure of 100 kPa, a settlement range of between 5 and 24 mm can be expected. The degree of expected soil consolidation upon loading is expected to decrease with depth due to the medium dense state of the underlying residuum.
- Special attention must be given to the selection of the correct material to be used for the bedding, fill material and the general backfill in the construction of pavement layers as well as foundations. On-site Materials:
    - As a whole, the soft materials sampled across the site are not deemed suitable for the re-use in the proposed construction.
    - These materials can be over-excavated and spoiled.
    - Material will need to be imported for the use in layer works.
- It is recommended that foundations be place on a uniform founding medium to limit the degree of differential settlement.
- Areas covered by roadways/paving should be over excavated and the soil replaced and compacted to remove the voided structure.
- Areas subjected to extensive fills need to be adequately modified/ compacted to limit soil movement over time (i.e., fill creep).
- Plumbing and service precautions will be necessary to prevent pipe rupture or joint leakages due to soil volume changes. It may be necessary to make use of flexible joints in pipes to minimise damage due to the movement of active soils.
- Good site drainage measures, on surface and subsurface, must be implemented to prevent moisture changes, which may add to the settlement and the development of perched groundwater tables. Drainage precautions are required to minimize differential movements, soil collapse and erosion.
- It is recommended that all earthworks be carried out in accordance with SABS 1200/SANS 10400 (current version). The imported material should be placed in layers not exceeding 200 mm in thickness and compacted to a minimum of 93% Modified AASHTO maximum dry density.
- Test pits were not compacted in layers when backfilled and differential settlements may occur across these features.
- Quality control testing should be undertaken by an accredited laboratory where possible.

6. Report Provisions

While every effort is made during the fieldwork phase to identify the different soil horizons, areas subject to a perched water table, areas of poor drainage, areas underlain by hard rock and to estimate their distribution, it is impossible to guarantee that isolated zones of poorer foundation materials, or harder rock have not been missed.

The presented geotechnical model is based on a data base of available information and available on-site exposures. Parcels of land within the developmental area which are free of excavations are modelled using on-site observations and surrounding exposures.

The design and implementation of the planned thick fills remains the responsibility of the consulting engineers- with GeoCalibre and its' employees carrying no liability in this regard. Adequate design and associated quality control measures should be implemented to ensure the longevity of the development as a whole.

The determination of flood lines and delineation of wetland areas were not part of this investigation scope and should be addressed by suitably competent professionals prior to the final site development plan is compiled, if deemed necessary. If the site or a portion thereof is situated within the 1:100-year flood line, or has been delineated as a wetland, it is the prerogative of the Civil Engineer or other suitably experienced specialist to overwrite the recommendations for such portions.

A competent person should inspect excavations for future structures at the time of construction or the open service trenches, to determine the variance from the above assessment of the site. The assumptions in this report are based on general knowledge of the soils in the area.

Although not anticipated at this site, it should be noted that this investigation did not include the assessment of any potential environmental hazards, or groundwater impacts that may be present, or ensue from the construction of the proposed structures.

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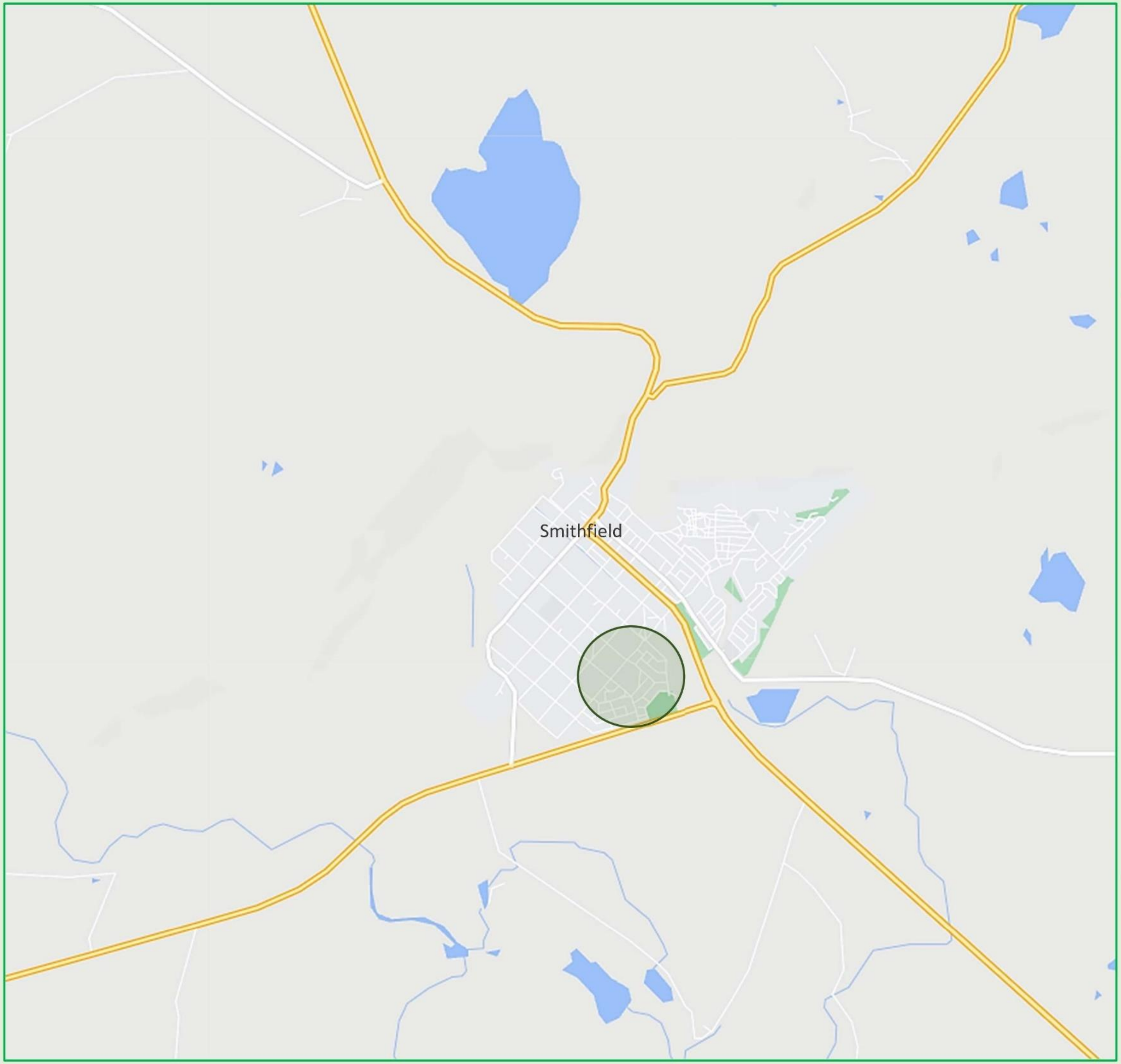
LAYOUT MAPS



GEOCALIBRE
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Figure 1

Study Area Location

LEGEND

Study Area

N

GeoCalibre Geotechnical Consultancy (PTY) LTD

Kevin Coertzen
Pri.Sci.Nat and MSAIEG

33 Denne Avenue
Bainsvlei
Bloemfontein

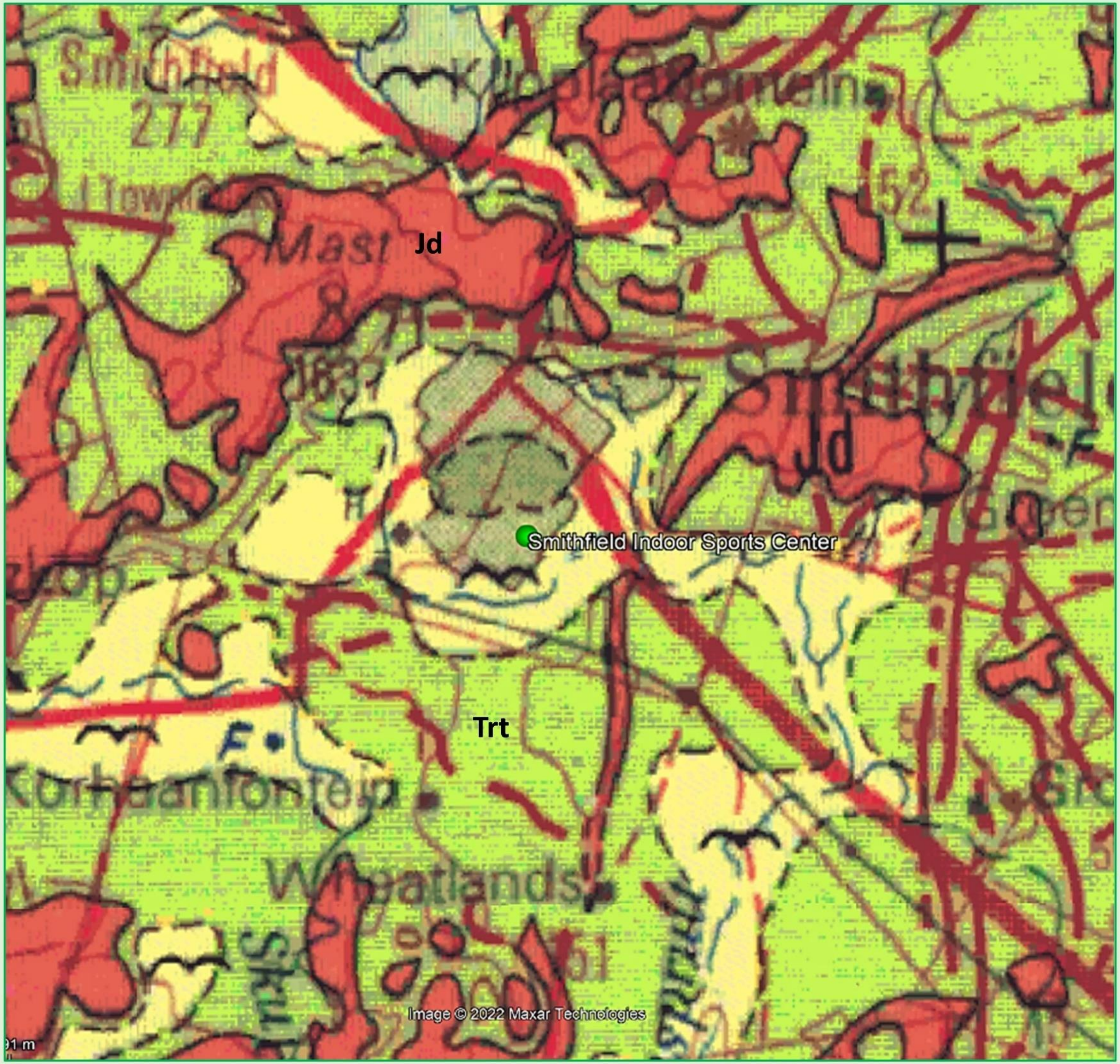

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Figure 2

Regional Geology Map

LEGEND

	Alluvium
Jd	Dolerite
Trt	Sandstone/Mudstone- Tarkastad Formation- Beaufort Group- Karoo Supergroup

Geological Series Map:
3026 Aliwal North ; Scale 1 : 250 000

N

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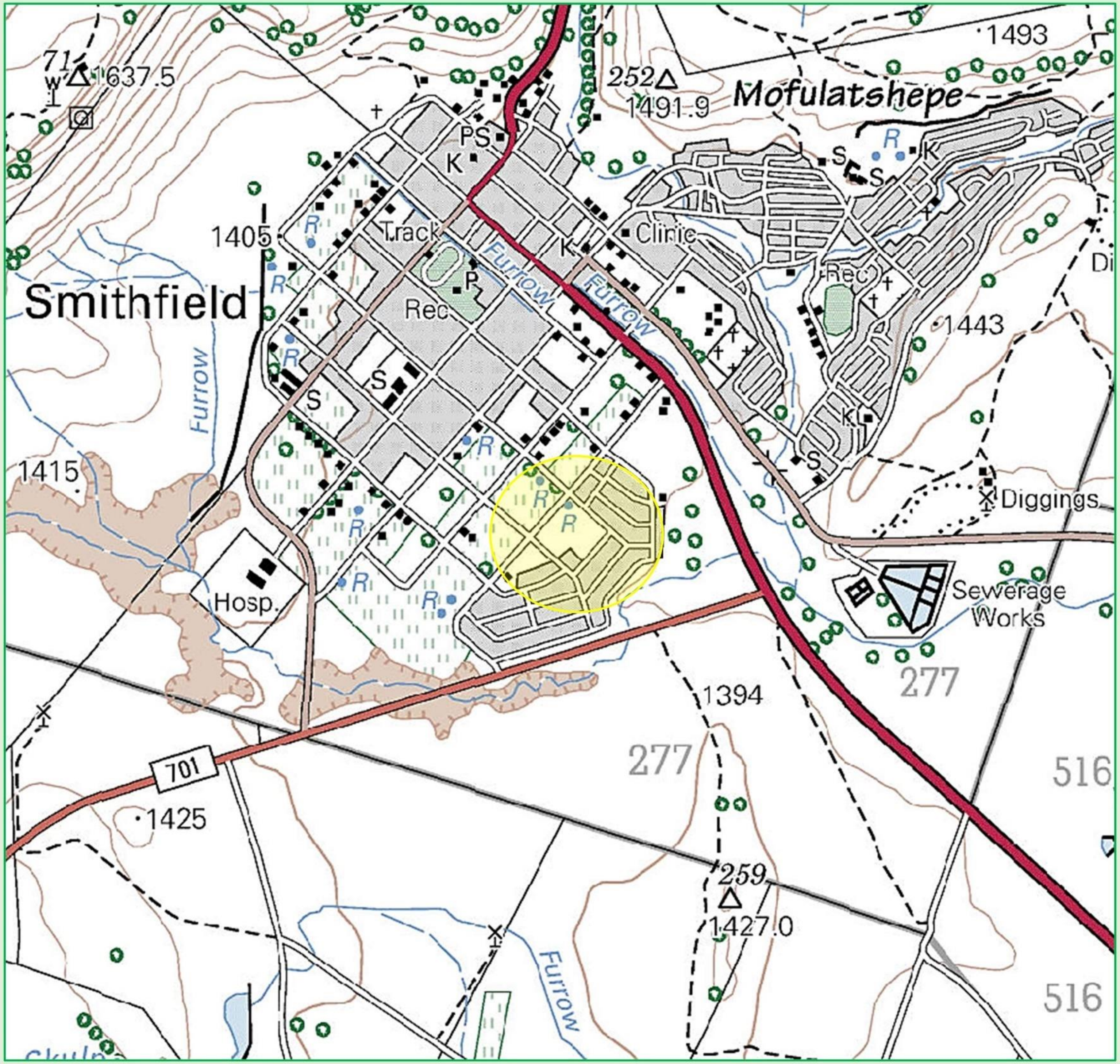

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Figure 3

Topocadastral Map

LEGEND

Study Area

Topocadastral Maps:
3026 Ba; Scale 1 : 25 000

N

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Figure 4

Site Location- Aerial View

LEGEND

Site Boundary

Fenced Areas

Annotated Stream Channel- 3026 Ba

N

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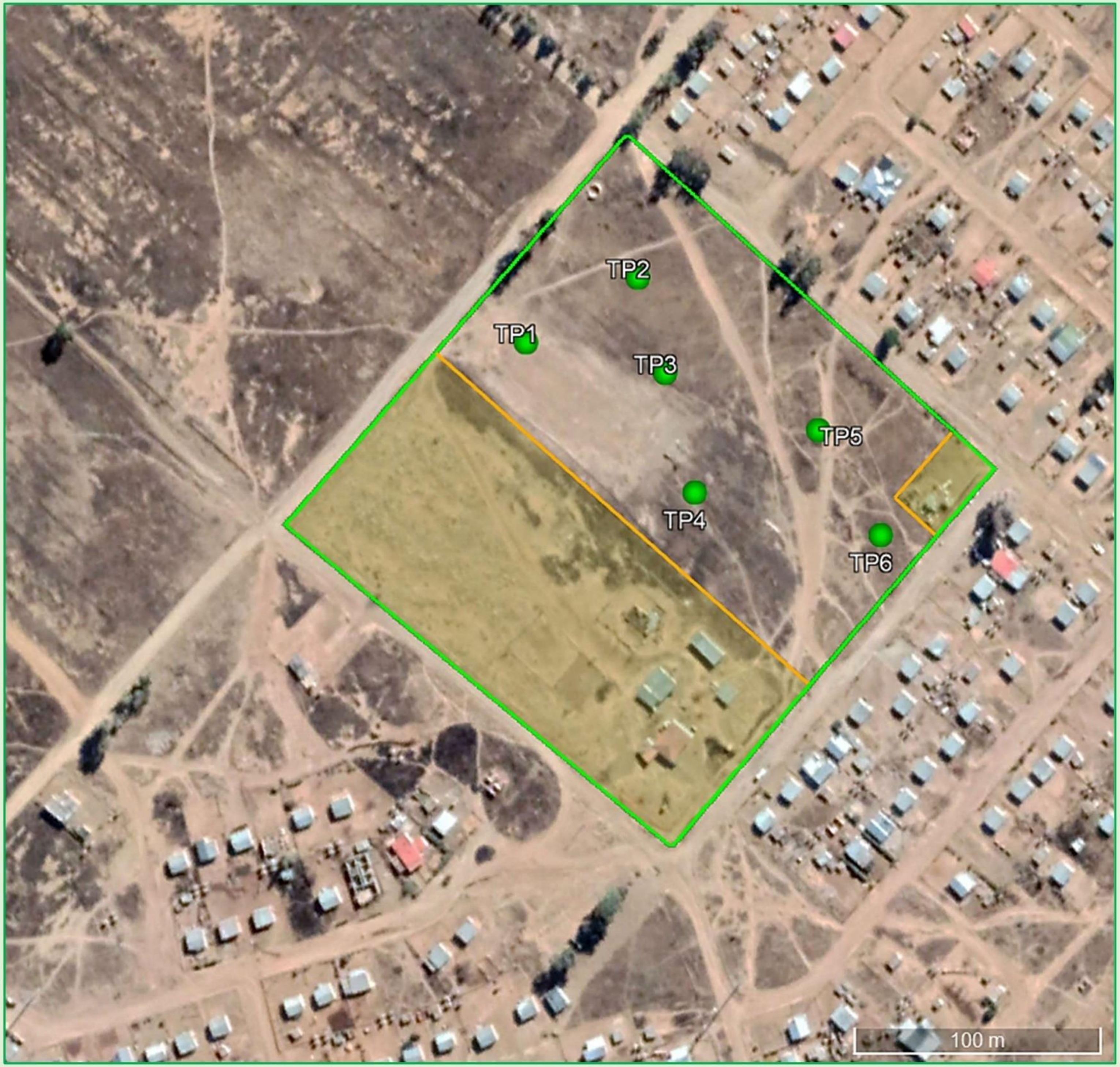

