SECTION 5.15: GEOTECHNICAL CHARACTERISATION	
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EXECUTIVE SUMMARY

Duynefontyn is a brown field site (the site) with two existing reactors of the Koeberg Nuclear Power Station (KNPS). Considerable investigative efforts went into the investigation and commissioning of KNPS. Although extensive, this pre-existing information has been supplemented with further detailed site-specific geotechnical investigations (drilling, geophysical investigations and laboratory testing) data analysis, the independent development of a probabilistic seismic hazard assessment (PSHA), monitoring and the construction of a three-dimensional geotechnical model. The additional investigations were carried out to meet current expectations with respect to geotechnical site characterisation and to address comments on the previous version of this report (SRK 2022/Rev1) from the National Nuclear Regulator (NNR). All of these efforts have been integrated to produce this section of this Site Safety Report (SSR).

Based on the outcomes of the geotechnical characterisation investigations, the following key conclusions are drawn:

- Geotechnical Profile:
    - The site is underlain by a poorly graded fine sand profile of *c.*25 m thickness consisting predominantly of aeolian deposits with a shallow groundwater table – this upper intergranular aquifer (the Sandveld Aquifer) is, in turn, underlain by rocks of the Malmesbury Group consisting primarily of greywacke and sandstone with interbedded siltstone, shale and mudstone units. These metasedimentary rocks have been upturned in the past and the bedding dips towards the coastline at *c.*75° with a strike of *c.*325°. Shearing/faulting and brecciated zones exist randomly with unknown orientations.
    - Characteristics of the sand profile are well investigated (historically and in the current investigation), and consist of an upper Bredasdorp formation, underlain by the Springfontein Formation and finally the Vaarswater Formation with occasional presence of completely weathered Malmesbury Group soils above the bedrock contact. These strata are poorly consolidated and the bulk of this horizon is saturated, resulting in these soils not presenting consistency of better than medium dense throughout the profile.
    - These soils have a mean shear wave velocity (Vs) of *c.*290 m/s (with a standard deviation of *c.*140 m/s), meaning that the minimum Vs = 304.8 m/s at the site will not be reliably met. Thus, a liquefaction potential risk is present over vast tracts of the site in soils below the groundwater table. It is anticipated that foundation improvement measures by mechanical means will not achieve bearing capacities of > *c.*200 kPa and it is probable that cement stabilisation of soils under critical infrastructure will be required (as was the case with the construction of a cement stabilised raft for the KNPS).

- The cement stabilised raft under KNPS is performing well and no liquefaction risk exists.
- The bedrock surface is a wave-cut platform, and due to the steep bedding dip, the bedrock surface manifests as regularly alternating variably metamorphosed greywacke/sandstone and interbedded shale layers. Investigating this buried surface in detail is not possible until the sands have been removed and the bedding surface can be mapped (as was the case when KNPS was developed). A secondary fractured rock aquifer (the Malmesbury Aquifer) exists in the site rocks.
- The sedimentary rocks, by depositional nature, present widely distributed parameters and it is postulated that exhaustive testing may even produce rock parameters trending towards a rectangular distribution. However, competent units (greywacke) dominate in large areas, and overall, the rock is described as ‘fair to good’ from a rock mass rating (RMR) perspective (42 per cent<RMR<86 per cent). Mean Vs is *c.*2000 m/s (standard deviation of *c.*700 m/s) and compression wave velocity (Vp) mean is *c.*3300 m/s (standard deviation of *c.*950 m/s).

- Interpretation of the geotechnical characteristics in the context of developing nuclear installation(s) at the site converge on the following:

  - Foundations of the nuclear island require a bearing capacity of at least *c.*720 kPa, meaning that these foundations will need to be carried through the poorly consolidated soils down to bedrock.
  - A large excavation requiring robust dewatering will therefore be required to found the nuclear island at construction stage. This is a proven strategy at the site as this was done for the KNPS, where an excavation was made and a cement stabilised raft backfilled to design founding levels. This founding strategy mitigates risks pertaining to liquefaction and bearing capacity for safety related structures within the nuclear island. Should excavations for founding be required into bedrock, detailed mapping of the exposed bedrock surface will be required once the excavation is made to reliably design such excavations. This was the strategy followed for the KNPS. Environmental impacts related to dewatering and disposal of large spoil volumes linked to the excavation must not be underestimated and designs will need to consider this important aspect.
  - Ancillary structure foundations (outside of the nuclear island) will require soil improvement measures, but it is estimated that improvement of the site soils by mechanical means will not realise bearing capacities > *c.*200 kPa. Individual structures will therefore require dedicated (localised) geotechnical investigations and foundation designs based on the outcomes of local conditions. The high groundwater table and liquefaction potential of the site soils will need to be accounted for.

- Care will need to be taken when designing founding in structures abutting the nuclear island to ensure that differential deformations (settlement and/or seismically induced deformations) are catered for considering that the nuclear island will have robust cement stabilised founding and abutting structures less robust founding (mechanical ground improvement).
- Rock excavation stability will be impacted by the orientation of cut faces relative to the bedding and primary joint sets, particularly if orientated within *c.*30° of bedding strike (*c.*325°). The inherent variability in rock quality will need to be carefully investigated to support design of such excavations, particularly if excavations into rock are required for safety related structures in the nuclear island (detailed mapping of the bedrock surface exposed by excavation will prove invaluable in this regard).
- Groundwater levels have received comment above, and the shallow groundwater table will need to consistently be catered for in geotechnical designs. Groundwater quality suggests that the site groundwater in the Sandveld (soils) and the Malmesbury (rock) aquifers is not anticipated to present aggressiveness towards concrete with low sulfate concentrations, but groundwater from both aquifers will be highly corrosive towards steel.

• Uncertainties:

- Persisting (from SRK, 2022 Rev1):
  - o Geotechnical characterisation of the offshore geotechnical profile is lacking, and should structures be required offshore, additional detailed geotechnical investigations will be required to support design.
- Removed (since SRK, 2022 Rev1):
  - o A draft probabilistic seismic hazard assessment (PSHA) report is available and the outcomes used to update liquefaction potential of site soils using updated assessment methods.

• Safety Comment:

- There are no new data to suggest that the geotechnical profile underlying the KNPS presents any safety concerns to the KNPS, and geotechnical performance monitoring of the cement stabilised soil foundation underlying the KNPS nuclear island indicates good performance of this founding solution.
- Sufficient geotechnical detail exists to suggest that development of nuclear installation(s) on this site will not present safety related challenges that cannot be mitigated by sound engineering.

- This SSR highlights those aspects of geotechnical design that may present safety risks and suggests rational engineering mitigations, many of which are proven for this site as they were applied to the development of the KNPS.

AMENDMENT RECORD			
Rev	**Draft**	**Date**	**Description**
0	2	4/07/13	Revised to comply with NBE comments – refer to responses in tracked changes version of this document as well as ‘responses to comments’ document.
0	3	25/03/2014	Included geotechnical parameters from existing references and added **(Brink, 1985)** from which additional geotechnical parameters are included.
1	0	27/01/2022	Updates to reflect NNR comments on the TSSR where relevant, additional fieldwork in support of broadening the site data footprint and assessing liquefaction potential.
1	1	30/03/2022	Revised to address Eskom comments on Draft 0
1a		15/03/2024	Revised to incorporate the PSHA and NNR comments

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5.15 GEOTECHNICAL CHARACTERISATION

5.15.1 Introduction

Duynefontyn (the site) is a brownfields site with two existing reactors for which extensive geotechnical data have been amassed over the years. The site has passed through a number of screening phases as described in ***Chapter 4*** (Site Investigation Approach) of this draft Site Safety Report (SSR) update. The existence of the Koeberg Nuclear Power Station (KNPS) on the southern extent of the site is testament to the site having undergone rigorous past investigation. The detailed investigations and the development of an SSR on the engineering geological characteristics and geotechnical profile for both the KNPS and the Duynefontyn site (north of KNPS), and comments on the suitability of the northern portion of the site are presented in this section. Historical information is accordingly supplemented in this section with additional site investigations, *in situ* and laboratory testing of the soils and rocks, the draft probabilistic seismic hazard assessment (PSHA) and groundwater level observations. These historical and new data are also presented and interpreted to support the suitability of the site and the potential impacts of the geotechnical characteristics of the site on nuclear installation safety, as prescribed in ***Section 5.15*** of the Technical Specification **(Eskom, 2010)**.

The investigations aimed at identifying potential geotechnical hazards at the site, taking into account the groundwater conditions at and around the site. Specific consideration is given to the rock and soil characterisation, potential hazards such as fault offsets, landslides, cavernous rocks, ground subsidence and soil liquefaction[2]. When a new nuclear installation(s) or the plant parameter envelope (PPE) are discussed in ***Section 5.15***, it is highlighted that the PPE is based on the UK EPR (European Pressurized Reactor – a NPS by EDF Energy and AP1000 (a NPS by Westinghouse) – consideration of any other technology shall require update of the PPE and this Section to investigate and document further evaluation of the external events.

This section is developed in collaboration with, and complements ***Section 5.11*** (Geohydrology). Integration with ***Section 5.13*** (Geology) and

[2]When a new nuclear power station (NNPS) or the plant parameter envelope (PPE) are discussed in this Section, it is highlighted that the PPE is based on the UK EPR (European Pressurized Reactor – a NPS by EDF Energy and AP1000 (a NPS by Westinghouse) – consideration of any other NNPS shall require update of the PPE and this Section to investigate and document further evaluation of the external events for other NNPS.

Section 5.14 (Seismic Hazard) in updating this section (***Section 5.15***) has been achieved in considering the approved 2024 Probabilistic Seismic Hazard Assessment (PSHA) **(Stamatakos & Watson-Lamprey, 2024)**. .

5.15.2 Purpose and Scope

Within the framework of the safety requirements and criteria presented in ***Subsection 5.15.4*** and in ***Appendix 5.15.A***, the purpose of this chapter is to:

- present the baseline geotechnical characteristics of the site (also referred to as the geotechnical profile) which describes the site capacity;
- make an assessment of how the site is likely to respond to loading or external events (demands);
- assess whether further development of nuclear installation(s) on the site is viable/feasible and whether such developments impact on the safe operation of the KNPS;
- describe mitigating measures (design principles) that may be considered to ensure safety in the context of a nuclear installation(s). These mitigation measures are based on the geotechnical characteristics identified at the site.

This section presents geotechnical characteristics of the site highlighting:

- overburden soils, their properties and distribution;
- underlying rocks, their properties and distribution;
- the position of the groundwater table relative to the soil/rock profile;
- an assessment of the soil and rock capacity under induced loads (i.e. when the nuclear installation(s) is built) during the nuclear installation lifetime;
- potential areas of concern and measures for improvement of the foundations and/or founding materials of the nuclear installation(s);
- monitoring programme/s for geotechnical parameters.

Detailed description of the activities carried out during the geotechnical characterisation of the site, including the detailed results achieved, are presented in the appendices of this section. The investigation of the site was conducted to demonstrate whether or not the geotechnical characteristics (capacity) support the acceptability of the site (on demand), hence addressing the site suitability.

As described in ***Subsection 5.15.4***, the geotechnical characterisation of the site follows a four-stage approach. This SSR update presents a summary of the results of the selection stage and confirms suitability of the southern portion of the site where the KNPS is situated. It further details the investigations carried out during the characterisation stage that integrates new data collated on the northern parts of the site with historical data relevant to KNPS. The pre-operational and operational stages relative to the northern parts of the site are future stages and it should be appreciated that the geotechnical characterisation for this portion of the site will evolve through all of the stages. It should also be pointed out that not all of the criteria and requirements (relative to the northern portion of the site) in ***Subsection 5.15.4*** are addressed in this SSR update. This is primarily due to the fact that certain design details of an additional proposed nuclear installation(s) were unknown at the time of writing this report.

This section exclusively addresses the selection and characterisation stages of the geotechnical investigation for this SSR update and serves no other purpose. It also does not address all of the requirements stipulated in ***Subsection 5.15.4*** where design details of proposed nuclear installation(s) are not yet known.

The geotechnical characteristics of the soils and rocks on the site apply to and inform only those aspects identified explicitly in this SSR update and no other aspect or condition that may be considered in, or affect subsequent stages of the project.

Additional data gathering will occur (and will be comprehensively addressed) in future investigations (see ***Subsection 5.15.4***) and present detail that is pertinent to design and implementation of a nuclear installation(s). These exclusions do not impact on this SSR update presenting sufficient evidence to make conclusive comment on the site suitability.

5.15.3 Regulatory Framework

The specific regulation relevant to the investigation of geotechnical site characteristics is The Regulations on Licensing of Sites for New Nuclear Installations (Department of Energy, 2010), Regulations 3 (2) (a) and 5 (3) requiring the submission of a site safety report and the site characterisation content thereof and 4 (5) accounting for natural phenomena and potential man-made hazards.

The national regulations specifically relevant to geotechnical investigations for an SSR are:

- The Regulations on Licensing of Sites for New Nuclear Installations (Department of Energy, 2010) Regulations 3 (2) (a) and 5 (3) requiring the submission of a site safety report and the site characterisation content thereof and 4 (5) accounting for natural phenomena and potential man-made hazards.
- PP-0014: Considerations for External Events for New Nuclear Installations (National Nuclear Regulator, 2012), specifically section 11.1 (4) Geological and Geotechnical hazards.

Since the above regulations are high level and do not explicitly target geotechnical characterisation of a nuclear site, the following supporting national legislation was used to further design the investigations done to produce this SSR:

- RG-0011: Interim Guidance on the Siting of Nuclear Facilities, Rev 0 (National Nuclear Regulator, 2016), specifically sections 7.2.1 (Seismic and geological considerations), 7.2.4 (Geotechnical hazards);
- RG-0016: Requirements for Authorisation Submissions Involving Computer Software and Evaluation Models for Safety Calculations (National Nuclear Regulator, 2006);
- PP-0014: Considerations for External Events for New Nuclear Installations (National Nuclear Regulator, 2012), specifically sections 11.1 (4) Geological and Geotechnical hazards;
- Eskom's Technical Specification for Site Safety Reports, NSIP01388 (Rev 0). Section 5.15: Geotechnical Characterisation (Eskom, 2010).

5.15.4 Reference Documents and Guides

Apart for the Technical Specification (Eskom, 2010), the above regulations are not specific in terms of geotechnical characterisation and so the technical specification and safety guides listed below were used to shape the approach to the geotechnical characterisation of the site:

- International Atomic Energy Agency (IAEA) Safety Guide No. NS-G-3.6, Geotechnical Aspects of Site Evaluation and Foundations for Nuclear Installations (International Atomic Energy Agency, 2004);

- United States Nuclear Regulatory Commission (US NRC) Regulatory Guide 1.132: Site Investigations for Foundations of Nuclear Power Plants (United States Nuclear Regulatory Commission, 2003);
- US NRC Regulatory Guide 1.138: Laboratory Investigations of Soils and Rocks for Engineering Analysis and Design of Nuclear Power Plants (United States Nuclear Regulatory Commission, 2003);
- United States Nuclear Regulatory Commission (US NRC), NUREG-0800 (United States Nuclear Regulatory Commission, 2007).

These guidelines, along with the regulations, present a framework in which the following should be presented in an SSR:

- identification and specification of the geotechnical characteristics of the site (i.e. the site capacity) in terms of external events of natural origin or human induced (i.e. demand) occurring in the region of the proposed site (United States Nuclear Regulatory Commission, 2007);
- monitoring of current and future uncertainties discussed in this SSR (United States Nuclear Regulatory Commission, 2007);
- events (or demands) that potentially can lead to radioactive exposure to be considered as part of the design features of the nuclear installation(s) (United States Nuclear Regulatory Commission, 2003).

(International Atomic Energy Agency, 2004) recommends that site characteristics that may affect the safety of the nuclear installation(s) be investigated and assessed. Characteristics of the natural environment in the region[3] that may be affected by potential radiological impacts in operational states and accident conditions shall also be investigated. All these characteristics shall be observed and monitored throughout the lifetime of the nuclear installation(s). In addition, proposed sites for nuclear installations shall be examined with regard to the frequency and severity of external natural and human induced events and phenomena that could

[3] The region is not clearly defined in this reference document, but in the context of this section, is assumed to be those areas on and around the site in which the geotechnical characteristics may have an influence on design or may have the potential to contribute to transporting radioactive materials away from the power plant.

affect the safety of the installation (***Chapter 6***, Evaluation of External Events). The foreseeable evolution of natural and human made factors in the region that may have a bearing on safety shall be evaluated for a time period that encompasses the projected lifetime of the nuclear installation(s).

(International Atomic Energy Agency, 2004) recommends development and implementation of the geotechnical site investigation programme in four stages, broadly described below:

- selection stage - One or more preferred candidate sites are selected after investigation of a large region, rejection of unsuitable sites, and screening and comparison of the remaining sites.
- characterisation stage (the stage relevant to this SSR) - This stage is further subdivided into:
- Verification, in which the suitability of the site to host a nuclear power plant is verified mainly according to predefined site exclusion criteria.
- Confirmation, in which the characteristics of the site necessary for the purposes of analysis and detailed design are determined[4].
- pre-operational stage[5] - Studies and investigations begun in the previous stages are continued after the start of construction and before the start of operation of the plant to complete and refine the assessment of site characteristics. The site data obtained allow a final assessment of the simulation models used in the ultimate design.
- operational stage[6] - Selected investigations are pursued over the lifetime of the plant.

(International Atomic Energy Agency, 2004), in essence, therefore, provides specific recommendations on the data needed, investigations, description of the geotechnical profiles and the parameters necessary for the geotechnical analysis and also for the design of nuclear installations.

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[4] A basis for conducting the characterisation stage is that detailed layouts have been finalised – this is not the case for this SSR update as only generalised layouts are known and as such, the characterisation stage is not completed in full

[5] This stage is not relevant to this SSR update as it only concerns the characterisation stage.

[6] This stage is not relevant to this SSR update as it only concerns the characterisation stage.

Consideration was also given to the US regulatory guidance (United States Nuclear Regulatory Commission, 2003), that specifies the geotechnical data needed for the design of nuclear installation foundations. The required scope of the laboratory testing programme for identification and classification of soil and rock, and the evaluation of the physical and engineering properties are specified by NUREG (United States Nuclear Regulatory Commission, 2003). Laboratory testing relevant to the characterisation phase has accordingly been implemented in this SSR.

It is also required by NUREG (United States Nuclear Regulatory Commission, 2007) that in the description of properties of underlying materials, state-of-the-art methods are used to determine the static and dynamic engineering properties of all foundation soil and rocks in the site area. Summary tables are also needed that catalogue the important results. A detailed discussion of laboratory sample preparation is required, in particular when there are critical samples. An additional requirement is the detailed discussion of the criteria used to determine that the samples are taken properly and sufficient number tested to define all critical soil parameters at the site, together with their potential variability. (United States Nuclear Regulatory Commission, 2007) also requires evaluation and presentation (primarily in the context of design) of:

- the relationship of foundations and underlying materials;
- dynamic characteristics of the soil or rock;
- data concerning the excavation, backfill, and earthwork analyses;
- analysis of groundwater conditions[7];
- response of soil and rock to dynamic loading;
- analysis of liquefaction potential;
- earthquake design basis (summary)[8];
- static analyses;

[7] Covered in ***Section 5.11***.

[8] Covered in ***Section 5.14.***

- techniques to improve subsurface conditions.

NUREG (United States Nuclear Regulatory Commission, 2003), requires that slope characteristics, design criteria and analysis, results of investigations including boring, shafts, pits, trenches and laboratory tests, and properties of borrow material, compaction and excavation specifications are presented in the license application for siting and construction of a nuclear installation(s). Not all of these issues have been addressed in great detail in this report for various reasons stated throughout the text.

Compliance of the site characteristics with the above safety requirements relevant to the site characterisation stage is evaluated in this section of this SSR.

Liquefaction potential for the site was evaluated in terms of the methodologies proposed by the National Centre for Earthquake Engineering Research (NCEER) (National Centre for Earthquake Engineering Research, 2001).

5.15.5 Historical and Desktop Research Data

The site is situated on the Cape West Coast approximately 25 km north of Cape Town along the R27 West Coast Road within the coastal plain of the Western Cape Province. The site incorporates the KNPS site and an area immediately to the north of the existing KNPS (the Duynefontyn Site).

Dunes, stabilised by vegetation and recent unconsolidated dunes, occupy large areas of the site. This Sandveld rises gently towards the east and southeast to an average elevation of between 100 and 200 m some 20 km east of the site. The south-eastern margin is demarcated by the Tygerberg Formation of the Malmesbury Group, whilst granites of the Darling Range intersect the coastal plain in the north and Blouberg Hill forms a prominent feature some 10 km to the south of the site. A few islands are present in the Atlantic Ocean within a 20 km radius, the most notable being Robben Island, 3.0 km × 1.5 km in extent and situated approximately 8 km west of the site.

The local site is typified by low lying dunes with the majority of the site covered by aeolian sand. The underlying material comprises recent marine sand beach deposits. There is no rock outcrop at surface. The dunes are slightly higher immediately adjacent to the beach. The area behind the coastal dunes forms a shallow plateau, colonised by grasses, scrub and fynbos. The site is at an elevation typically of about 0 to 20 metres relative to mean sea level (m amsl).

The site has undergone many investigations pre and post the development of the KNPS. **(Brink, 1981)**, **(Brink, 1985)** and **(Eskom, 2006)** are a select few of the geotechnical/geological studies carried out at the site. These references form the backdrop to the summary of available data that follows. These historical investigations focused on many aspects of geotechnics, but in essence, the following data were reviewed within their collective context:

- topographic maps;
- geological maps;
- geological reports and other geological literature;
- geophysical references;
- geotechnical reports and other geotechnical literature;
- geohydrological reports.

However, these historical investigations did not provide sufficient detail in the northern areas of the site to provide the required level of information and understanding of this portion of the site characteristics, justification of the suitability of this portion of the site and hence the development of an SSR encompassing this portion of the site. ***Subsection 5.15.6*** covers this shortfall.

5.15.5.1 **Landform – Geomorphology** (Illenberger, 2010)

The dunes at Duynefontyn form part of the Atlantis corridor dunefield, and the dune varieties found are mobile transverse dunes, transverse dunes artificially stabilised with alien vegetation, and naturally vegetated parabolic dunes. Groundwater only "daylights" at Duynefontyn in one or two small ephemeral interdune hollows, so there are no significant impacts related to the interaction between groundwater and dune dynamics at this site.

Access roads and transmission lines can be built across the mobile dunes with medium operational impacts (easily mitigated with drift fences and pioneer indigenous vegetation) and other dunes with insignificant operational impacts.

Topsoil and spoils stockpiles located on the mobile dunes will have medium operational impacts (easily mitigated with drift fences and pioneer indigenous vegetation) and other dunes with low operational impacts.

At Duynefontyn, 25 percent of the specific variety of mobile dunes will be lost if nuclear installation(s) are developed, but this is not a fatal flaw in terms of safety and is more of an environmental impact consideration.

5.15.5.2 Geology

The local geology beneath the site comprises Quaternary-age soils overlying rocks of the Tygerberg Formation of the Malmesbury Group and this is summarised in ***Table 5.15.1***. The bedrock underlying the footprint area of the KNPS, which was exposed during construction of that facility, is described in **(Brink, 1981)** together with a detailed geological map, cross-section and aerial photography of the exposed formation. **(Brink, 1981)** is a case history and notes the following:

- Bedrock comprises a variety of rocks of the Tygerberg Formation, Malmesbury Group which has been variably metamorphosed (steeply dipping, interlaminated and bedded succession of greywackes/metagreywackes, shales, siltstones and mudstones with occurrences of phyllites and schists; with a dominant strike north-northwest-south-southeast).
- Beds grade from coarse to fine in upward fining successions with abundant synformational bedding sequences and rocks are considered typical of a marine turbidite succession.
- Bedrock is described as a fluted, wave cut terrace of Pre-Tertiary age situated at an average -10 m msl.
- The degree and depth of weathering is described as highly variable with un-weathered greywacke present within 6 m of bedrock surface but weathering in the mudstones and siltstones extending to 26 m depth in places. Around fault zones, the rock is brecciated and weathering extends to ‘great depth’.
- The rocks are intensely jointed and often sheared along fault planes whilst quartz veins, pyrite and clay gouge are ubiquitous along joints and faults especially where the wall rocks to faults are brecciated.
- The rock was deeply weathered in the eastern section of the KNPS excavation but largely unweathered on the western side.
- The age of the overlying sand was dated and it was concluded that there has been no significant movement on any of the joints or faults within the last two million years.

- Permeability at depths of between 3 and 17 m into rock is in the range 6×10^{-6} to 9×10^{-9} m/s, and artesian water pressure was recorded in several boreholes drilled into bedrock – artesian head was estimated to be 4 m msl.
- The characteristics of seismic wave propagation through bedrock revealed relatively lower values to a depth of 70 m below ground surface.
- Tunnelling for cooling water supply was ruled out due to the high permeability and fractured nature of the rock[9].

[9] Further nuclear installation(s) development on this site present (in concept) a high risk of recirculation of cooling water back to the existing KNPS intake hence tunnelling is an option that cannot be ignored and this is the reason for this option being addressed in some detail in this SSR.

Table 5.15.1
Regional Geology

<table>
<tr><th colspan="2"></th><th>Lithology</th><th>Formation</th><th>Group</th></tr>
<tr><td rowspan="5">Quaternary</td><td rowspan="2">Holocene</td><td>Sandy soil, poorly sorted and slightly clayey</td><td>-</td><td></td></tr>
<tr><td>Aeolian/calcareous quartzose sand</td><td>Witsand</td><td rowspan="6">Sandveld</td></tr>
<tr><td rowspan="3">Pleistocene</td><td>Aeolian/calcrete-capped calcareous sandstone</td><td>Langebaan</td></tr>
<tr><td>Littoral coquina and sandstone</td><td>Velddrif</td></tr>
<tr><td>Aeolian/quartzose sand with intermittent peaty layers</td><td>Springfontyn</td></tr>
<tr><td rowspan="2">Neogene</td><td>Pliocene</td><td>Quartzose and muddy sand, and shelly gravel, phosphate-rich</td><td>Varswater</td></tr>
<tr><td>Miocene</td><td>Silcrete</td><td>Bellville</td></tr>
<tr><td colspan="2">Jurassic</td><td>Dolerite</td><td>Karoo dolerite</td><td></td></tr>
<tr><td colspan="2" rowspan="2">Ordovician to Lower Carboniferous</td><td>Sandstones and shales</td><td>-</td><td>Cape Supergroup</td></tr>
<tr><td>Dolerite</td><td>Pre-Cape dolerite dykes</td><td></td></tr>
<tr><td colspan="2">Cambrian</td><td>Sandstones and conglomerates</td><td>Klipheuwel</td><td></td></tr>
<tr><td colspan="2" rowspan="2">Late Precambrian</td><td>Granites with hybrid and porphyritic varieties;diorites; augen gneisses</td><td>-</td><td>Cape Granite Suite</td></tr>
<tr><td>Greywacke, sandstone, mudstone and shale; metamorphosed equivalents.</td><td>-</td><td>Malmesbury</td></tr>
</table>

(Eskom, 2006) contains a detailed assessment of the geology of the area and this has been summarised briefly in the following sub-sections.

5.15.5.2.1 Regional Geology

The site is situated in the Tygerberg Terrane on an anticline formed by rocks of the approximately 950 Ma old Malmesbury Group. The consolidated hard-rock geology of the region ranges in age from the Precambrian to the Palaeozoic. The rocks present within the region comprise the Malmesbury Group and Klipheuwel Formation, post-Malmesbury intrusive Cape Granites and the lower parts of the Cape Supergroup.

The Tygerberg Formation of the Malmesbury Group and granite from the Cape Granite Suite that intrudes the Malmesbury rocks comprise most of the bedrock on which the younger Quaternary- Cenozoic sediments of the Western Cape were deposited. The sandstones of the Table Mountain Group (TMG) of the Cape Supergroup form the highland areas in the interior, east of the coastal plains.