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Figure 5

Testing Locations

LEGEND

Test Pit

Site Boundary

Fenced Areas

N

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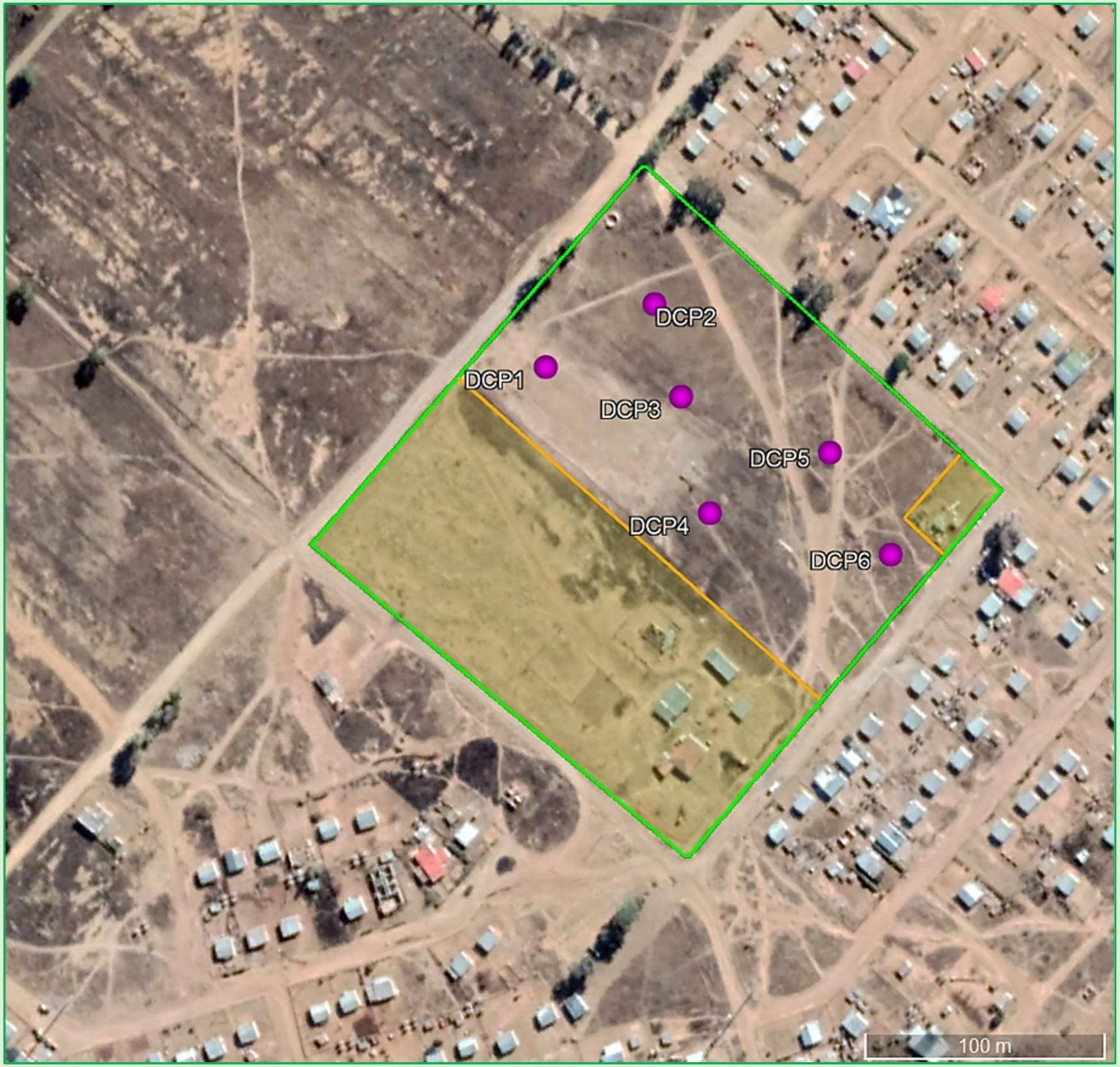

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Figure 6

DCP Testing Locations

LEGEND

DCP Test

Site Boundary

Fenced Areas

N

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Testing Locations

GC-22-104
Smithfield Indoor Sports Centre

BVi Consulting Engineers- Bloemfontein

Test Pit Locations

Test Pit Number	Location		Elevation (meters above mean sea level)	Final Excavation Depth with TLB (m)
	Latitude (South)	Longitude (East)		
TP1	30.22198	26.53383	1397	1,10
TP2	30.22176	26.53430	1396	1,30
TP3	30.22210	26.53442	1395	1,20
TP4	30.22253	26.53454	1394	0,80
TP5	30.22230	26.53506	1393	1,30
TP6	30.22268	26.53532	1392	1,60

DCP Locations

DCP Number	Location		Elevation (meters above mean sea level)	Final Penetration Depth (mm)
	Latitude (South)	Longitude (East)		
DCP1	30.22198	26.53383	1397	776
DCP2	30.22176	26.53430	1396	934
DCP3	30.22210	26.53442	1395	617
DCP4	30.22253	26.53454	1394	395
DCP5	30.22230	26.53506	1393	928
DCP6	30.22268	26.53532	1392	1384

Please note that all GPS co-ordinates are extracted from Garmin Oregon 600 tm and elevation data from Google Earth PRO

Appendix A

Soil Profiles

GEOCALIBRE GEOTECHNICAL CONSULTANCY

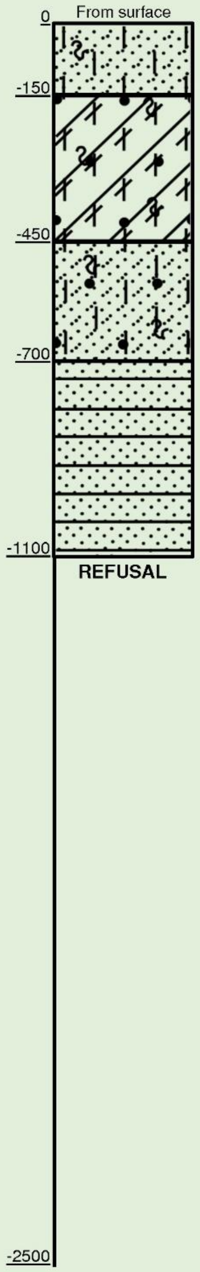
Test Pit TP1

GC-22-104

Smithfield Indoor Sports Center

BVi Consulting Engineers- Bloemfontein

Soil Profile for Test Pit TP1



slightly MOIST; dark and light BROWN; LOOSE; PINHOLED; silty SAND with traces of fine gravel; ALLUVIUM; organic rich topsoil; minor anthropogenic contamination; abundant fine roots; Not Sampled.

MOIST; dark BROWN, mottled black; FIRM; PINHOLED and FISSURED; sandy CLAY with traces of sandstone gravels; ALLUVIUM; traces of fine roots; strongly structured; traces of ferricrete nodules; Disturbed Sample No. 1.

slightly MOIST; light BROWN, blotched white, stained red; DENSE; INTACT; silty SAND with minor angular sandstone gravels; RESIDUAL sandstone; traces of fine roots; traces of ferricrete nodules and ferruginisation on gravels; Not Sampled.

slightly MOIST; light olive CREAM, stained red and brown, banded white; DENSE; FINE grained; thinly bedded; HIGHLY weathered; MEDIUM HARD ROCK; SANDSTONE of the Tarkastad Formation; minor calcification along bedding planes; open joints with clay infilling; Not Sampled.

Profile Notes		Excavation Description	
Profiled by:	KS Coertzen (Pri.Sci.Nat)	**Contractor:**	Hire Cor
Groundwater Seepage:	N/A	**Machine:**	JCB 3CX- TLB Type Light Mechanical Excavator
Excavation Stability:	Sidewalls stable	**Excavation Character:**	Refusal on weathered sandstone bedrock
Samples Extracted:	1 Disturbed Sample	**Date Profiled:**	25/01/2022
Elevation (MAMSL):	1397	**Coordinates:**	30.22198°S 26.53383°E

GC-22-104

Smithfield Indoor Sports Centre

Profile Photo: TP1

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GC-22-104

Smithfield Indoor Sports Centre

Material Present in Test Pit: TP1

Surroundings of Test Pit: TP1

Facing West

Facing South

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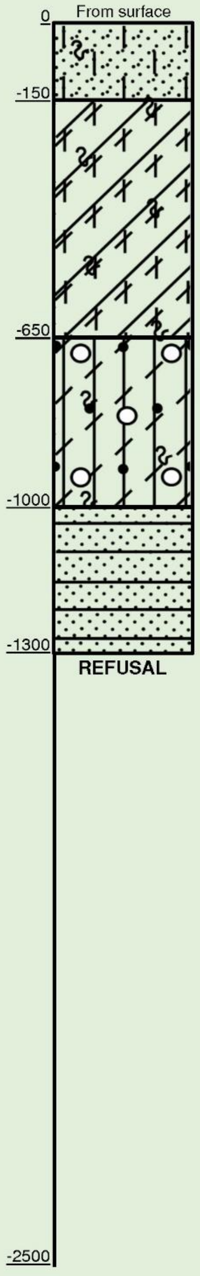
Test Pit TP2

GC-22-104

Smithfield Indoor Sports Centre

BVi Consulting Engineers- Bloemfontein

Soil Profile for Test Pit TP2



slightly MOIST; dark BROWN; medium DENSE with loose pockets; PINHOLED; silty SAND with traces of fine gravel; ALLUVIUM; organic rich topsoil followed by minor fine roots; minor anthropogenic contamination; Not Sampled.

MOIST; dark reddish BROWN, mottled white; FIRM; PINHOLED; sandy CLAY with traces of fine sandstone gravel; ALLUVIUM; traces of fine roots; Bulk Sample No. 2.

Gradual contact

MOIST; dark yellowish orangey BROWN, mottled white and black; FIRM; SLICKENSIDED; clayey SILT with traces of fine sandstone gravel; ALLUVIUM; traces of fine roots; traces of ferricrete nodules; Bulk Sample No. 2; Undisturbed Sample No. 1.

slightly MOIST; light olive CREAM, banded white; DENSE; FINE grained; thinly bedded; HIGHLY weathered; MEDIUM HARD ROCK; SANDSTONE of the Tarkastad Formation; minor ferricrete nodules along bedding planes; rapid transition into less weathered sandstone bedrock; Not Sampled.

Profile Notes		Excavation Description	
Profiled by:	KS Coertzen (Pri.Sci.Nat)	Contractor:	Hire Cor
Groundwater Seepage:	N/A	Machine:	JCB 3CX- TLB Type Light Mechanical Excavator
Excavation Stability:	Sidewalls stable	Excavation Character:	Refusal on weathered sandstone bedrock
Samples Extracted:	1 Bulk Sample, 1 Undisturbed Sample	Date Profiled:	25/01/2022
Elevation (MAMSL):	1396	Coordinates:	30.22176°S 26.53430°E

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GC-22-104
Smithfield Indoor Sports Centre

Profile Photo: TP2

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GC-22-104

Smithfield Indoor Sports Centre

Material Present in Test Pit: TP2

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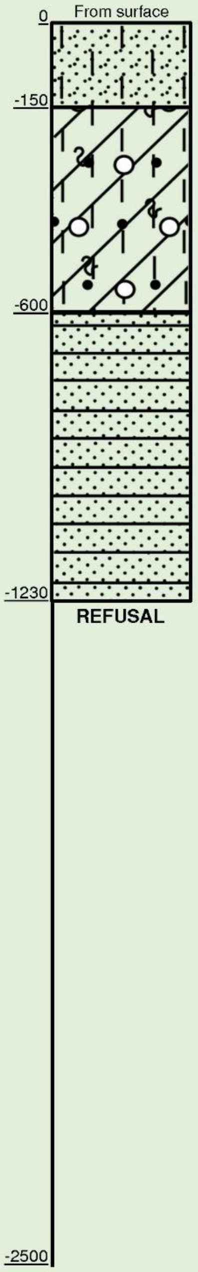
Test Pit TP3

GC-22-104

Smithfield Indoor Sports Centre

BVi Consulting Engineers- Bloemfontein

Soil Profile for Test Pit TP3



slightly MOIST; light BROWN; LOOSE; PINHOLED; silty SAND with traces of fine gravel; ALLUVIUM; organic rich topsoil; minor anthropogenic contamination; Not Sampled.

Gradual contact

slightly moist to MOIST at the base; dark BROWN, blotched white, mottled black; FIRM; SLICKENSIDED; sandy CLAY with minor sandstone gravels; ALLUVIUM; traces of fine roots; traces of ferricrete nodules; Disturbed Sample No. 3; Undisturbed Sample No. 2.

slightly MOIST; light olive CREAM, stained red, banded grey; DENSE to very dense at the base; FINE grained; thinly bedded; HIGHLY weathered; MEDIUM HARD ROCK; SANDSTONE of the Tarkastad Formation; minor calcification along bedding planes and joints; minor decomposed organic materials; Not Sampled.

Profile Notes		Excavation Description	
Profiled by:	KS Coertzen (Pri.Sci.Nat)	**Contractor:**	Hire Cor
Groundwater Seepage:	N/A	**Machine:**	JCB 3CX- TLB Type Light Mechanical Excavator
Excavation Stability:	Sidewalls stable	**Excavation Character:**	Refusal on weathered sandstone bedrock
Samples Extracted:	1 Disturbed Sample, 1 Undisturbed Sample	**Date Profiled:**	25/01/2022
Elevation (MAMSL):	1395	**Coordinates:**	30.22210°S 26.53442°E

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GC-22-104

Smithfield Indoor Sports Centre

Profile Photo: TP3

Material Present in Test Pit: TP3

Surroundings of Test Pit: TP3

Facing North

Facing South

GEOCALIBRE
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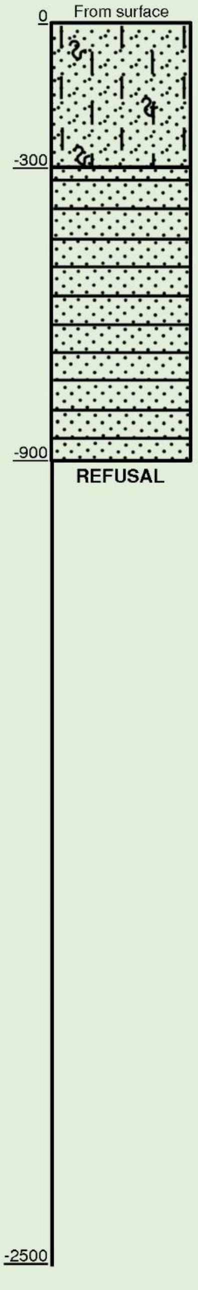
Test Pit TP4

GC-22-104

Smithfield Indoor Sports Centre

BVi Consulting Engineers- Bloemfontein

Soil Profile for Test Pit TP4



slightly MOIST; dark BROWN, blotched cream; LOOSE; PINHOLED; clayey SAND with minor sub-angular sandstone gravel and cobbles; ALLUVIUM; organic rich topsoil followed by minor fine roots; minor anthropogenic contamination; Not Sampled.

~Undulating contact

slightly MOIST; dark greyish OLIVE, stained white and black, banded brown; DENSE to very dense at the base; FINE grained; closely jointed; HIGHLY weathered; MEDIUM HARD ROCK; SANDSTONE of the Tarkastad Formation; traces of decomposed roots; rapid transition into sandstone less decomposed bedrock; ferruginisation and calcification along open joints; Bulk Sample No. 4.

Profile Notes		Excavation Description	
Profiled by:	KS Coertzen (Pri.Sci.Nat)	**Contractor:**	Hire Cor
Groundwater Seepage:	N/A	**Machine:**	JCB 3CX- TLB Type Light Mechanical Excavator
Excavation Stability:	Sidewalls stable	**Excavation Character:**	Refusal on weathered sandstone bedrock
Samples Extracted:	1 Bulk Sample	**Date Profiled:**	25/01/2022
Elevation (MAMSL):	1394	**Coordinates:**	30.22253°S 26.53454°E

GEOCALIBRE
GEOTECHNICAL CONSULTANCY

GC-22-104
Smithfield Indoor Sports Centre

Profile Photo: TP4

GEOCALIBRE
GEOTECHNICAL CONSULTANCY

GC-22-104

Smithfield Indoor Sports Centre

Material Present in Test Pit: TP4

Surroundings of Test Pit: TP4

Facing North

Facing South West

GEOCALIBRE
GEOTECHNICAL CONSULTANCY

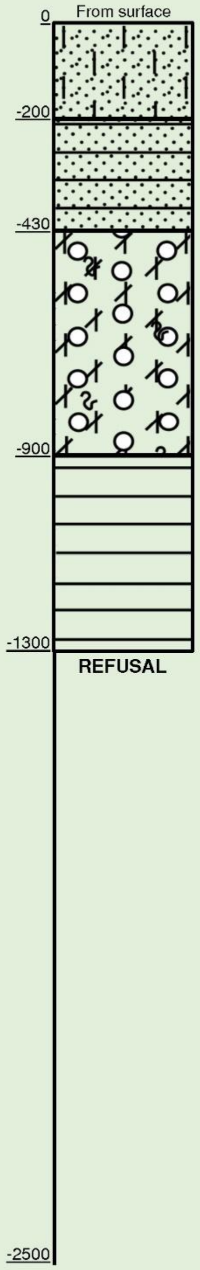
Test Pit TP5

GC-22-104

Smithfield Indoor Sports Centre

BVi Consulting Engineers- Bloemfontein

Soil Profile for Test Pit TP5



slightly MOIST; dark BROWN; stained cream; LOOSE; PINHOLED; clayey SAND with minor sub-angular sandstone gravel and cobbles; ALLUVIUM; organic rich topsoil; Not Sampled.

~Undulating contact

slightly MOIST; light greyish OLIVE, blotched white, stained brown and red; medium DENSE; FINE grained; closely jointed and thinly bedded; COMPLETELY weathered; SOFT ROCK; SANDSTONE of the Tarkastad Formation; thin band of sandstone bedrock interbedded with the mudstone; highly variable exposure of the decomposed protolith; Not Sampled.

MOIST; dark olive GREY; mottled white, banded purple; medium DENSE to dense at the base; RELICT bedding; angular clayey silty GRAVEL; RESIDUAL mudrock; traces of calcification along relict bedding; traces of fine roots; Bulk Sample No. 5.

Gradual contact

slightly MOIST; dark greyish OLIVE, banded cream, brown and purple; DENSE; very FINE grained; thinly bedded and closely jointed; COMPLETELY weathered; SOFT ROCK; MUDROCK of the Tarkastad Formation; traces of decomposed roots; Not Sampled.