The Malmesbury Group is predominantly a marine sedimentary assemblage, showing great variation of litho-facies. The Malmesbury orogeny commenced with the folding of the Malmesbury sediments into synclines and anticlines around an almost horizontal, northwest-striking fold axes.

Table 5.15.1 summarises the geological formations present in the south-western Cape Province in approximate chronological order.

5.15.5.2.2 Site Response

The following brief overview is extracted from **(Eskom, 2006)**. The Tygerberg Terrane is separated from the adjacent Swartland and Boland terranes by the Saldanha–Franschhoek and Piketberg–Wellington fault zones, respectively.

The Saldanha–Franschhoek Fault Zone is approximately 30 km to the east of the site. The Malmesbury rocks, due to their tectonic history, are extensively faulted and fractured, as excavations for the KNPS revealed. ***Figure 5.15.1*** **(Eskom, 2006)** represents this graphically.

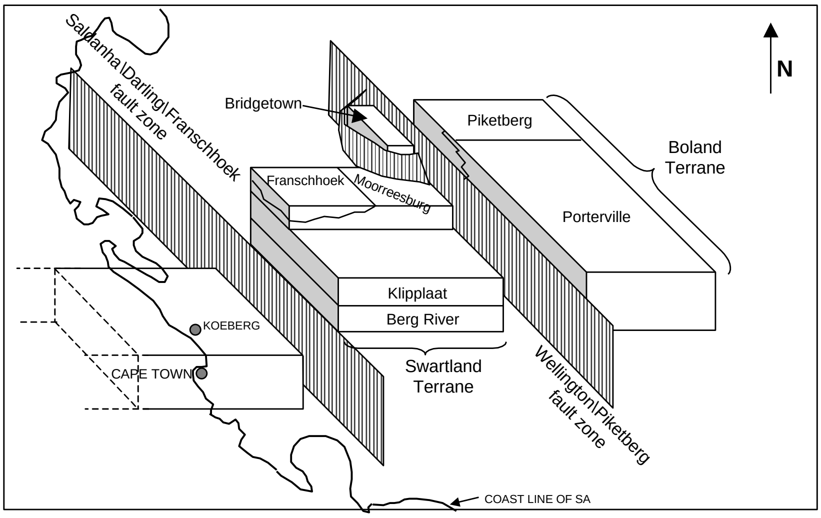


Figure 5.15.1
Three Dimensional Representation – WC Regional Geology

Extensive faults with a north-northwest strike are known to exist in the Malmesbury Group. The present day stress field generated at the mid−Atlantic ridge is at roughly right angles to these faults and is larger than the stress field generated at the southern African plate margin. Since the inclination of the stress field is also sub−horizontal, the stress field locks these faults into position and negates movement along them.

There is some controversy as to the existence or otherwise of the postulated Cape Hangklip-Milnerton Fault. **(Eskom, 2006)**, indicated a postulated fault between Silver Sands and Rooiels on the Cape Hangklip peninsula, but the PSHA study showed the presence of this risk to be inconclusive. Should this fault exist, its postulated north-northwest extension would be the nearest fault to the site. The KNPS was designed for this, and the possibility of a magnitude 7.0 seismic event on this potential feature was considered. In contrast, however, **(Eskom, 2006)** states that, from the evidence of rock boring lamellibranches, the site has not been the subject of destructive tectonic forces in the last five million years.

During the preparation of **(Eskom, 2006)**, existing data were reviewed and updated with new information including additional geophysical coverage. The previous data were also supplemented and re-interpreted utilising more modern software and a new seismic hazard assessment was undertaken using a probabilistic basis rather than the earlier deterministic approach. The conclusion of these studies was that the design basis for the KNPS remained valid.

5.15.5.2.3 Site Specific Geology

The geology of the Precambrian-age bedrock and superficial deposits of the site area was the subject of comprehensive investigations prior to construction of the KNPS.

Extracts from the findings of these investigations and considered pertinent to this SSR follow.

(a) Stratigraphy and Lithology

The stratigraphic succession at the site comprises the following elements (***Table 5.15.2***) as revealed from drill cores and subsequent excavations.

Table 5.15.2
Lithology at the KNPS Site

Formation	Stratigraphic Succession	
Witsand Formation	(Members not identified)	Top
Langebaan Formation	Atlantis Dune Member Milnerton Beach Member	
Springfontyn Formation	Upper Arenaceous Member Lower Gastropod Member	
Varswater Formation	Peaty Sand Member Upper Bioturbated Sand Member/Shark Tooth Bed Lower Arenaceous Member Basal Gravel Member	
Tygerberg Formation	(Members not named)	Base

The succession listed in ***Table 5.15.2*** is described below from the base upwards:

- Tygerberg Formation:
  - The bedrock comprises a variety of rock types of the Tygerberg Formation and include steeply dipping, interlaminated, variably weathered, jointed greywackes, siltstones and mudstones and

metamorphosed equivalents with a north-northwest – south-southeast strike (*c.325*°).

- Gradational sequences and contacts are characteristic, and the beds grade mainly from coarse to fine in grain size in upward-fining successions.
- Argillaceous horizons, interbedded with the greywacke occur in the site area, and are normally less massive – they are further distinguished from the greywacke by a fine-grained even texture, grey to grey/green colour and a generally more noticeable bedding.
- Distinct lithological boundaries are not always present and gradational changes are common in borehole cores, although it must be noted that vertical boreholes drilled into steeply inclined bedding will not efficiently intersect bedding transitions except at high angles.
- The bedrock surface consists of a fluted, wave-cut terrace located at approximately 10.0 m msl.

• Varswater Formation:

  - This formation rests unconformably on the wave-cut terrace and consists of ubiquitous basal gravels overlain by a succession of fine sands.
  - The Basal Gravel Member comprises well-rounded to angular pebbles of quartzite and vein quartz set.
  - The Lower Arenaceous Member consists of fine sandstones characteristically composed of well-sorted, fine-grained sands and polished quartz grains accompanied by phosphatised shell fragments and primary structures are weakly developed, consisting mainly of sub-horizontal bedding planes.
  - An undulating boundary separates the 1.5 to 1.8 m thick Lower Arenaceous Member from the Upper Bioturbated Sand Member, which is about 9.5 m thick and is completely bioturbated.
  - In contrast to the rest of the members, the lower portion of the Upper Bioturbated Sand Member contains large numbers of sharks' teeth, fish (vertebrae, teeth, scales, and spines) and whale debris (vertebrae, ear bones, and ribs).
  - Overlying the Upper Bioturbated Sand Member is the Peaty Sand Member, which varies in thickness from 10 to 15 cm and is present throughout the excavation areas at the KNPS site.

• Springfontyn Formation:

- This formation is continuous throughout the KNPS excavation and is sub-divided into two members namely 1) a lower Gastropod Member and 2) an Upper Arenaceous Member;
- The lower Gastropod Member is composed of fossiliferous fine, medium and coarse sand with scattered fine gravels.
- The upper Arenaceous Member comprises mainly well-sorted sand of fine to coarse, grain size.

- Langebaan and Witsand formations - The Langebaan and Witsand formations have been identified at the site.

(b) Structural Geology and Tectonics

Except for the southwestern corner of the KNPS SEC pump house excavation, the bedding beneath the KNPS dips steeply west-southwest at angles of about 75° with a strike varying between 320° and 330° from true north **(Brink, 1981)**.

In the southwestern corner of the KNPS pump house excavation, a portion of a synclinal fold structure is present and this feature represents the only observed deviation from the general structural pattern present in the bedrock at KNPS. The synclinal fold predates all ages of faulting found to exist in the bedrock of the excavations (see ***Section 5.13***).

5.15.5.3 Groundwater Conditions

Historical investigations indicate that the permeability of the basement rock is categorised as low and therefore borehole yields are expected to be low. However, SSR geohydrological investigations (see ***Section 5.11***) have indicated higher borehole yields at the site, with some boreholes yielding >4 ℓ/s. The underlying bedrock materials are varied in type and can be expected to have variable permeability values, which are largely dependent on the degree of jointing that exists. Secondary permeability (water movement through joints) has produced values in the region of 3.0×10^{-6} m/s, and similar values can be inferred where borehole core is very fractured and has a low Rock Quality Designation (RQD).

The groundwater table is located 2 to 5 m below ground surface judging by previous investigations. In two historical boreholes drilled, artesian conditions were encountered, but whether such conditions are commonplace in the particular area of interest is conjectural. Perched water tables can be expected to occur above clayey horizons or well developed calcrete deposits.

Hydraulic conductivity for sand Layer 1 and 2 sands are very similar and approximately 1.0 × 10^{-4} m/s. By contrast Layer 3 has a lower permeability (1.0 × 10^{-7} m/s). This feature was attributed to the fact that the sand fraction is considerably finer in Layer 3 than in Layers 1 and 2.

The most significant engineering geological characteristics emanating from previous studies relating to groundwater are that the groundwater table is shallow and saturates the predominantly medium dense overburden soils.

With specific reference to ***Section 5.11,*** the following is relevant.

5.15.5.3.1 Physical Conditions

The site overlies two aquifer systems:

- the southern extent of the upper-lying primary or intergranular Sandveld Aquifer which is known locally as the Atlantis Aquifer;
- the deeper-lying weathered and basal fractured-rock (secondary) aquifer system of the Malmesbury Group.

With reference to ***Subsection 5.11.5*** of this SSR, the following regional characteristics pertain to the fractured rock Malmesbury Aquifer and to the Sandveld Aquifers:

- Fractured rock Malmesbury Aquifer:
  - Groundwater potential is predominantly low (<2 ℓ/s) but some areas of greater groundwater potential exist (>2 ℓ/s) where the Sandveld Aquifer is present, at contacts with granite intrusions, at the unconformity between the Malmesbury Group and TMG and where faulting and jointing extend from the TMG into the underlying Malmesbury Group.
  - Springs are rare in the Malmesbury Aquifer region.
  - Groundwater chemistry varies considerably but is generally of a sodium-chloride and alkaline type.
  - On a regional scale, this aquifer is classified as a minor aquifer of moderate to low vulnerability (***Subsection 5.11.5***);
  - Groundwater flow direction is predominantly to the west.
- Sandveld Aquifer:
  - This is a major aquifer of high vulnerability (***Subsection 5.11.5***) and shows high storage capacity and good groundwater supply potential.

- The aquifer extends to below sea level in places and is thus vulnerable to saline-water intrusion in coastal areas should over-abstraction occur.
- Groundwater storage occurs mainly in the sands and aeolianite.
- Recharge percolates rapidly through the highly porous, fine sandy and calcareous material to the pebble/gravel/shell beds.
- Minor dune-slack wetlands occur parallel to the coastline and are seasonal.
- Build-up of groundwater levels is unlikely to occur because of the high porosity and hydraulic conductivity of the Sandveld Group formations (see ***Subsection 5.11.5***) and discharge to the nearby ocean.
- Borehole yields are typically 0.1 to 5 ℓ/s and groundwater levels are between 2 and 5 m msl, but it is noted that yields of up to 88 ℓ/s have been measured in the Atlantis region.

Properties of the Sandveld Aquifer (***Subsection 5.11.6***) are:

- Transmissivity ranges from 16 to 140 m^2/d, averaging 59 m^2/d.
- Storativity values range from 0.2 (in the Sandveld Aquifer proper) to 0.001 (in the Lower Sandveld and weathered Malmesbury Aquitard).
- Hydraulic conductivity (K) values (horizontal) obtained from testing of the SSR boreholes range from 0.9 to 5.6 m/d, averaging 2.5 m/day.
- Water from this aquifer is of mixed NaCl, $Ca(HCO_3)_2$, $MgSO_4$ type with neutral to alkaline pH.

The average hydraulic properties of the Malmesbury Aquifer as defined in ***Subsection 5.11.6*** are:

- This fractured rock aquifer is highly anisotropic and aquifer parameters vary significantly across the site.
- Transmissivity ranges up to 180 m^2/d, averaging 70 m^2/d.
- K values range up to 37.7 m/d, averaging 6.2 m/day.
- Storativity values range from 0.0001 to 0.0029 (averaging 0.0012), indicating confined to semi-confined conditions.
- Water from this aquifer is of NaCl type with acidic to neutral pH.

Groundwater management during construction will be variably affected by the Sandveld and Malmesbury aquifers based primarily on the ranges in transmissivity.

5.15.5.3.2 Groundwater Chemistry

This section draws from the groundwater quality assessment (***Subsection 5.11.5.5***). Groundwater chemistry from the regional hydrocensus is summarised in ***Table 5.15.3***.

Table 5.15.3
Regional Groundwater Chemistry

Determinand	Units	Sandveld Aquifer sampling points				Malmesbury Aquifer sampling points			
		E08	GCS01	PBMR-BH	TW2	SRK-KG01	SRK-KG04	SRK-KG06	SRK-KG09
Calcium	mg/ℓ	97.4	109.4	242.8	71.8	**139.9**	**91.9**	**99.8**	**86.2**
Magnesium	mg/ℓ	34.4	45.0	56.1	8.6	**42.6**	**28.6**	**28.7**	**31.9**
Sodium	mg/ℓ	156.7	423	288.6	95.9	**468.1**	**336.2**	**329.5**	**347.2**
Potassium	mg/ℓ	6.5	3.3	18.9	44.4	**6.2**	**3.8**	**4.2**	**3.6**
Alkalinity	mg/ℓ as $CaCO_3$	260	194	327	221	**116.2**	**193.8**	**186.1**	**236.9**
Chloride	mg/ℓ	300	211	155	155	**1031.0**	**610.1**	**610.7**	**560.8**
Sulphate	mg/ℓ	44.5	95.6	227.8	42.3	**4.7**	**48.1**	**41.5**	**81.6**
Nitrate	mg/ℓ as N	0.47	<0.025	14.3	3.24	**3.5**	**1.5**	**1.3**	**0.7**
Fluoride	mg/ℓ	0.25	0.15	0.32	0.13	**0.1**	**0.1**	**0.3**	**0.3**
Total Iron	mg/ℓ	1.7	1.52	0.10	0.03	**2.1**	**1.4**	**3.2**	**0.3**
Total Manganese	mg/ℓ	0.63	0.06	<0.05	<0.05	**0.2**	**0.1**	**0.1**	**0.1**
Ammonia	mg/ℓ as N	<0.025	0.068	<0.025	<0.025	**0.3**	**0.5**	**0.3**	**0.2**
Phosphorus (Ortho-P)	mg/ℓ as PO_4	0.153	0.156	0.171	0.845	**0.6**	**2.1**	**2.2**	**1.1**
pH	pH units	7.7	7.3	7.4	8.5	**7.1**	**7.5**	**7.5**	**7.4**
EC	mS/m	148	247	250	98	**337.1**	**231.0**	**231.5**	**223.9**

Chemical characteristics of groundwater from the different aquifers in the area can be summarised as follows (***Section 5.11***):

- Sandveld Aquifer:
    - Chemical characteristics are mixed NaCl, $Ca(HCO_3)_2$, $MgSO_4$ type and neutral to alkaline pH.
    - Langelier Saturation Indices vary from 0.21 to 0.32: this groundwater is likely to cause scaling.
    - Sulfate at concentrations >200 mg/ℓ becomes aggressive towards concrete, and ranges from 44 to 228 mg/ℓ, but averages 102 mg/ℓ at the site with PBMR-BH being an outlier.
    - The Larson-Skold corrosion indices for mild steel for groundwater sampled from boreholes in the Sandveld Aquifer range from 1.4 to 5.8 (median of 2.6), which indicates that a tendency towards high corrosion rates should be expected.
- Malmesbury Aquifer:
    - Chemical characteristics are NaCl type and acidic to neutral pH.
    - Langelier Saturation Indices vary from -1 to 0.46; this groundwater is likely to cause mild scaling.
    - Sulfate concentrations range between 1.8 and 77 mg/ℓ.
    - Larson-Skold indices range between 3.6 to 144.8 (median of 5.1); this water will be highly corrosive to steel components.

5.15.5.3.3 Summary Findings from Previous Investigations

The historical data used in this SSR update are listed and the geotechnical profile (including groundwater conditions) as interpreted from these data is described below. In addition, comments on seismology and liquefaction potential as documented in historical information are described to create a linkage to the investigative work carried out for the KNPS. It should be noted that the historical information generated in support of the KNPS is voluminous and this SSR update has attempted to capture only those outcomes of previous studies considered necessary to update the geotechnical characterisation of the site.

(a) Data used in this report

The historical data analysed in this SSR is described below:

- borehole drilling and excavations:
  - Data was analysed from a range of historical data including the original investigation boreholes and investigations pertaining to the now shelved Pebble Bed Modular reactor (PBMR). ***Table 5.15.4*** details the number of boreholes used in the desktop analysis phase of this SSR compared to new data as discussed in ***Section 5.15.6***.
  - It is noted that many other historical boreholes have been drilled on the KNPS site (and surrounds), but not all of these data can be reliably georeferenced and/or borehole logs are not available for all of the boreholes. That said, the database of known boreholes (with approximate positions in some cases) is indicated in ***Drawing 5.15.1***.

Table 5.15.4
Summary Borehole Drilling and Excavation

Timing	Site / Study	# Boreholes
Historical	KNPS	81
	PBMR	38
Current	SSR Geotechnical	41
	SSR Hydrogeological	13
	Draft PSHA	20

- penetration testing - Two principal methods of penetration testing were adopted during the original investigations on the site:

  - Static cone penetration tests (CPTs) were performed, but their success was limited by the tendency of the cone to refuse often prematurely in a gravel layer, on individual boulders or cobbles or on competent shallow pedogenic layers – all of which occur randomly across the site.
  - Standard penetration tests (SPTs), using a Raymond split-spoon sampler, were routinely carried out at regular intervals in the small diameter boreholes that were drilled during the investigations.
  - SPT test data on summary statistics pertaining to historical boreholes are shown in ***Table 5.15.5***

Table 5.15.5
Historical Penetration Testing Results (SPT N Values)

	Layer 1 (Bredasdorp Formation)	Layer 2 (Springfontyn Formation)	Layer 3 (Varswater Formation)
Maximum N	refusal	refusal	refusal
Minimum N	3	3	3
Mean N	26	**34**	**37**
Stan Deviation	23,84	25,00	24,02

  - Additional penetration testing was carried out recently in support of developing this SSR as presented in ***Table 5.15.6*** – the current and historical results correlate well.

Table 5.15.6
Summary Penetration Testing Results (SPT N Values) - Current

	Layer 1 (Bredasdorp Formation)	Layer 2 (Springfontyn Formation)	Layer 3 (Varswater Formation)
Maximum N	refusal	refusal	refusal
Minimum N	4	4	8
Mean N	20	**31**	**37**
Stan Deviation	22	26	24

- laboratory testing – Numerous historical laboratory tests were undertaken on soils and rocks, including:
  - characterisation testing (particle size distribution and Atterberg Limits), shear strength properties (cohesion and friction angle), density and moisture content determinations on soils;
  - unconfined compressive strength, density, Young’s Modulus and Poisson’s Ratio on rocks.
- Geophysical Investigations to measure:
  - Shear (Vs) and compression (Vp) wave velocity in soils and rock using downhole and crosshole techniques, Multichannel Analysis of Surface Waves (MASW) and seismic refraction surveys - Poison’s ratio was also measured.
  - subsurface delineation through electrical resistivity tomography (ERT) and seismic refraction surveys.

(b) Site Geology and Subsurface Materials

Site soils and rocks are previously described as follows.

- soils:
  - In broad terms, the materials underlying the area consist of a sequence of variably calcareous and fossiliferous aeolian, estuarine and marine sands of considerable thickness (approximately 20 – 30 m) as described in ***Subsection 5.15.5.2.3***.
  - Three distinct sand layers are present and comprise aeolian calcareous sand (Layer 1 – Bredasdorp Formation), estuarine dark

grey sand (Layer 2 – Springfontyn Formation) and Marine green fine sand (Layer 3 – Varswater Formation) – the extent of these layers is shown in ***Table 5.15.7***, but considerable variability can occur locally.

Table 5.15.7
Historically Derived Average Thickness of the Principal Sand Layers

	Layer 1 (m)	Layer 2 (m)	Layer 3 (m)
Max (m)	13	8,3	12,97
Min (m)	3	0	3,8
Mean (m)	6,56	3,79	8,47

- By comparison, ***Table 5.15.8*** updates ***Table 5.15.7*** with data from the current investigations. The data do not vary significantly even though the current investigations cover a considerably broader geographical footprint.
- Layer 1 is mainly light grey to light brown, generally fine-grained with numerous interbedded medium and coarse-grained lenses and layers, as well as random calcrete which varies in its degree of development from a white dusty colouration to a well cemented bouldery layer and variable, but significant, amounts of shell debris – the thickness variability in Layer 1 is largely due to the rapid changes in elevation of the undulating ground surface.
- Layer 2 is grey to dark grey due to the presence of organic matter and was reported to be more variable in thickness indicating that it has been eroded in places.
- Layer 3 is greenish-grey and is the most homogeneous in thickness and widespread in distribution, being found along the west coast as far north as Hondeklip Bay.
- The most significant engineering geological characteristics emanating from previous studies are that the overburden soils are thick (in the region of 20 – 30 m), have randomly distributed gravel / cobble / boulder / calcrete horizons reducing the efficacy of penetration testing and increasing marginally in consistency with depth from medium dense becoming dense (sometimes very dense) near the bedrock contact.
- Geophysical parameters of beach sands and Tygerberg formation rocks extracted from (Brink, 1981) appear in ***Table 5.15.9.***

- ***Table 5.15.10*** lists the relative density data for the soil Layers 1, 2 and 3 from (Brink, 1985).
- (Brink, 1985) found no significant variability in shear strength parameters between Layers 1, 2 and 3 and found the peak friction angle for initial sample density varied between 34˚ and 41˚ whilst the angle of friction at constant volume ranged between 30˚ and 33˚.

Table 5.15.8
Current Average Thickness of the Principal Sand Layers

	Layer 1 (m)	Layer 2 (m)	Layer 3 (m)
Max (m)	19.5	25.1	24.9
Min (m)	0	0	0
Mean (m)	7.3	10.0	8.0

Table 5.15.9
Literature Survey Geophysical Parameters

Indicative Depth Below Ground Level (m)	Description	Compressional Wave Velocity (m/s)	Poisson's Ratio	Shear Wave Velocity (m/s)
0 to 3.0	Very loose, fine grained beach sand	549 to 610	0,42 to 0,46	152 to 229
3.0 to 16.0	Dense, fine-to-medium grained beach sand	1 615 to 1 737	0,36	762 to 823
16.0 to 70.1	Weathered rocks of Tygerberg Formation	3 353 to 3 413	0,32 to 0,33	1 707
70.1 plus	Fresh rocks of Tygerberg Formation	5 029 to 6 096	0,28 to 0,29	2 743 to 3 352

Table 5.15.10
Literature Survey Geotechnical Parameters

Soil Layer	Average Maximum Dry Density (kg/m^3)	Average Minimum Dry Density (kg/m^3)	Average Field Dry Density (kg/m^3)	Average Relative Density (%)
1	1 770	1 480	1 670	71
2	-	-	1 670	-
3	1 790	1 420	1 650	68

- Rocks:
  - Bedrock materials encountered in the boreholes consist, in the unweathered state, of massive, fine to medium-grained, quartzitic, occasionally cross-bedded, indurated, grey, extremely hard rock greywackes and fine-grained, even textured, moderately hard rock, grey or grey-green, bedded mudstones with subordinate micaceous laminated shale bands that are rare in occurrence and usually less than a metre in thickness.
  - The most significant engineering geological characteristics emanating from previous studies of the site rocks is their variability linked to the variable distribution of these rocks.
  - Geophysical parameters of Tygerberg formation rocks extracted from (Brink, 1981)

(c) Seismology

The site lies on the western branch of the Cape Fold Belt adjacent to the syntaxis zone. The closest known fault is the Mamre Fault (17 km north of the site) while a possible shear zone tentatively called the Milnerton Fault was historically proposed to occur between Bloubergstrand and Cape Town, but the PSHA concludes the evidence for this risk is inconclusive. The nearest proven faults are those displacing Table Mountain Group rocks in the Cape Peninsula some 30 km from the site.

The maximum possible earthquake that was projected for the region was M 6.60 ± 0.3 with a Peak Ground Acceleration (PGA) of 0.27 ± 0.14 g (Eskom, 2006).

(d) Liquefaction Potential

Liquefaction and intense ground deformation in the area between Melkbosstrand and Cape Town are well documented in historical data (Eskom, 2006). The closest position where liquefaction features were reported is at Blauweberg's Vlei some 10 km south of the site.

5.15.6 Site Investigation for Supplementary Data

This section deals with new information that was collected to primarily characterise that portion of the site lying to the north of the KNPS and to integrate these data with the KNPS data. Investigation planning, field investigation descriptions, laboratory testing, liquefaction potential assessments, surface investigations and human induced conditions receive comment.

It is necessary to point out that the logistics (drilling in particular) required to access large parts of the northern portion of the site are challenging in the dune environment, and that the density of intrusive investigation positions therefore varies substantially across the site.

The data collated for this SSR draws from the drilling findings of ***Section 5.11*** to some extent.

5.15.6.1 Investigation Planning and Process

The field investigations were implemented as presented in ***Drawing 5.15.1***.

This section covers the strategic planning of the investigation in order to pre-empt any uncertainties that may exist and to provide a basis for considering mitigation measures.

Figure 5.15.2 shows the geotechnical data gathering approach schematically, indicating the significance of investigation planning and refinement throughout.

Drawing 5.15.1
Duynefontyn Site Layout and Fieldwork Data Positions

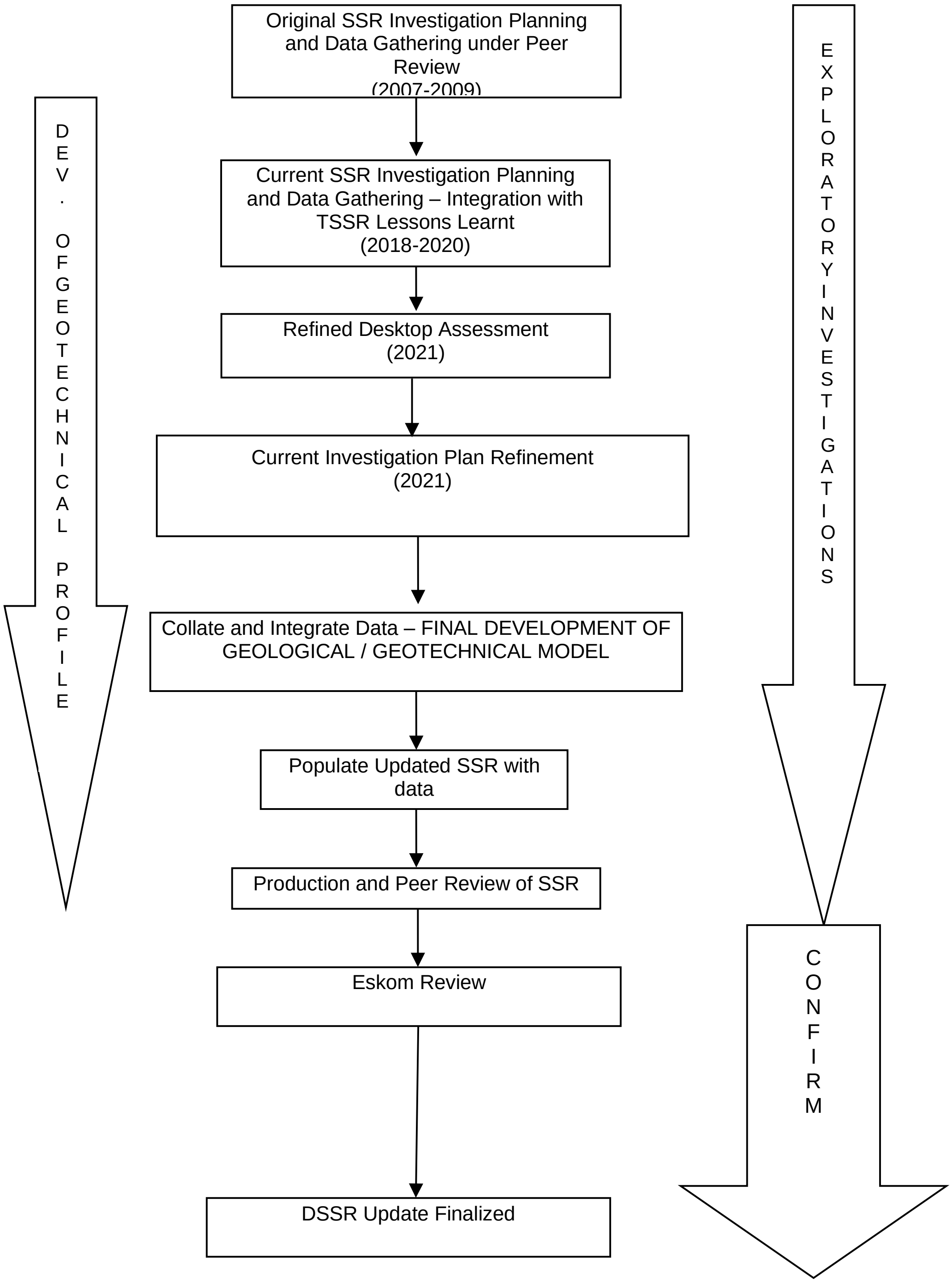


Figure 5.15.2
Geotechnical Process Applied for the Duynefontyn Site

Notes on *Figure 5.15.2*:

- Supervision and quality control on all aspects of field investigations was a feature of the data collation work carried out on the site as per project QA/QC plans.
- Drilling was carried out under the supervision of an experienced drilling foreman.
- Sampling was carried out by experienced field technicians.
- Data analysis and reporting on data were carried out by an experienced geotechnical engineer.

5.15.6.2 Intrusive Investigations

Intrusive geotechnical investigations were carried out to supplement the historical data presented in ***Subsection 5.15.5***. These historical data only provided insight into the geotechnical characteristics of the southern portion of the site in proximity to the KNPS. The intrusive investigations presented in this section are complementary and targeted that portion of the site to the north of KNPS (***Drawing 5.15.1***) in two phases: 2008 and 2021 campaigns.

(a) Spacing (Grid)

In the 2008 campaign, borehole drilling depths and positions were planned on a 110 m x 110 m grid with varying target drilling depths. ***Table 5.15.9*** describes the target boreholes. (American Society for Testing and Materials International, 2007) gives general guidelines on borehole spacing and target depths with specific reference to safety-related structures and the following general guidelines apply:

- There should be at least one boring at the location of every safety-related structure.
- All boreholes should extend at least 10 m into bedrock.

However, in exploring geotechnical characteristics on a more detailed basis to inform design (i.e. to provide sufficient information for the pre-operational and operational stages); (International Atomic Energy Agency, 2004) gives more specific guidance including:

- Under safety-related structures, there should be at least one borehole approximately on a 30 m grid under buildings and earth dams and one borehole approximately every 30 m along linear structures.
- Additional boreholes must be specifically situated along the periphery, at corners and other selected locations.
- Drilling depths should ensure that at least one quarter of boreholes are drilled into fresh rock and the remaining boreholes to a depth under

founding level equal to the width of structure or to a depth equal to the foundation depth below the original ground surface, whichever is greater.

In the context of the site where details of the nuclear installation(s) and locality of safety-related structures are not finalised, the initially planned grid system (110 m x 110 m) with varying target drilling depths was employed to capture a general site geotechnical profile. The pre-operational and operational stages of the project will target specific boreholes and borehole depths targeting safety related structures of known position as set out in (International Atomic Energy Agency, 2004).

The intended and actual boreholes drilled in the 2008 campaign, the defining target and the actual drilling depths are summarised in ***Table 5.15.11***. The table also provides information about the positioning and drilling depths, as well as on any variations from initially planned activities.

In the 2021 campaign, additional boreholes were drilled to expand the geographical footprint of geotechnical data to better comply with the Environmental Impact Assessment (EIA) footprint.

(b) Target Information

Boreholes were logged describing moisture content (in the case of soils), colour (soils and rocks), consistency (soils) and hardness (rocks), soil/rock type, soil/rock structure and geological origin.

As required, (International Atomic Energy Agency, 2004) boring logs contain:

- the date when the boring was made;
- the location of the boring (X and Y coordinates);
- the depths of boreholes;
- the borehole collar elevations with respect to a permanent benchmark.

The logs also include:

- the elevations of the top of boreholes and depth to boundaries of soil or rock strata;
- the classification and description of soil and rock layers;
- blow count values obtained from SPTs;

- per cent recovery of rock core;
- quantity of core not recovered for each core interval or drill run;
- rock quality designation (RQD);
- laboratory test results for rocks and soils (2018 campaign logs).

(c) Drilling Procedures

The following guiding procedures were followed in carrying out the drilling programme:

- 2008 Campaign:
  - Thirty-three boreholes were drilled vertically and all drilling was of N (60 mm core diameter) size, employing the use of diamond or tungsten bits (***Table 5.15.11)***.
  - Conventional double-tube core barrels and split-inner double-tube swivel type core barrels were used to improve core recovery.
  - N-size borehole casing was used to maintain borehole integrity through overburden horizons and to allow appropriate groundwater monitoring piezometers to be installed.
  - Groundwater level monitoring devices (piezometers) were installed in all geotechnical boreholes drilled.
  - Other intrusive investigations carried out are detailed in ***Table 5.15.12*** and the actual programme achieved in ***Table 5.15.13.***
- 2021 Campaign:
  - Eight vertical boreholes were drilled P-size (85 mm core diameter) in order to construct boreholes for downhole testing.
  - Conventional double-tube core barrels and split-inner double-tube swivel type core barrels were used to improve core recovery, and in some cases PQ drilling was employed to assure better core recovery.
  - P-size borehole casing was used to maintain borehole integrity through overburden horizons.
- 2023 PSHA campaign (see ***Table 5.15.16***):
  - Twenty vertical boreholes were drilled using a combination of percussion, mud rotary and sonic drilling techniques.

- These boreholes were drilled to between *c.*20 and *c.*120m depth targeting both the soil and rock profiles.
- Vs profiles were measured in selected boreholes using PS Suspension logging techniques.

(d) *In situ* Testing

SPTs were carried out in accordance with the international reference test procedure laid down in the proceedings of the First International Symposium on Penetration Testing (ISOPT) 1988 (Campanella & Robertson, 1988) with tests performed in all boreholes and at 1.5 m frequency in soils. SPT tests were carried out to refusal in dense strata or bedrock and provide extensive data laterally across the site and with depth. This resulted in recovery of approximately 0.45 m long samples where possible, with intermediate washbore samples.

SPT tests carried out were thus the most representative measurement of soil consistency across the site.

'Refusal' of the SPT was defined to be when the number of blows necessary to penetrate 75 mm exceeded 25.

Dynamic Penetrometer Super Heavy (DPSH) tests were carried out to determine the nature of soil materials intermediate to boreholes. The DPSH tests were carried out in accordance with (Campanella & Robertson, 1988). DPSH tests did not provide extensive data due to shallow refusal on dense layers/pedogenic horizons.

(e) Sampling

Undisturbed sampling within soil horizons at the site proved challenging as sample recovery was inhibited by the cohesionless nature of the soils and by the presence of shallow groundwater. Disturbed soil samples were obtained at regular intervals from borehole core and SPT samples for soil indicator testing and to provide an extensive assessment of the soil grading and Atterberg Limits laterally and vertically across the site. Shelby tube sampling was attempted in the 2021 campaign with marginal success as Shelby tubes tended to buckle in dense subsurface horizons.

Rock samples were recovered, but sampling was restricted by the fractured nature of the material and the difficulties that this introduced in sample integrity during transport to laboratories. All rock samples were transported as 'fragile' items under controlled sample chain of custody. Reasonable representation of the site rock characteristics was obtained through sampling in a high percentage of boreholes and in different geological formations.

(f) Preservation of Borehole Core

Both soil and rock borehole core were packed in depth sequence in marked core boxes. The soil core was extracted from the drill barrels or from borehole washwater directly into plastic sleeves that were subsequently slit open to allow photography and logging. The rock core was systematically arranged in the core boxes with all broken joints closed as accurately and tightly as possible.

Standards followed for sample preservation included clear definition of core storage box construction, marking, temporary storage and handling prior to photographing and storage in core sheds on the site.

(g) Transportation and Storage of Samples

Both rock and soil samples were taken and transported under a formal chain of custody system linked to the quality data pack. Rock and soil samples were transported in boxes. Since this SSR is produced in the context of the geotechnical investigation stage of characterisation, no specific borrow or construction material samples were targeted. The suitability of *in situ* materials for construction will be investigated prior to the pre-operational stage once detailed design of structures emerges. In maintaining pace with the project programme, sample storage time was limited wherever practically possible and transport under the chain of custody expedited.

Table 5.15.11
2008 Campaign Planned and Drilled Boreholes

Borehole ID	Coordinates			Depth		Depth Explored		Remarks
	X	Y	Z	Planned	Actual	Soil	Rock	
KB01	53585.103	3726105.684	5.82	30	**30.02**	17.2	12.82	Outlying BH to assess conditions at depth away from the site centroid
KB05	52883.806	3727221.241	12.54	30	**29.96**	22.25	7.71	Gridline borehole targeting intermediate depth (30 m) and exploring a target of 10 m into bedrock
KB06	52985.997	3727017.974	12.24	40	**40.52**	14	26.52	Gridline borehole targeting deeper depth (40 m) and exploring a target of 20 m into bedrock
KB07	53069.914	3726815.117	10.50	50	**54.25**	17.75	36.5	Gridline borehole targeting deeper depth (40 m) and exploring a target of 20 m into bedrock
KB08	53162.192	3726614.417	10.38	40	**40.45**	20.2	20.25	Gridline borehole targeting deeper depth (40 m) and exploring a target of 20 m into bedrock
KB09	53258.424	3726417.107	12.74	30	**30.00**	24.45	5.55	Gridline borehole targeting intermediate depth (30 m) and exploring a target of 10 m into bedrock
KB10	53060.803	3726329.754	18.11	20	**26.92**	20.51	6.41	Gridline borehole targeting shallow depth (20 m) and exploring a target of 5 m into bedrock
KB11	52967.352	3726527.384	16.59	30	**30.13**	26.46	3.67	Gridline borehole targeting shallow depth (20 m) and exploring a target of 5 m into bedrock
KB12	52859.935	3726736.263	16.68	40	**40.14**	24.45	15.69	Gridline borehole targeting deeper depth (40 m) and exploring a target of 20 m into bedrock
KB13	52768.018	3726931.630	15.86	30	**28.50**	22.95	5.55	Gridline borehole targeting shallow depth (20 m) and exploring a target of 5 m into bedrock
KB14	52676.960	3727135.092	16.03	30	**27.05**	19.95	7.1	Gridline borehole targeting shallow depth (20 m) and exploring a target of 5 m into bedrock
KB17	53079.818	3727058.022	7.44	20	**21.58**	16.02	5.56	Gridline borehole targeting shallow depth (20 m) and exploring a target of 5 m into bedrock
KB18	52881.178	3726978.932	14.77	30	**30.10**	19.95	10.15	Gridline borehole targeting intermediate depth (30 m) and exploring a target of 10 m into bedrock
KB19	53167.492	3726858.326	10.47	30	**29.60**	23.82	5.78	Gridline borehole targeting shallow depth (20 m) and exploring a target of 5 m into bedrock
KB21	53284.895	3726674.657	6.26	50	**54.95**	18.5	36.45	Gridline borehole targeting deeper depth (40 m) and exploring a target of 20 m into bedrock
KB25	53361.561	3726697.643	5.18	20	**24.00**	18	6	Gridline borehole targeting shallow depth (20 m) and exploring a target of 5 m into bedrock
KB26	53180.532	3727099.398	5.65	30	**30.00**	18.5	11.5	Gridline borehole targeting intermediate depth (30 m) and exploring a target of 10 m into bedrock
KB27	53090.174	3727181.334	6.07	20	**22.63**	18.5	4.13	Gridline borehole targeting shallow depth (20 m) and exploring a target of 5 m into bedrock
KB29	53267.456	3726781.598	6.85	30	**30.00**	18.45	11.55	Gridline borehole targeting intermediate depth (30 m) and exploring a target of 10 m into bedrock

Borehole ID	Coordinates			Depth		Depth Explored		Remarks
	X	Y	Z	Planned	Actual	Soil	Rock	
KB30	53356.720	3726580.470	5.27	20	**25.50**	18	7.5	Gridline borehole targeting shallow depth (20 m) and exploring a target of 5 m into bedrock
KB31	53093.098	3727292.052	6.91	30	**30.18**	22.95	7.23	Gridline borehole targeting shallow depth (20 m) and exploring a target of 5 m into bedrock
KB32	53267.650	3726897.190	7.35	40	**40.20**	19.95	20.25	Gridline borehole targeting deeper depth (40 m) and exploring a target of 20 m into bedrock
KB33	53449.088	3726498.541	5.05	30	**30.00**	13.95	16.05	Gridline borehole targeting intermediate depth (30 m) and exploring a target of 10 m into bedrock
KB34	53249.274	3726541.484	8.26	30	**30.14**	18.6	11.54	Gridline borehole targeting intermediate depth (30 m) and exploring a target of 10 m into bedrock
KB36	53080.390	3726939.137	10.60	30	**30.04**	15	15.04	Gridline borehole targeting intermediate depth (30 m) and exploring a target of 10 m into bedrock
KB37	52987.447	3727139.662	11.57	30	**30.17**	19.72	10.45	Gridline borehole targeting intermediate depth (30 m) and exploring a target of 10 m into bedrock
KB38	52876.429	3727093.424	14.43	20	**25.56**	20.58	4.98	Gridline borehole targeting shallow depth (20 m) and exploring a target of 5 m into bedrock
KB40	53065.497	3727398.605	7.11	30	**30.00**	19.95	10.05	Gridline borehole targeting intermediate depth (30 m) and exploring a target of 10 m into bedrock
KB41	53158.993	3726491.498	10.28	20	**23.75**	18.75	5	Gridline borehole targeting shallow depth (20 m) and exploring a target of 5 m into bedrock
KB42	53041.468	3726452.932	18.01	30	**30.04**	27.04	3	Gridline borehole targeting shallow depth (20 m) and exploring a target of 5 m into bedrock
KB43	52844.077	3727317.798	12.51	30	**31.46**	24.07	7.39	Gridline borehole targeting intermediate depth (30 m) and exploring a target of 10 m into bedrock
KB44	52868.650	3726855.616	17.54	50	**54.78**	19.95	34.83	Gridline borehole targeting deeper depth (40 m) and exploring a target of 20 m into bedrock
KB45	52777.735	3727058.860	14.11	30	**30.06**	20.98	9.08	Gridline borehole targeting intermediate depth (30 m) and exploring a target of 10 m into bedrock

Notes

BH= borehole

KB= Borehole ID prefix

Table 5.15.12
2008 Campaign Other Intrusive Geotechnical Investigations Carried Out

Activity	Purpose	Relevant Standard/Method
Standard Penetration Tests (SPT)	Explore *in situ* consistency of site soil horizons and liquefaction potential of site soils	In accordance with international reference test procedure laid down in proceedings of First International Symposium on Penetration Testing (ISOPT) 1988 (Campanella & Robertson, 1988)
Dynamic Penetrometer Super Heavy Tests (DPSH)	Explore *in situ* consistency of site soil horizons intermediate to boreholes and SPTs	In accordance with international reference test procedure laid down in proceedings of First international Symposium on Penetration Testing (ISOPT) 1988 (Campanella & Robertson, 1988)
Measurement of the Site Groundwater Table Elevation	Inform liquefaction potential assessment and define site groundwater table	Measured in piezometers installed in boreholes from borehole collar elevations established by a formal land survey
Point Load Testing	Measurement of rock point load strength index and calibration of visual (log) descriptions	Measure with calibrated field instrument

Table 5.15.13
2008 Campaign Other Intrusive Geotechnical Investigations Carried Out (Achieved Programme)

Exploratory Method	ID	Depth		Remarks
		Planned	Actual	
DPSH	KDP1	Refusal/Bedrock	2.7	Presumed refusal on calcrete/dense sand
DPSH	KDP2	Refusal/Bedrock	N/A	Omitted due to consistent shallow refusal of DPSH tests
DPSH	KDP3	Refusal/Bedrock	N/A	Omitted due to consistent shallow refusal of DPSH tests
DPSH	KDP4	Refusal/Bedrock	3.6	Presumed refusal on calcrete/dense sand
DPSH	KDP5	Refusal/Bedrock	1.8	Presumed refusal on calcrete/dense sand
DPSH	KDP6	Refusal/Bedrock	2.7	Presumed refusal on calcrete/dense sand
DPSH	KDP7	Refusal/Bedrock	N/A	Omitted due to consistent shallow refusal of DPSH tests
DPSH	KDP8	Refusal/Bedrock	N/A	Omitted due to consistent shallow refusal of DPSH tests
DPSH	KDP9	Refusal/Bedrock	3.3	Presumed refusal on calcrete/dense sand
DPSH	KDP10	Refusal/Bedrock	3.0	Presumed refusal on calcrete/dense sand
DPSH	KDP11	Refusal/Bedrock	N/A	Omitted due to consistent shallow refusal of DPSH tests
DPSH	KDP12	Refusal/Bedrock	N/A	Omitted due to consistent shallow refusal of DPSH tests

Exploratory Method	ID	Depth		Remarks
		Planned	Actual	
DPSH	KDP13	Refusal/Bedrock	3.3	Presumed refusal on calcrete/dense sand
DPSH	KDP14	Refusal/Bedrock	2.4	Presumed refusal on calcrete/dense sand
DPSH	KDP15	Refusal/Bedrock	1.8	Presumed refusal on calcrete/dense sand
DPSH	KDP16	Refusal/Bedrock	N/A	Omitted due to consistent shallow refusal of DPSH tests
DPSH	KDP17	Refusal/Bedrock	N/A	Omitted due to consistent shallow refusal of DPSH tests
DPSH	KDP18	Refusal/Bedrock	1.8	Presumed refusal on calcrete/dense sand
DPSH	KDP19	Refusal/Bedrock	1.5	Presumed refusal on calcrete/dense sand
DPSH	KDP20	Refusal/Bedrock	3.3	Presumed refusal on calcrete/dense sand
DPSH	KDP21	Refusal/Bedrock	N/A	Omitted due to consistent shallow refusal of DPSH tests
DPSH	KDP22	Refusal/Bedrock	2.7	Presumed refusal on calcrete/dense sand
DPSH	KDP23	Refusal/Bedrock	3.3	Presumed refusal on calcrete/dense sand
DPSH	KDP24	Refusal/Bedrock	0.6	Presumed refusal on calcrete/dense sand
DPSH	KDP25	Refusal/Bedrock	4.2	Presumed refusal on calcrete/dense sand
DPSH	KDP26	Refusal/Bedrock	N/A	Omitted due to consistent shallow refusal of DPSH tests
DPSH	KDP27	Refusal/Bedrock	1.5	Presumed refusal on calcrete/dense sand
DPSH	KDP28	Refusal/Bedrock	2.4	Presumed refusal on calcrete/dense sand
DPSH	KDP29	Refusal/Bedrock	3.3	Presumed refusal on calcrete/dense sand
DPSH	KDP30	Refusal/Bedrock	N/A	Omitted due to consistent shallow refusal of DPSH tests
DPSH	KDP31	Refusal/Bedrock	1.5	Presumed refusal on calcrete/dense sand
DPSH	KDP32	Refusal/Bedrock	3.6	Presumed refusal on calcrete/dense sand
DPSH	KDP33	Refusal/Bedrock	3.6	Presumed refusal on calcrete/dense sand
DPSH	KDP34	Refusal/Bedrock	N/A	Omitted due to consistent shallow refusal of DPSH tests
DPSH	KDP35	Refusal/Bedrock	N/A	Omitted due to consistent shallow refusal of DPSH tests
DPSH	KDP36	Refusal/Bedrock	2.1	Presumed refusal on calcrete/dense sand
DPSH	KDP37	Refusal/Bedrock	N/A	Omitted due to consistent shallow refusal of DPSH tests
DPSH	KDP38	Refusal/Bedrock	2.7	Presumed refusal on calcrete/dense sand
DPSH	KDP39	Refusal/Bedrock	N/A	Omitted due to consistent shallow refusal of DPSH tests
DPSH	KDP40	Refusal/Bedrock	0.6	Presumed refusal on calcrete/dense sand

Notes:

Presumed refusal refers to a likely scenario but was not confirmed as DPSH probes do not recover samples.

Poor outcomes of DPSH testing screened out the use of any other probing methods (e.g. CPTu probing) due to high likelihood of shallow refusal.

Table 5.15.14
2021 Campaign Planned and Drilled Boreholes

Borehole ID	Coordinates			Depth		Depth Explored		Remarks
	X	Y	Z	Planned	Actual	Soil	Rock	
KB46	-51866	-3726651	24.75	80	80	29.45	50.55	Drilled to explore geotechnical profile, carry out SPT tests, obtain laboratory testing samples and construct boreholes for downhole geophysical testing. Target depth was 50 m into bedrock.
KB47	-52168	-3726484	27.25	83	81.6	31.6	50	
KB48	-52667	-3726223	19.5	78	80	30	50	
KB49	-52932	-3726055	17.1	82	80	31.05	48.95	
KB50	-52809	-3725522	25.9	84	82.89	35.23	47.66	
KB51	-53196	-3725713	20.75	78	83	32.45	50.55	
KB52	-53468	-3725763	15.25	69	72.5	22.55	49.95	
KB53	-53163	-3725884	17.25	77	80	39	41	

Table 5.15.15
2021 Campaign Other Investigations Carried Out (Planned vs Achieved Programme)

Exploratory Method	Number		Remarks
	Planned	Actual	
MASW Single source soundings	100	56	Vs measurements of (primarily) the soil horizons targeted to support the liquefaction potential assessment of the site
MASW Multi-source soundings	0	6	Single source measurements were sacrificed to carry out multi-source measurements to support the probabilistic seismic hazard assessment (***Section 5.14***)
Downhole geophysical testing	8 boreholes	8 boreholes	Downhole geophysics performed on all boreholes drilled.

Table 5.15.16
2023 PSHA Drilled Boreholes

Borehole ID	Coordinates			Depth	Depth Explored		Remarks
	X	Y	Z		Soil	Rock	
CG_DA100	-52810.6	-3726603	17	100	27	73	Drilled to support the PSHA and supplement the geotechnical profile database.
CG_DA40	-52815.5	-3726604	17	30	26	4	
CG_M1	-52908.8	-3726308	19.1	33.8	30	3.8	
CG_M10	-53154.7	-3726819	10.8	21.1	18.0	3.1	
CG_M11	-52739.4	-3727170	15.1	21	20.1	0.9	
CG_M2	-52829.4	-3726620	16.4	28.62	24.8	3.82	
CG_M3	-52961.5	-3727261	10	22.46	21	1.46	
CG_M3b	-52961.5	-3727261	10	22.46	21.9	0.56	
CG_M4	-52265.1	-3726721	21.8	29.13	24.35	4.78	
CG_M5	-52964.7	-3726645	17	30.42	25.56	4.86	
CG_M6	-53266.7	-3726547	7.5	21.71	18.0	3.71	
CG_M7	-53024.9	-3726475	16.1	31.6	28.3	3.3	
CG_M8	-52996	-3726939	10.9	15.88	12.0	3.88	
CG_M9	-53497.9	-3726297	5.5	19.63	16.3	3.33	
CG_SA30	-53197.4	-3725807	18.2	34	29.0	5.0	
CG_SA90	-53194.3	-3725810	18.4	90	26.0	64.0	
CG_ST1	-52776.9	-3727101	13	120	17.0	103.0	
CG_ST2	-52249.4	-3726811	21	80	16.0	64.0	
CG_ST3	-53031	-3726445	17	80	30.0	50.0	
CG_ST4	-53826.8	-3726082	0	80	26.0	54.0	

5.15.6.4 Geophysical Investigations

5.15.6.4.1 SSR Scope

The geophysical investigation scope is detailed in Table 5.15.11. the following techniques were used:

- MASW data was acquired with a 24 channel Geode landstreamer system employing 1m or 2m geophone spacing. A 6.2kg hammer and plate source and ESS-100 Weight Drop were used. The 1D sounding positions were surveyed using a Trimble R8 GPS system in RTK (base station) and/or VRS Trignet mode. Accuracy of these survey systems is better than 5cm for X, Y and Z. MASW data was processed using Surfseis v6 software.

- In collaboration with the seismic hazard assessment team, single source MASW soundings were complimented with several multi-source soundings.

- Downhole seismic testing was carried out using a Geostuff 3-component geophone with a 6.2 kg hammer source at surface to generate P and S waves.

The detailed report describing these investigations is contained in ***Appendix 5.15.I***, and the following comment in addition to that contained in ***Appendix 5.15.I*** is relevant:

- The site soils have a mean shear wave velocity (Vs) of *c.*290 m/s (with a standard deviation of *c.*140m/s) and the mean Vs for the site rocks is *c.*2000 m/s (standard deviation of *c.*730 m/s) and compression wave velocity (Vp) mean is *c.*3300 m/s (standard deviation of *c.*950 m/s).

- The Vs profile for the site is visually documented in the sections shown in ***Drawings 5.15.3*** to ***5.15.12***.

In addition, it is noted that the Vs monitoring at the KNPS indicates that the cement stabilised soil raft foundation is performing well – this is best illustrated in Drawing 5.15.4 that indicates a Vs profile under the KNPS nuclear footprint of >750 m/s, but trending towards 1 000 m/s.

5.15.6.4.2 PSHA Scope

From the 20 boreholes drilled in this study, 6 boreholes were selected to measure Vs using PS suspension logging methods.

5.15.6.5 Laboratory Investigations

The purpose of the laboratory investigation programme was to identify and classify soils and rock from the site, and to evaluate their physical and engineering properties defining the site capacity and the influence that this capacity has on the safety of the planned nuclear installation(s). The evaluation for this SSR covered investigation, description, classification, testing and analysis of soil and rock to determine their interaction with structures planned to be built in or upon them or with them. Descriptions of the physical properties of soil and rock were improved through laboratory testing.

To confirm field investigation data collected, a laboratory testing programme for soils and rocks was implemented and was developed in observance with (American Society for Testing and Materials International, 2008), (Committee of State Road Authorities, 1986), (British Standards Institution, 1990), (International Society for Rock Mechanics, 1979). Soil and rock samples obtained from the intrusive investigations were subjected to a laboratory testing programme targeting selected parameters listed in ***Table 5.15.17*** and the resulting testing programme is detailed in ***Table 5.15.18.*** This section covers the specifications used for the laboratory investigations and the methods by which they were carried out.

Sample representativeness proved to be a challenge at the site. Within the site rocks, prescribed laboratory testing methods (***Table 5.15.18***) require rock samples of at least 200 mm in length. The sampling process was affected by the available lengths of intact core (see ***Appendix 5.15.C***). In particular, the steeply dipping bedding joints (75˚) rendered much of the recovered core unsuitable for testing. This resulted in sampling of less frequently jointed core sticks and may have biased the results. Within the site soils, difficulties were experienced in obtaining undisturbed soil samples as discussed in ***Subsections 5.15.6.1*** and ***5.15.6.2***. Shelby tube samples were obtained in the 2021 campaign, but for the most part, Shelby tubes refused and physically buckled when executing the sampling activities.

The tests performed were:

- classification tests,
- engineering properties tests.

Table 5.15.17
Recommended Soil and Rock Properties

Soil		Rock
Water Content	Permeability	Porosity
Unit Weights	Consolidation	Permeability
Void Ratio	Shear Strength	Seismic Velocity
Porosity	Triaxial Compression	Direct Tensile Strength
Saturation	Unconfined Compression	Direct Shear
Atterberg Limits	Humid*, Dry* and Relative Densities	Unconfined Compression
Specific Gravity	Grain Size Analysis	Triaxial Compression
Erodibility	Compaction	Slake Durability
Carbonates and Sulphates content*	Salt content*	Specific Gravity
Oedometric, Young's modulus*		

The tests undertaken in this SSR investigation aimed at describing parameters to sufficiently support an assessment of the suitability of the site that will be subjected to loads typical of nuclear installation(s). No specific structure layouts or loading information was available at the time of developing this SSR.