Profile Notes		Excavation Description	
Profiled by:	KS Coertzen (Pri.Sci.Nat)	Contractor:	Hire Cor
Groundwater Seepage:	N/A	Machine:	JCB 3CX- TLB Type Light Mechanical Excavator
Excavation Stability:	Sidewalls stable	Excavation Character:	Refusal on weathered mudrock bedrock
Samples Extracted:	1 Bulk Sample	Date Profiled:	25/01/2022
Elevation (MAMSL):	1393	Coordinates:	30.22230°S 26.53506°E

GEOCALIBRE
GEOTECHNICAL CONSULTANCY

GC-22-104
Smithfield Indoor Sports Centre

Profile Photo: TP5

Material Present in Test Pit: TP5

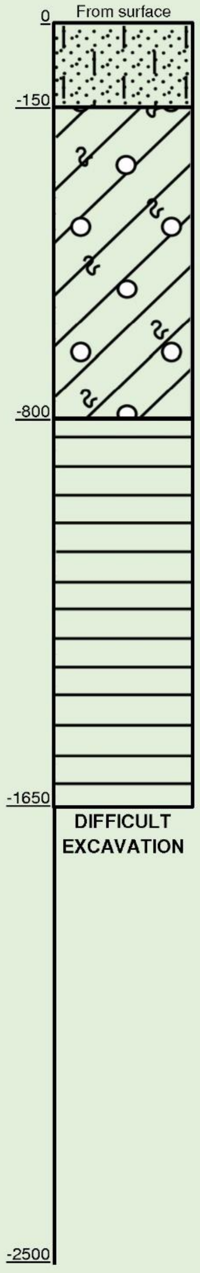
Surroundings of Test Pit: TP5

Facing East

Facing South West

GEOCALIBRE GEOTECHNICAL CONSULTANCY	**Test Pit TP6** GC-22-104 Smithfield Indoor Sports Centre BVi Consulting Engineers- Bloemfontein

Soil Profile for Test Pit TP6



slightly MOIST; dark reddish BROWN; LOOSE; PINHOLED; clayey SAND with traces of gravel; ALLUVIUM; organic rich topsoil; minor anthropogenic contamination; Not Sampled.

~Undulating contact

MOIST; dark olive GREY; mottled white, banded purple and red; FIRM; RELICT bedding; sandy CLAY with minor angular mudstone gravels; RESIDUAL mudrock; traces of calcification along relict bedding; traces of fine decomposed roots; minor bands of less decomposed protolith; Disturbed Sample No. 6.

Gradual contact

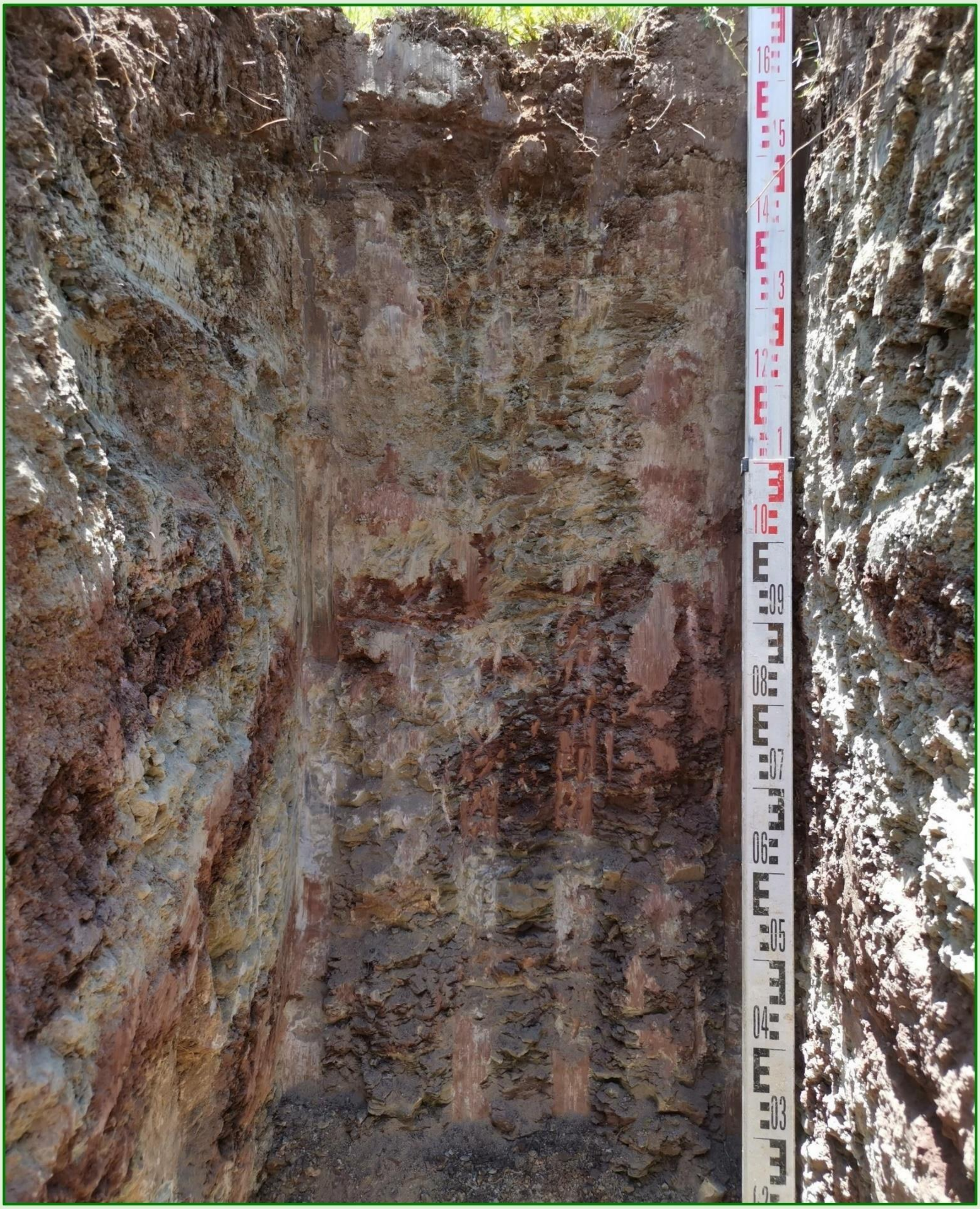
slightly MOIST; dark greyish OLIVE, banded cream, brown and purple; medium dense to DENSE at the base; very FINE grained; thinly bedded and closely jointed; COMPLETELY weathered; SOFT ROCK; MUDROCK of the Tarkastad Formation; traces of decomposed roots; gradual increase in consistency with depth; Not Sampled.

Profile Notes		Excavation Description	
Profiled by:	KS Coertzen (Pri.Sci.Nat)	**Contractor:**	Hire Cor
Groundwater Seepage:	N/A	**Machine:**	JCB 3CX- TLB Type Light Mechanical Excavator
Excavation Stability:	Sidewalls stable	**Excavation Character:**	Difficult excavation in weathered mudrock bedrock
Samples Extracted:	1 Disturbed Sample	**Date Profiled:**	25/01/2022
Elevation (MAMSL):	1392	**Coordinates:**	30.22268°S 26.53532°E

GEOCALIBRE
GEOTECHNICAL CONSULTANCY

GC-22-104
Smithfield Indoor Sports Centre

Profile Photo: TP6



Material Present in Test Pit: TP6

GEOCALIBRE GEOTECHNICAL CONSULTANCY

Surroundings of Test Pit: TP6

Facing West

Facing South

Appendix B

Lab Results

Disturbed and Bulk Samples

LETABA LABORATORIES AND SURVEYORS (Pty) Ltd
CIVIL ENGINEERING MATERIALS LABORATORY

6 Gruis Street, Hilton
Bloemfontein, 9301
P.O Box 22874,
Exton Road, 9313

T0864

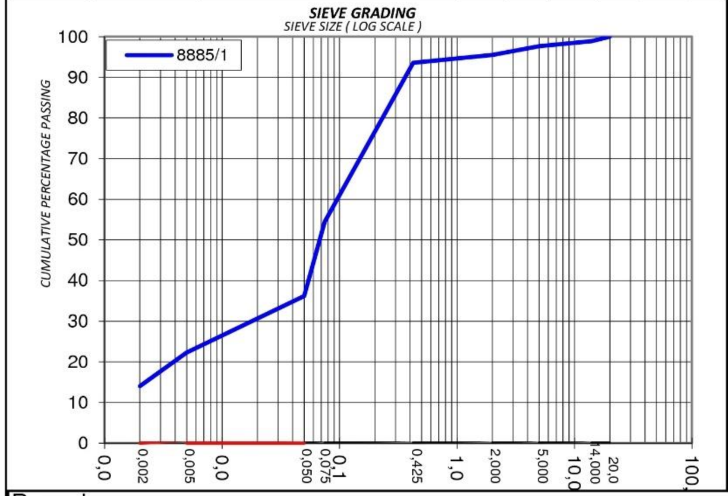
Tel. No: 087 285 0816 **e-mail :** letaba@letabalab.co.za www.letabalab.co.za

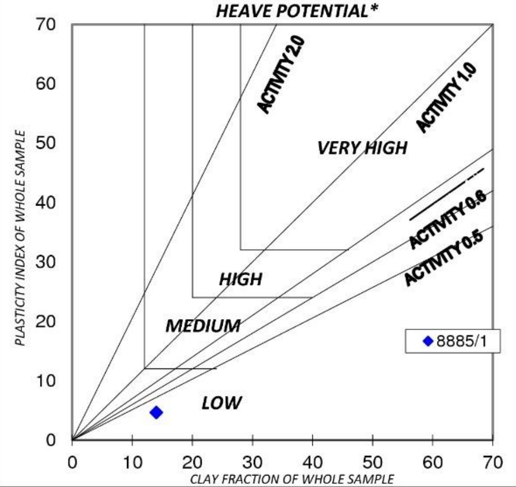
GRAVEL, SOIL AND SAND ANALYSIS : FOUNDATION INDICATOR TEST REPORT

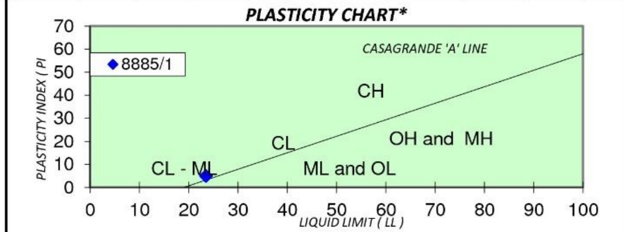
SANS 3001 Methods GR1, GR3, GR5, GR10, GR20, GR30 & GR40

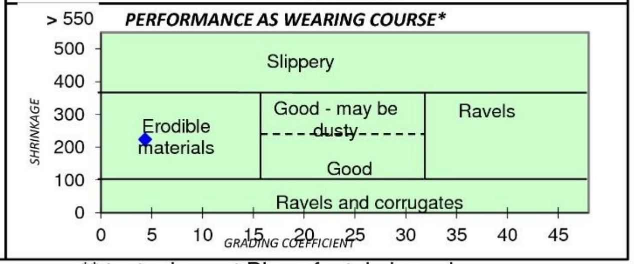
Client : **GeoCalibre (Pty) Ltd**	Address: 33 Denne Avenue,Donegal SH, Bainsvlei, Bloemfontein, 9301	**Date Sampled: 25-Jan-22**
Contract : **Smithfield Indoor Sport Centre**		**Doc No:** 8885/1(i)
Description : **TP 1 from 0.10 - 0.55 m below existing ground level - uncrushed material**		**Date Tested: 02-Feb-22**

Depth (m)	Sample No.	Description (Unified Soil Classification - ASTM D2487*)	Sieve analysis Cumulative percentage passing 50,0 mm	37,5 mm	28.0 mm	20,0 mm	14.0 mm	5.0 mm	2,00 mm	0,425 mm	0,075 mm	0,05 mm*	0.005 mm*	0.002 mm*	Soil Mortar Analysis % of mat. <2,00 mm* Coarse sand <2,0 >0,425mm	Fine sand <0,425 >0,075mm	Silt <0,05 >0,005mm	Clay <0,005 mm	Effective size*	Uniformity - coef.*	Curvature coef.*	Grading modulus*	Atterberg Limits Liquid Limit	Plasticity Index	Linear Shrinkage	Classifications* Unified Soil	COLTO	US.Highway	Group Index
0.10 - 0.55	8885/1	dusky Red Inorganic clay				100	99	98	96	94	54	36,2	22,3	14,0	2,1	40,9	14,6	23,3	<0.002	96	3,3	0,6	24	5	2,4	CL	N/A	A-4	4









Remarks:

The Shrinkage Product of this material was 225

** tests done at Bloemfontein branch

Date Issued: 17/Feb/22

Technical signatory (Name) : **O. Bodumele**

Signature:

Digitally signed by Obakeng Bodumele
Date: 2022.02.17 15:18:31 +02'00'

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Exton Road, 9313

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Tel. No: 087 285 0816 **e-mail :** letaba@letabalab.co.za www.letabalab.co.za

GRAVEL, SOIL AND SAND ANALYSIS : FOUNDATION INDICATOR TEST REPORT

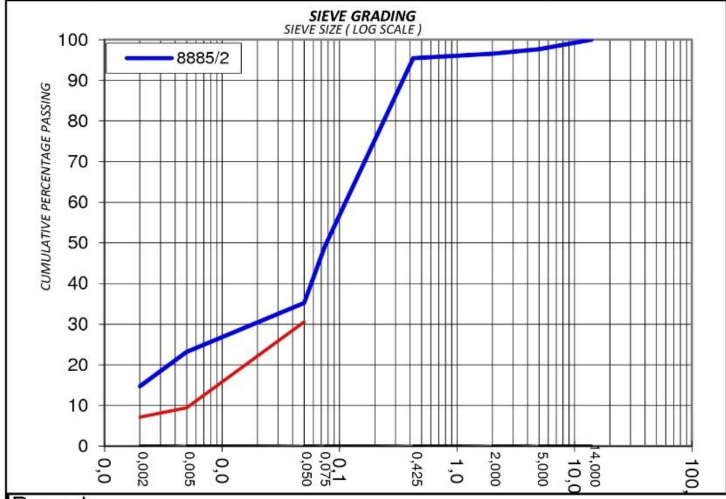
SANS 3001 Methods GR1, GR3, GR5, GR10, GR20, GR30 & GR40

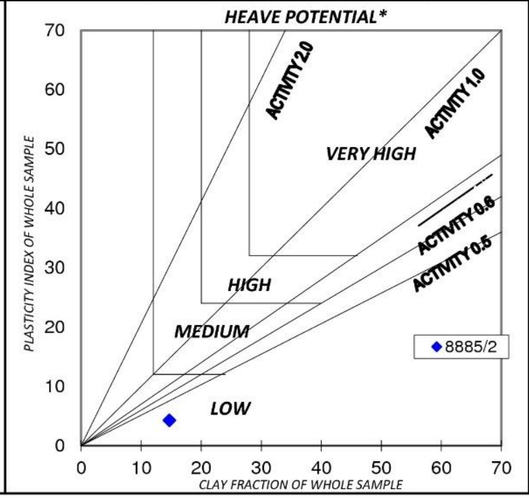
Client : **GeoCalibre (Pty) Ltd** Address: 33 Denne Avenue,Donegal SH, Bainsvlei, Bloemfontein, 9301 **Date Sampled: 25-Jan-22**

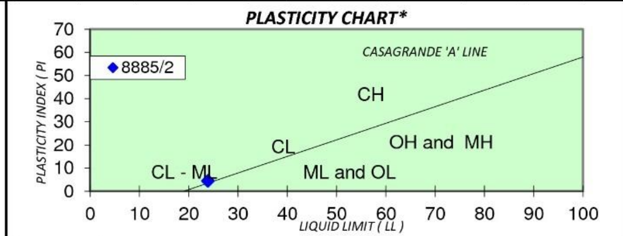
Contract : **Smithfield Indoor Sport Centre** **Doc No:** 8885/2(i)

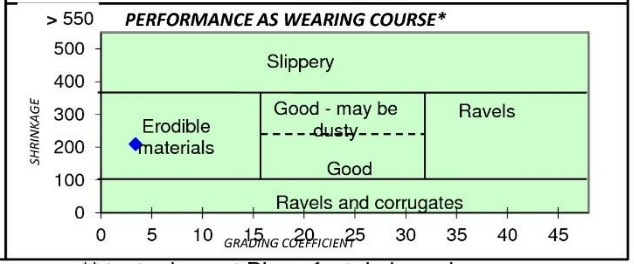
Description : **TP 2 from 0.20 - 1.10 m below existing ground level - uncrushed material** **Date Tested: 02-Feb-22**

Depth (m)	Sample No.	Description (Unified Soil Classification - ASTM D2487*)	Sieve analysis Cumulative percentage passing												Soil Mortar Analysis % of mat. <2,00 mm*				Effective size*	Uniformity - coef.*	Curvature coef.*	Grading modulus*	Atterberg Limits			Classifications*			
			50,0 mm	37,5 mm	28.0 mm	20,0 mm	14.0 mm	5.0 mm	2,00 mm	0,425 mm	0,075 mm	0,05 mm*	0.005 mm*	0.002 mm*	Coarse sand <2,0 >0,425mm	Fine sand <0,425 >0,075mm	Silt <0,05 >0,005mm	Clay <0,005 mm					Liquid Limit	Plasticity Index	Linear Shrinkage	Unified Soil	COLTO	US.Highway	Group Index
0.20 - 1.10	8885/2	drk Red Silty/clayey sand					100	98	97	95	49	35,2	23,2	14,7	1,1	48,1	12,4	24,0	<0.002	113	3,0	0,6	24	5	2,2	sm/sc	<G9	A-4	3
						*Double Hydrometer Values:						30,5	9,4	7,1															









Remarks: ** tests done at Bloemfontein branch

The Shrinkage Product of this material was 210

***DOUBLE HYDROMETER RESULT : 41 %** If Double Hydrometer result is above 40%, the material is considered to be dispersive.