Selected parameters listed in ***Table 5.15.17*** were measured through intrusive investigation methods (***Subsection 5.15.6.2***) and laboratory investigations generally aligned with the programme shown in ***Table 5.15.18*** and further expanded on in ***Subsection 5.15.6.4***.

Table 5.15.18
Laboratory Testing Programme

Laboratory Test	Purpose	Standard/method
Soil		
Soil Characterisation Testing	Characterisation of the site soils including full hydrometer analysis	ASTM D422 (American Society for Testing and Materials International, 2007)
	Characterisation of the site soils	BS1377: Part 2: 1990 9.2 (British Standards Institution, 1990)
	Soil density testing	(South African National Standard, 2014), (South African National Standard, 2014b), (South African National Standard, 2015), (South African National Standard, 2015b), (South African National Standard, 2019), (South African National Standard, 2014c), (South African National Standard, 2014d)
Atterberg Limits	Standard determination of Atterberg Limits (Liquid Limit, Plastic Limit and Plasticity Index)	TMH1 A2, A3 and A4 (Committee of State Road Authorities, 1986)
	Determination of Atterberg Limits (Liquid Limit, Plastic Limit and Plasticity Index) with special emphasis on Liquid Limit in lower ranges	BS1377: Part 2: 1990 Clauses 4.4 and 5 (British Standards Institution, 1990)
Rock		
Uniaxial Compressive Strength (UCS) Testing with Young's Modulus (E) and Poisson's (v) Ratio	Determination of rock strength under laboratory conditions. Measurement of deformation parameters (E, v) for future use in modelling	ISRM Suggested Methods 1981 (International Society for Rock Mechanics, 1979)
Bulk and Dry Density	Parameters measured in conjunction with UCS, E and v	ISRM Suggested Methods 1981 (International Society for Rock Mechanics, 1979) (American Society for Testing and Materials International, 2018)

5.15.6.6 Liquefaction Potential Assessment

Analysis of the field investigation and laboratory testing programme results indicated that the site has soils that exhibit a liquefaction potential. This is primarily supported by the fact that confined strata consisting of non-cohesive soils of variable consistency were identified below the groundwater table by SPT tests and Vs profiling.

Utilising methodologies proposed by the NCEER (National Centre for Earthquake Engineering Research, 2001), a liquefaction potential assessment was undertaken for the site and the methodology used is summarised in this section.

The following SPT data were available for the liquefaction potential assessment:

- 408 SPT tests from 2008 campaign;
- 159 SPT tests from 2021 campaign;
- 685 SPT tests from historical data.

These SPT data provide a good geographical representation of the nature of the site soils. Based on the methodologies recommended in (National Centre for Earthquake Engineering Research, 2001), the SPT results were corrected by applying scaling factors for the following physical influences:

- overburden Pressure – CN;
- SPT hammer Energy Ratio – CE;
- borehole diameter – CB;
- drilling rod length – CR;
- sampling method – CS;
- soil fines content.

The resultant corrected SPT N blow counts, referred to as the $(N_1)_{60CS}$ blow counts were used to calculate the Cyclic Resistance Ratio (CRR)[10] for each SPT test. The CRR represents the capacity of the soils to withstand liquefaction under certain loads (demands) resulting from ground shaking.

The Cyclic Stress Ratio (CSR)[11], representing loads from ground shaking (or demand) was calculated for an event as described in the PSHA (Stamatakos & Watson-Lamprey, 2024) which concludes that, at a probability of exceedance of 10^{-4}, the PGA for the site is conservatively 0.4g with an earthquake magnitude of $M_{6.5}$.

If the CSR exceeds the CRR, the soils are potentially liquefiable. However, another prerequisite for liquefaction to occur is that the soils must be saturated (i.e. be located below the groundwater table). The details of proposed foundations for the nuclear installation(s) are not yet known and proposed construction methodology is also not known (except that a large excavation requiring dewatering will be made). Since construction methodology could involve implementing subsurface hydraulic barriers to support dewatering, scenarios may develop during construction that alter the groundwater table (upwards) in certain areas on the site (e.g. the groundwater table may rise up-gradient of hydraulic barriers). For this reason, the liquefaction potential assessment carried out for this SSR assumes that liquefaction could in fact occur both above and below the current groundwater table as fixed at the date of this report.

5.15.6.7 Geomorphology Assessment

(Illenberger, 2010) describes the dune systems at the Duynefontyn site in detail as referenced in ***Figure 5.15.3*** (a direct extract from (Illenberger, 2010)). The following is noted from Figure 5.15.3 and (Illenberger, 2010):

- Mobile dunes (active transverse dunes) occur in the northern portion of the Duynefontyn site. These active transverse dunes migrate in a northerly direction (away from KNPS and any proposed new nuclear installation(s)) driven by the predominantly southerly winds.

- Mitigation involves stabilising the mobile dunes with drift fences, brushwood and with pioneer indigenous dune vegetation – the artificially stabilised dunes just north of KNPS are testament to the efficacy of such

10 As a function of corrected SPT N value and hypothetical earthquake magnitudes.

11 As a function of hypothetical PGA and the stress environment in which the SPT tests was carried out.

mitigation measures.

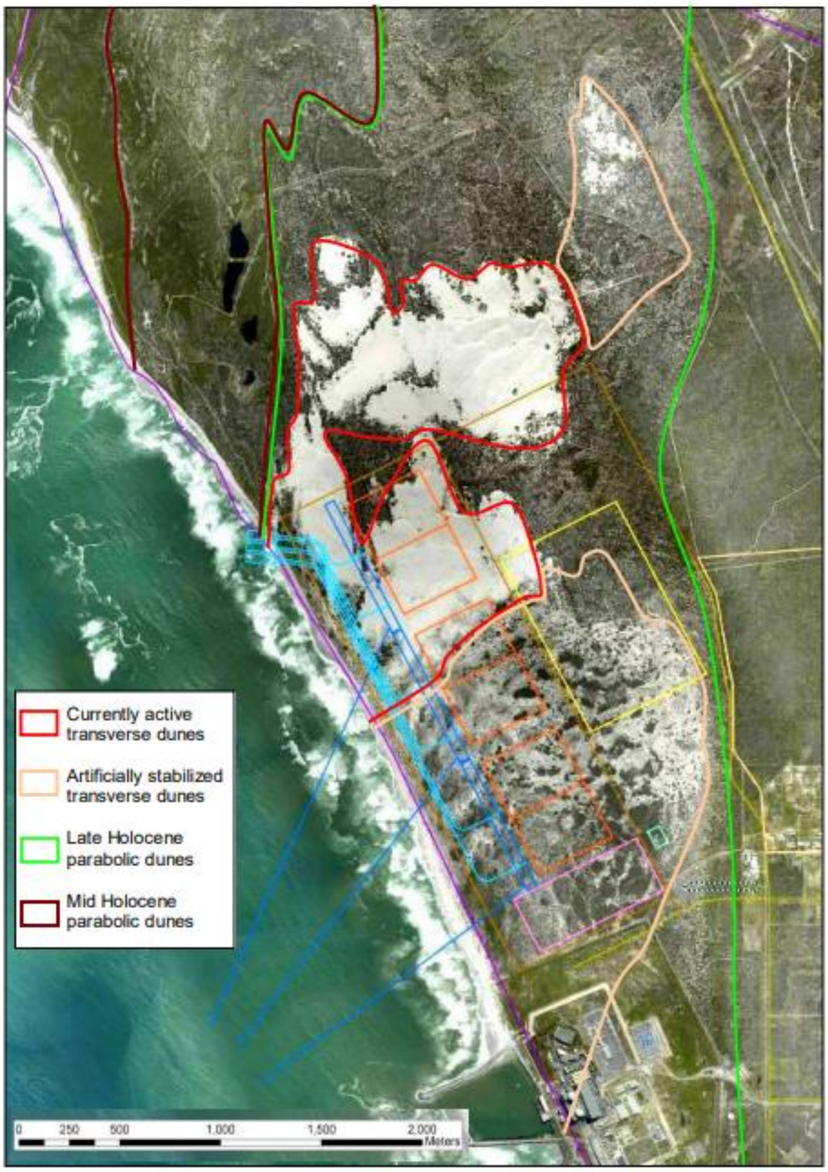


Fig 2.2. Dunefield varieties at Duynefontein and surroundings. The dunefield forms part of the Atlantis corridor dunefield. In order to make the map more legible, the zones are not shaded. The area inside each demarcation line is the corresponding zone.

Figure 5.15.3: Duynefontyn Dune Distribution (Illenberger, 2010)

5.15.6.8 Surface Investigations

The excessive thickness of the overburden sand at the site precluded any extensive surface investigations at the site (e.g. test pits) and drilling supported by geophysical investigations was the preferred method of exploring the characteristics of this thick overburden horizon.

As has been previously mentioned, rock outcrop is not present on the site for detailed surface mapping of rock structure. It is envisaged that this will be done when the foundations excavations are made for new nuclear installation(s), emulating the experience at KNPS.

5.15.6.9 Human Induced Conditions

(United States Nuclear Regulatory Commission, 2003) highlights the following human induced issues that may impact on site safety in the geotechnical context:

- the location of infrastructure, together with dams or reservoirs whose locations may cause a flooding hazard or produce loading effects at the site;
- past or on-going activities, such as mining or oil and gas production, and other fluid extraction or injection that may have impacted on subsurface conditions;
- the presence of former industrial sites, underground storage tanks, or landfills and the potential for hazardous, toxic, or radioactive waste being present.

Apart from the potential that subsurface hydraulic barriers may alter the groundwater table (see Section 5.15.6.6), the site currently does not present any human induced conditions that could impact on site safety in the geotechnical context with specific reference to the above and such are not expected to be imposed during the life of the proposed nuclear installation(s). This statement is supported by the presence of the KNPS and the mandatory exclusion zones that are implicit in the development of nuclear sites in South Africa.

5.15.6.10 Summary of Current and Future Investigations

(a) Current Investigations (Characterisation Stage)

Additional geotechnical characterisation of the site was launched in October 2007 (culminating in 2008) to supplement the historical investigations carried out and an SSR was developed at that stage. A current investigation campaign (the 2021 campaign) as detailed in this report further supplemented the database as did an investigation conducted for the PSHA (Stamatakos & Watson-Lamprey, 2024) including drilling and Vs profiling in 2023. These programmes focussed on the verification and, where possible, the confirmation stages and included:

- review of available information and in particular reports (Brink, 1981), (Brink, 1985) and (Eskom, 2006) in the 2008 campaign, supplemented with (Eskom, 2017) and (Eskom, 2020) in the 2021 campaign;
- intrusive field investigations including:
  - drilling of geotechnical core boreholes;
  - SPT tests in soils;
  - point load testing of rocks;
  - geophysical investigations to supplement the SPT data for liquefaction potential assessments (MASW and downhole seismic as well as PS suspension log profiling);
- interpretation of field information in the form of detailed borehole logs and rock joint condition assessments;
- liquefaction[12] potential investigations that comprised of analysis of SPT and Vs tests in site soils;
- laboratory testing of selected soil and rock samples, including:
  - soil characterisation tests with grading analyses, hydrometer and Atterberg Limits[13] analyses;
  - soil density testing on undisturbed samples;
  - rock uniaxial compressive strength (UCS) testing with measurement of Young’s Modulus (E) and Poisson’s ratio (v) and rock bulk and dry density;

[12] Liquefaction: A process, in which, some sandy, water-saturated soils can behave like liquids rather than solids. Liquefaction is caused by a sudden loss of shear strength and rigidity of saturated, cohesionless soils.

[13] Atterberg Limits: A basic measure of the nature of a fine-grained soil, the behaviour of which depends on the water content of the soil – the soil may appear in four states: solid, semi-solid, plastic and liquid, in each state the consistency and behaviour of a soil is different and thus so are its engineering properties.

- monitoring of groundwater levels and integration with monitoring results presented in ***Section 5.11***;
- analysis of data obtained from historical, field and laboratory sources;
- review of the above processes by an independent peer reviewer.

(b) Pre-operational Stage

Prior to this stage, investigations will continue until the foundation excavations are completed and the bedrock is exposed in this/these excavation/s and mapped. This will complete the data collation allowing the pre-operational phase to commence.

(c) Operational Stage

During the operation of the nuclear installation(s), the settlement of structures and water table levels will be measured (International Atomic Energy Agency, 2004). These data will be compared with predictions to enable an updated safety assessment for the nuclear installation(s) to be made. The type of parameters, the frequency of their measurement and all related activities will be addressed in the maintenance programme of the nuclear installation foundations (International Atomic Energy Agency, 2004).

5.15.7 Investigation Results

The results of the intrusive investigation are voluminous. For this reason a summary of the results of these investigations is presented here which is supported by detailed information in the following appendices:

- borehole logs of the 2008 and 2021 campaigns in ***Appendix 5.15.B***;
- borehole core photographs of the 2008 and 2021 campaigns in ***Appendix 5.15.C***;
- joint condition logs of the 2008 and 2021 campaigns in ***Appendix 5.15.D***;
- SPT test result plots of the 2008 and 2021 campaigns in ***Appendix 5.15.E*** and individual results in ***Appendix 5.15.B***;
- DPSH test result plots of the 2008 campaign in ***Appendix 5.15.F***;
- Point Load test results of the 2008 and 2021 campaigns in ***Appendix 5.15.B***;

- groundwater table measurements in ***Section 5.15.5.3***.

5.15.7.1 Soil Profile

The geographical distribution of the soils at the site can be summarised as follows (see also ***Drawings 5.15.1*** to ***5.15.12***):

- The sand overburden depth varies from a minimum of 12.0 m to a maximum of 39.0 m, averaging about 21.0 m (see ***Drawings 5.15.3*** to ***5.15.12***). Indications are that there is a general trend of the sand overburden thickness increasing with increasing distance from the coast.
- The average elevation of the bedrock at the site is -9.7 m msl with localised levels varying between 6.5 m msl and -24.4 m msl indicating the undulating nature of the wave cut terrace (see ***Drawing 5.15.2***), and a marginal increase in bedrock level moving away from the coastline.
- Within these sand deposits, the water table is generally between 3 and 5 m from surface and the soils at this site are therefore mostly saturated (see ***Drawings 5.15.3*** to ***5.15.12***).
- The lithology of the site soil profile is described in ***Section 5.15.5.2.3*** and ***Drawings 5.15.3*** to ***5.15.12***.
- Occasional gravel or cobbles (typical of a wave-cut platform environment) and/or residual Malmesbury Group soils (generally manifesting as thin horizons of clay) occur at the sand/bedrock contact.
- The consistency of the sand deposits is variable and is a function of the depositional environment – measured consistencies range from loose to very dense where calcrete occurs but medium dense to depth where medium dense consistency is predominantly not surpassed throughout the soil profile. A trend that is evident is that soil consistency generally starts off as very loose to loose at surface (in the dunes) and gradually improves to on average medium dense (dense and very dense in paces). See ***Appendix 5.15.E*** for more detail.
- The soil profile has been variably calcretised and thin hardpan calcrete layers or concretions randomly occur nearer the surface rendering probe testing (DPSH) largely unsuccessful due to shallow refusal.
- There appears to be no trend in soil density between the aeolian and water deposited soils. Some of the SPT plots show a marked increase in consistency when entering the marine sands and some show a

decrease, with the remainder showing no change outside of a gradual improvement in density with depth due to normal consolidation.

- Residual soils derived from weathering of the underlying Malmesbury Group rocks are not prominent at the site and were presumably eroded off in geological history.

Borehole samples from the drilling programme were logged and colour, consistency (guided by SPT tests), soil type (guided by laboratory testing, see ***Subsection 5.15.6.4***), structure and origin were documented (see ***Appendices 5.15.B*** and ***5.15.C***). The site soils are dominated by aeolian and marine sand deposits.

SPT plots indicating corrected N (i.e. ($N1_{60CS}$)) are presented in ***Appendix 5.15.E*** (where they are superimposed on the position of the groundwater table described in ***Section 5.11***). Individual SPT results are shown on the borehole logs in ***Appendix 5.15.B***.

Numerous SPT profiles show only a gradual (or no) increase with depth and soil consistency ranges from loose (surface) to medium dense (bedrock). The position of the groundwater table and the different soil horizons (aeolian and marine) do not appear to influence the SPT test results to the point where any trends are evident. SPT profiles provided the best assessment of the soil characteristics as DPSH tests tended to refuse on random (but normally relatively shallow) dense/pedogenic horizons.

Refinement of the geotechnical characterisation process was strongly influenced by the variability in site soil consistency, particularly in soil horizons below the groundwater table where loose to medium dense material was widely encountered. Representative sampling proved practically impossible in the 2008 campaign and SPT tests therefore became the primary source of characterisation data within the soils (backed up by index tests of these disturbed samples).

The 2021 campaign attempted Shelby tube sampling with marginal success, and the majority of Shelby tubes physically buckled during the sampling task (particularly at depth in confined soils). Shelby samples successfully taken were subjected to laboratory testing aimed at determining *insitu* density.

Drawing 5.15.2
Wave-cut Platform and Ground Elevation Contours

5.15.7.2 Rock Profile

Borehole core was logged (***Appendix 5.15.B***), photographed (***Appendix 5.15.C***) and subjected to point load testing to calibrate physical descriptions in the borehole logs. Logging procedures included a description of rock colour, weathering profile, structure, hardness and geological origin. This logging profile was extended by the peer review influence during the 2008 campaign to include a detailed assessment of the rock joint conditions (***Appendix 5.15.D***).

The results of the rock investigations demonstrate that the site is underlain by a wave-cut platform consisting of Malmesbury Group rocks characterised with randomly alternating greywacke, shales, mudstones, sandstones and siltstones and metamorphosed equivalents (see ***Section 5.13*** of this SSR).

5.15.7.2.1 Wave-cut Platform

The wave-cut platform has the following physical characteristics as presented in ***Drawings 5.15.1*** to ***5.15.12***:

- The wave-cut platform consists of rocks of the Malmesbury Group dipping steeply (75˚) at 320˚<strike<330˚ (note that cross sections presented in the drawings are exaggerated 5 times vertically).

- The average elevation of the bedrock at the site is -9.7 m msl with localised levels varying between 6.75 m msl and -24.4 m msl. This large variance is ascribed to the fact that the exposed wave cut platform consists of alternating rocks of varying type and integrity that have been subjected to varying degrees of weathering and erosion. It can be noted from ***Drawings 5.15.2*** that where borehole density is high, the bedrock contours tend to indicate sharp transitions in the elevation of the wave cut platform – it is postulated that where borehole density is high across the entire site, similar transitions in wave cut platform elevation would be measured. It is noted from ***Drawings 5.15.2*** that several depressions exist in the wave cut platform as depicted by the bedrock contours.

- The steeply dipping rocks are dominated by greywacke and to a lesser extent sandstone, but interbedded mudstone/siltstone/shale sequences are regularly present. The greywacke (and to a lesser extent sandstone when quartzitic) is more competent than the other rocks and wave-cut platform levels are probably influenced by this.

- There is a marginal trend of the bedrock level decreasing towards the coastline (see ***Drawings 5.15.2*** to 5***.15.12*** noting that the sections in these drawings are exaggerated five times vertically).

- In summary, the site bedrock surface consists of an undulating wave-cut platform variably positioned with respect to mean sea level and presenting peaks and troughs in some areas.

Drawing 5.15.3
North-South Section 52 000: Lithology, Vs and Liquefaction FoS

Drawing 5.15.4
North-South Section 52 500: Lithology, Vs and Liquefaction FoS

Drawing 5.15.5
North-South Section 53 000: Lithology, Vs and Liquefaction FoS

Drawing 5.15.6
North-South Section 53 500: Lithology, Vs and Liquefaction FoS

Drawing 5.15.7
East-West Section 3 275 500: Lithology, Vs and Liquefaction FoS

Drawing 5.15.8
East-West Section 3 276 000: Lithology, Vs and Liquefaction FoS

Drawing 5.15.9
East-West Section 3 276 500: Lithology, Vs and Liquefaction FoS

Drawing 5.15.10
East-West Section 3 277 000: Lithology, Vs and Liquefaction FoS

Drawing 5.15.11
East-West Section 3 277 500: Lithology, Vs and Liquefaction FoS

Drawing 5.15.12
East-West Section 3 278 000: Lithology, Vs and Liquefaction FoS

5.15.7.2.2 Point Load Index

Summary statistics of point load test results carried out on geotechnical core are shown in ***Table 5.15.19.*** Point load tests were carried out during the fieldwork programme primarily to calibrate visual description of the site rock hardness only, and not as a primary means of establishing rock strength. Point load tests were consistently orientated such that the tests were conducted perpendicular to bedding and/or jointing evident in the core section being tested – all of the data/statistics presented in this section are therefore relevant to testing done perpendicular to bedding and/or jointing.

Table 5.15.19
Statistics of Point Load Test Results - Summary

Parameter	UCS Derived from PLI[14] (MPa)
Mean	77.1
Standard Deviation	71.6
Sample[15]	248
Range	409
Minimum	0.85
Maximum	410

Comments on the point load test results are:

- The mean PLI derived UCS is 77 MPa with a standard deviation of 72 MPa and there is thus a wide range in values.
- Test results have a positively skewed distribution (see ***Figure 5.15.4***).
- Besides informing physical descriptions and indicating potential anisotropy[16] (***Appendix 5.15.B***), the point load test results lend no other value to the geotechnical characterisation of the site.
- It should be observed that the sampling process was affected by the available lengths of intact core (see ***Appendix 5.15.C***). This resulted in point load tests being carried out on samples less frequently jointed and

[14] Point Load Index
[15] Number of tests carried out.
[16] Anisotropy is the property of being directionally dependent.

may have biased higher results.

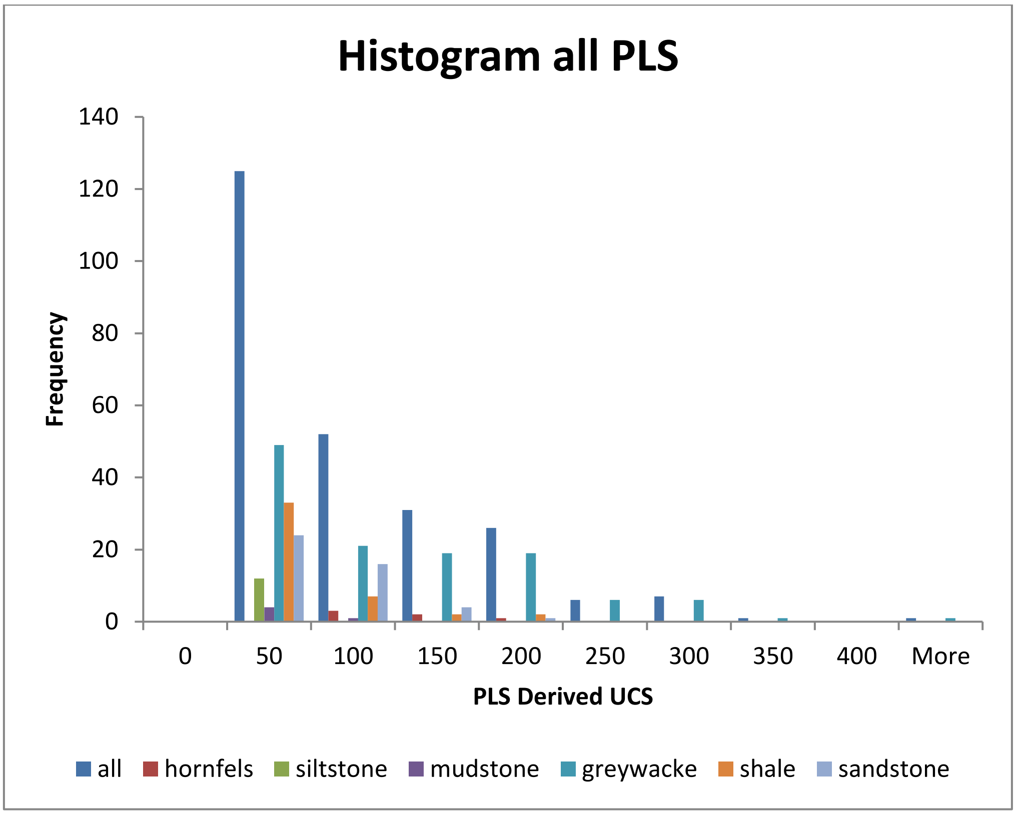


Figure 5.15.4: Point Load Index

5.15.7.2.3 Rock Discontinuities

(a) Jointing

The high variability in the geotechnical properties of the rocks underlying the site has been confirmed. Discontinuities in the site rocks contribute to this variability. Refinement of the geotechnical characterisation approach (***Figure 5.15.1***) recognised this and a greater emphasis was placed on more detailed description of the rock joint conditions in the 2008 field investigation campaign.

Boreholes have therefore been logged with the additional aim of describing joint conditions in greater detail and these logs are included in ***Appendix 5.15.D***. Over 4 000 joints were described in the Malmesbury Group rocks across the 2008 and 2021 field investigation campaigns. ***Table 5.15.20*** summarises the joint conditions from the voluminous information contained in ***Appendix 5.15.D***. ***Table 5.15.20*** represents statistics on the entire data set contained in ***Appendix 5.15.D***.

To further explore the influence that rock jointing has on the geotechnical profile at the site, the data contained in ***Appendix 5.15.D*** was evaluated in various other ways as presented in:

- ***Figure 5.15.5:*** showing a histogram of the fracture frequency of the entire data set.
- ***Figure 5.15.6:*** showing a histogram of the measured RQD.

Utilising these data, (Laubscher, 1990) was then used to calculate rock mass ratings (RMRs). Means were calculated for the Malmesbury Group rocks based on weighting each geotechnical interval according to its logged length, resulting in longer intervals contributing more to the overall mean. The results of the RMR calculations are presented in ***Table 5.15.21***.