***pH =7.6 , *Electrical Conductivity =0.04 S/m, tested on the whole sample**

Date Issued: 22/Feb/22 Technical signatory (Name) : **O. Bodumele** Signature:

Digitally signed by Obakeng Bodumele
Date: 2022.02.22 12:34:03 +02'00'

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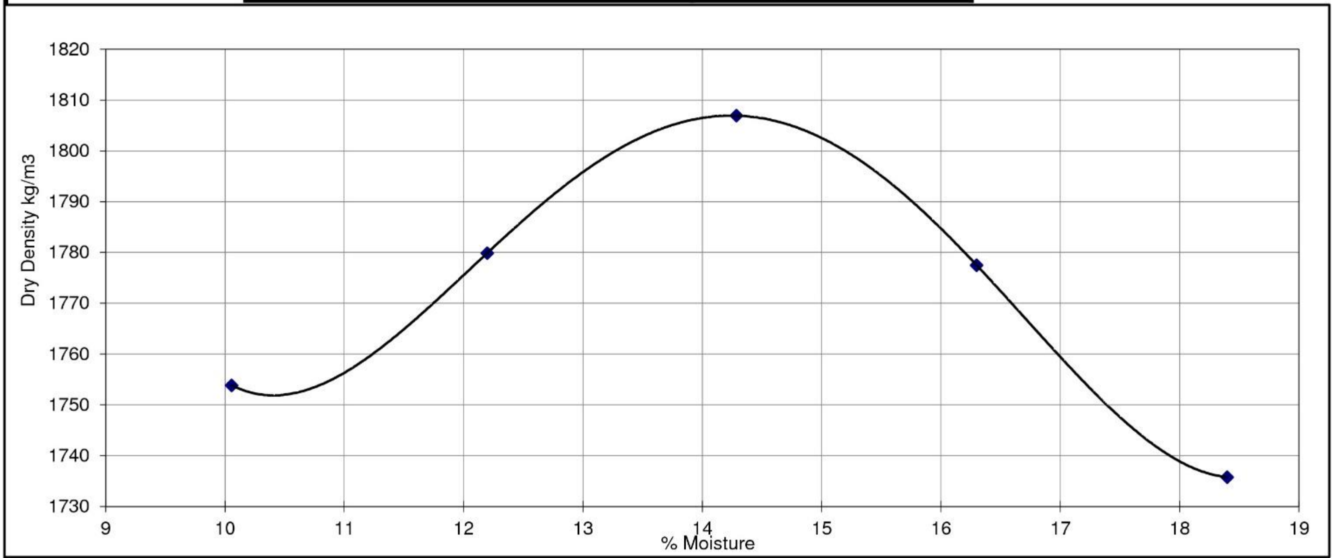
Tel. No: 087 285 0816 e-mail : letaba@letabalab.co.za www.letabalab.co.za

CBR and Maximum Dry Density Test Report

SANS 3001 Methods GR20, GR30 & GR40

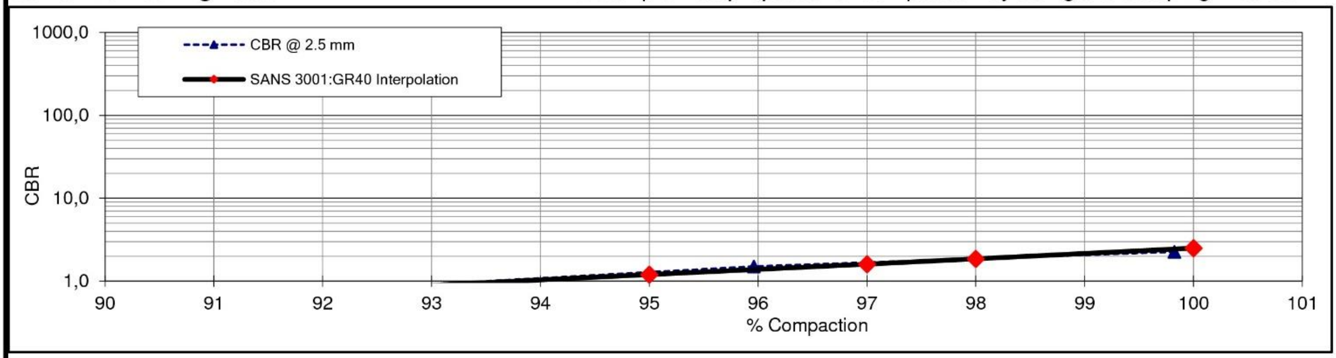
Client:	GeoCalibre (Pty) Ltd	Date tested:	02-Feb-22
Contract :	Smithfield Indoor Sport Centre	Date Sampled:	25-Jan-22
Description:	TP 2 from 0.20 - 1.10 m below existing ground level - uncrushed material	Sample No.	8885/2
		Doc no:	8885/2(ii)

*Maximum Dry Density =	1807	kg/m³
Optimum moisture content =	14,3	%



California Bearing Ratio

* This sample was prepared for compaction by using the Scalping method



% Compaction	100	98	97	95	93	90
CBR of 13.3 kN	2	2	2	1	1	1

** tests done at Bloemfontein branch

REMARKS

Briquette Information			
Compaction (%)	99,8	96,0	92,9
Dry Density (kg/m³)	1804	1734	1679
Compaction Moisture (%)	14,2	14,3	14,3
% Swell	0,09	0,28	0,60

Date Issued: 22-Feb-22 Technical signatory (Name) : O. Bodumele Signature:

Digitally signed by Obakeng Bodumele
Date: 2022.02.22 12:34:35 +02'00'

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Exton Road, 9313

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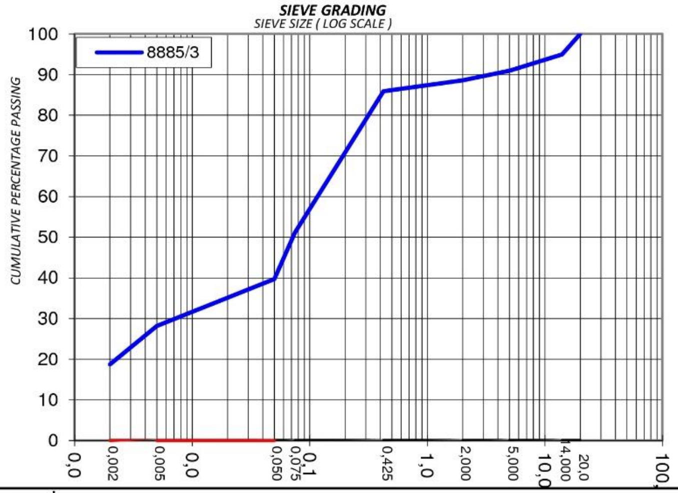
Tel. No: 087 285 0816 e-mail : letaba@letabalab.co.za www.letabalab.co.za

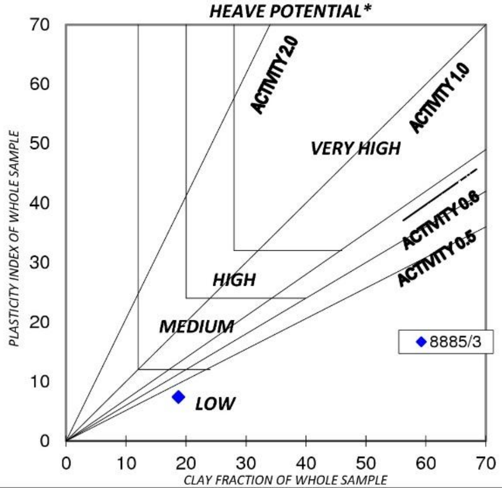
GRAVEL, SOIL AND SAND ANALYSIS : FOUNDATION INDICATOR TEST REPORT

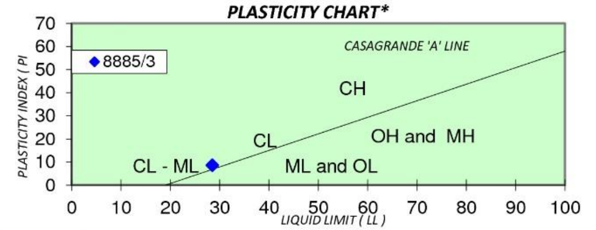
SANS 3001 Methods GR1, GR3, GR5, GR10, GR20, GR30 & GR40

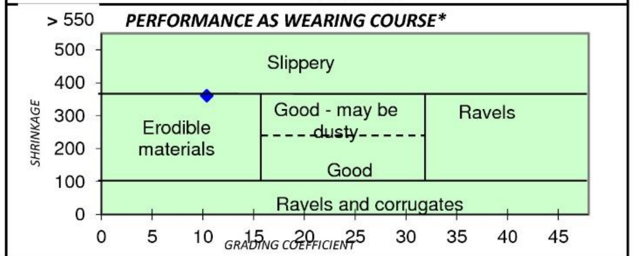
Client : **GeoCalibre (Pty) Ltd**	Address: 33 Denne Avenue,Donegal SH, Bainsvlei, Bloemfontein, 9301	**Date Sampled: 25-Jan-22**
Contract : **Smithfield Indoor Sport Centre**		**Doc No:** 8885/3(i)
Description : **TP 3 from 0.15 - 0.65 m below existing ground level - uncrushed material**		**Date Tested: 02-Feb-22**

Depth (m)	Sample No.	Description (Unified Soil Classification - ASTM D2487*)	Sieve analysis Cumulative percentage passing 50,0 mm	37,5 mm	28,0 mm	20,0 mm	14,0 mm	5.0 mm	2,00 mm	0,425 mm	0,075 mm	0,05 mm*	0.005 mm*	0.002 mm*	Soil Mortar Analysis % of mat. <2,00 mm* Coarse sand <2,0 >0,425mm	Fine sand <0,425 >0,075mm	Silt <0,05 >0,005mm	Clay <0,005 mm	Effective size*	Uniformity - coef.*	Curvature coef.*	Grading modulus*	Atterberg Limits Liquid Limit	Plasticity Index	Linear Shrinkage	Classifications* Unified Soil	COLTO	US.Highway	Group Index
0.15 - 0.65	8885/3	dusky Red Inorganic clay				100	95	91	89	86	51	39,7	28,2	18,7	3,0	39,3	13,0	31,8	<0.002	117	0,4	0,7	29	9	4,2	CL	N/A	A-4	3









Remarks:

The Shrinkage Product of this material was 361

** tests done at Bloemfontein branch

Date Issued: 22/Feb/22 Technical signatory (Name) : **O. Bodumele** Signature:

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Date: 2022.02.22 12:43:01 +02'00'

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GRAVEL, SOIL AND SAND ANALYSIS : FOUNDATION INDICATOR TEST REPORT

SANS 3001 Methods GR1, GR3, GR5, GR10, GR20, GR30 & GR40

Client : **GeoCalibre (Pty) Ltd** Address: 33 Denne Avenue,Donegal SH, Bainsvlei, Bloemfontein, 9301

Date Sampled: 25-Jan-22

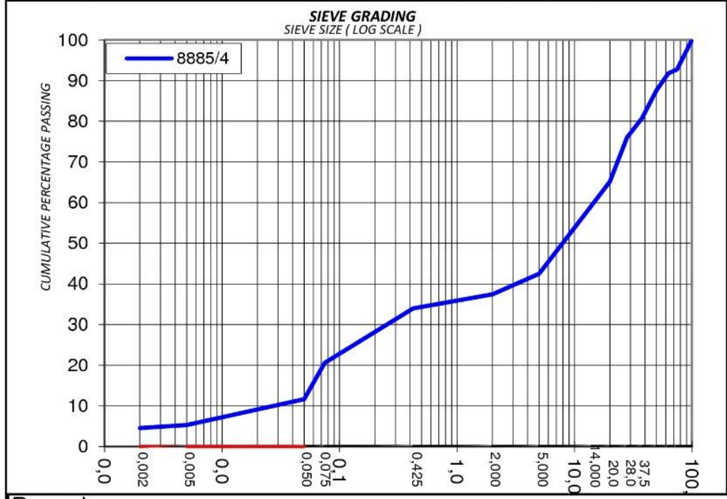
Contract : **Smithfield Indoor Sport Centre**

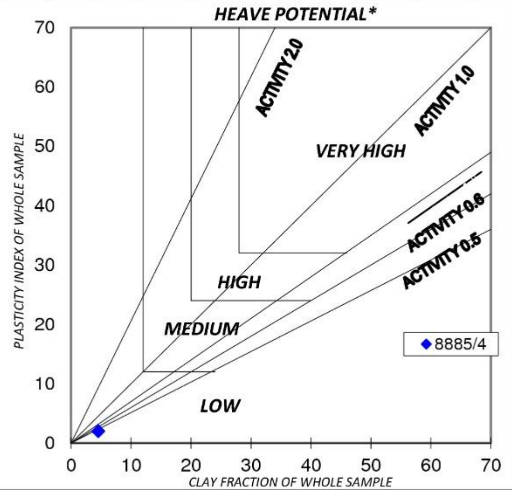
Doc No: 8885/4(i)

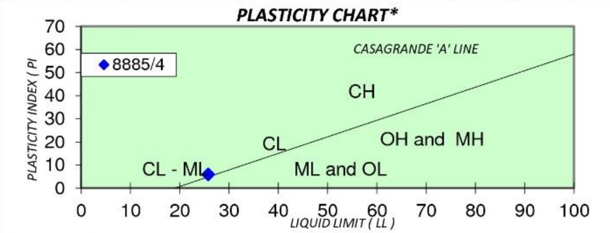
Description : **TP 4 from 0.30 - 0.80 m below existing ground level - uncrushed material**

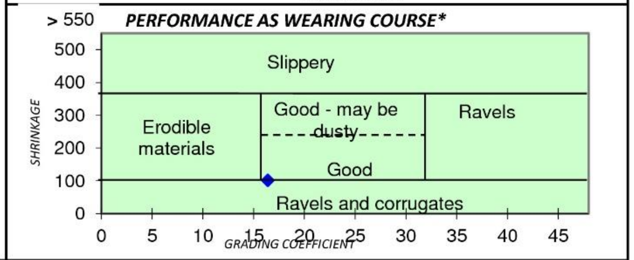
Date Tested: 02-Feb-22

Depth (m)	Sample No.	Description (Unified Soil Classification - ASTM D2487*)	Sieve analysis Cumulative percentage passing 50,0 mm	37,5 mm	28.0 mm	20,0 mm	14.0 mm	5.0 mm	2,00 mm	0,425 mm	0,075 mm	0,05 mm*	0.005 mm*	0.002 mm*	Soil Mortar Analysis % of mat. <2,00 mm* Coarse sand <2,0 >0,425mm	Fine sand <0,425 >0,075mm	Silt <0,05 >0,005mm	Clay <0,005 mm	Effective size*	Uniformity - coef.*	Curvature coef.*	Grading modulus*	Atterberg Limits Liquid Limit	Plasticity Index	Linear Shrinkage	Classifications* Unified Soil	COLTO	US.Highway	Group Index
0.30 - 0.80	8885/4	lt Brown Silty/Clayey gravel with sand	88	81	76	65	59	42	37	34	21	11,6	5,3	4,5	9,2	35,8	17,0	14,0	0,028	524	0,2	2,1	26	6	3,0	GM/GC	<G9	A-1-b	0









Remarks:

The Shrinkage Product of this material was 102

** tests done at Bloemfontein branch

Date Issued: 22/Feb/22 Technical signatory (Name) : **O. Bodumele** Signature:

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Date: 2022.02.22
12:57:10 +02'00'

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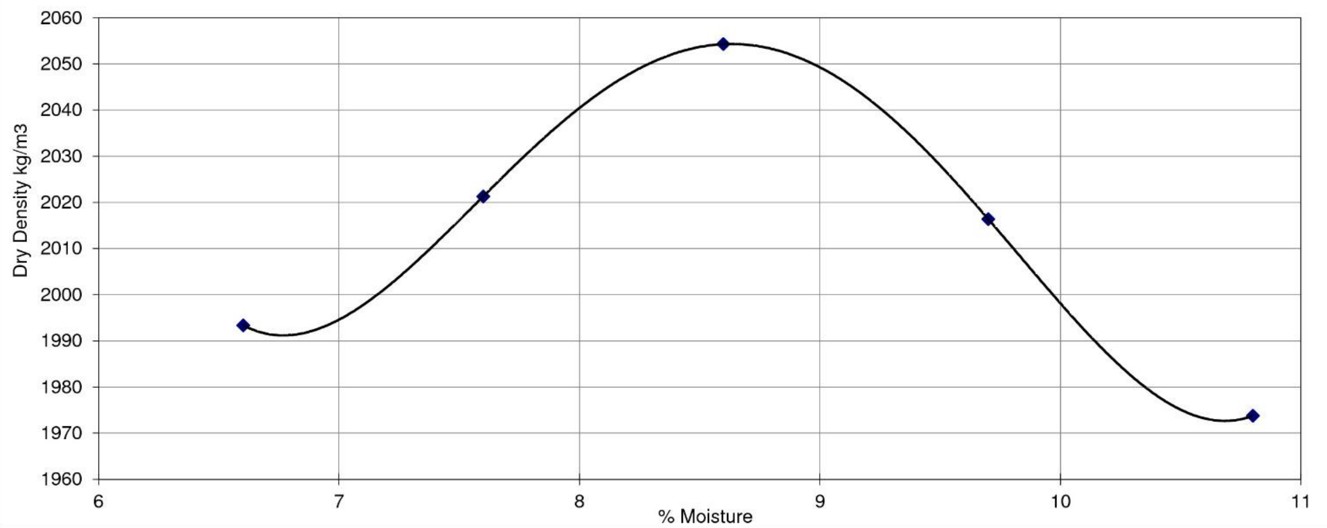
CBR and Maximum Dry Density Test Report

SANS 3001 Methods
GR20, GR30 & GR40

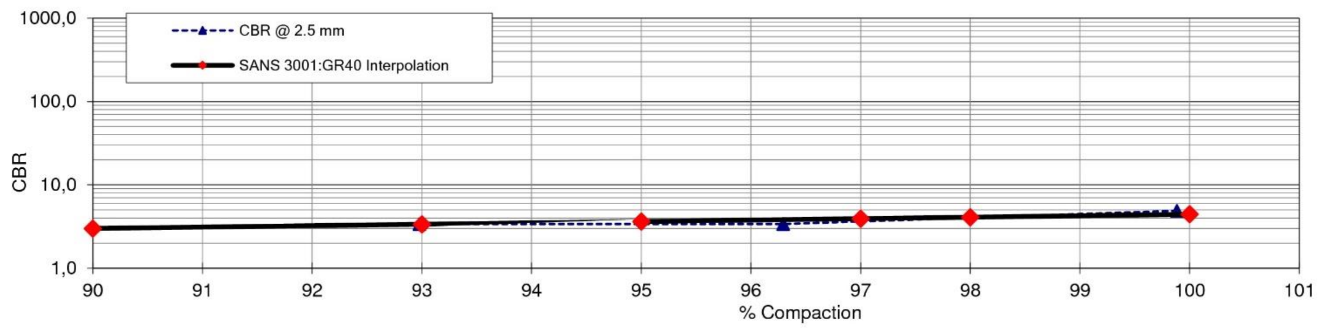
Client:	GeoCalibre (Pty) Ltd	Date tested:	02-Feb-22
Contract :	Smithfield Indoor Sport Centre	Date Sampled:	25-Jan-22
Description:	TP 4 from 0.30 - 0.80 m below existing ground level - uncrushed material	Sample No.	8885/4
		Doc no:	8885/4(ii)

*Maximum Dry Density =	2054	kg/m³
Optimum moisture content =	8,6	%



California Bearing Ratio

* This sample was prepared for compaction by using the Scalping method



% Compaction	100	98	97	95	93	90
CBR of 13.3 kN	4	4	4	4	3	3

** tests done at Bloemfontein branch

REMARKS

Briquette Information			
Compaction (%)	99,9	96,3	93,0
Dry Density (kg/m³)	2052	1978	1910
Compaction Moisture (%)	8,5	8,6	8,6
% Swell	0,07	0,11	0,23

Date Issued: 22-Feb-22 Technical signatory (Name) : O. Bodumele Signature:

Digitally signed by Obakeng Bodumele
Date: 2022.02.22 12:57:24 +02'00'

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AND SURVEYORS (Pty) Ltd
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Bloemfontein, 9301
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Tel. No: 087 285 0816 e-mail : letaba@letabalab.co.za www.letabalab.co.za

GRAVEL, SOIL AND SAND ANALYSIS : FOUNDATION INDICATOR TEST REPORT

SANS 3001 Methods GR1, GR3, GR5, GR10, GR20, GR30 & GR40

Client : **GeoCalibre (Pty) Ltd** Address: 33 Denne Avenue,Donegal SH, Bainsvlei, Bloemfontein, 9301

Contract : **Smithfield Indoor Sport Centre**

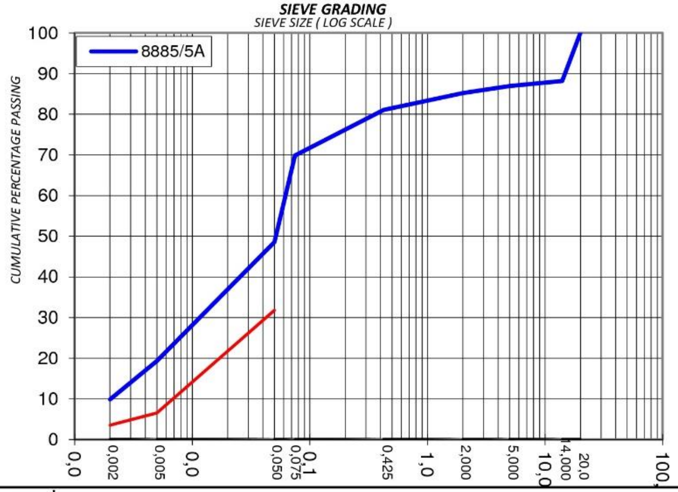
Description : **TP 5 from 0.50 - 0.90 m below existing ground level - uncrushed material**