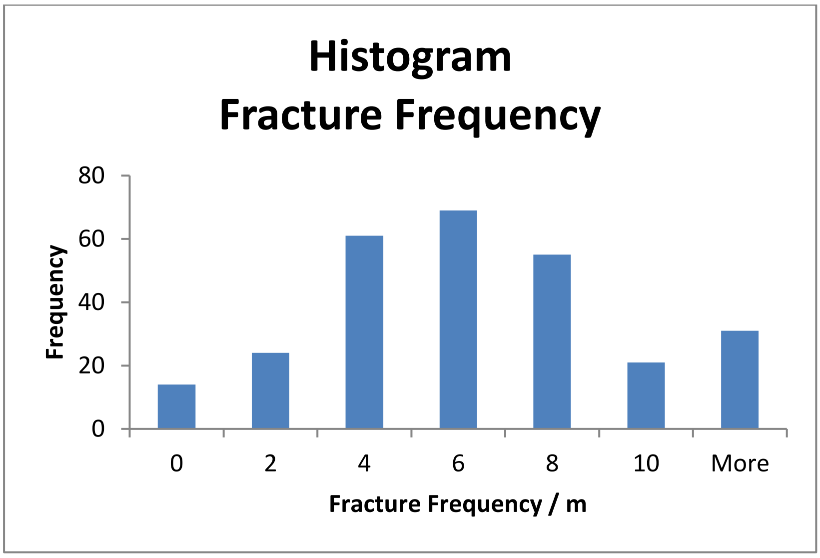


Figure 5.15.5
Fracture Frequency

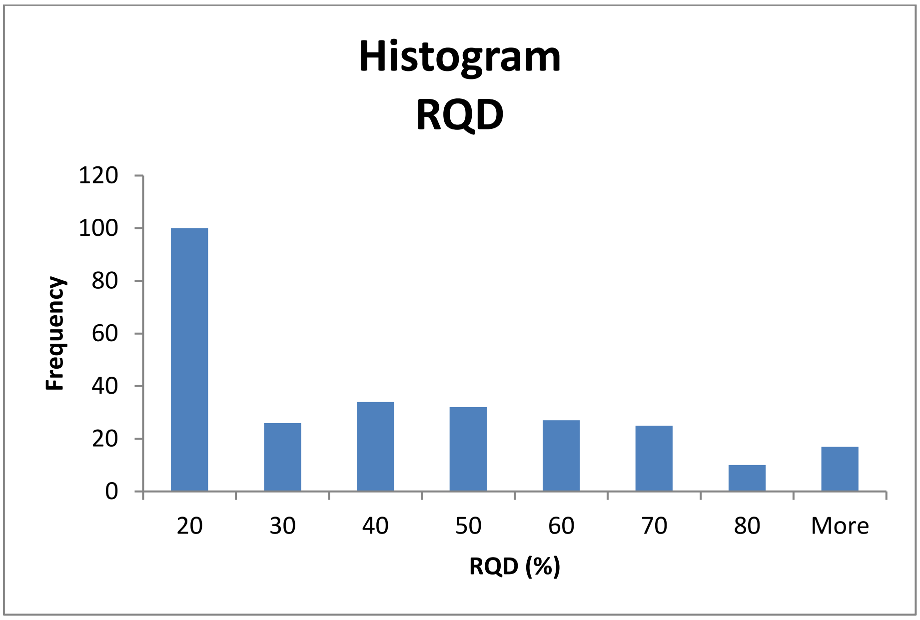


Figure 5.15.6
Rock Quality Designation

Table 5.15.20
Summary of Rock Joint Conditions

Parameter	Malmesbury Group			Remarks
	Min	Average	Max	
RQD (%)	0.00	34.31	100.00	RQD = % of intact core > 100 mm in length in a geotechnical interval or run
Recovered Core				
% solids			68.07%	'Solids' = recovered core still in an intact cylindrical state
% matrix			31.93%	'Matrix' = recovered core consisting of broken rock fragments
Joint Distribution				
	JT1 sub-hor.	JT2	JT3 sub-vert.	
Number Logged	1663	1247	1309	The physical number of each joint set logged
Percentage (%)	39%	30%	31%	The % of the total number of joints logged
Joint Surface Condition				
Macro (%)	JT1 sub-hor.	JT2	JT3 sub-vert.	The macro joint cond. denoted as a % of the total number of joints logged. All three joint sets are dominated by straight joints (and to a lesser extent slightly undulating) on a macro scale.
Straight	70%	62%	66%	
Slight undulating	14%	19%	14%	
Curved	8%	11%	12%	
Uni-directional, wavy	7%	7%	5%	
Multi-directional wavy	1%	2%	3%	
Micro (%)	JT1 sub-hor.	JT2	JT3 sub-vert.	The micro joint condition denoted as a % of the total number of joints logged. All three joint sets are dominated by smooth planar (and to a lesser extent smooth undulating) joints on a micro scale.
Polished	2%	0%	0%	
Smooth Planar	63%	63%	55%	
Rough Planar	7%	2%	11%	
Slickensided Undulating	0%	0%	0%	
Smooth Undulating	19%	22%	21%	
Rough Undulating	3%	6%	5%	
Slickensided Stepped	0%	0%	0%	
Smooth Stepped	4%	5%	3%	
Rough Stepped/Irregular	3%	3%	4%	
Joint Infill (%)	JT1 sub-hor.	JT2	JT3 sub-vert.	The joint infill type denoted as a % of the total number of joints logged. Joints commonly have no infill, but when present infill is sheared and stained.
Irregular	0%	0%	0%	
Soft, Sheared Fine	3%	6%	6%	
Soft, Sheared Medium	0%	0%	0%	
Soft, Sheared Coarse	0%	0%	0%	
Non-Softening, Sheared Fine	8%	10%	3%	
Non-Softening, Sheared Medium	1%	0%	0%	
Non-Soft., Sheared Coarse	0%	0%	0%	
Non-Soft., Sheared Staining	19%	19%	13%	
None	69%	66%	77%	
Wall Alteration (%)	JT1 sub-hor.	JT2	JT3 sub-vert.	Describing the joint wall alteration and denoting this as a % of the total number of joints logged. Joint walls have the same hardness as the rock.
Wall = rock hardness	100%	100%	100%	
Wall >rock hardness	0%	0%	0%	
Wall < rock hardness	0%	0%	0%	

Table 5.15.21
Rock Mass Ratings

Rock	Parameter	RMR [17]	Class	Blasting	Weather.	Orient.	Stress	MRMR [18]	Class	Typical Excavation Support Required	
GRY 114 Intervals	Min	30	4B	0.94	0.96	0.8	0.97	21	4B	c+1	Bolts (1 m spacing) + straps & mesh if rock is finely jointed
	Weighted Mean	49	3B	0.94	0.96	0.8	0.97	34	4A	d	Bolts (1 m spacing) + mesh/steel-fibre reinforced shotcrete bolts as lateral restraint
	Max	84	1B	0.94	0.96	0.8	0.97	59	3A	b	Bolts (1 m spacing)
	Weighted S. Dev.	8						6			
HRN 5 Intervals	Min	42	3B	0.94	0.96	0.8	0.97	29	4B	f	Bolts (1 m spacing) + mesh/steel-fibre reinforced shotcrete bolts as lateral restraint + straps in contact with or shotcrete in and cable bolts as reinforcing and lateral restraint
	Weighted Mean	49	3B	0.94	0.96	0.8	0.97	35	4A	d	Bolts (1 m spacing) + mesh/steel-fibre reinforced shotcrete bolts as lateral restraint
	Max	63	2B	0.94	0.96	0.8	0.97	44	3B	b	Bolts (1 m spacing)
	Weighted S. Dev,	6						4			
MGR 1 Interval	Min	50	3B	0.94	0.96	0.8	0.97	35	4A	d	Bolts (1 m spacing) + mesh/steel-fibre reinforced shotcrete bolts as lateral restraint
	Weighted Mean	50	3B	0.94	0.96	0.8	0.97	35	4A	d	Bolts (1 m spacing) + mesh/steel-fibre reinforced shotcrete bolts as lateral restraint
	Max	50	3B	0.94	0.96	0.8	0.97	35	4A	d	Bolts (1 m spacing) + mesh/steel-fibre reinforced shotcrete bolts as lateral restraint
	Weighted S. Dev,	-						-			
MSH 1 Interval	Min	42	3B	0.94	0.84	0.8	0.97	26	4B	f	Bolts (1 m spacing) + mesh/steel-fibre reinforced shotcrete bolts as lateral restraint + straps in contact with or shotcrete in and cable bolts as reinforcing and lateral restraint
	Weighted Mean	42	3B	0.94	0.84	0.8	0.97	26	4B	f	Bolts (1 m spacing) + mesh/steel-fibre reinforced shotcrete bolts as lateral restraint + straps in contact with or shotcrete in and cable bolts as reinforcing and lateral restraint
	Max	42	3B	0.94	0.84	0.8	0.97	26	4B	f	Bolts (1 m spacing) + mesh/steel-fibre reinforced shotcrete bolts as lateral restraint + straps in contact with or shotcrete in and cable bolts as reinforcing and lateral restraint
	Weighted S. Dev,	-						-			

[17] Rock Mass Rating
[18] Mining Rock Mass Rating

Rock	Parameter	RMR[17]	Class	Blasting	Weather.	Orient.	Stress	MRMR[18]	Class		Typical Excavation Support Required
MUD 7 Intervals	Min	28	4B	0.94	0.84	0.8	0.97	17	5A	h+f/p	Spilling plus Bolts (1 m spacing) + mesh/steel-fibre reinforced shotcrete bolts as lateral restraint + straps in contact with or shotcreted in and cable bolts as reinforcing and lateral restraint
	Weighted Mean	45	3B	0.94	0.84	0.8	0.97	28	4B	f	Bolts (1 m spacing) + mesh/steel-fibre reinforced shotcrete bolts as lateral restraint + straps in contact with or shotcrete in and cable bolts as reinforcing and lateral restraint
	Max	60	2B	0.94	0.84	0.8	0.97	37	4A	c	Bolts (1 m spacing) + straps + mesh if rock is finely jointed
	Weighted S. Dev,	12						7			
QST 78 Interval	Min	40	3B	0.94	0.96	0.8	0.97	28	4B	f	Bolts (1 m spacing) + mesh/steel-fibre reinforced shotcrete bolts as lateral restraint + straps in contact with or shotcrete in and cable bolts as reinforcing and lateral restraint
	Weighted Mean	40	3B	0.94	0.96	0.8	0.97	28	4B	f	Bolts (1 m spacing) + mesh/steel-fibre reinforced shotcrete bolts as lateral restraint + straps in contact with or shotcrete in and cable bolts as reinforcing and lateral restraint
	Max	40	3B	0.94	0.96	0.8	0.97	28	4B	f	Bolts (1 m spacing) + mesh/steel-fibre reinforced shotcrete bolts as lateral restraint + straps in contact with or shotcrete in and cable bolts as reinforcing and lateral restraint
	Weighted S. Dev,	-						-			
SHL 55 Intervals	Min	28	4B	0.94	0.84	0.8	0.97	17	5A	h+f/p	Spilling plus Bolts (1 m spacing) + mesh/steel-fibre reinforced shotcrete bolts as lateral restraint + straps in contact with or shotcreted in and cable bolts as reinforcing and lateral restraint
	Weighted Mean	52	3A	0.94	0.84	0.8	0.97	32	4A	c	Bolts (1 m spacing) + straps + mesh if rock is finely jointed
	Max	85	1B	0.94	0.84	0.8	0.97	52	3A	b	Bolts (1m spacing)
	Weighted S. Dev,	10						6			
SLT 13 Intervals	Min	44	3B	0.94	0.84	0.8	0.97	27	4B	f	Bolts (1 m spacing) + mesh/steel-fibre reinforced shotcrete bolts as lateral restraint + straps in contact with or shotcrete in and cable bolts as reinforcing and lateral restraint
	Weighted Mean	52	3A	0.94	0.84	0.8	0.97	32	4A	c	Bolts (1 m spacing) + straps + mesh if rock is finely jointed
	Max	81	1B	0.94	0.84	0.8	0.97	50	3B	b	Bolts (1m spacing)
	Weighted S. Dev,	8						5			
VUG 1 Interval	Min	86	1B	0.94	0.96	0.8	0.97	60	2B	a	Local bolting at joint intersections
	Weighted Mean	86	1B	0.94	0.96	0.8	0.97	60	2B	a	Local bolting at joint intersections
	Max	86	1B	0.94	0.96	0.8	0.97	60	2B	a	Local bolting at joint intersections
	Weighted S. Dev,	-						-			

Table 5.15.22
Fracture Frequency Statistics

Fracture Frequency Statistics	
Mean	5.75
Median	5.38
Standard Deviation	3.14
Range	18.12
Minimum	0.40
Maximum	18.52

Other highlighted results from ***Tables 5.15.20*** to ***5.15.22*** as well as ***Figures 5.15.5*** and ***5.15.6*** are:

- The RQD has a mean of 34 per cent and borehole core recovery (i.e. core successfully extracted out of boreholes) was on the whole good, highlighting generally good quality drilling, but it is noted that 32 per cent of the core recovered consisted of broken rock fragments.

- Bedding joints, sub-vertical joints and sub-horizontal joints appear to be equally developed/distributed and none of these joint sets totally dominate.

- All joints on a macro-scale are dominantly straight, but slightly undulating on occasion and on a micro scale predominantly smooth planar, but occasionally smooth undulating.

- Joint infill is largely not present, but when present is non-softened sheared material that is stained.

- Joint wall alteration is globally equal to rock hardness in all joint sets.

- Joints are spaced 5.75 m on average (standard deviation of 3.14 m).

(b) Faulting and Shear Zones

The intrusive investigations identified shearing and faulting in the following boreholes:

- faulting/brecciated zones:

  - KB21 (33.24 - 33.80 m): evidence of faulting;
  - KB26 (26.01 - 26.28 m): fault/shear zone;
  - KB33 (15.25 - 16.37 m): containing breccia;

- KB44 (47.53 – 49.05 m; 47.53 - 49.05 m): containing breccia;
- KB46 (73.85 - 74.88): fault/shear zone;
- KB47 (38.47 - 39.60 m; 56.94 - 58.00 m; 61.88 – 62.72 m; 71.95 - 79.90 m): fault/shear zone;
- KB48 (32.00 - 35.24 m): fault/shear zone;
- KB50 (at 65.00 m): fault/shear;
- KB53 (59.30 – 64.20 m; 67.24 - 73.30 – 76.20 m): brecciated material, shear/fault.

- shear zones:

  - KB1 (24.40 – 25.80 m): sheared;
  - KB8 (29.35 – 33.10 m; 33.65 – 35.95m): minor shearing in places;
  - KB12 (24.45 - 32.54 m): shear zones present;
  - KB14 (~25.00 m): sheared;
  - KB17 (20.50 – 20.60 m): sheared;
  - KB21 (19.95 - 29.45 m; 28.29 – 28.82 m): shear zones present;
  - KB26 (26.01 - 26.28 m): fault/shear zone;
  - KB27 (19.75 - 22.63 m): occasional shear zones;
  - KB29 (23.25 - 24.80 m): probable shear zones present;
  - KB33 (21.00 – 24.00 m): healed shear zones;
  - KB36 (26.76 - 27.04 m): shear zone;
  - KB40 (22.5 – 30.00 m): sheared in places;
  - KB44 (23.10 - 23.60 m; 49.05 – 54.78 m): shear zone / sheared in places;
  - KB46 (44.56 – 47.56 m; 73.85 - 74.88): fault/shear zone
  - KB47 (38.47 - 39.60 m; 43.20 – 43.93 m; 56.94 - 58.00 m; 61.88 – 62.72; 66.97 – 68.70 m; 71.95 - 79.90 m): fault/shear zone;
  - KB48 (32.00 - 35.24 m; 40.36 m): fault/shear zone;
  - KB49 (35.35 m; 36.23 m) clear shear zones;
  - KB50 (at 65.00 m): fault/shear;
  - KB51 (35.78 – 36.22 m; 36.27 – 40.05 m; 60.90 m; 61.35 m; 72.44 – 73.46 m; 73.68 – 73.77 m; 74.60 – 75.25 m): shear zone;
  - KB52 (22.55 – 38.09 m; 52.26 – 58.10 m): sheared.

Special mention of the faulted/sheared/brecciated zones in borehole KB53 from 59.30 – 76.20 m is made. The orientation of this brecciate/faulted zone is not traceable with the available data – ***Drawing 5.15.8*** indicates the position of this feature.

5.15.7.2.4 Rock Slope and Tunnel Stability

A site characteristic is the presence of three joint sets that are predominantly straight and smooth. There is little doubt that any excavations in rock on this site where the dominant joints daylight into cut slopes will require lateral support measures. Bedding is steeply dipping (75° striking at *c.*325°). Such lateral support measures will not be unduly onerous to design and will consist of conventional lateral support (e.g. as indicated in ***Table 5.15.21***). It is noted, however, that the Malmesbury Group rocks are characterised by randomly alternating beds of varying rock type/quality, and for this reason, it is anticipated that for practical purposes, a single lateral support type will probably be used as predicting the frequency of occurrence of the various rock types in order to design varying rock support methods to suit will not be possible (even when the foundation excavations are mapped in detail). Predominantly, ***Table 5.15.21*** suggests that rock bolts will be required at approximately 1 m spacing with mesh and shotcrete to secure smaller blocks.

Potential modes of failure resulting from the intersection of the dominant joint sets are discussed below and the following is noted:

- Any excavations can fail by sliding along the bedding plane if a rock face is oriented within 30° of the bedding strike. In such cases, the sub-vertical joints will form lateral release and the sub horizontal joints will provide upper release in the case of tunnels.
- Toppling slope failure can occur on the sub-vertical joints/bedding if slopes are oriented within 30° of the strike of the joints/bedding.
- Rock falls can occur from tunnel roofs on the sub-vertical joints, released by bedding or the sub-horizontal joints.

These failure mechanisms can be contained using standard rock support techniques as suggested in ***Table 5.15.21*** in conjunction with surface support like mesh and shotcrete and/or fibre reinforced shotcrete. As the joint conditions are generally consistent, the likelihood of failure will be influenced by the geometrical relationship between the joints and excavation orientation rather than based on the preferential weakness off a single joint set or rock type.

The weighted RMR results (***Table 5.15.21***) indicate 42 per cent < RMR < 86 per cent, which is interpreted as fair to good quality rock. The prevalence of smooth joint conditions adversely impacts the RMR. Adjusted rock mass ratings were calculated by reducing the RMR values to take account of induced impacts such as blasting, post exposure weathering, orientation of joints and stress, which were estimated for shallow excavations in these rocks types (utilising **(Laubscher, 1990)**). The resulting Mining Rock Mass Rating (MRMR) were used for estimating excavation support requirements as shown in ***Table 5.15.21***.

Empirical estimates **(Laubscher, 1990)** of allowable slope angles determined from the average adjusted rock mass rating indicate that rock slopes in the Malmesbury rocks should not be excavated at angles steeper than 45° for slopes up to 50 m in height in the greywacke, meta-greywacke, sandstone and hornfels units and 40° for the mudstone, siltstone and shale units. These are indicative slope angles for conceptual design purposes only, and individual excavations would require detailed design based on orientation and height of the cut. In addition, surface support may be required depending on the localised fracture spacings and/or blast/excavation induced fractures. Ravelling and toppling of small blocks and fragments that will require step by step excavation and support with rock bolts and mesh reinforced shotcrete could result. Steeper slope angles than those suggested above would be possible utilising appropriate lateral support.

5.15.7.3 Groundwater Monitoring Results

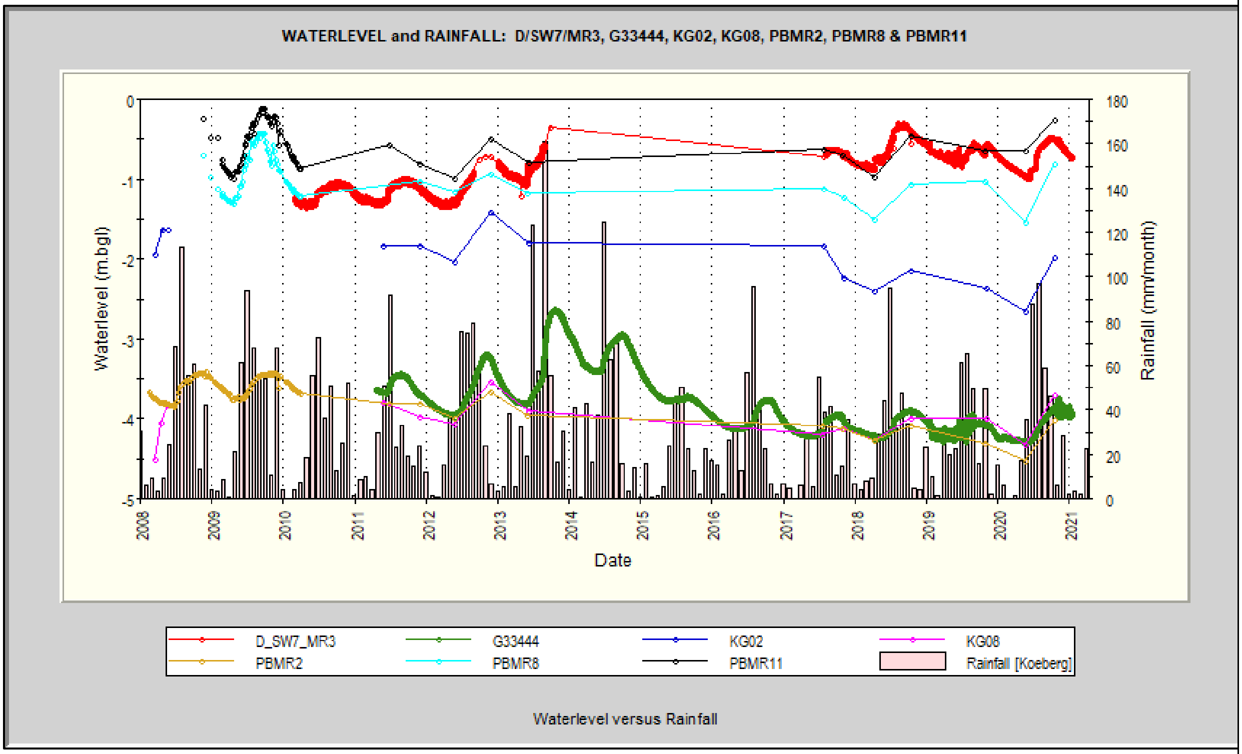
5.15.7.3.1 Groundwater Levels

Groundwater levels are monitored as detailed in ***Section 5.11.5*** (Geohydrology) – a section of this SSR that is supported by a continuous monitoring programme. ***Figure 5.15.7*** is an extract from the monitoring programme and shows:

- Groundwater level monitoring on a continuous basis using digital loggers and dipmeter measurements at sporadic intervals – the positions of the individual boreholes monitored are shown on ***Drawing 5.15.13.***

- Groundwater fluctuation with time against rainfall measured at the KNPS weather station – trends linked to groundwater table rise after the rainfall season is noted.

The groundwater level monitoring results (along with the calibrated hydrogeological model – see ***Section 5.11.7)*** demonstrate that the groundwater table contours are parallel to the coast and groundwater therefore flows perpendicular to the coast (see ***Drawing 5.15.5*****)**. to emphasise groundwater level and rainfall linkage, it is notable to see the gradual decline in groundwater table level during the 2015 to 2018 drought.



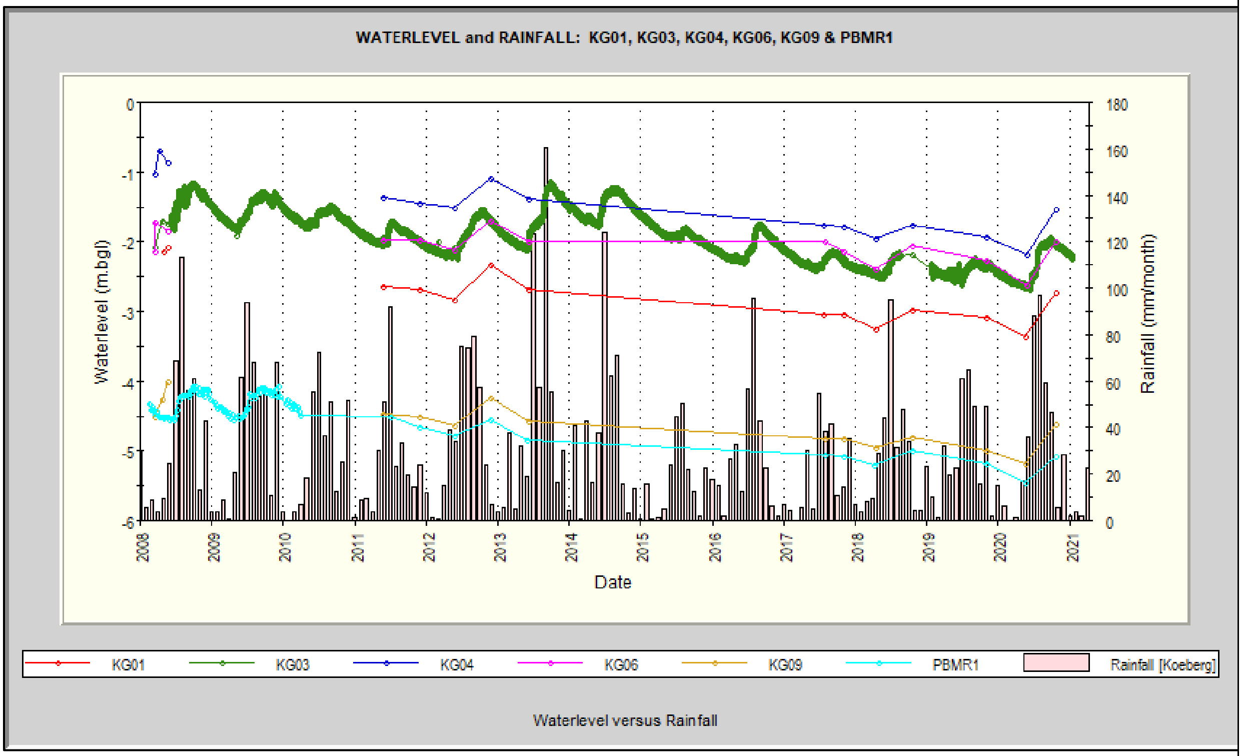


Figure 5.15.7
Measured Water Levels vs Rainfall (Ref Section 5.11)

Drawing 15.5.13 Groundwater Elevation Contours

5.15.7.4.1 Groundwater Chemistry

Details pertaining to the groundwater chemistry at the site are contained in ***Section 5.15.5.3***. Groundwater emanating from the Sandveld (soils) and the Malmesbury (rock) aquifers is not anticipated to present aggressiveness towards concrete with low sulfate concentrations. Groundwater from both aquifers, however, will be highly corrosive towards steel and concrete mix designs will need to consider restricting the permeability of concrete exposed to groundwater as a primary mitigation. In addition, steel reinforcing coatings are available and epoxy coatings are the most commonly used in industry.

5.15.7.5 Results of Laboratory Investigations

Laboratory test results are presented to confirm the geotechnical characteristics described in ***Subsection 5.15.7*** as follows:

- results of soil characterisation tests (including Atterberg Limits) in ***Appendix 5.15.G***;
- results of rock characterisation tests in ***Appendix 5.15.H***.

5.15.7.5.1 Soil Classification and Strength

The laboratory test results represent a wide geographical distribution of data points at the site (see ***Drawing 5.15.1*** and individual test results in ***Appendix 5.15.G***). Samples were taken and analysed to provide a three-dimensional data set representative of the site soils.

Characterisation of the site soils was a primary goal for this SSR. ***Table 5.15.23*** shows the summary statistics of the laboratory soil characterisation test results. The results of grading analyses are shown only as the soils are predominantly non plastic (with a few exceptions of ‘slightly plastic’ soils) and as such, Atterberg Limits were not measured. Grading statistics in ***Table 5.15.23*** are arranged by the typical soil units encountered at the site and as described in ***Section 5.15.5.3.3***.

Table 5.15.23
Summary Statistics of Laboratory Soil Characterisation Test Results

	Grading				Grading				Grading			
	Clay	Silt	Sand	Gravel	Clay	Silt	Sand	Gravel	Clay	Silt	Sand	Gravel
	% Passing	% Passing	% Passing	% Passing	% Passing	% Passing	% Passing	% Passing	% Passing	% Passing	% Passing	% Passing
	Bredasdorp Formation (Layer 1)				Springfontyn Formation (Layer 2)				Varswater Formation (Layer 3)			
Mean	1.9	1.8	95.5	0.8	1.9	2.9	94.3	0.8	2.5	4.9	92.2	0.3
Median	1	1	97	0	2	2.5	95.5	0	2	4.5	93.5	0
Standard Deviation	1.54	1.87	4.95	4.49	0.77	3.17	5.88	4.50	1.74	4.27	5.90	1.07
Range	7	10	27	25	2	17	32	27	6	15	20	4
Minimum	1	0	71	0	1	0	66	0	1	0	78	0
Maximum	8	10	98	25	3	17	98	27	7	15	98	4
Count	31	31	31	31	36	36	36	36	14	14	14	14

The site soils are:

- homogenous across the site and dominated by sand (of aeolian and marine origin primarily) with some fines present in the form of silt and marginal clay – a few exceptions are encountered where bedrock has weathered to clay or silt at the contact;
- non-plastic and poorly graded;
- the soils have an average SG of 2.56 with a standard deviation of 0.02.

Figure 5.15.8 shows the grading curves for the three typical soil units encountered at the site. The homogeneity of the site soils is a feature of ***Figure 5.15.8*** apart from a few outlier test results in the Springfontyn and Varswater Formations.

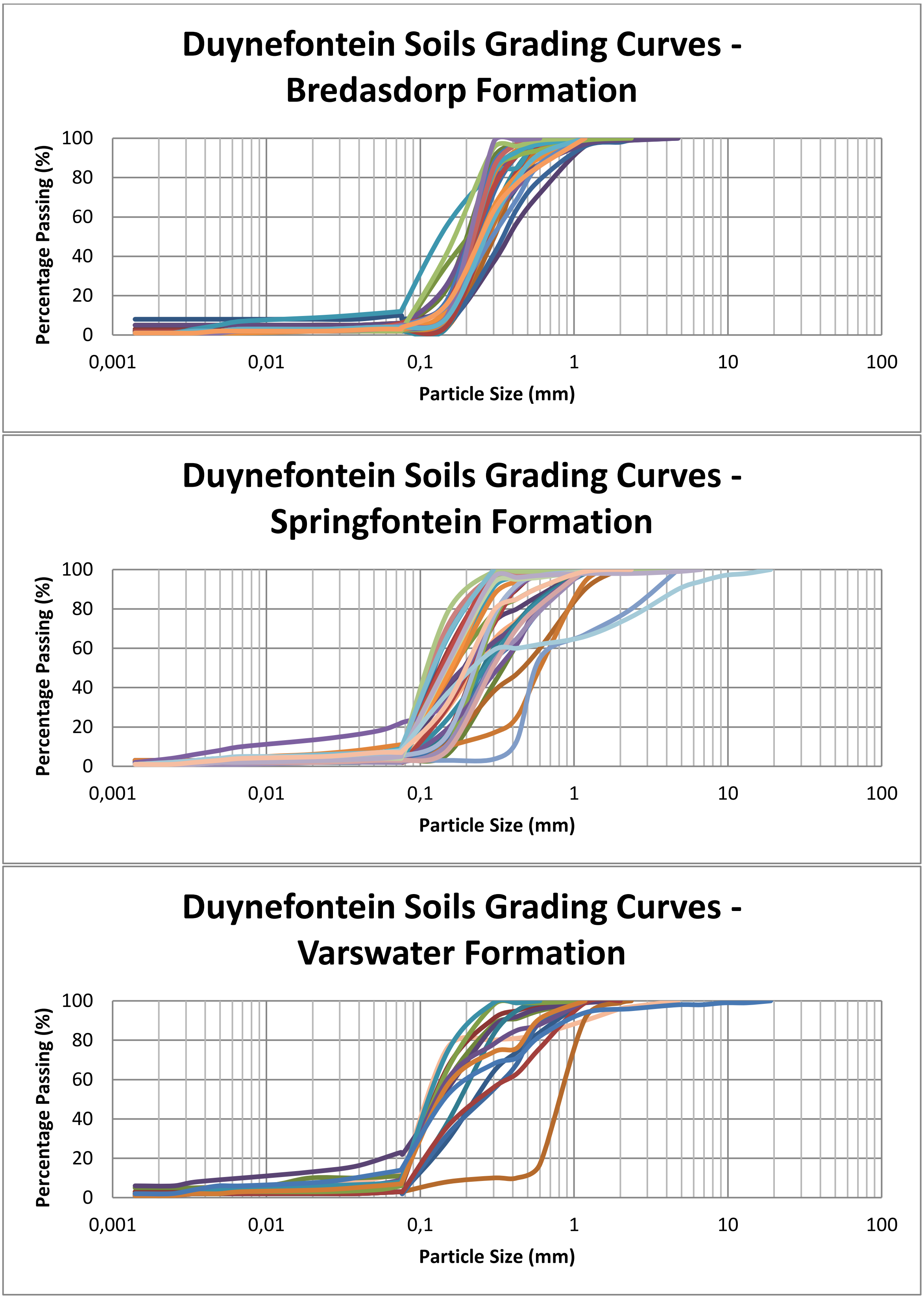


Figure 5.15.8
Grading Curves for Duynefontyn Soils

(a) Soil density

Because of the difficulty in obtaining undisturbed soil samples (see ***Subsection 5.15.6.2***), the laboratory testing programme did not target the *in situ* density of site soils in the 2008 campaign. To gain an understanding of the *in situ* density of site soils, an assessment of *in situ* density was carried out based on the SPT test results. ***Figure 5.15.9*** shows the average particle size of the site soils as this is a key input parameter into the density assessment using SPT data.

(Hogegentogler, et al., 1937) and (Cubrinovski & Ishihara, 2001) were used as a basis for this assessment and ***Table 5.15.24*** shows the results. ***Figure 5.15.10*** and ***Figure 5.15.11*** show these results graphically.

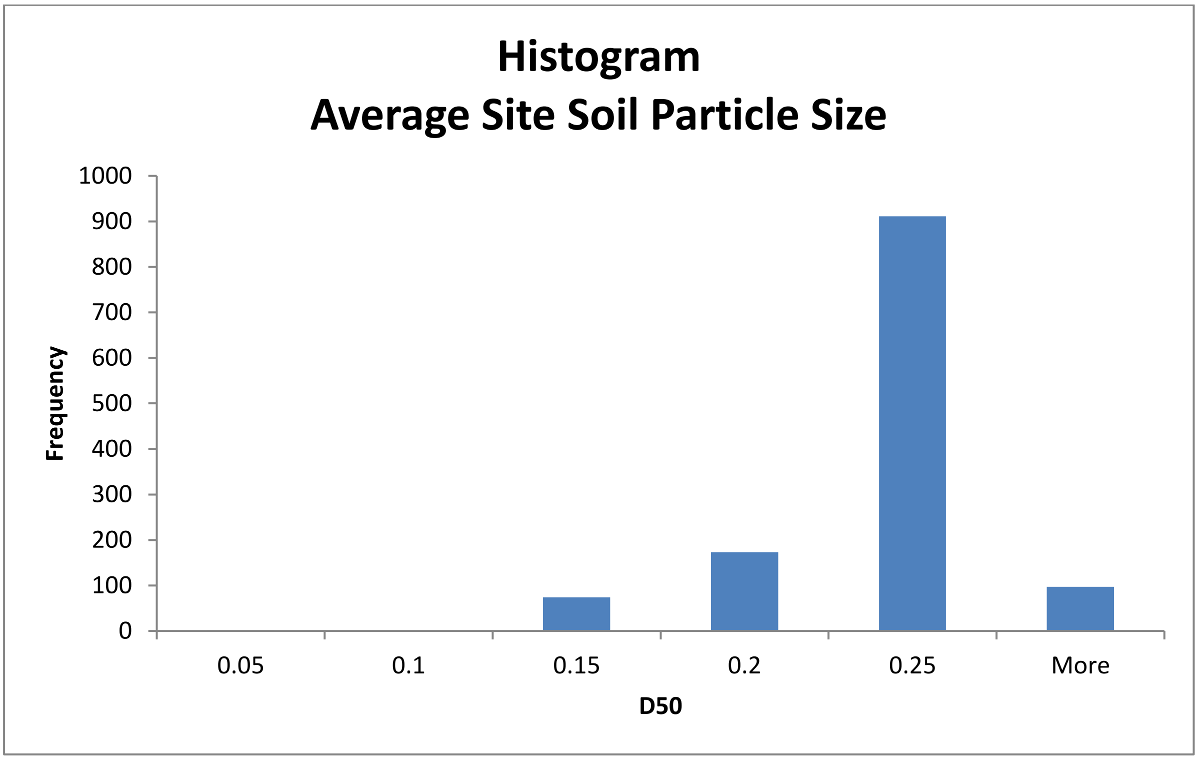


Figure 5.15.9
Average Particle Size (D_{50}) all Site Soils

Table 5.15.24
***In situ* SPT Derived Relative Density (as %) for all Site Soils**

Lithological Unit	Average Depth	Dr (%)
Bredasdorp	-2.5	47%
	-7.5	63%
	-12.5	77%
	-17.5	98%
Springfontein	-2.5	43%
	-7.5	57%
	-12.5	63%
	-17.5	74%
	-22.5	93%
Varswater	-7.5	52%
	-12.5	65%
	-17.5	71%
	-22.5	76%
	-30.0	92%

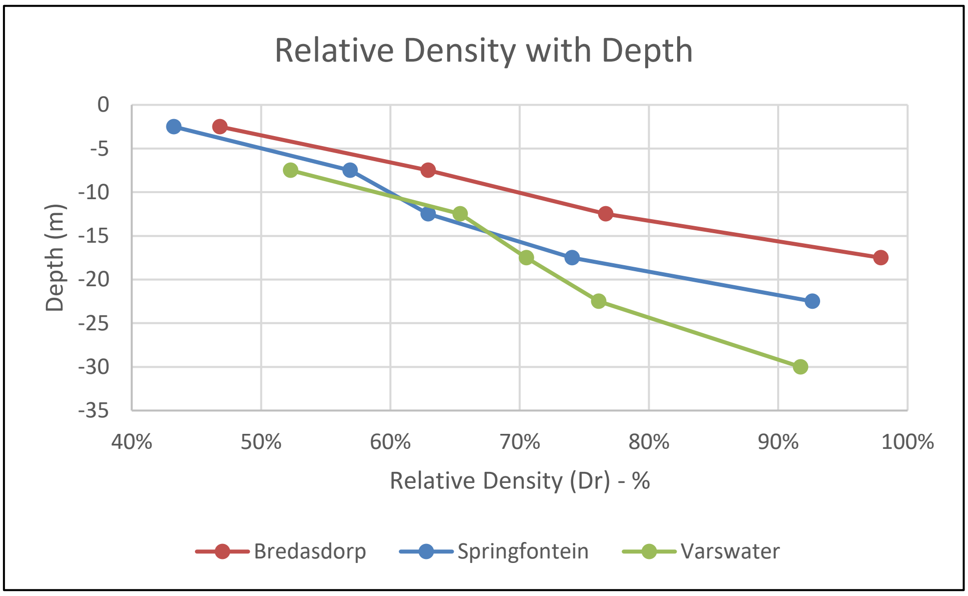


Figure 5.15.10
SPT Derived Relative Density (as %) for all Site Soils

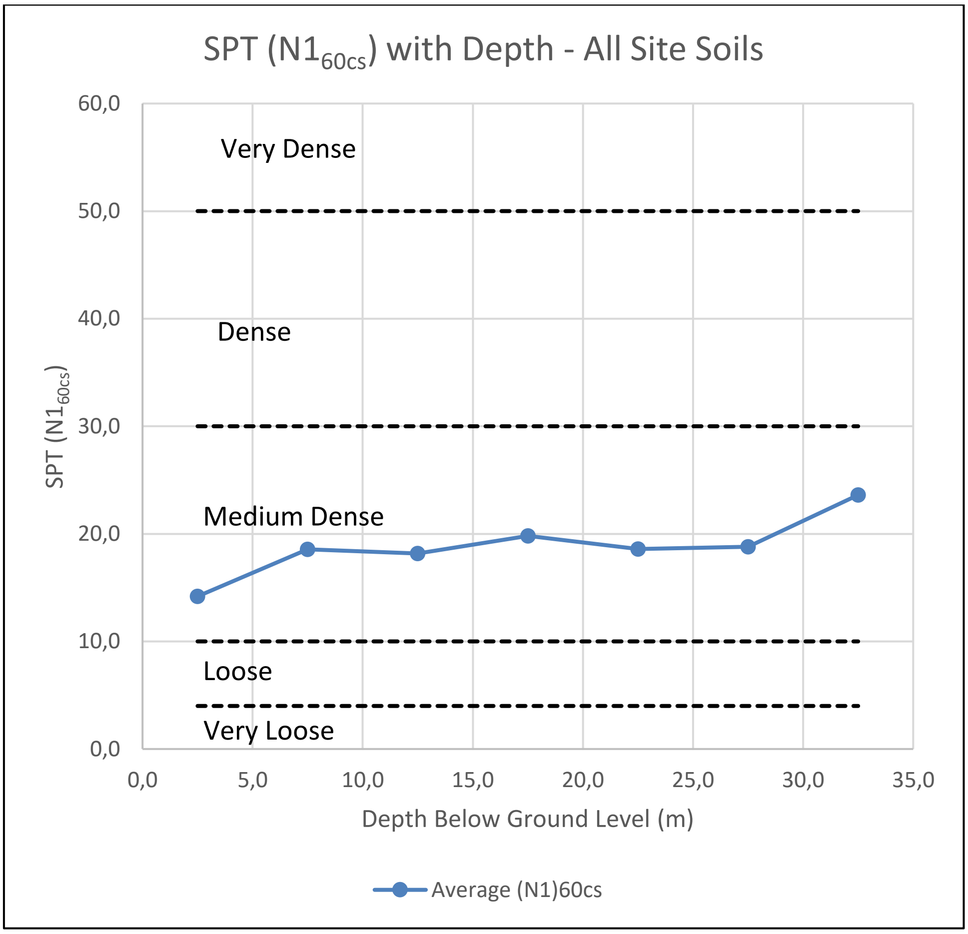


Figure 5.15.11
SPT Derived Consistency for all Site Soils

The homogeneity of the site soil particle size distribution is evident in the consistency in the D_{50} (or average grain size) of the soils (***Figure 5.15.9***). From the calculated *in situ* density ranges, the following is noted:

- In general, soil density (expressed as a percentage) and consistency (expressed as corrected SPT $N1_{60SC}$) as shown in ***Figure 5.15.10*** and ***Figure 5.15.11*** increase with depth, but corrected SPT N values indicate that the site soil consistency does not improve to better than medium dense. Soils in this consistency range will be challenging to found in, particularly if localised lateral variability is encountered as differential settlement challenges will arise.
- Soil improvement to increase consistency and reduce variability will be required on this site.

(b) Soil Slope Stability

The soils at the site are homogenous and dominated by aeolian and marine sand deposits. They are poorly graded and non-plastic. Friction angles estimated from SPT tests (>1 200 tests) as per (Clayton, 1993) indicate friction angles ranging from 27° to 42° with a mean of 32.4°.

The cohesionless nature of the soils, the thick deposits encountered at the site and the presence of groundwater raises stability of any constructed or excavated slopes on the site as a critical and environmentally far reaching consideration. In order to maintain a factor of safety of 1.95 for slopes (International Atomic Energy Agency, 2004) it is not conservative to assume that permanent slopes may be designed at slope angles less than one in three (18°) to minimise risks to site safety. In addition, robust dewatering systems will be required to maintain the long term drawdown of the groundwater table in excavations; depending on how reliable such dewatering systems are, slope angles could be increased or decreased.

At the proposed depth of excavation (i.e. to bedrock) a shallow slope angle of ≤18º will result in an extensive excavation footprint that will impact considerably on construction cost and the environment. Dewatering will be a mandatory requirement for slope stabilisation and this will have an impact on the drawdown of groundwater as per ***Section 5.11***. As per construction methods carried out for the KNPS, excavations will not be permanent, but will be open for a considerable time period considering the scale of the excavation.

(c) Founding Conditions

Founding of conventional structures (i.e. non-safety related structures) is an important consideration as the longevity of these structures is important in supporting the nuclear installation(s). The design engineers will aim to limit foundation instability and designs will need to consider the following:

- Material consistency at surface is variable and ranges from loose (in unconsolidated aeolian sands) to very dense where calcrete has formed – this leads to differential settlement risk for structures founded in the shallow geotechnical profile (at surface).
- All conventional foundations (e.g. pad foundations, strip foundations) must be founded on at least 'dense' material to minimise settlement risks, or alternatively appropriately designed foundations are to be founded on consistent medium dense horizons to mitigate differential settlement risks.

- Analysis of the site-wide SPT results indicates average SPT N = 15 in the upper 2 m and N = 21 between 2 m and 3 m depth – this translates to medium dense consistency in cohesionless soils.

- Since large portions of the site are covered by medium dense sand dunes (loose at surface), ground improvement measures may thus be required for conventional foundations. It is unlikely that mechanical ground improvement will result in bearing capacity exceeding 200 kPa.

- Assessment of the site soils bearing capacity against the PPE (***Chapter 1***, Introduction) indicates that structures within the nuclear island shall be founded on materials with a minimum bearing capacity of 718.2 kPa. In their current form, the distribution of calculated bearing capacity of the site soils based on SPT data is shown in ***Figure 5.15.12*** and ***Figure 5.15.13***. It is clear to see from this figure that soils beneath the nuclear island will need to be removed and considerably improved (e.g. by cement stabilising as was done for KNPS).

- Other site structures (outside of the nuclear island and outside of the zone of founding improvement for the nuclear island structures) will encounter medium dense material with an average bearing capacity of 172 kPa (standard deviation of 139 kPa). Again, variability in founding conditions comes to the fore, and individual structures will require dedicated geotechnical investigations to assess localised site bearing capacity, soil improvement mitigations to develop consistency and reduce differential settlement risks – such strategies will support foundation designs.

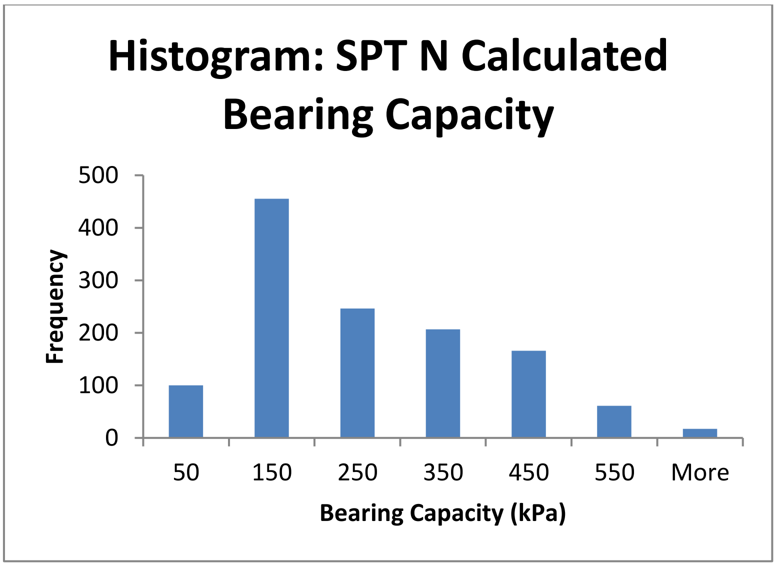


Figure 5.15.12
SPT Derived Bearing Capacity

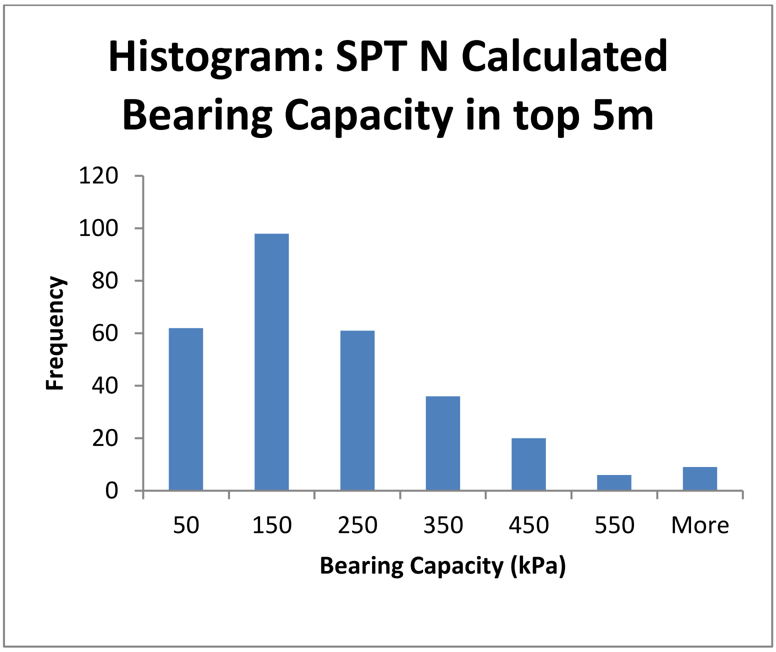


Figure 5.15.13
SPT Derived Bearing Capacity

5.15.7.5.2 Rock Strength, Elastic Parameters and Density

The measurement of rock properties and the results of laboratory testing are presented in this section.

The influence of bedding and closed discontinuities on test results was anticipated at the outset of the programme and selected rock samples analysed were therefore photographed before and after testing to visually gauge this influence (see ***Appendix 5.15.H***). Where not photographed, particular detail was given to documenting failure modes as can be seen in ***Appendix 5.15.H***.

A statistical summary of the rock laboratory test results is presented in ***Table 5.15.25*** and the detailed results, along with pre and post-failure photographs and/or mode of failure description, in ***Appendix 5.15.H***.

Table 5.15.25
Summary Statistics of Rock Laboratory Tests

	UCS (MPa)						E (GPa)						ν					
	h/fels	s/sto	m/sto	g/wack	shale	sands st.	h/fels	s/sto	m/sto	g/wack	shale	sands st.	h/fels	s/sto	m/sto	g/wack	shale	sands st.
Sample	1	9	1	28	18	35	1	9	1	28	18	35	1	9	1	28	18	35
Range		65.0	0.0	193.8	42.5	86.8		55.3	0.0	203.7	31.6	65.1		0.227		0.724	0.381	0.279
Mean	123.0	20.4	21.5	46.3	21.5	30.8	101.5	12.2	9.9	47.1	15.2	23.7	0.313	0.177	0.155	0.288	0.235	0.198
Min.	123.0	2.8	21.5	1.2	4.4	4.5	101.5	1.5	9.9	0.2	3.4	1.1	0.313	0.107	0.155	0.022	0.112	0.094
Max.	123.0	67.8	21.5	195.0	46.9	91.3	101.5	56.8	9.9	203.9	35.0	66.1	0.313	0.334	0.155	0.746	0.493	0.373

	Bulk Density (kg/m^3)						Dry Density (kg/m^3)					
	h/fels	s/sto	m/sto	g/wack	shale	sands st.	h/fels	s/sto	m/sto	g/wack	shale	sands st.
Sample	1	9	1	28	18		1	9	1	28	18	35
Range		2620	0	650	2480			770		820	670	651
Mean	2700	1056	2360	2519	406		2690	2325	2260	2485	2479	2536
Min.	2700	0	2360	2120			2690	1840	2260	1930	2254	2106
Max.	2700	2620	2360	2770	2480		2690	2610	2260	2750	2924	2756

Notes regarding Poisson's Ratio (ν): some results were obtained > 0.5 and this 'anomaly' is addressed in the bullet points below.

The summary statistics presented in ***Table 5.15.25*** are graphically presented in ***Figure 5.15.14*** and ***Figure 5.15.15***. The results are variably distributed, but in general exhibit a positively skewed distribution.

On the basis of the work and analyses performed, the following can be concluded:

- general:
    - The test results exhibit a positively skewed distribution of key parameters and this confirms variability in the site rock parameters particularly between the different rock types constituting the Malmesbury Group. This variability will have to be factored into the design and design specific geotechnical investigations.
    - Prior to the pre-operational stage of the geotechnical characterisation, variability with respect to specific safety-related structures will need to explored and the works affected must be designed to meet the range of values. Of particular importance will be detailed mapping of the exposed bedrock surface once the excavation for the nuclear installation(s) has been completed and bedrock exposed.
    - It should be observed that the sampling process was affected by the available lengths of intact core[19] with high integrity (see ***Appendix 5.15.C***). This resulted in sampling being carried out on samples less frequently jointed and may have biased the results to represent more competent rock.
- uniaxial compressive strength (UCS):
    - The results are positively skewed.
    - UCS of the greywacke, sandstone and hornfels (only a single sample) rocks is higher than the shales, siltstones and mudstones as would be expected.
    - Lower sample size was obtained in the rocks outside of the greywacke and sandstone units.
    - For each rock type analysed, the significance of the results lies in the range and thus the variability.
- Young's Modulus (E): the results exhibit a positively skewed distribution.
- Poisson's Ratio (ν):

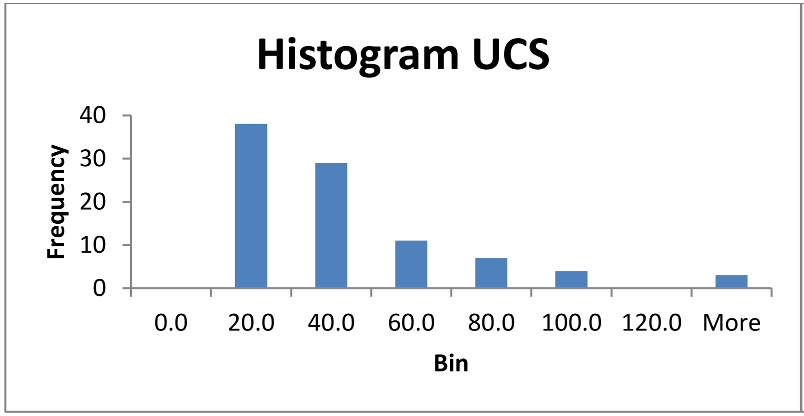
[19] Meaning core that, in the field technicians opinion, was capable of remaining intact through transport to the laboratory.

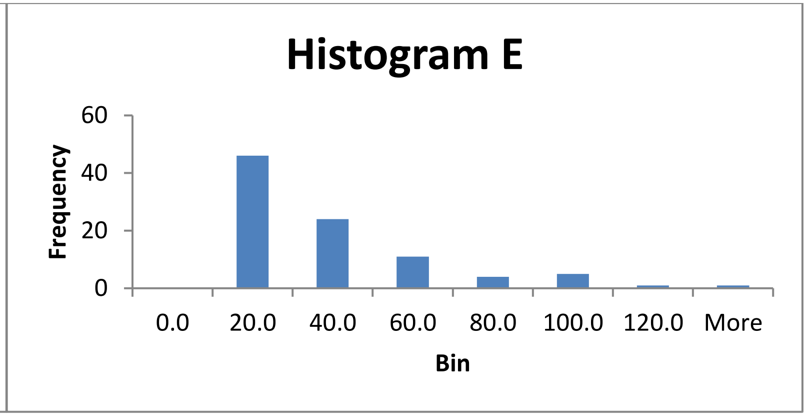
- At first glance, the results appear to be uniformly distributed, but it is noted that several results appear in the tail of the distribution.
- There are spurious values, most probably related to a combination of testing methods and the influence of rock fractures on the measurement of volumetric changes. Examination of the post analysis rock sample photographs lends some credibility to this theory in that samples exhibiting high measured ν values appear to have undergone significant deformation along joints.
- Pseudo Poisson's Ratios[20] well in excess of the theoretical maximum of 0.5 are possible in sheared rock. In the context of this project, Poisson's ratio for the rock as an un-fractured elastic medium is required.

- Bulk and dry density - The results appear to produce less variability than other parameters investigated.

In summary, the laboratory analyses confirm that the site rocks display a high variability in parameters influenced by different rock types within the Malmesbury depositional sequence. These different rock types experience variations in rate and extent of weathering due to their individual resistance to weathering. This does not present undue constraints to the geotechnical characterisation as sufficient information can be obtained through targeted investigations in the pre-operational and operational phases to inform design on localised sites.

[20] Values of Poisson's ratio are expected to range from 0.25 to 0.5 and recorded values in excess thereof are unusual. There is precedent for the occurrence of high values documented in work by (Barton & Bandis, 1982), where it was demonstrated that heavily jointed rock masses can show 'expansion ratios' or pseudo-Poisson's ratios far in excess of 0.5 and even in excess of 1.0 as (shear) failure is approached. This is due to the fact that elastic continuum theory is 'violated' by dilating shear displacements tending to occur on the failing joint surfaces.