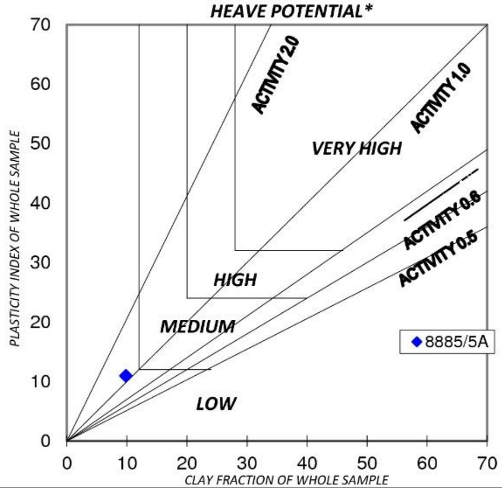
Date Sampled: 25-Jan-22

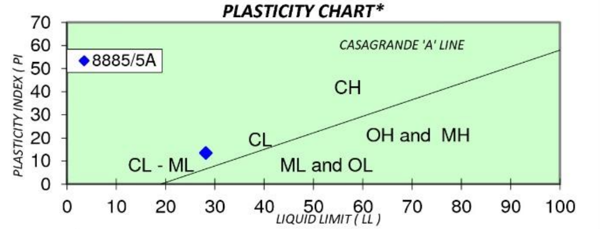
Doc No: 8885/5A(i)

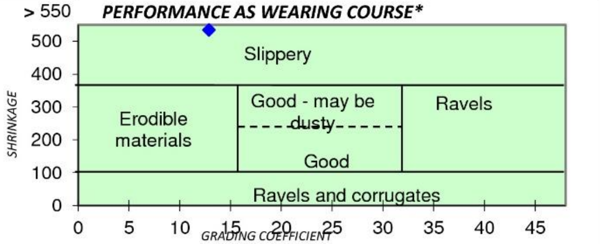
Date Tested: 02-Feb-22

Depth (m)	Sample No.	Description (Unified Soil Classification - ASTM D2487*)	Sieve analysis Cumulative percentage passing 50,0 mm	37,5 mm	28.0 mm	20,0 mm	14.0 mm	5.0 mm	2,00 mm	0,425 mm	0,075 mm	0,05 mm*	0.005 mm*	0.002 mm*	Soil Mortar Analysis % of mat. <2,00 mm* Coarse sand <2,0 >0,425mm	Fine sand <0,425 >0,075mm	Silt <0,05 >0,005mm	Clay <0,005 mm	Effective size*	Uniformity - coef.*	Curvature coef.*	Grading modulus*	Atterberg Limits Liquid Limit	Plasticity Index	Linear Shrinkage	Classifications* Unified Soil	COLTO	US.Highway	Group Index
0.50 - 0.90	8885/5A	drk Reddish Brown Inorganic clay				100	88	87	85	81	70	48,6	19,3	9,9	4,9	13,1	34,4	22,6	0,002	31	1,1	0,6	28	14	6,6	CL	<G9	A-6	7
					*Double Hydrometer Values:							31,8	6,5	3,5															









Remarks:

** tests done at Bloemfontein branch

The Shrinkage Product of this material was 535

***DOUBLE HYDROMETER RESULT : 34 %** If Double Hydrometer result is above 40%, the material is considered to be dispersive.

***pH =7.6 , *Electrical Conductivity =0.035 S/m, tested on the whole sample**

This test report is supplement to test report no. 8885/5

Date Issued: 9/Mar/22 Technical signatory (Name) : **O. Bodumele** Signature:

Digitally signed by Obakeng Bodumele
Date: 2022.03.09 10:59:23 +02'00'

LETABA LABORATORIES
AND SURVEYORS (Pty) Ltd
CIVIL ENGINEERING MATERIALS LABORATORY

6 Gruis Street, Hilton
Bloemfontein, 9301
P.O Box 22874,
Exton Road, 9313

T0864

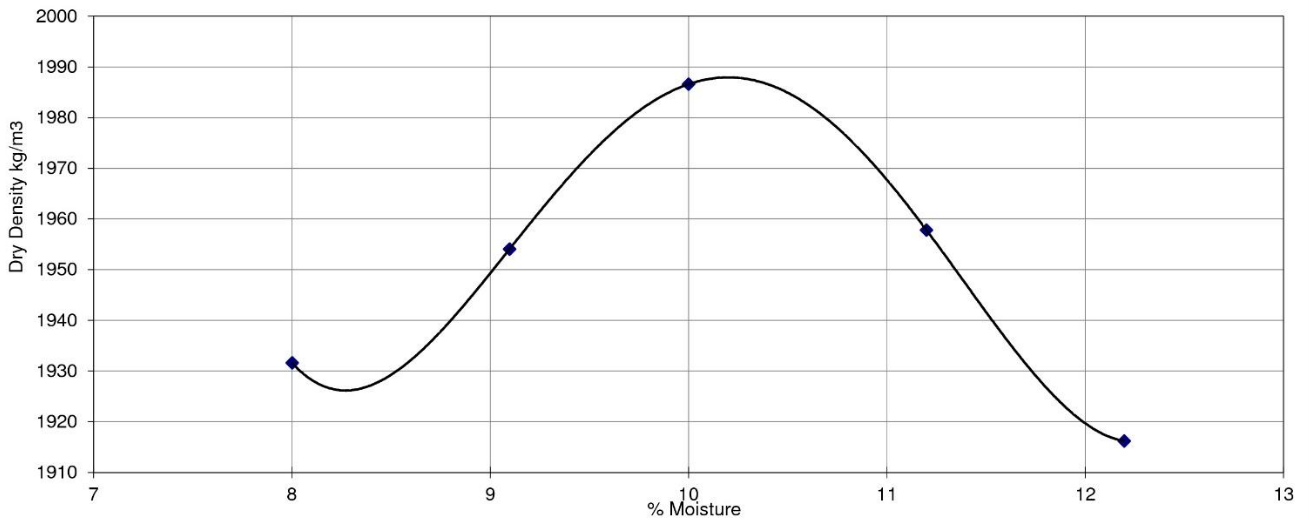
Tel. No: 087 285 0816 e-mail : letaba@letabalab.co.za www.letabalab.co.za

CBR and Maximum Dry Density Test Report

SANS 3001 Methods
GR20, GR30 & GR40

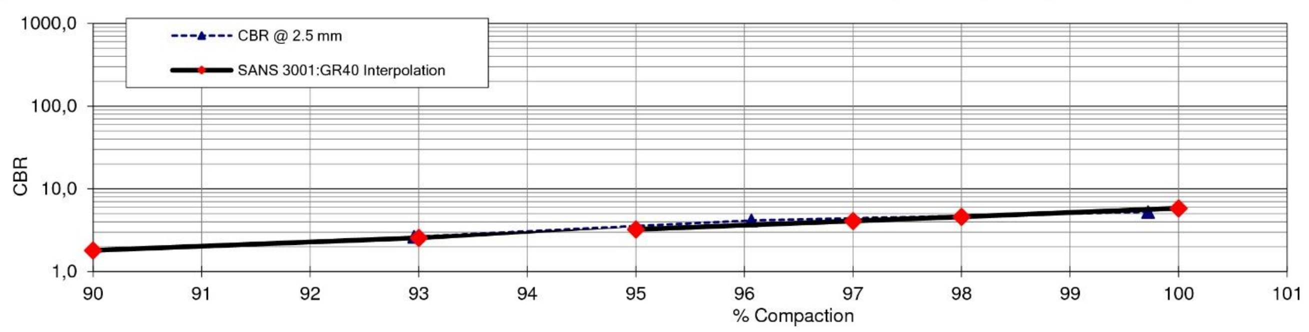
Client:	GeoCalibre (Pty) Ltd	Date tested:	02-Feb-22
Contract :	Smithfield Indoor Sport Centre	Date Sampled:	25-Jan-22
Description:	TP 5 from 0.50 - 0.90 m below existing ground level - uncrushed material	Sample No.	8885/5A
		Doc no:	8885/5A(ii)

*Maximum Dry Density =	1987	kg/m³
Optimum moisture content =	10,2	%



California Bearing Ratio

* This sample was prepared for compaction by using the Scalping method



% Compaction	100	98	97	95	93	90
CBR of 13.3 kN	6	5	4	3	3	2

** tests done at Bloemfontein branch

REMARKS

Briquette Information			
Compaction (%)	99,7	96,1	93,0
Dry Density (kg/m³)	1982	1909	1847
Compaction Moisture (%)	10,0	10,1	10,1
% Swell	0,09	0,27	0,43

This test report is supplement to test report no. 8885/5

Date Issued: 09-Mar-22 Technical signatory (Name) : **O. Bodumele** Signature:

Digitally signed by Obakeng Bodumele
Date: 2022.03.09 10:59:46 +02'00'

LETABA LABORATORIES AND SURVEYORS (Pty) Ltd
CIVIL ENGINEERING MATERIALS LABORATORY

6 Gruis Street, Hilton
Bloemfontein, 9301
P.O Box 22874,
Exton Road, 9313

T0864

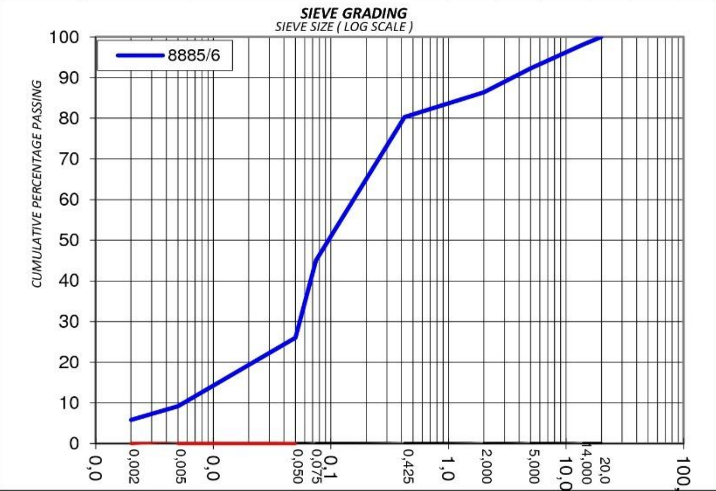
Tel. No: 087 285 0816 e-mail : letaba@letabalab.co.za www.letabalab.co.za

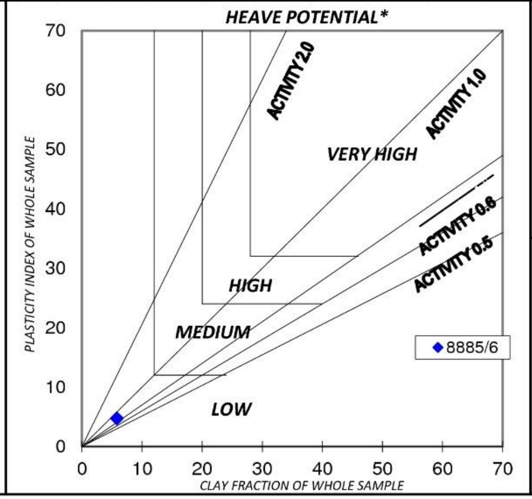
GRAVEL, SOIL AND SAND ANALYSIS : FOUNDATION INDICATOR TEST REPORT

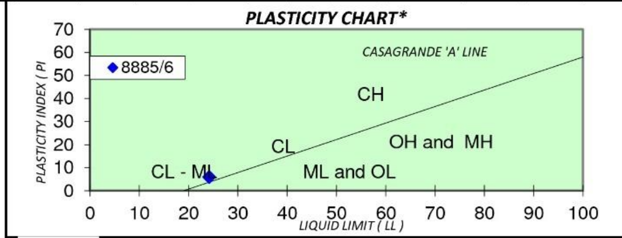
SANS 3001 Methods GR1, GR3, GR5, GR10, GR20, GR30 & GR40

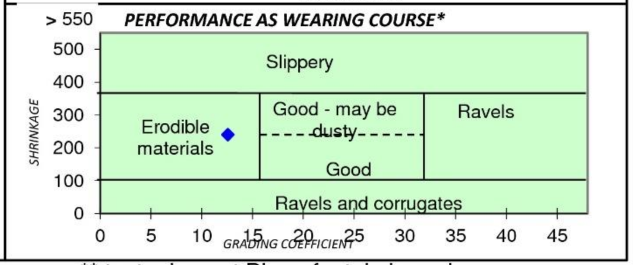
Client : **GeoCalibre (Pty) Ltd** Address: 33 Denne Avenue,Donegal SH, Bainsvlei, Bloemfontein, 9301	**Date Sampled: 25-Jan-22**
Contract : **Smithfield Indoor Sport Centre**	**Doc No:** 8885/6(i)
Description : **TP 6 from 0.50 - 0.90 m below existing ground level - uncrushed material**	**Date Tested: 02-Feb-22**

Depth (m)	Sample No.	Description (Unified Soil Classification - ASTM D2487*)	Sieve analysis Cumulative percentage passing												Soil Mortar Analysis % of mat. <2,00 mm*				Effective size*	Uniformity - coef.*	Curvature coef.*	Grading modulus*	Atterberg Limits			Classifications*			
			50,0 mm	37,5 mm	28.0 mm	20,0 mm	14.0 mm	5.0 mm	2,00 mm	0,425 mm	0,075 mm	0,05 mm*	0.005 mm*	0.002 mm*	Coarse sand <2,0 >0,425mm	Fine sand <0,425 >0,075mm	Silt <0,05 >0,005mm	Clay <0,005 mm					Liquid Limit	Plasticity Index	Linear Shrinkage	Unified Soil	COLTO	US.Highway	Group Index
0.50 - 0.90	8885/6	lt Brown Silty/clayey sand				100	98	92	86	80	45	26,0	9,1	5,8	7,1	40,8	19,5	10,6	0,006	28	3,4	0,9	24	6	3,0	sm/sc	N/A	A-4	2









Remarks:

The Shrinkage Product of this material was 241

** tests done at Bloemfontein branch

Date Issued: 22/Feb/22 Technical signatory (Name) : **O. Bodumele** Signature:

Appendix C

Undisturbed Sample Results

STEYN WILSON GEOTECHNICAL
CIVIL ENGINEERING TESTING LABORATORIES

sanas Testing Laboratory T0835

11 Gooderson Road Blackheath
PO Box 58 Blackheath 7581
Tel: 021 905 0435
Fax: 086 499 9482
Email: geotech@steynwilson.co.za
Web: www.steynwilson.co.za

Client:	Geocalibre
Project:	Smithfield Indoor Sport Centre
Attention:	Kevin Coertzen
Address	Bloemfontein, South Africa, ,
Contact No.:	083 608 4426
Your Ref. No:	SWL20305
Date Reported	07 March 2022

TEST REPORT REFERENCE NUMBER / JOB NUMBER : SWG00278

Dear Sir / Madam

Herewith please find the original reports pertaining to the above mentioned project.

Test Requested

1	Shearbox	Pg 2 - 4
1	Oedometer & Swell	Pg 5 - 5
1	Double Oedometer	Pg 6 - 6

Site Sampling and Materials Information

Sampling Method	As per Client
Environmental Conditions	As per Client
Deviations from prescribed method	None
Responsibility of information disclaimer	As per Client

FINAL REPORT

We would like to take this opportunity to thank you for your valued support.
Should you have any further enquiries please don't hesitate to contact me.

Yours Faithfully
Steyn Wilson Geotechnical

Frank Coetzee
Technical Signatory

Remarks:

1. Information contained herein is confidential to Steyn Wilson Geotechnical and the addressee
2. Opinions & Interpretations are not included in our schedule of Accreditation.
3 The results reported relate only to the sample tested, Further use of the attached information is not the responsibility or liability of Steyn Wilson Geotechnical.
4. This document is the correct record of all measurements made, and may not be reproduced other than with full written approval from a director of Steyn Wilson Geotechnical.
5. Measuring equipment is traceable to SI Units (Where applicable).
6 Should there be any deviation from the prescribed test method comments will be made thereof, pertaining to the test on the relevant materials report.
7 Uncertainty of measurement is calculated and corresponds to a coverage probability of approximately 95%. Available on request.
8. The decision rule states that the measurement of uncertainty can be applied by the customer to the test results, on request. It is not the responsibility or liability of Steyn Wilson Geotechnical.
9. All tests marked with (*) means that those test methods are not accredited.