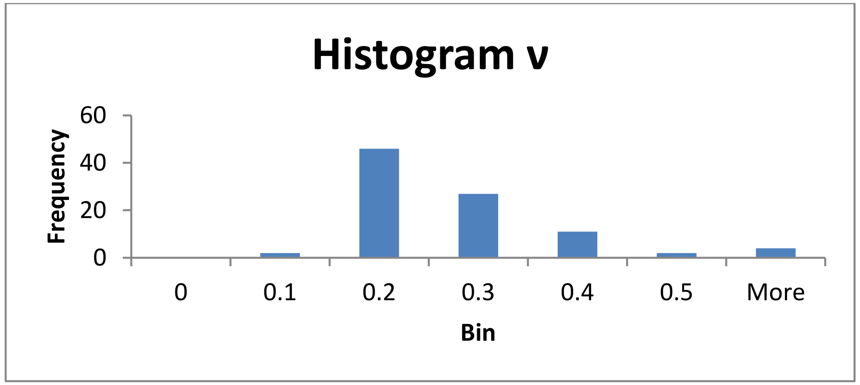
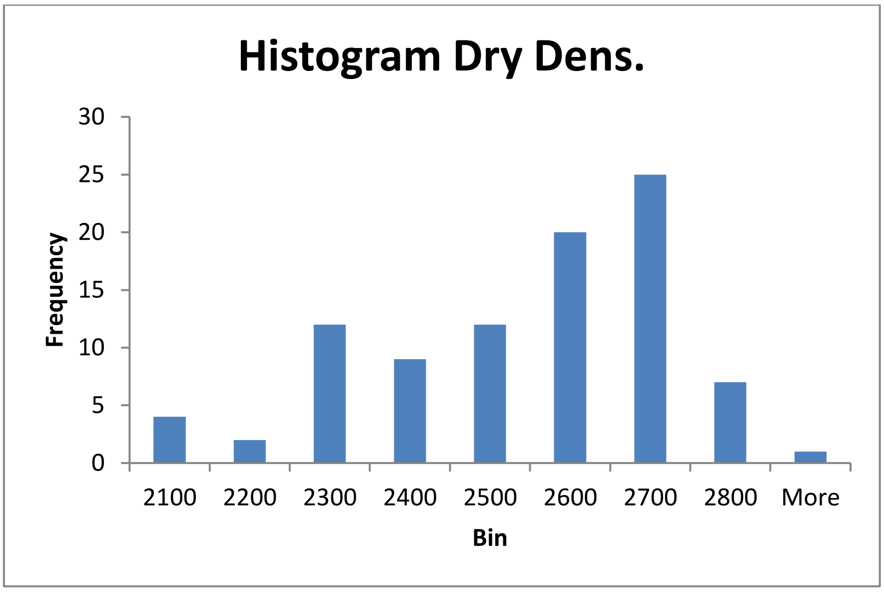


Figure 5.15.14
UCS, Young's Modulus and Poisson's Ratio from Laboratory Tests



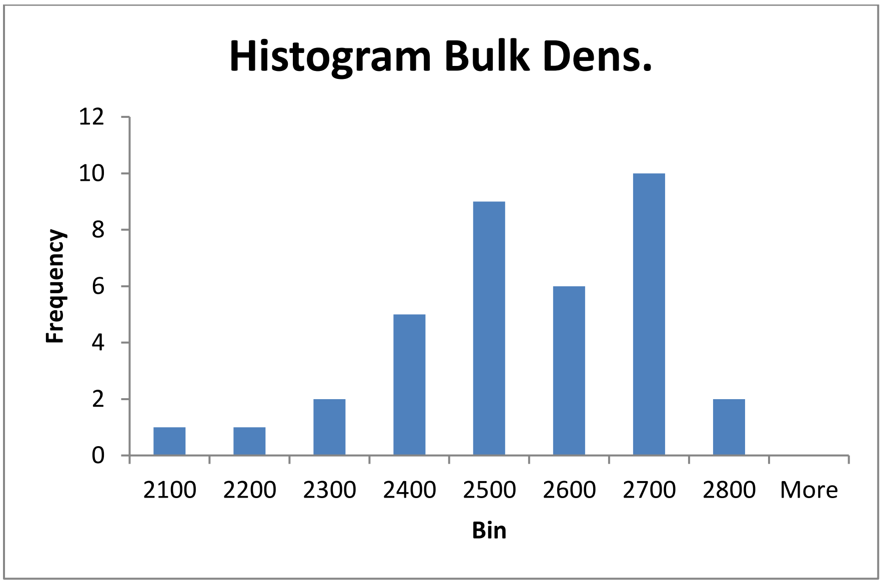


Figure 5.15.15
Bulk and Dry Density from Laboratory Tests

5.15.7.5.3 Results of the Liquefaction Potential Assessment

Section 5.15.6.6 indicates that the current site response is an $M_{6.5}$ event with a PGA of 0.4 g as per (Stamatakos & Watson-Lamprey, 2024). The liquefaction potential assessment was carried out according to (National Centre for Earthquake Engineering Research, 2001), and calculations were done as follows:

- SPT N values were corrected to $(N1)_{60CS}$ values and the cyclic resistance ratio (CRR) scaled for an $M_{6.5}$ event. The cyclic stress ratio (CSR) was determined for a PGA of 0.4 g and a factor of safety (FoS) calculated per SPT test, where FoS = scaled CRR/CSR.

- Shear wave velocity (Vs) measurements from MASW, downhole and PS suspension logging seismic testing were corrected to Vs_1 measurements considering the effective stresses at each measurement position. Vs1 was plotted against CSR (PGA of 0.4g $M_{6.5}$ event), and superimposed on the boundary line determined by equation:

  $CRR = 0.03(Vs_1/100)^2 + 0.9[1/(Vs_{1c}\text{-}Vs1)\text{-}1/Vs_{1c}]$

  where Vs_{1c} = 220 m/s for sands with fines content < 5 per cent.

The results are shown graphically in:

- ***Figure 5.15.16***: where SPT $(N1)_{60CS}$ is plotted against CSR with a liquefaction threshold line calculated for a $M_{6.5}$ event. Values plotting above the CRR($M_{6.5}$) line indicate materials with a propensity to liquefy with this triggering seismic event.

- ***Figure 5.15.17***: where a histogram of FoS results is plotted – it is noted that a FoS <1 indicates that the CSR > CRR, and liquefaction becomes a risk. It is noted that there are FoS that are high and this is linked to cement stabilised soil zones and zones where soils are cemented/partially cemented and the SPT refused.

- ***Figure 5.15. 18:*** where values plotting above and left of the red CRR line indicate a propensity to liquefy.

The SPT $(N1)_{60CS}$ based liquefaction potential assessment gives a clear indication that there are soils on this site that show a potential to liquefy upon the triggering seismic event ($M_{6.5}$ and PGA = 0.4g). The Vs_1 based liquefaction potential assessment indicates much lower liquefaction potential.

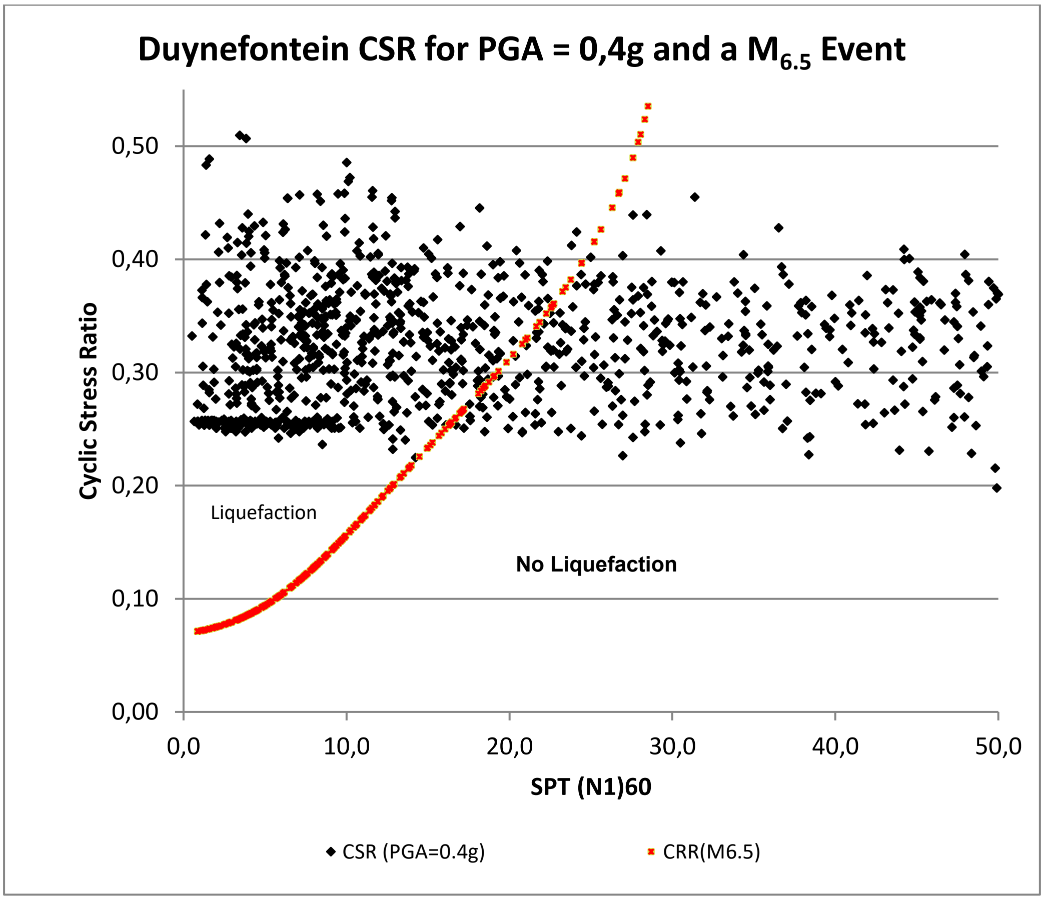


Figure 5.15.16
SPT $(N1)_{60CS}$ Derived Liquefaction Potential

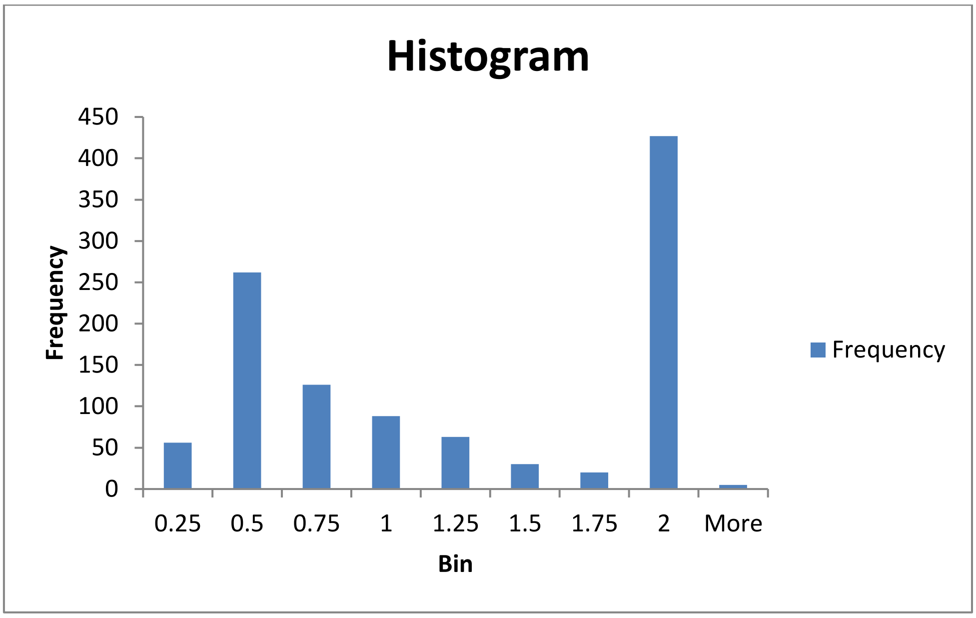


Figure 5.15.17
SPT $(N1)_{60CS}$ Derived Liquefaction FoS Histogram

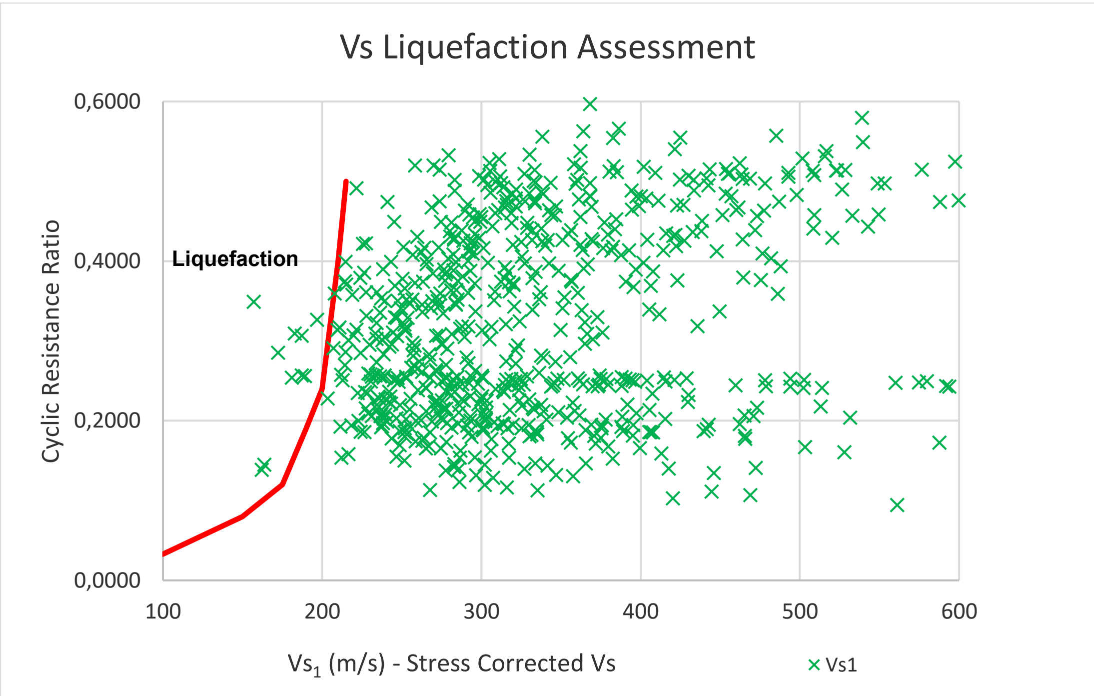


Figure 5.15.18
Vs_1 Derived Liquefaction Potential

It is useful to view the results of the liquefaction potential assessment by referring to the cross sectional plots of FoS against liquefaction contained in ***Drawings D5.15.3*** to ***5.15.12*** and ***Drawing 5.15.14***. The following comments are made with reference to these drawings:

- There are numerous instances shown on the cross sections where the liquefaction potential assessment indicates zones where FoS < 1 as well as FoS<1.5 (an industry minimum norm).
- Several liquefiable zones underlie the Duynefontyn site where a nuclear installation(s) is proposed, and strategies used in the past for KNPS involving excavation of soils to bedrock and replacement with cement stabilised soil backfill cannot be avoided under the proposed nuclear island(s).
- ***Drawing 5.15.12*** shows the positions of KNPS Units 1 and 2, and the positive impact that cement stabilised soil founding has had on liquefaction FoS. It is necessary to view both the FoS cross section and the Vs cross section on this drawing, as well as the oblique view on ***Drawing 5.15.14*** to fully appreciate this statement.

It is reasonable to conclude that the site soils exhibit a potential to liquefy in large areas across the site under the loads/triggers imposed by seismically induced ground shaking – it is also reasonable to conclude that the cement stabilised raft under the KNPS rules out liquefaction as a risk in this area.

Drawing 15.5.14
Oblique View of Liquefaction FoS

(a) Concluding Remarks on Liquefaction Potential

Based on the analyses performed above, the following conclusions are drawn:

- SPT tests and a measured Vs profile across the site were used to assess the liquefaction potential according to recognised international procedures (National Centre for Earthquake Engineering Research, 2001). Analysis of these data using the recognised method is considered appropriate for the liquefaction potential assessment at the site.
- Wide distributions of soils investigated on the site have a high liquefaction potential, including areas within the Duynefontyn site earmarked for future nuclear installation(s) development and soil improvement from bedrock will be required.
- Historical soil improvement measures under the KNPS nuclear island are effective to date.

5.15.7.6 Environmental and Cultural Considerations

5.15.7.6.1 Introduction

(International Atomic Energy Agency, 2004) highlights protection of cultural resources, such as archaeological sites and artefacts.

Environmental considerations, although dealt with in the EIA need to be mentioned in this SSR as the geotechnical profile of the site suggests that excessive site disturbance could result when excavations are made. These excavations (coupled with the need to dewater), positioned practically anywhere on the proposed site could carry high environmental damage risks. The design engineers will therefore be pressured into limiting this risk and in so doing should be required to optimise designs. An obvious optimisation to mitigate environmental impact will be to steepen cut/fill slopes. This will, in turn, impact on the stability of slopes. Secondary considerations related to spoil areas should also be mentioned as bulk excavations will be required and this material will need to be spoiled somewhere (potentially on the site). (Illenberger, 2010) specifically points out the sensitivity of the site to receiving spoil heaps/temporary stockpiles. (International Atomic Energy Agency, 2004) highlights risks related to placement of fill into wetlands. The site has several wetlands (***Section 5.3***, Ecology) and management of excavated material (spoil) should take this into account.

To place these environmental risks in context, a one hectare excavation for the proposed nuclear installation foundation footprints was considered in the central area of the site. ***Table 5.15.26*** shows the calculated disturbed surface area and excavated volumes that would be associated with this hypothetical excavation on the assumption that the founding level will be approximately at the average bedrock level (i.e. ~-10 m msl). The calculation in ***Table 5.15.26*** is also based on the assumption that excavation slopes will be battered back to angles in the region of 18º for stability reasons.

Table 5.15.26
One Hectare Foundation Footprint Excavation Scenario

Resulting Disturbed area (ha)	Resulting Excavated Volume (m^3)
5.9	860 000

From ***Table 5.15.26*** it can be seen that a one hectare base footprint excavation results in a disturbed surface area of 5.9 ha and produces a spoil volume of 860 000 m^3. It is reasonable to assume that, at best, spoil stockpiles will equal the disturbed areas resulting from the excavations, but will in all likelihood supersede the excavation areas for the following reasons:

- The excavated material may bulk up (potentially by a factor of 1.5), resulting in the need to stockpile a greater volume (effectively) of material than is excavated. Alternatively, spoil stockpiles will require controlled construction to an appropriate compaction specification.
- Environmental considerations for the siting of spoil stockpiles will, in all likelihood, consider visual and erosion (wind and water) impacts as well as geomorphological impacts (Illenberger, 2010) and could result in limitations on stockpile heights, slope angles and geographical positioning on the site – this potentially resulting in unacceptably large surface disturbances in sensitive areas.

5.15.7.6.2 Construction Impacts on the KNPS

Linked closely to the potential environmental impact concerns are concerns related to the fact that an existing nuclear installation (KNPS) exists on the site. Further developments in close proximity to the KNPS should take cognisance of the following:

- The identified groundwater table and groundwater drawdown at the

KNPS which is dealt with in greater detail in ***Section 5.11***, but it needs to be emphasised that this remains a primary concern to the safe operation of KNPS as lack of control over, e.g. introducing a fluctuating groundwater table, could impact on a host of geotechnical issues such as liquefaction potential of soils under infrastructure supporting the nuclear island but outside of the nuclear island (see ***Section 5.15.7.5.3*** indicating this risk does not persist under the nuclear island). Robust dewatering design in conjunction with predictive dewatering modelling, as has been undertaken in ***Section 5.11***, will be an important design consideration to mitigate any risks to the foundations at KNPS.

- Construction induced hazards, particularly excessive dust generation during excavations, spoil stockpiling and construction activities as the site soils contain fines (albeit at a low content of <5 per cent on average) and haul roads will degrade with high construction traffic volumes and exacerbate dust generation. This operational challenge was encountered by drilling traffic in the drilling campaigns. Ventilation systems at KNPS could be impacted and this is a safety concern. Standard dust control mitigation measures can, however, be employed to overcome any potential risks to the KNPS ventilation systems.

- From a geotechnical perspective, unreliable dewatering systems will present a risk to construction personnel and construction programme when the excavation to bedrock is open as a rise in phreatic surface (e.g. due to failure/periodic failure of the dewatering system) will lead to slope instability which may manifest only a gradual creep/sloughing, but this will cut off access to the excavation for construction plant and present health and safety risks to construction workers.

The cumulative impacts of site disturbance from excavations/spoil stockpiles and groundwater table drawdown during dewatering (***Section 5.11***) could result in unacceptable environmental damage at this site if the nuclear installation excavation is not carefully designed and implemented. Strict policing and control of mitigation measures will be an important consideration to ensure that impacts on the KNPS are minimised. Reliable dewatering systems will be required to ensure that excavation slopes remain serviceable and do not pose a health and safety risk to construction workers.

5.15.7.6.3 Seismic Conditions

(Stamatakos & Watson-Lamprey, 2024) documenting the PSHA covers the seismic conditions for the site and surrounds in great detail.The site response spectra are defined in great detail in the PSHA and this will provide adequate information for design engineers to mitigate any risks. Of particular importance in this regard is that a seismic event of $M_{6.5}$ (and PGA = 0.4g) will trigger liquefaction in large areas across the site as has been demonstrated in ***Subsection 5.15.6.6.***

5.15.8 Geotechnical Profile of the Site

5.15.8.1 Inherent Natural Characteristics

The geotechnical profile presented in this SSR includes the following as recommended in (International Atomic Energy Agency, 2004):

- data gathering point positions (***Drawing 5.15.1***);
- a geometrical description, such as subsurface stratigraphic descriptions, lateral and vertical extents, number of layers and layer thicknesses (***Appendix A-5.15.B*** and ***Drawings 5.15.2*** to ***5.15.12***);
- the physical properties of soil and rock and values used for classification (***Appendices A-5.15.B***, ***A-5.15.C*** and ***A-5.15.D***);
- shear wave velocities in the site rocks and soils documented in ***Drawings 5.15.2*** to ***5.15.12*** and ***Appendix 5.15.I***;
- mechanical properties of site materials obtained by *in situ* or laboratory tests (***Subsections 5.15.6.2*** and ***5.15.7.1***);
- characteristics of the groundwater table (***Drawing 5.15.13*** and ***Subsection 5.15.7.3***);
- a description of the surface topography (in the form of contours) (***Drawing 5.15.2***).

Based on this information, the site geotechnical characteristic (profile) was developed. In summary, the geotechnical profile for the site is comprised of the following:

- site soils:
  - The site has an average 21 m thick (in vertical extent) aeolian and marine sand deposit, with a minimum thickness of 12 m (near the sea) and a maximum of 39 m (inland).
  - Marine and aeolian soils are homogenous in grading and are poorly graded with a very high sand size fraction and low fines content (on average <5 per cent) as demonstrated by the laboratory test results.
  - The upper aeolian sands have been variably calcretised and thin hardpan calcrete layers or concretions occur close to surface in places which essentially resulted in a largely ineffectual DPSH probing programme as shallow refusal was common;

- The aeolian and marine soils together constitute a primary intergranular aquifer, the Sandveld Aquifer.
- The groundwater table marks the top of the Sandveld Aquifer and is situated between 3 and 5 m from ground surface but can fluctuate with fluctuating rainfall patterns and topography.
- Groundwater flow within this aquifer is perpendicular to the coastline.
- Soil consistency generally increases with depth but there are places where the consistency remains at medium dense throughout the soil horizon.
- These extensive, poorly consolidated soils within an intergranular aquifer exhibit zones of liquefaction potential across this site.

Mechanical ground improvement in these soils does not normally produce bearing capacities in excess of 200 kPa. Soil bearing capacities required in excess of this for supporting infrastructure development will only be achieved with cement stabilisation improvement measures or with piling solutions. Slope stability in these soils will require robust dewatering design and battering back of slopes to safe angles (in the region of <1:3 or18˚).

- site rocks:

  - In general, the bedrock consists of variably weathered greywacke and sandstone with interbedded shale and mudstone layers (and metamorphosed equivalents) of the Malmesbury Formation.
  - Bedding planes are steeply dipping with the greywacke layers tending to be less weathered and more competent and the interbedded layers tending to be more weathered and of softer rock quality.
  - The rock quality generally improves with increasing depth and softer layers are generally not encountered once good quality rock has been encountered.
  - The bedrock is jointed with localised shear zones (and minor fault zones/brecciated zones) which occur randomly throughout the rock mass, possibly accompanied with the intrusion of quartz veins. In these zones, the jointing tends to be very closely spaced and there may be some alteration or more advanced weathering along the joint planes resulting in clayey silt joint infill.
  - Seepage of water is also evident within these zones (iron stained) and secondary, vuggy quartz, also occurs within thin shear zones. The extent of these preferential flow paths varies, and drilling water return averaged *c.*75 per cent, but reduced to as low as *c.*10/20 per cent in boreholes KB 47 and 52.

- Laboratory and point load test results indicate high variability in rock strength with greywacke and sparsely occurring hornfels exhibiting significantly higher rock strength than the interbedded rocks (siltstones/mudstones).
- Rock joint conditions indicate that block failure (sliding, toppling) in conjunction with surface ravelling failure in zones of closely spaced jointing, is likely in all manner of excavations oriented within 30° of joint/bedding strike.

Founding improvement measures can only be achieved by removing weathered/poor material and replacing this with e.g. mass concrete. Excavated rock slopes can be supported with standard lateral support systems consisting of bolts and/or anchors in conjunction with surface support (mesh, mesh and shotcrete or fibre reinforced shotcrete).

5.15.8.2 Extraneous Natural Hazards

The geotechnical profile presented above does not specifically refer to the following features which could be potentially challenging to the design (International Atomic Energy Agency, 2004):

- previous use of the site (e.g. mining activities);
- gas pockets, and swelling rocks[21];
- zones of weakness or discontinuities in crystalline rocks;
- indicators of potential cavities and susceptibility to ground collapse in the context of:
  - sinks, sink ponds, caves and caverns;
  - sinking streams;
  - historical ground subsidence;
  - natural bridges;
  - surface depressions;
  - springs;

[21] However, interbedded shales are commonly encountered in the profile and these rocks are sometimes prone to weathering down to expansive clays

- rock types such as limestone, dolomite, gypsum, anhydrite, halite, terra rossa soils[22], lavas, weakly cemented clastic rocks, coal or ores;
- non-conformities in soluble rocks.

However, information gathered to date through intrusive investigations carried out historically and currently do not indicate that any of these features are present on the site bar the presence of less competent interbedded rocks in the geological sequence. The exposed bedrock in the foundation excavations done prior to the pre-operational stage of the geotechnical investigation will need to specifically investigate the occurrence of these features on localised foundation footprints, particularly the distribution of shale and mudstone.

5.15.8.3 Concerns Exposed During Investigative and Interpretive Phases

The concerns exposed during the investigative and interpretive phases are briefly as follows:

- difficulty to obtain undisturbed soil samples;
- low to average soil consistency with depth in saturated conditions;
- deep cohesionless sand overburden together with an inter-granular aquifer giving rise to likely need for extensive excavations and dewatering requirements demanding robust dewatering systems, these in turn potentially resulting in major environmental impacts and/or slope instability in foundation excavations;
- high potential for liquefaction of sands in areas;
- disposal of extensive volumes of excavated sand potentially causing notable environmental impacts;
- jointed nature of site rocks with dominant joints potentially ‘daylighting’ into cut slopes;
- presence of sheared/faulted/brecciated rocks;
- bias in rock sampling due to nature of jointing and further bias to greywacke rock sampling due to the dominance of greywacke and the

[22] A red soil produced by the weathering of limestone

higher strength of this rock returning a greater frequency of intact drilling core;

- variation in geology within the wave-cut platform;
- high variability in geotechnical properties of the rock related to depositional characteristics;
- number of potential modes of cut slope failure in rocks;
- differential movement across rock structures and soil-rock interfaces;
- the potential for excessive dust to be generated on a heavily trafficked site during construction potentially resulting in concerns to the integrity of the KNPS ventilation systems.

The above concerns are highlighted to prompt appropriate reaction to these issues by the design engineers. None of these concerns presents any particular challenge to the suitability of the site as mitigation can be achieved using tried and tested engineering solutions and by adhering to lessons learnt in the construction of KNPS. Were any particular concern to be placed ahead of the rest, it would be to ensure that appropriate planning is put in place to fully explore the variability in site rocks in the foundation excavations prior to pre-operational phase, and to ensure robust dewatering of excavations. A similar approach was used in developing the KNPS and therefore does not present any obvious challenges.

5.15.9 Aspects of Works Requiring Geotechnical Design

This SSR is positioned within the characterisation stage. Since fixed plant layouts/designs are pending, there is reference within the text above of concluding certain information prior to the pre-operational phase. Notwithstanding these highlighted data gaps, this section does provide a clear understanding of the geotechnical setting of the site.