Direct Shear Test

Initial Sample Details

		Specimen 1	Specimen 2	Specimen 3
Height	(mm)	20.0	20.0	20.0
Diameter	(mm)	60.0	60.0	60.0
Mass	(g)	102.9	108.9	104.1
Moisture	(%)	21.0	21.0	21.0
Dry Density	(Mg/m^3)	1.50	1.59	1.52
Bulk Density	(Mg/m^3)	1.82	1.93	1.84
Void Ratio		0.763	0.665	0.742
Particle Density	(Mg/m^3)	2.65		
Sample Method		Bag		
Disturbed/Undisturbed		Undisturbed		
Remoulded Desity	(Mg/m^3)	-		

Consolidation Details

		Specimen 1	Specimen 2	Specimen 3
Vertical Displacement	(mm)	1.121	2.093	3.084
Void Ratio After Consolidation		0.664	0.491	0.474

Maximum Shear Stress Results

		Specimen 1	Specimen 2	Specimen 3
Normal Stress	(kPa)	50	100	200
Peak Shear Stress	(kPa)	26.9	47.7	87.4
Horizontal Strain at Failure	(mm)	1.4	6.9	6.7
Verical Strain at Failure	(mm)	0.183	0.509	0.802
Rate of Shear	(mm/min)	0.006	0.014	0.010
Friction Angle (ϕ)	(°)	21.9		
Cohesion (c)	(kPa)	7.21		

Final Sample Details

		Specimen 1	Specimen 2	Specimen 3
Mass	(g)	106.2	109.0	100.8
Moisture	(%)	33.1	30.1	27.9
Void Ratio		0.648	0.449	0.404

STEYN-WILSON GEOTECHNICAL

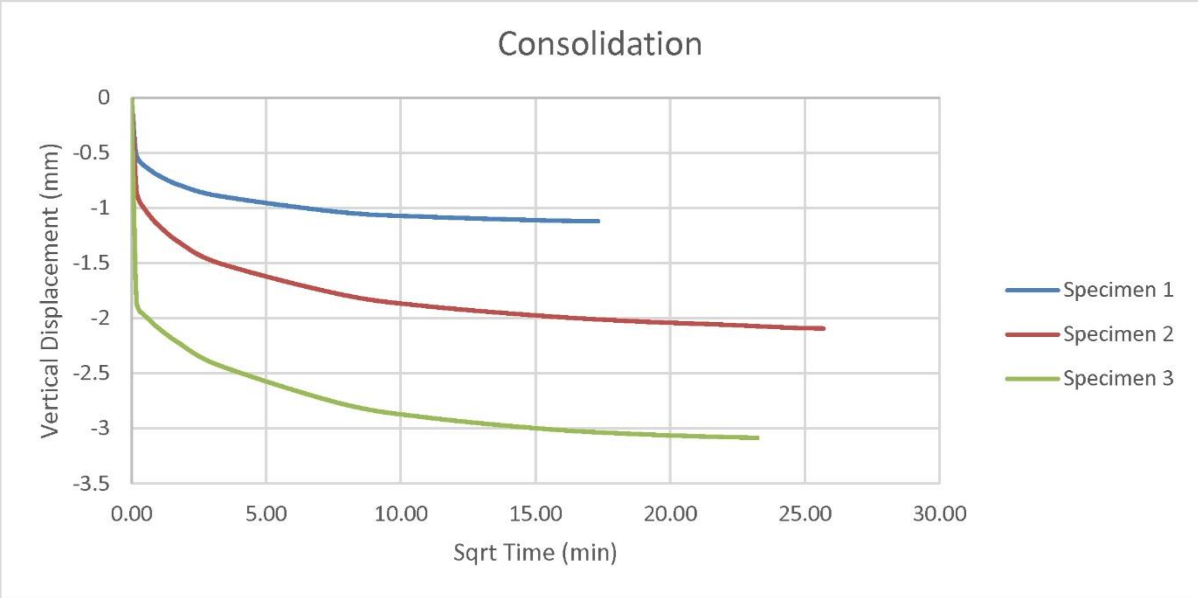
sanas Testing Laboratory T0835

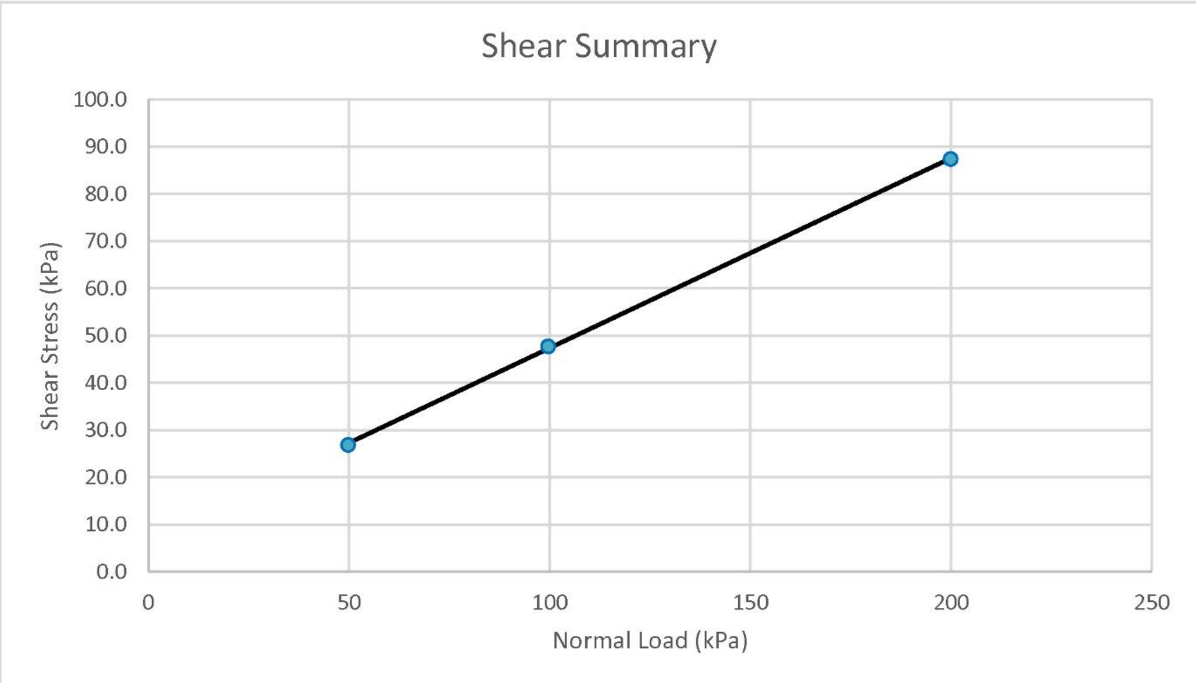
Project	**Smithfield Indoor Sports Centre**		
Sample	**TP.2_0.80 - 1.00m**		
Client	Geocalibre	Test Method	BS1377 - 7: 1990
Jobfile	SWG00278	Test Date	08/02/2022

Direct Shear Test

Graphs

Friction Angle (ϕ)	(°)	21.9
Cohesion (c)	(kPa)	7.21





STEYN-WILSON GEOTECHNICAL

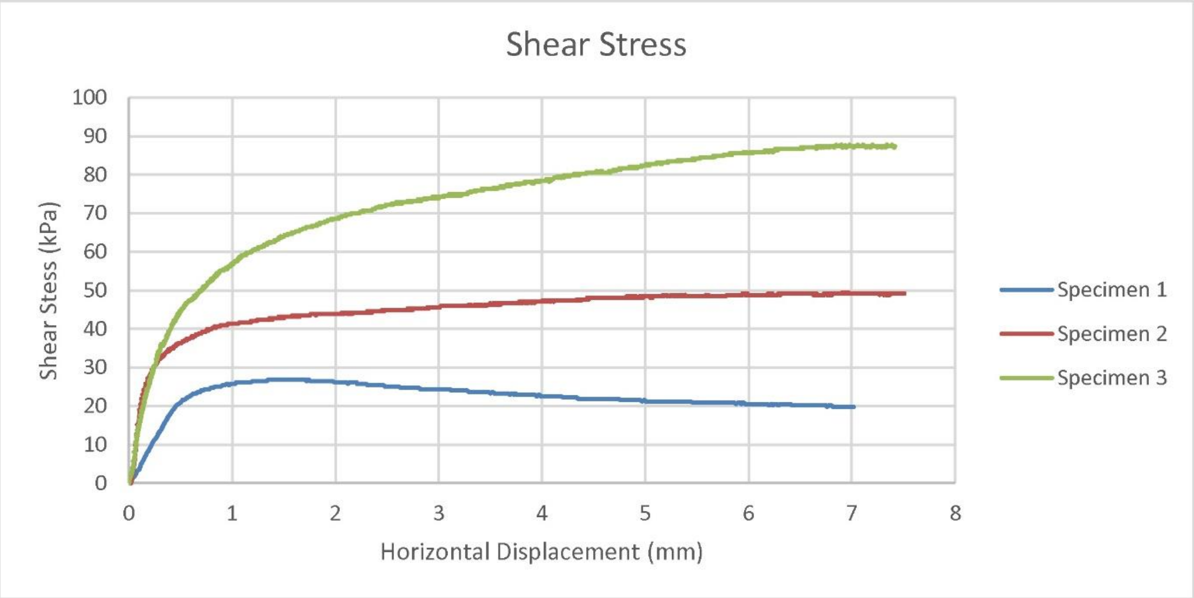
sanas Testing Laboratory

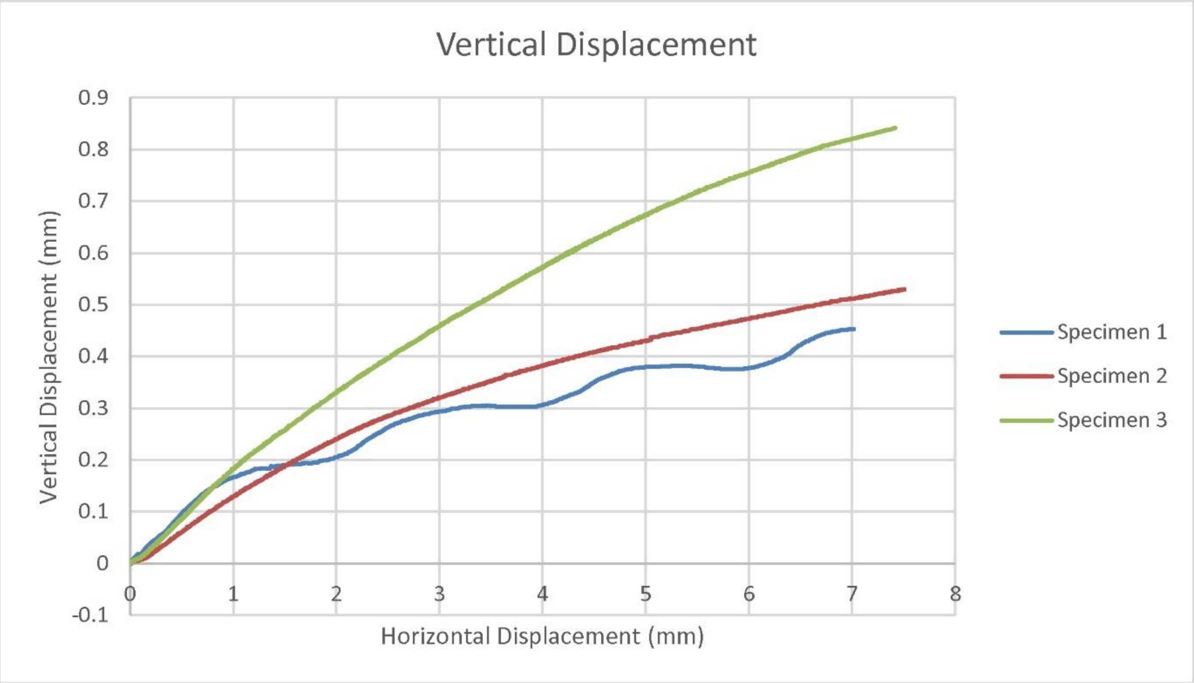
Project	**Smithfield Indoor Sports Centre**		
Sample	**TP.2_0.80 - 1.00m**		
Client	Geocalibre	Test Method	BS1377 - 7: 1990
Jobfile	SWG00278	Test Date	08/02/2022

Direct Shear Test

Graphs

Friction Angle (ϕ)	(°)	21.9
Cohesion (c)	(kPa)	7.21





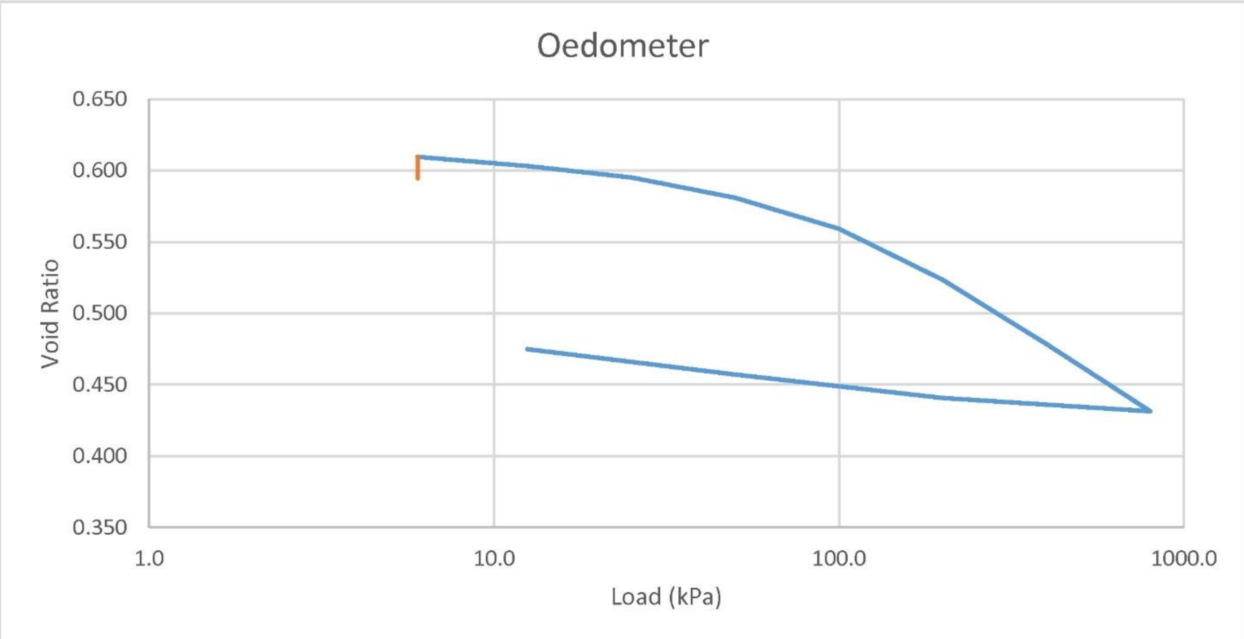
Project	**Smithfield Indoor Sports Centre**		
Sample	**TP.2_0.80 - 1.00m**		
Client	Geocalibre	Test Method	BS1377 - 7: 1990
Jobfile	SWG00278	Test Date	08/02/2022

Oedometer Swell Test

Sample Detail		Initial	Final
Height	(mm)	20.3	18.8
Diameter	(mm)	63.5	63.5
Weight	(g)	128.7	128.2
Moisture	(%)	21.8	22.4
Dry Density	(Mg/m^3)	1.64	1.76
Bulk Density	(Mg/m^3)	2.00	2.16
Void Ratio		0.595	0.475
Particle Density	(Mg/m^3)	2.62	
Disturbed/Undisturbed		Undisturbed	
Remoulded Density	(Mg/m^3)	-	

Load (kPa)	Height (mm)	Void Ratio
6.0	20.300	0.595
6.0	20.492	0.610
12.5	20.410	0.603
25	20.307	0.595
50	20.127	0.581
100	19.851	0.559
200	19.396	0.524
400	18.823	0.479
800	18.223	0.431
200	18.341	0.441
50	18.552	0.457
12.5	18.776	0.475

Swell Results		
Swell Percentage	0.9	%
Swell Pressure	19	kPa



STEYN-WILSON GEOTECHNICAL

Project	**Smithfield Indoor Sports Centre**		
Sample	**TP3_0.40 - 0.60**		
Client	Geo Calibre	Test Method	BS1377 - 5: 1990
Jobfile	SWG00278	Test Date	28/02/2022

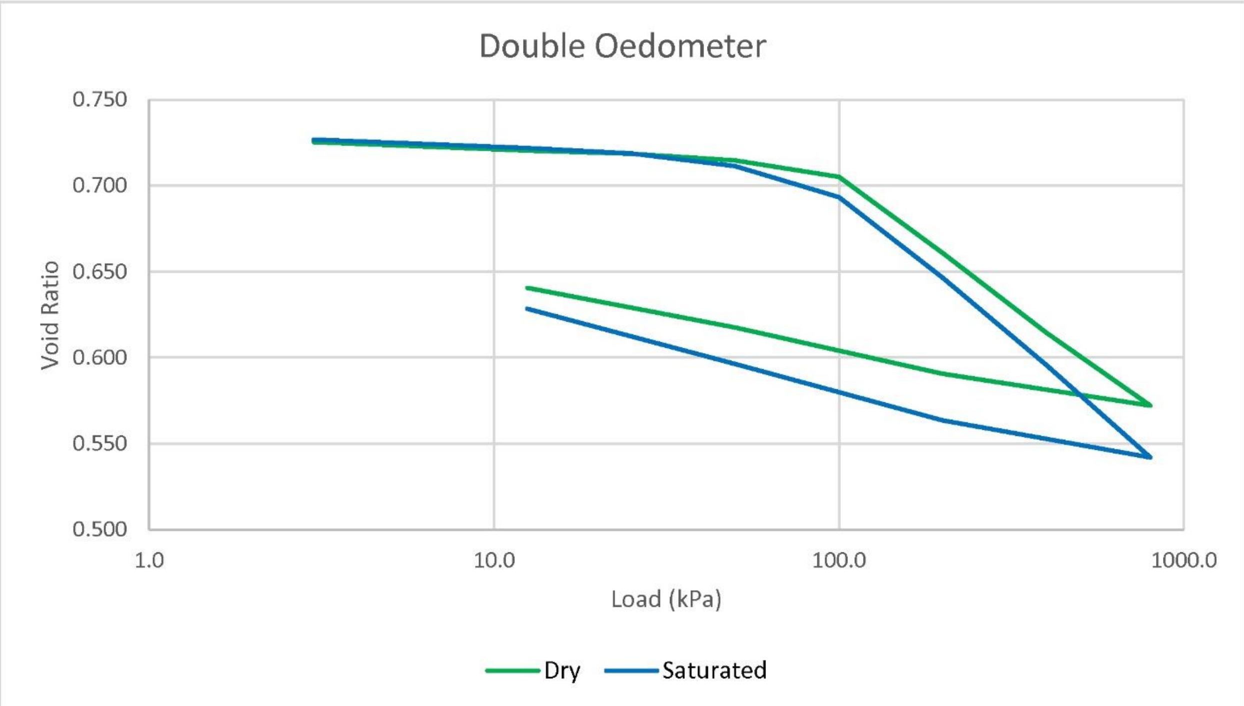
Double Oedometer Test

Dry Sample Detail		Initial	Final
Height	(mm)	20.3	19.3
Diameter	(mm)	63.5	63.5
Weight	(g)	119.5	121
Moisture	(%)	21.0	26.8
Dry Density	(Mg/m^3)	1.54	1.56
Bulk Density	(Mg/m^3)	1.86	1.98
Void Ratio		0.725	0.641
Particle Density	(Mg/m^3)	2.65	
Disturbed/Undisturbed		Undisturbed	
Remoulded Density	(Mg/m^3)	-	

Saturated Sample Detail		Initial	Final
Height	(mm)	20.3	19.1
Diameter	(mm)	63.5	63.5
Weight	(g)	119.4	120.0
Moisture	(%)	21.0	29.2
Dry Density	(Mg/m^3)	1.53	1.53
Bulk Density	(Mg/m^3)	1.86	1.98
Void Ratio		0.727	0.628
Particle Density	(Mg/m^3)	2.65	
Disturbed/Undisturbed		Undisturbed	
Remoulded Density	(Mg/m^3)	-	

Dry Sample		
Load (kPa)	Height (mm)	Void Ratio
3.0	20.300	0.725
12.5	20.242	0.721
25.0	20.221	0.719
50.0	20.175	0.715
100.0	20.061	0.705
200.0	19.540	0.661
400.0	18.993	0.614
800.0	18.496	0.572
200.0	18.713	0.591
50.0	19.031	0.618
12.5	19.302	0.641

Saturated Sample		
Load (kPa)	Height (mm)	Void Ratio
3.0	20.300	0.727
12.5	20.242	0.722
25.0	20.206	0.719
50.0	20.118	0.711
100.0	19.907	0.693
200.0	19.356	0.647
400.0	18.756	0.596
800.0	18.126	0.542
200.0	18.380	0.564
50.0	18.765	0.596
12.5	19.142	0.628



Project	Smithfield Indoor Sports Centre		
Sample	TP2_0.80-1.00m		
Client	Geo Calibre	Test Method	BS1377 - 5: 1990
Jobfile	SWG00278	Test Date	10/02/2022

Appendix D

DCP Test Results

DCP Test DCP 1

GC-22-104

Smithfield Indoor Sports Center

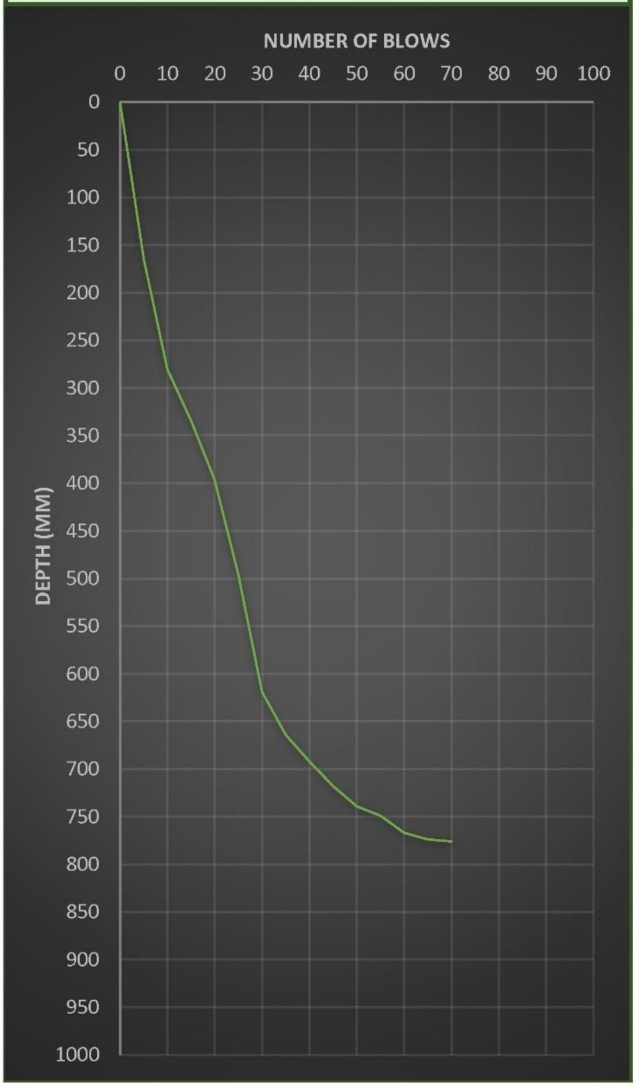
BVi Consulting Engineers- Bloemfontein

DCP 1

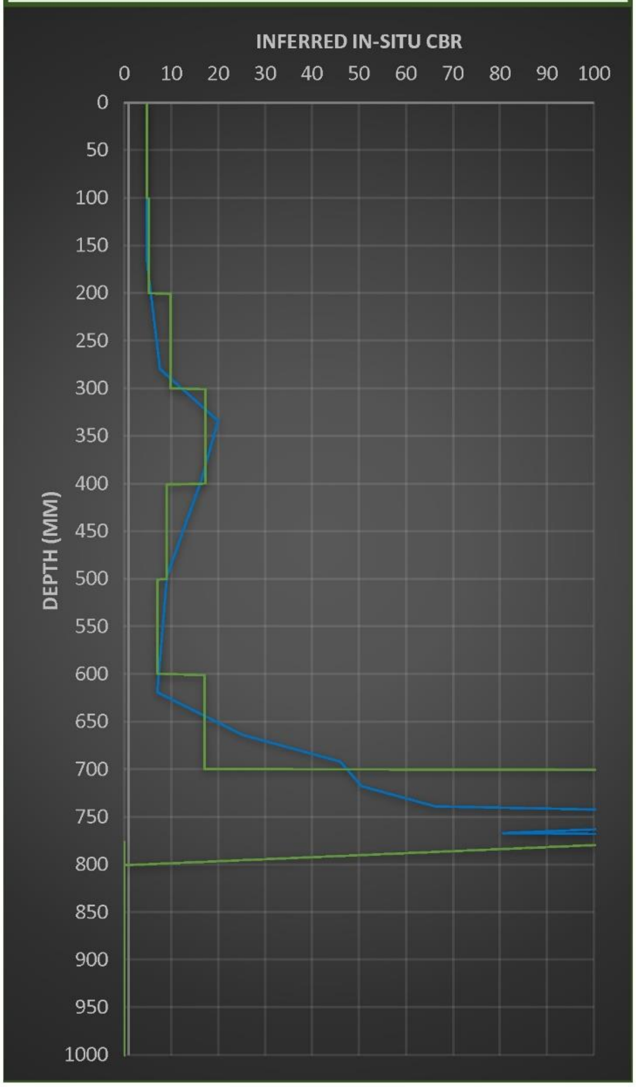
DCP conducted from E.G.L

Test Date: 25/01/2022

Number of blows vs penetration depth



In-situ CBR vs penetration depth



Penetration Rate (mm/blow) @ 100 mm intervals			
0 to 100	101 to 200	201 to 300	301 to 400
33,0	30,7	18,8	12,1
401 to 500	501 to 600	601 to 700	701 to 800
20,1	24,4	12,2	2,7

Average in-situ CBR @ 100 mm intervals			
0 to 100	101 to 200	201 to 300	301 to 400
4,8	5,3	9,9	17,4
401 to 500	501 to 600	601 to 700	701 to 800
9,1	7,1	17,1	117,8

Terminated Depth: 776 mm

Termination Character: Refusal

GEOCALIBRE GEOTECHNICAL CONSULTANCY

DCP Test DCP 2

GC-22-104

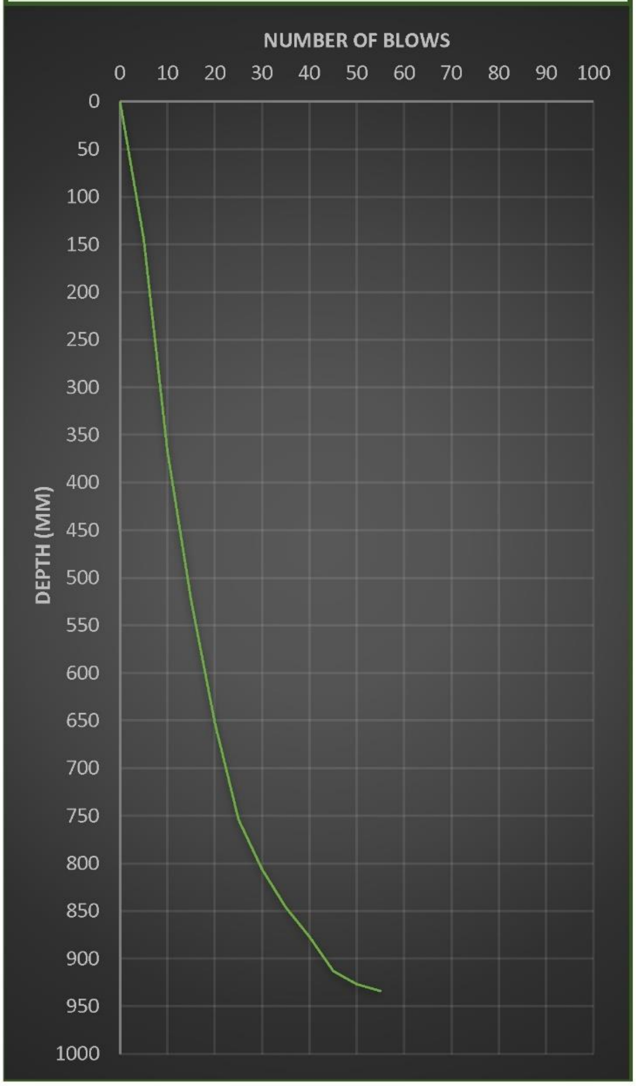
Smithfield Indoor Sports Center

BVi Consulting Engineers- Bloemfontein

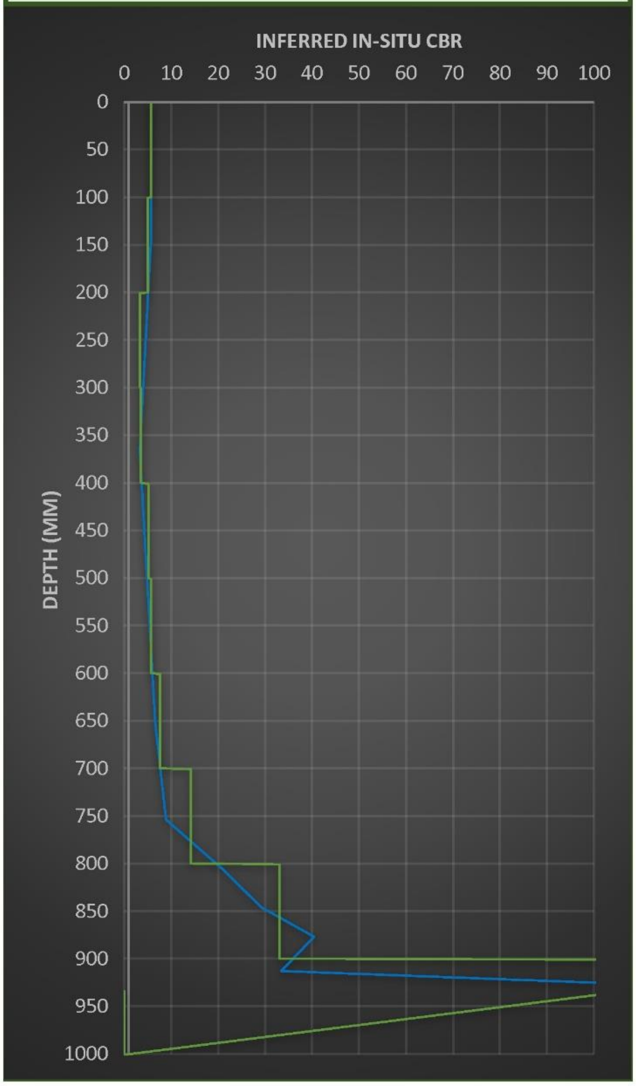
DCP 2

DCP conducted from E.G.L	Test Date: 25/01/2022

Number of blows vs penetration depth



In-situ CBR vs penetration depth



Penetration Rate (mm/blow) @ 100 mm intervals			
0 to 100	101 to 200	201 to 300	301 to 400
29,0	32,0	44,2	41,3
401 to 500	501 to 600	601 to 700	701 to 800
31,4	29,0	22,9	14,1

Average in-situ CBR @ 100 mm intervals			
0 to 100	101 to 200	201 to 300	301 to 400
5,7	5,0	3,3	3,6
401 to 500	501 to 600	601 to 700	701 to 800
5,1	5,7	7,7	14,2

Terminated Depth: 934 mm

Termination Character: Refusal

DCP Test DCP 3

GC-22-104

Smithfield Indoor Sports Center

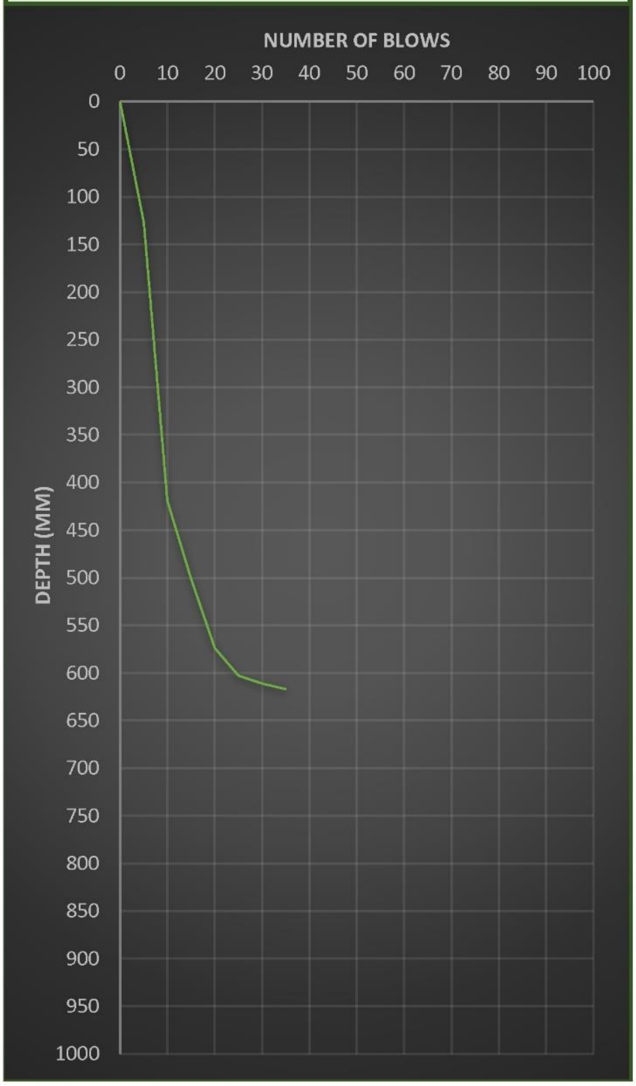
BVi Consulting Engineers- Bloemfontein

DCP 3

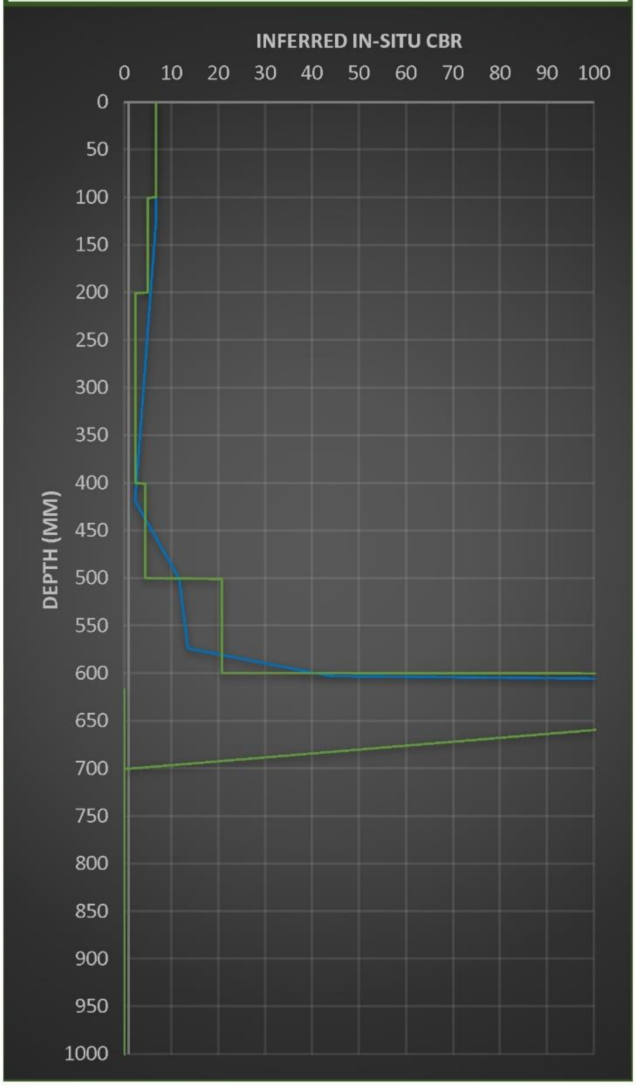
DCP conducted from E.G.L

Test Date: 25/01/2022

Number of blows vs penetration depth



In-situ CBR vs penetration depth



Penetration Rate (mm/blow) @ 100 mm intervals

0 to 100	101 to 200	201 to 300	301 to 400
25,4	32,0	58,4	58,4
401 to 500	**501 to 600**	**601 to 700**	**701 to 800**
34,5	10,5	1,6	

Average in-situ CBR @ 100 mm intervals

0 to 100	101 to 200	201 to 300	301 to 400
6,7	5,0	2,3	2,3
401 to 500	**501 to 600**	**601 to 700**	**701 to 800**
4,6	20,8	204,1	

Terminated Depth: 617 mm

Termination Character: Refusal

GEOCALIBRE
GEOTECHNICAL CONSULTANCY

DCP Test DCP 4

GC-22-104

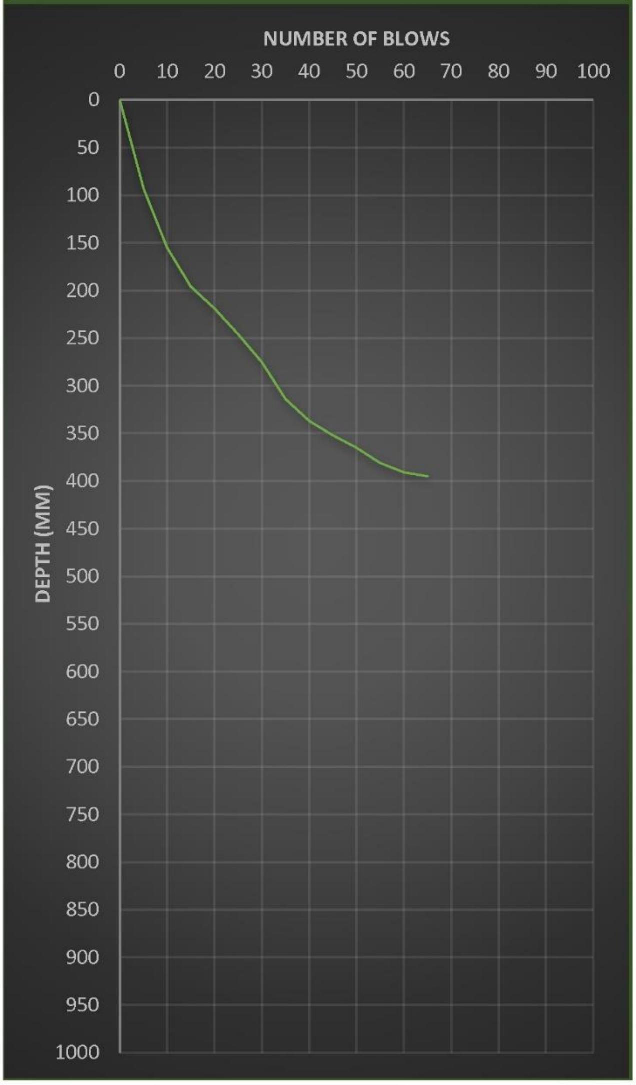
Smithfield Indoor Sports Center

BVi Consulting Engineers- Bloemfontein

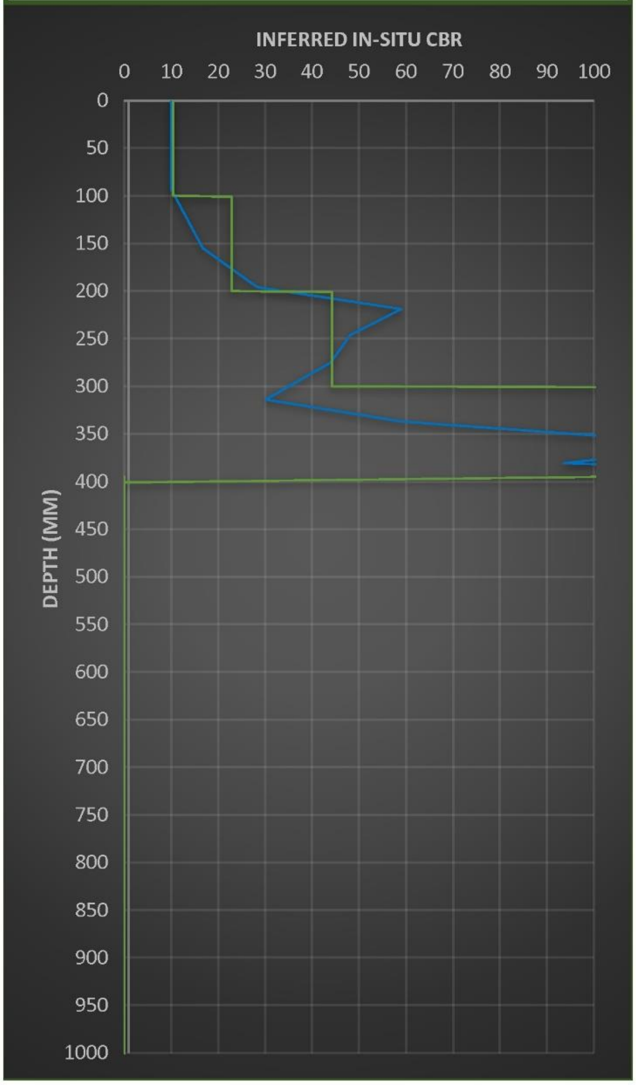
DCP 4

DCP conducted from E.G.L	Test Date: 25/01/2022

Number of blows vs penetration depth



In-situ CBR vs penetration depth



Penetration Rate (mm/blow) @ 100 mm intervals

0 to 100	101 to 200	201 to 300	301 to 400
18,0	9,7	5,8	3,0
401 to 500	501 to 600	601 to 700	701 to 800

Average in-situ CBR @ 100 mm intervals

0 to 100	101 to 200	201 to 300	301 to 400
10,5	22,9	44,3	102,1
401 to 500	501 to 600	601 to 700	701 to 800

Terminated Depth: 395 mm	Termination Character: Refusal

GEOCALIBRE
GEOTECHNICAL CONSULTANCY

DCP Test DCP 5

GC-22-104
Smithfield Indoor Sports Center

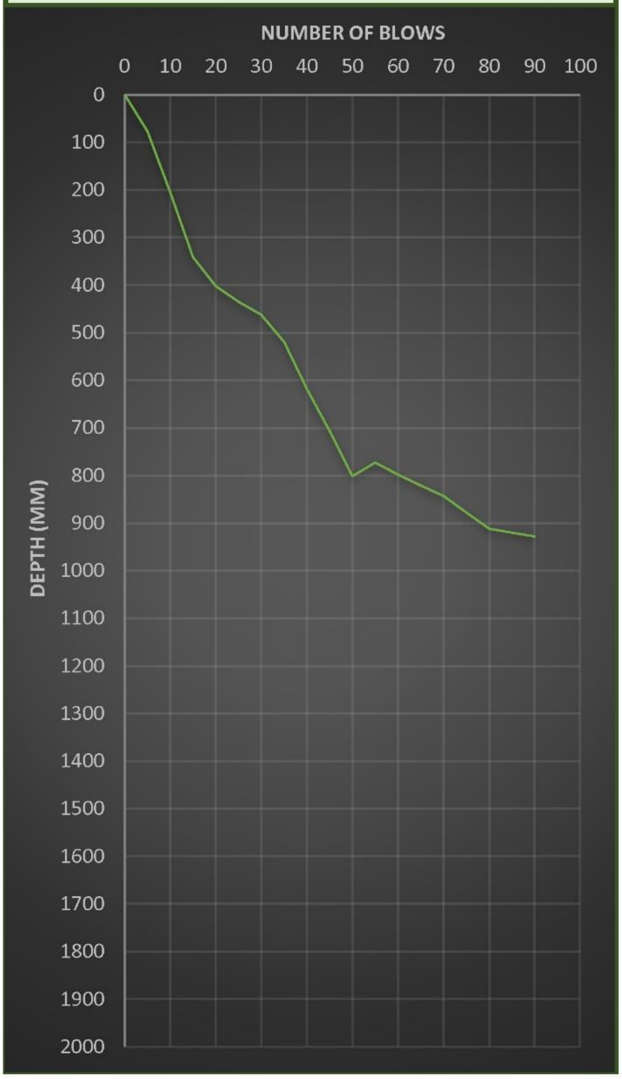
BVi Consulting Engineers- Bloemfontein

DCP 5 (2m)

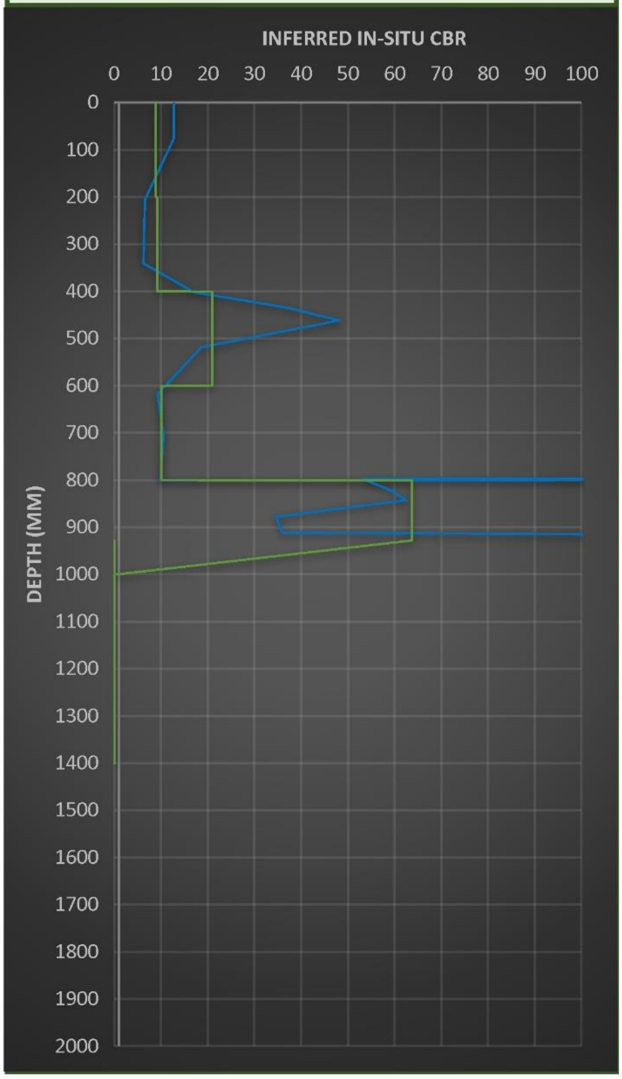
DCP conducted from E.G.L

Test Date: 25/01/2022

Number of blows vs penetration depth



In-situ CBR vs penetration depth



Penetration Rate (mm/blow) @ 200 mm intervals			
0 to 200	201 to 400	401 to 600	601 to 800
20,5	19,9	10,4	18,4
801 to 1000	1001 to 1200	1201 to 1400	1401 to 1600
4,3			

Average in-situ CBR @ 200 mm intervals			
0 to 200	201 to 400	401 to 600	601 to 800
8,9	9,2	21,0	10,1
801 to 1000	1001 to 1200	1201 to 1400	1401 to 1600
63,8			

Terminated Depth: 928 mm

Termination Character: Refusal

DCP Test DCP 6

GC-22-104

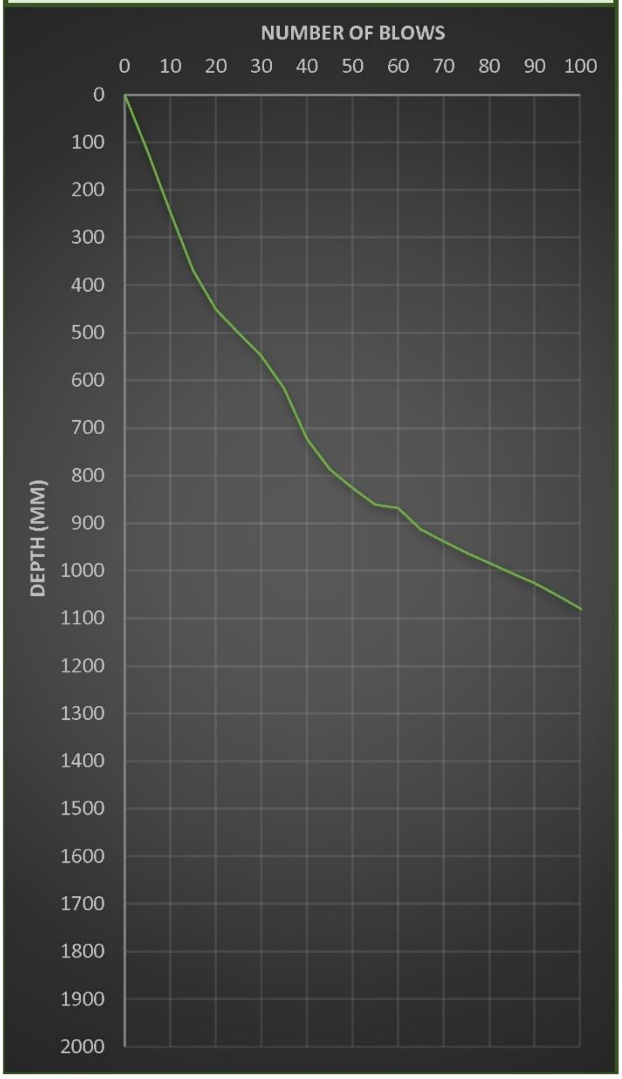
Smithfield Indoor Sports Center

BVi Consulting Engineers- Bloemfontein

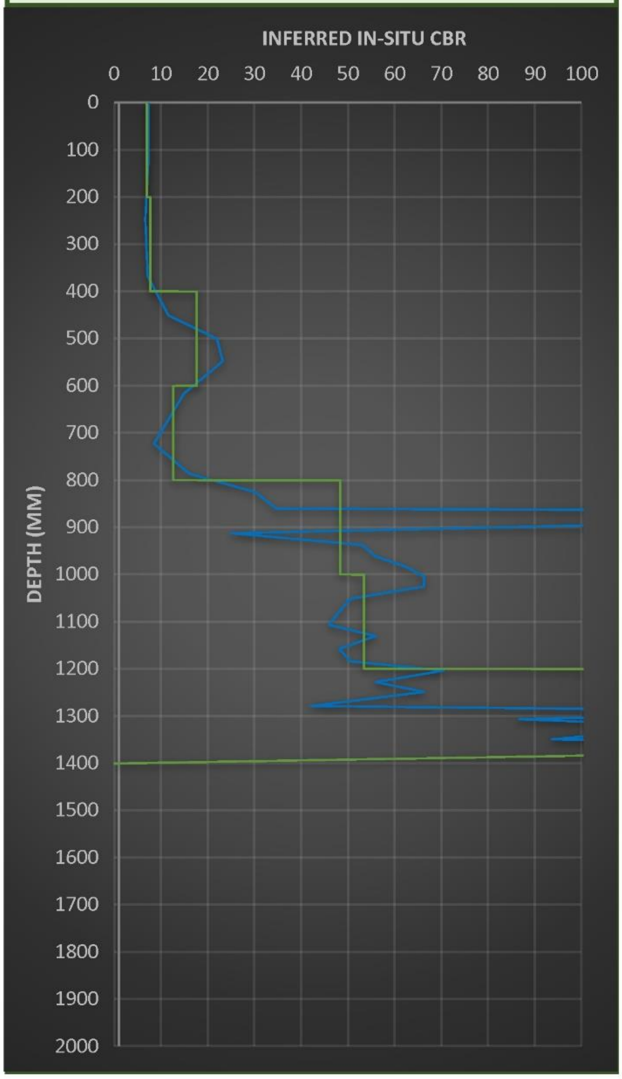
DCP 6 (2m)

DCP conducted from E.G.L	Test Date: 25/01/2022

Number of blows vs penetration depth



In-situ CBR vs penetration depth



Penetration Rate (mm/blow) @ 200 mm intervals

0 to 200	201 to 400	401 to 600	601 to 800
24,5	22,8	11,9	15,5
801 to 1000	**1001 to 1200**	**1201 to 1400**	**1401 to 1600**
5,4	5,0	3,0	

Average in-situ CBR @ 200 mm intervals

0 to 200	201 to 400	401 to 600	601 to 800
7,1	7,7	17,7	12,6
801 to 1000	**1001 to 1200**	**1201 to 1400**	**1401 to 1600**
48,3	53,4	100,9	

Terminated Depth: 1384 mm	Termination Character: Refusal

Appendix E

Site Classification Reference Table

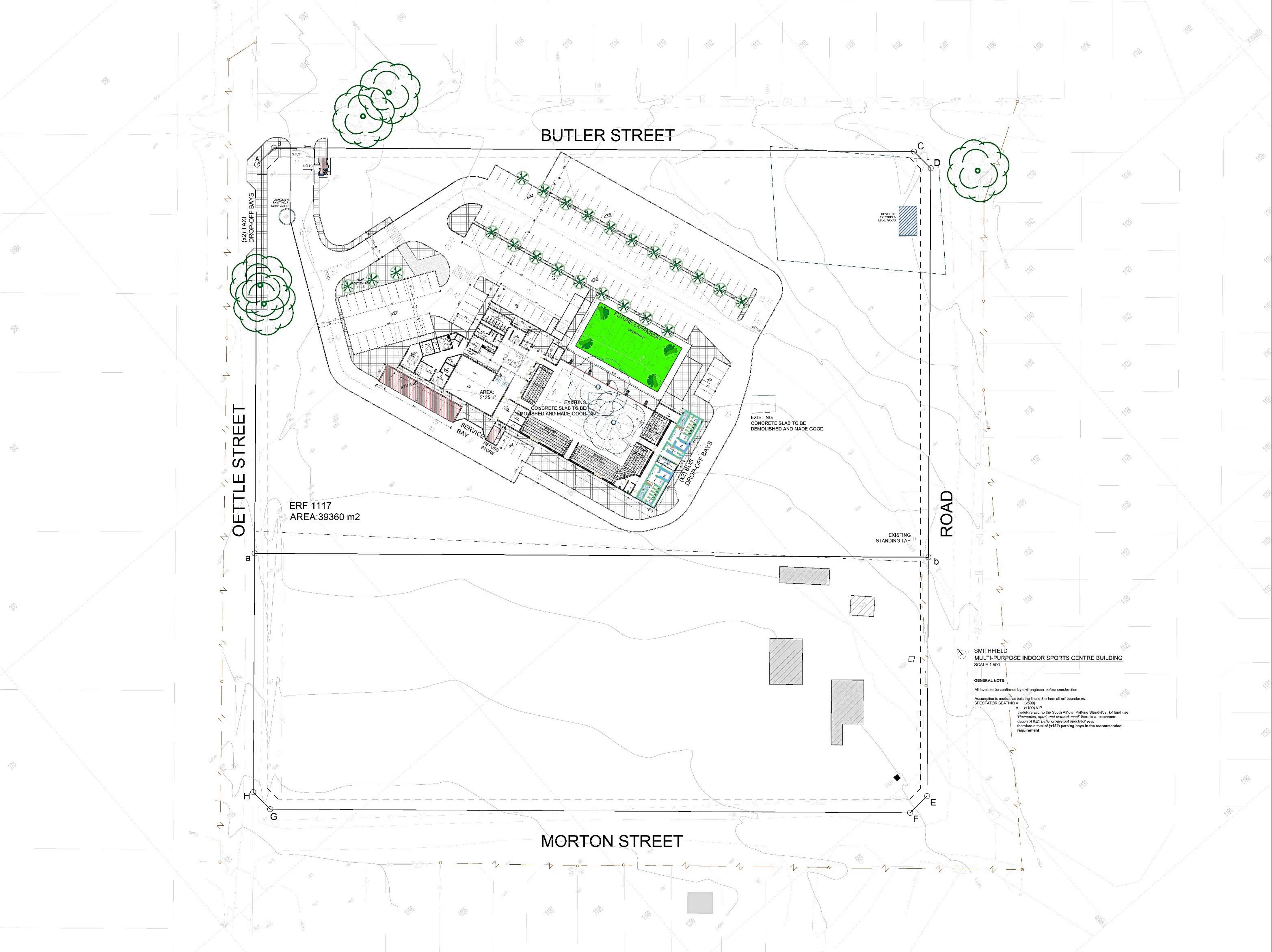
Table 7. Geotechnical Constraints in Urban Development (SANS 634:2012)

CONSTRAINT		DESCRIPTOR		
DESCRIPTION		1 (most favourable)	2 (intermediate)	3 (least favourable)
A	Collapsible soil	Any collapsible horizon or consecutive horizons totalling depth of less than 750 mm in thickness	Any collapsible horizon or consecutive horizons totalling depth of more than 750 mm in thickness	n/a
B	Seepage	Permanent or perched water table more than 1.5 m below ground surface	Permanent or perched water table less than 1.5 m below ground surface	Swamps and marshes
C	Active soil	Low soil-heave potential anticipated	Moderate soil-heave potential anticipated	High soil-heave potential anticipated
D	Highly compressible soil	Low soil compressibility anticipated	Moderate soil compressibility anticipated	High soil compressibility anticipated
E	Erodibility of soil	Low	Intermediate	High
F	Difficulty of excavation to 1.5 m depth	Scattered or occasional boulders less than 10% of total volume	Rock or hardpan pedocretes between 10% and 40% of total volume	Rock or hardpan pedocretes more than 40% of total volume
G	Undermined ground	Undermining at a depth greater than 200 m below surface	Old, undermined areas to a depth of 200 m below surface	Mining within less than 200 m of surface with total extraction
H	Stability (dolomite land)	Possibly stable	Potentially instable	Known sinkholes and dolines
I	Steep slopes	2-6 degrees	< 2 degrees or 6-18 degrees	> 18 degrees
J	Unstable natural slopes	Low risk	Intermediate risk	High risk
K	Seismic activity	10% probability of an event less than 100 cm/s^2 in 50 years	Mining-induced seismicity > 100 cm/s^2	Natural seismicity > 100 cm/s^2
L	Flooding	n/a	Adjacent to known drainage or channel with slope < 1%	Areas within drainage channel or floodplain

(After Partridge, Wood & Brink, 1993)

Appendix F

Site Layout Plan



SMITHFIELD
MULTI-PURPOSE INDOOR SPORTS CENTRE BUILDING
SCALE 1:500

GENERAL NOTE:

All levels to be confirmed by civil engineer before construction.

Assumption is made that building line is 3m from all erf boundaries.
SPECTATOR SEATING = (x500)
= (x100) VIP
therefore acc. to the South African Parking Standards, for land use *'Recreation, sport, and entertainment'* there is a recommendation of 0.25 parking bays per spectator seat
therefore a total of (x150) parking bays is the recommended requirement

Drawing Size A1

FOR INFORMATION

Revisions

IMPLEMENTING AGENT

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Regus Business Centre
Uni Park Building
062 Voddom Lane
Brandwag
Bloemfontein
9301

Postal
PO Box 71487
Bryanston
2021

Contact
tel: +2757 008 5205

Branches
Gauteng | Free State
Eastern Cape | Western Cape

SMITHFIELD: MULTI-PURPOSE INDOOR SPORT CENTRE
FOR
DEPARTMENT OF SPORTS, ARTS, CULTURE & RECREATION

ERF 1117, Mohokare Municipality

INFORMATION & ILLUSTRATIVE PURPOSES ONLY: CONCEPT SDP

21-100-MULTI-SPORT	SMITH SDP-000	REV A

SCALE	DATE	DESIGNED	DRAWN	CHECKED	STAGE
					INFO