This makes conceptual evaluation[23] of the sufficiency of the site in meeting the prospective loads of the proposed nuclear installation(s) possible. A systematic presentation of aspects of work requiring geotechnical design is summarised below and presented in detail in ***Appendix A-5.15.A***. Subjective engineering judgment is sometimes relied upon in this appendix

[23] Preliminary evaluation based on site data but not necessarily representative of a specific foundation footprint.

to assess the ability of the various aspects of work to appropriately withstand the prospective loads, to assess the comparative simplicity of the ultimate design process and to assess whether all uncertainties are definitively addressed.

To summarise, this SSR makes generalised recommendations with respect to future geotechnical design and safety issues based on the available geotechnical data.

The aspects of work requiring geotechnical design considered relevant to assessing the site suitability are summarised as follows (to be read in conjunction with ***Appendix 5.15.A*** where considerably more detail is supplied):

- soil slope stability design mitigating:
  - erosion failure;
  - classic slip-circle failure;
  - liquefaction failure;
  - dewatering design;
  - piping failure;
- rock slope stability design mitigating:
  - toppling failure;
  - wedge failure;
  - planar failure;
  - surface ravelling failure;
  - classic slip-circle failure in weathered/soft rock;
- foundation design mitigating:
  - bearing, sliding, settlement and differential settlement failure;
  - failure due to dynamic and differential dynamic loading;
  - problems arising between structure interfaces (e.g. the nuclear island/monolithic structures) and ancillary structures (e.g. cooling water intake/outlet structures);
  - concentrated loading from towers and stacks and the effects on foundation materials of vibratory loading;
  - potentially challenging foundation cleaning and preparation sequences;

- potential concerns relating to damped foundations of the nuclear island;
- mass foundations required to bridge suitable founding level and safety related structures;

- cooling water supply and outfall structure design mitigating:

  - potential problems related to linear structures (tunnels, canals and shafts) traversing geological discontinuities;
  - deformations related to rock mass controlled behaviour as opposed to structure controlled behaviour;
  - interface design between these structures and the nuclear island;

- selected infrastructure design mitigating:

  - pipeline failure;
  - cyclic hydrodynamic loading on breakwaters;

- construction planning mitigating:

  - environmental impacts related to bulk excavations, bulk soil disposal, dewatering of bulk excavations and groundwater table drawdown;
  - dust generation and groundwater table drawdown impacts on the safe operation of the KNPS.

Subjective engineering judgment of the suitability of the various aspects of the work with regard to future demands as described in (International Atomic Energy Agency, 2004) was informed/based on the following conditions:

- foundations:

  - The nuclear installation island and turbine hall will be founded on or in competent bedrock as was the case with the KNPS, and will be founded on materials with an allowable bearing capacity of at least 718.2 kPa (***Chapter 1***, Introduction).
  - Founding materials must be of such a nature that design of foundations can be simplified, uncertainty reduced and risks to safety thus removed through design.
  - Alternatively, variability in founding materials must be sufficiently understood such that foundation design can be simplified, uncertainty removed and risks to safety minimised through design.

- excavations:

- Excavations through soil overburden and rock are required to reach good founding and the site geotechnical profile should pose a low risk to excavation stability provided that appropriate excavation designs (including dewatering) are carried out.
- Soil overburden depth increases with distance from the sea.
- The geotechnical profile (including the groundwater regime see ***Section 5.11***) must be sufficiently understood such that excavation design can be simplified, uncertainty reduced and risks to safety removed through design.
- Excavation design must take cognisance of environmental conservation without risking site safety.

• site locality and access to cooling water:

- The locality of the nuclear installation(s) could be anywhere on the site, but within a reasonable distance from the sea as a critical safety aspect of the nuclear installation(s) will be the reliable intake and outlet of cooling water.
- The cooling water intake could be via a conventional cooling water intake system should the nuclear installation(s) be located at the sea or via a tunnel intake should they be located further inland but within reach of the sea.

• auxiliary works (infrastructure supporting the nuclear installation(s)) will need to support the safety of the nuclear installation(s).

None of the above aspects requiring geotechnical design present unduly onerous design challenges within the context of the site geotechnical profile described herein.

5.15.10 Subsurface Site Characteristics, Areas of Uncertainty and Conceptual Mitigation Measures

Appendix A-5.15.A lists aspects of work requiring geotechnical design, design parameters obtained to date and outstanding parameters to be collated in the pre-operational and operational stages of the geotechnical characterisation. It also describes potential mechanisms of failure and suggests design criteria to overcome these mechanisms of failure. Lastly, ***Appendix A-5.15.A*** makes a conceptual assessment of the site capacity related directly to the potential demands (loads)[24] that could be imposed on it.

This section of this SSR integrates all of the preceding sections (as well as the appendices, and in particular ***Appendix A-5.15.A***) to form a precursor to future design (or a design concept). Since the site is under scrutiny from regulatory authorities for further development of nuclear installation(s), the design concept is required to justify supporting or contesting the suitability of the site. As such, both positive and negative aspects of the site characteristics in the context of structures likely to be built there must be described. Prior to subsequent phases of the geotechnical investigations (see ***Subsection 5.15.4***), certain design / geotechnical parameters that are required to support design and ultimately to confirm detailed design will be required.

Highlighting uncertainties will assist in the robust planning of further geotechnical investigations (see ***Subsection 5.15.4***) just as screening investigation and the KNPS investigation outcomes informed the planning of investigations for this SSR. Forewarning on the likely investigative requirements for future data gathering phases is a critical aspect in further confirming the geotechnical suitability of the site. Acknowledging that the plant layout will be known prior to the pre-operational and operational phase geotechnical investigations, the opportunity exists to close out any persisting uncertainty.

This section draws from the remainder of this report to highlight various site specific characteristics and areas of uncertainty where characteristics have not been confirmed for various reasons. The areas of concern presented for various aspects of the works requiring geotechnical design will require appropriate design mitigation measures to ensure that the potential risks during the nuclear installation lifetime will be acceptable – as detailed in the conceptual assessment in ***Appendix A-5.15.A***. This section also discusses these mitigation measures (International Atomic Energy Agency, 2004) and their potential effectiveness in ensuring that nuclear installation(s) safety will not be compromised, which is the primary requirement for demonstrating acceptability of the site.

[24] Loads presented in the Plant Parameter Envelope are used where available.

5.15.10.1 Founding Conditions

5.15.10.1.1 Subsurface Site Characteristics and Areas of Uncertainty

The main foundations of the future nuclear installation(s) will be placed on bedrock and, depending on where the nuclear installation(s) is positioned, this may give rise to temporary/permanent excavations in sands that may exhibit liquefaction potential and may be susceptible to slope failure. Groundwater management will be critical to the integrity of such foundation excavations.

The preceding sections (supported by the appendices) describe the geotechnical profile and highlight the following:

- The consistency of site soils is seen to generally increase with depth, but does not reliably exceed medium dense consistency throughout the soil profile. This, in the presence of the Sandveld intergranular aquifer indicates that certain areas of this site have a high liquefaction potential.

- Depth to bedrock varies across the site (shallower near the sea and deeper inland). Therefore, positioning the nuclear installation(s) practically anywhere on this site will require on average ~20 m deep excavations through the Sandveld Aquifer, and such excavation depth increases as one moves away from the sea.

- Lateral and vertical variability exists in bedrock founding materials, noting the following:

  - Sharp transitions in the steeply dipping site geology results in a lateral variance in rock quality with greywacke (and to a lesser extent sandstone) dominating and being of higher strength than interbedded shales, mudstones and siltstones.
  - Weathering profiles vary depending on the rock type with shales, mudstones and siltstones generally more prone to weathering.
  - The distribution of competent and incompetent rocks, which appears to be random, highlights variability and this is no more evident than in the histogram plots of the laboratory test results.

- The liquefaction potential of founding soils in certain areas across the site, noting that liquefaction potential is influenced by both soil characteristics and seismic hazard[25].

- The need for founding material improvement measures in medium dense (or less) soils throughout the soil profile.

The following characteristics of site soils are highlighted:

- Site soils are granular and the potential for heave/expansion is thus absent, but liquefaction risks persist under the site dynamic triggers.
- Settlement of structures founded in soils can be limited by employing soil improvement measures, particularly aimed at homogenising founding horizons and reducing differential settlement risks.

At present, the primary concern for the nuclear installation(s) foundation safety remains the variability in the geotechnical profile and the potential for site soils to liquefy. The site rock properties vary throughout as does soil consistency. Impedance to/amplification of shear waves travelling from bedrock into overburden is an unknown, but a reliable Vs profile of the site is presented in this SSR update and a reliable definition of the seismic loading in the PSHA to support such an assessment.

This variability creates a challenge for the nuclear installation design and differential rock bearing capacity[26] and potential for differential dynamic loading conditions should be noted.

The site rock variability, however, does not present unduly onerous challenges to the design of the nuclear installation foundations considering that the KNPS development was able to overcome similar challenges. In addition, this SSR highlights data that will be gathered to close out uncertainty. Once footprints for safety related structures are selected and the foundation excavations done, this variability can be investigated in detail on a localised scale to assess how founding may be locally affected and mitigations designed.

Additional uncertainty may be introduced should safety related structure foundations be required to straddle shears / faults identified in the foundation

[25] It is noted that the PSHA is concluded, and that dynamic triggering of liquefaction has been considered in this SSR update.

[26] Bearing Capacity: The carrying capacity of soil/rock materials as an indicator of what loads can safely be placed on such materials.

excavations. In this context, it would be necessary to relocate such structures or investigate specially designed foundations to withstand differential loading. Similar uncertainties were faced in the development of the KNPS and were effectively mitigated.

Considering the above discussion, ***Table 5.15.27*** provides a systematic presentation of founding conditions for various conceptual structures typical of nuclear installation projects. It provides summary of:

- design concepts for the works requiring geotechnical design in the context of the measured geotechnical profile as well as the current confidence in this profile;
- the impact that the geotechnical profile may have on the design of conceptual structures;
- data gaps (uncertainties) that require closing out – data that is critical to informing design and confirming detailed design;
- design precautions and potential construction difficulties that may be encountered in future phases (i.e. design and construction phases);
- potential operational impacts and environmental impacts related to the works requiring geotechnical design.

The aim of ***Table 5.15.27*** is to present a conceptual assessment of potential geotechnical and construction related issues at the site. This then allows in-principle conclusions to be drawn on the site suitability from a geotechnical characterisation viewpoint.

Table 5.15.27
Nuclear Installation Foundation Concepts and Geotechnical Data Acquisition Needs

Aspect / Structure	Potential Extent of Structure (Geometry)	Geotechnical Characteristics Affecting Founding	Reliability of Existing Geotechnical Data	Data Uncertainties	Design Precautions	Construction Difficulties	Operational Impacts	Operational and Environmental Impacts
Small, conventional spread and strip foundations (founding in soils).	Localised and ideally shallow founding in the geotechnical profile. Adjacent foundations potentially spanning medium dense to loose soils and/or dense cemented soils near surface. Spread foundations potentially requiring founding at or within the groundwater table.	Settlement ranges. Liquefaction potential in founding materials and the groundwater table characteristics. Variations in vertical consistency of soils and the random presence of cemented/calcretised zones near surface. Founding in medium dense soils will not reliably carry loads to confidently maintain settlements within acceptable limits. These medium dense soils also have a greater response to saturation effects in the dry SA climate.	Site data are reliable and describe the homogeneity and/or variability of soils (particle size distribution and consistency) across the site as well as the consistency of soils down the profile. Data lack site specific focus on a local scale.	Location of structures and site specific vertical extent of compressible soils. Site specific vertical variability in soil consistency needs to be investigated for specific structure footprints.	Dewatering to allow access to founding materials and to arrest temporarily liquefaction of soils and/or instability of soil slopes. Soil stabilisation will be required to arrest permanent liquefaction under safety related structures. Differential settlement may occur – mechanical soil improvement measures will be required even for lightly loaded structures, but bearing capacity in excess of 200 kPa will not be attainable without cement stabilisation. Soil improvement will be optimal if soil is removed and replaced in engineered layers not exceeding 200 mm thick and structures founded on rafts. Founding within/near the groundwater table may introduce seasonal responses in founding soil consistency. Founding to be specified in dense soils, improved soil horizons or on piles for settlement sensitive structures or for structures with design bearing loads > 200 kPa.	Effective/reliable dewatering and disposal of extracted groundwater. Soil improvement will require excavation and backfilling in engineered layers as mechanical techniques from surface will not achieve improvement to an acceptable depth (i.e. will not create a reliable soil raft adequate for founding). Compaction grouting is an option for ground improvement, but this will need to be appropriately designed and the efficacy thereof monitored during construction. Extent of overall excavation if sides are not shored. Excavations deeper than 1.5 m will require shoring. Founding levels will require inspection and approval immediately prior to pouring blinding concrete.	Minimal impacts provided correct founding methods are selected and ground improvement measures are carried out correctly. The integrity of soils stabilised to arrest liquefaction will need to be monitored to ensure that their integrity remains intact. Dust Control impacting KNPS ventilation systems.	Dust control

Aspect / Structure	Potential Extent of Structure (Geometry)	Geotechnical Characteristics Affecting Founding	Reliability of Existing Geotechnical Data	Data Uncertainties	Design Precautions	Construction Difficulties	Operational Impacts	Operational and Environmental Impacts
Adjoining small, conventional spread and strip foundations (founding in soils) to monolithic structures like the nuclear island (founded on rock/cement stabilised raft)	Transition zones between monolithic structure foundations and adjoining structure foundations (i.e. on a localised scale). Founding depths may vary between monolithic structures (founded in rock) and adjoining structures (founded in soil).	Liquefaction potential in founding materials and the groundwater table characteristics, but this may have a low probability of occurring considering the nuclear island excavation and soil improvement will be a bulk activity extending over a large lateral extent. Differentials resulting from different founding regimes (rock and soil) – introducing both differential bearing capacity and differential dynamic loading regimes. Monolithic structure foundation excavations will require over-excavation to ensure slope stability – adjoining structures will be founded in fill horizons. Dynamic loading regime and seismic wave impedance / amplification when propagating from rock into soil horizons (i.e. changes in seismic wave propagation through different mediums).	Site data are reliable but may require additional investigations on a local scale (e.g. if no boreholes have been drilled at proposed localised structures).	Site specific vertical variability in soil consistency needs to be investigated for specific structure interfaces.	Dewatering to allow access to founding materials and to temporarily reduce liquefaction potential of soils and/or slope stability. Soil stabilisation will be required to arrest permanent liquefaction in safety related structures. Specially designed interfaces between monolithic and adjoining structures to cater for differential settlement and differential seismic wave propagation (impedance effects to be noted). Holistic/integrative design of monolithic structure foundations and adjoining structure foundations, noting construction sequencing/methodology. Backfill around monolithic structures will not have bearing capacity in excess of 200 kPa unless cement stabilisation is employed or backfill material imported.	Effective dewatering and disposal of extracted groundwater. Over-excavation of monolithic structure footprints requiring engineered backfill in close proximity to monolithic structures. Construction programme / sequencing may be impacted by design and/or localised geotechnical conditions. Founding levels will require inspection and approval immediately prior to pouring blinding concrete.	Minimal impacts provided correct founding methods are selected and backfilling is carried out correctly. In addition, impacts will be limited if holistic design approach is taken. The integrity of soils stabilised to arrest liquefaction will need to be monitored to ensure that their integrity remains intact. Dust Control impacting KNPS ventilation systems.	Dust control
Towers and stacks	Localised and lateral variability not likely to be an issue in soils, but may be an issue if founded on rock.	Liquefaction potential of slope excavations and the groundwater table characteristics. Variations in vertical consistency of soils and the random presence of cemented / calcretised zones near surface. Lateral (and vertical) variability in rocks. Dynamic loading regime and seismic wave impedance / amplification when propagating from	Site data are reliable but may lack site specific focus on a local scale (relevant to both soils and rocks) depending on positioning of towers/stacks.	Location of structures and site specific vertical extent of compressible soils. Site specific vertical variability in soil consistency and 3D variability in rocks needs to be investigated in individual structure footprints.	High concentrated loads will require special foundations (e.g. piles) in soils. Ground improvement techniques will not assist founding in soils – bearing loads will have to be transferred to deeper (denser) soil horizons, to depths where pile skin friction supports structure or to soft rock/bedrock horizons where bearing capacity will surpass that of soils.	None – if correct founding methods are employed (e.g. piles). Pile testing should be supervised. Quality control should be vigorously pursued.	No impacts provided correct founding methods are selected. The integrity of soils stabilised to arrest liquefaction will need to be monitored to ensure that their integrity remains intact. Dust Control impacting KNPS ventilation systems.	No obvious impacts unless dewatering is required

Aspect / Structure	Potential Extent of Structure (Geometry)	Geotechnical Characteristics Affecting Founding	Reliability of Existing Geotechnical Data	Data Uncertainties	Design Precautions	Construction Difficulties	Operational Impacts	Operational and Environmental Impacts
		rock into soil horizons (i.e. changes in seismic wave propagation through different mediums).						
Structure containing heavy vibratory machinery and/or settlement sensitive structures	Potentially linear (e.g. crane bays), but probably rectangular mechanical / nuclear process / related structures (industrial scale).	Liquefaction potential of slope excavations and the groundwater table characteristics. Consolidation of loose soils triggered by plant vibrations. Variations in vertical and horizontal consistency of soils and the random presence of cemented / calcretised zones near surface. Soils will not have bearing capacity >200 kPa even with mechanical ground improvements – desired bearing capacity in excess of this will require cement stabilisation and/or piling. Dynamic loading regime and seismic wave impedance / amplification when propagating from rock into soil horizons (i.e. changes in seismic wave propagation through different mediums).	Site data are reliable but lack site specific focus on a local scale. Data does not appropriately document the 3D variability in rock characteristics as the site rocks are covered with overburden and only accessible by drilling.	Location of structures and site specific vertical extent of compressible soils. Impedance / amplification of seismic waves propagating from bedrock into soil horizons impacting on differential dynamic loading scenarios and triggering liquefaction.	High concentrated loads will require special foundations (e.g. piles or deep excavations to bedrock) and cannot be founded in soils unless bearing loads are considerably <200 kPa Differential settlement, depth to bedrock, groundwater influences on design and construction. Seismic response of structures, systems and components.	Slope stability of excavations. Effective dewatering. Environmentally acceptable disposal of extracted groundwater and spoil. The potential impact of spoil stockpiles on nuclear installation safety due to instability if located near the nuclear installation works.	No impacts provided correct founding methods are selected. The integrity of soils stabilised to permanently arrest liquefaction will need to be monitored to ensure that their integrity remains intact. Dust Control impacting KNPS ventilation systems.	Groundwater drawdown effecting localised freshwater ecological environment. Excavation footprints and cumulative impact of temporary stockpile footprints.
Linear structures (e.g. cooling water canals) Tunnels are treated separately in ***Table T-5.15.27***	From cooling water source (sea) to the nuclear island founded in soil, rock or a combination of soil and rock (pending final design levels of the nuclear installation(s)). Potentially on an extensive linear scale.	Liquefaction potential and the position of the groundwater table. Variations in vertical and horizontal consistency of soils and the random presence of cemented / calcretised zones near surface may introduce localised bearing capacity variability and/or excavation difficulties. Dynamic loading regime	Site data are reliable but lack site specific focus on a local scale. Data do not appropriately document the 3D variability in rock characteristics as the site rocks are covered with overburden and only accessible by drilling.	Location of structures and site specific vertical extent of compressible soils. Extent of variability along linear structure alignment (not yet known). Impedance / amplification of seismic waves propagating from bedrock into soil	Differential settlement and tolerances. Earthworks requirements through undulating dune surface topography and impacts on dewatering, design against liquefaction failure and slope stability. Ground improvement measures will not increase bearing capacity above 200 kPa by mechanical means.	Earthworks and materials handling/double handling to limit environmental impacts. Dewatering over extended distances. Disposal of spoil. Stabilisation of deep temporary trenches.	Plant shutdown due to post construction failure of (e.g.) cooling water supply. The integrity of soils stabilised to arrest liquefaction will need to be monitored to ensure that their integrity remains intact. Leaks may erode trench bed and cause	Failure of structures leading to interrupted cooling water supply and/or surface/groundwater contamination. Earthworks and materials handling (temporary stockpiling) scars leading to general environmental degradation.

Aspect / Structure	Potential Extent of Structure (Geometry)	Geotechnical Characteristics Affecting Founding	Reliability of Existing Geotechnical Data	Data Uncertainties	Design Precautions	Construction Difficulties	Operational Impacts	Operational and Environmental Impacts
		and seismic wave impedance / amplification when propagating from rock into soil horizons – seismic waves arriving at the site from deep sources may undergo amplification when travelling from bedrock (with high V_s) into soils with V_s an order of magnitude lower.		horizons impacting on differential dynamic loading scenarios and triggering liquefaction. Occurrence and importance of secondary faults and geological discontinuities. Differential dynamic response in different geology and across lithological boundaries – dynamic response parameters may need to describe the rock mass as lithological boundaries are closely spaced in the Malmesbury Group Rocks.	Stabilisation of soils to arrest liquefaction failure. Strike slip on faults crossing linear structures. Proactively cut-off drainage paths of leaking water along trench beds.		differential settlement Dust Control impacting KNPS ventilation systems.	
Damped foundations	Under the nuclear island and relatively local (i.e. non-linear). Situated in areas of high overburden thickness (average 20 m) anywhere on the site. Structures will be founded in bedrock and on competent (slightly/unweathered rock).	Liquefaction potential and the position of the groundwater table impacting on slope stability. Variable 3D rock characteristics due to weathering profile and/or geology (wide distribution of parameters) – bearing capacity may vary from low strength (in highly weathered siltstones, mudstones and shales) to high strength in unweathered material (greywacke and hornfels). Overburden profile and necessary foundation excavations (influence on project programme, excavation feasibility / design / construction and environmental consequence) Dynamic loading regime	Site data are reliable but lacks site specific focus on a local scale. Data does not appropriately document the 3D variability in rock characteristics as the site rocks are covered with overburden and not easily accessible.	Detailed mapping of bedrock underlying foundations – to be carried out after foundation excavations to bedrock are completed. Position of the proposed footprint. Differential dynamic response in different geology and across lithological boundaries – dynamic response parameters may need to describe the rock mass as lithological boundaries are closely spaced in the Malmesbury Group Rocks. Data lacking in northern areas.	Slope stability of foundation excavation cuts and environmental impacts due to cutback angles and dewatering requirements. Integration of damped structure foundations with auxiliary / supporting structure foundations and issues relating to differential settlement, dynamic loading and liquefaction of soils. Damping/impedance within foundation soils. Tension/shear failure over lithological boundaries influencing integration of auxiliary structures. Stability of spoil stockpiles.	Effective dewatering, disposal of extracted groundwater and excavated slope stability. Earthworks and materials handling to limit environmental impacts. Deeply weathered rock horizons that require removal (to depth) and backfilling with e.g. mass concrete/cement stabilised soil. Construction programme / sequencing may be impacted by design and/or localised	Plant shutdown due to post construction failure. The integrity of soils stabilised to arrest liquefaction will need to be monitored to ensure that their integrity remains intact.	Groundwater drawdown effecting regional freshwater ecological environment and groundwater users. Cumulative impact of excavation and spoil footprints. Impacts on the receiving environment of potentially large scale spoil disposal if not stockpiled.

Aspect / Structure	Potential Extent of Structure (Geometry)	Geotechnical Characteristics Affecting Founding	Reliability of Existing Geotechnical Data	Data Uncertainties	Design Precautions	Construction Difficulties	Operational Impacts	Operational and Environmental Impacts
		and seismic wave impedance / amplification when propagating from rock into soil horizons (i.e. changes in seismic wave propagation through different mediums). Seismic response spectra.				geotechnical conditions.		

5.15.10.1.2 Conceptual Mitigation Measures

This report presents sufficient information to make an assessment that the site capacity with respect to founding materials will not require unduly onerous designs. In addition, later investigations prior to the pre-operational phase will not be unduly onerous. The repeated reference to variability of site founding materials cannot go unchecked. It will be necessary to finalise designs to take account of site specific conditions and variability to provide data related to the uncertainties listed in ***Table 5.15.27***.

Such investigations will include (with reference to (International Atomic Energy Agency, 2004):

- additional drilling;
- mapping of exposed bedrock once foundation excavations have been completed.

Foundations can then be appropriately designed based on localised (or site specific) information gleaned from these investigations or can be relocated if unfavourable conditions are encountered (e.g. extensive weathering under the proposed footprint).

Certain mitigation measures can, however, be proposed at this stage of the investigation based on current knowledge of the geotechnical profile and the concepts presented in ***Table 5.15.27***. These are:

- small, conventional spread and strip foundations (founding in soils):
  - Liquefaction failure of founding materials carries a high risk across the site and dewatering of these materials will reduce the probability of liquefaction to zero. Dewatering systems will, however, be required to maintain the drawdown achieved during construction for extended time periods as re-saturation of site soils will re-introduce high liquefaction risks.
  - Soil improvement measures to reduce variability are likely to provide solutions for many shallow founding concerns and remove liquefaction potential as has been successfully done at the KNPS. It is critical that the design engineer employs robust soil improvement measures and, in general, improvement (and homogenisation) of soil consistency will best be achieved by removing material and replacing in engineered layers not exceeding 200 mm per layer compacted to 100 per cent mod AASHTO with composite layers not less than 2 m

in vertical thickness – soil rafts so created will not support loads greater than 200 kPa unless cement stabilisation is employed.

- In certain instances (e.g. where the groundwater table drawdown cannot be guaranteed for the nuclear installation life), soil improvement measures should include stabilising soils with cement as was achieved at the KNPS.
- Ground improvement to reduce liquefaction potential will require integrity checking (monitoring) throughout the lifetime of the nuclear installation(s) as is done at the KNPS.
- More heavily loaded structures or settlement sensitive structures will require piling probably to bedrock level to support end bearing piles.

• adjoining small, conventional spread and strip foundations (founding in soils) to monolithic structures (founded in rock):

  - Special articulation design allowing higher differential movement tolerances between these structures is advised.
  - Backfill around monolithic structures must be carried out as recommended for soil improvement measures above and must not be confined to small inaccessible spaces where engineering of backfill in layers becomes impractical.
  - Cement stabilisation of backfill layers is recommended to reduce differential settlement and liquefaction risks.

• towers and stacks:

  - As these structures will be situated in thick overburden areas, they should be piled as any attempts at soil improvement may result in large excavations with associated slope stability concerns, dewatering concerns and environmental impacts.
  - The response of safety related structures to earthquake induced dynamic loading will require specialised design consideration pending the finalisation of the SHA.

• structure containing heavy vibratory machinery and/or settlement sensitive structures:

  - Soil improvement measures are not advised as these structures will be founded in areas of thick overburden deposits – these structures should be piled.

  - The response of safety related structures to earthquake induced dynamic loading will require specialised design consideration pending the finalisation of the SHA.

- linear structures:

  - These structures will be founded in soil, and depending on their imposed loads, conventional soil improvement measures should be employed should bearing capacity <200 kPa be required. For more heavily loaded structures (i.e. imposed loads >200 kPa), cement stabilised methods should be considered (trial mixes with *in situ* materials should be conducted) and for settlement sensitive structures, piled foundations will be required to transfer loads to bedrock in most instances.
  - If structures traverse areas where deep excavations in soil are required (e.g. traversing the dune area), slope stability issues will be primary design drivers – soil slopes should be battered back to 1:3 vertical to horizontal after dewatering.
  - The response of safety related structures to earthquake induced dynamic loading will require specialised design considering the PSHA outcomes as it may be required that e.g. damped foundations be considered in design.
  - Proactively interrupt or cut-off potentially leaking water flow underneath linear structures along trench bottoms.

- damped foundations:

  - These foundations will be in rock and the stability of excavated slopes carries a high risk.
  - Such slopes may consist of soil, rock, or composite slopes. Soil slopes should be battered back to 1:3 vertical to horizontal after dewatering, and rock slopes running within 30° of bedding strike (320° to 330°) will require conventional lateral support (e.g. rock anchors).
  - Soil slopes should not contribute high superimposed loads to the crest of rock slopes and should be set back from the crest.
  - Dewatering is the most critical mitigation measure required to stabilise excavations in this high liquefaction potential environment. Dewatering design will have many design drivers and it is imperative that the behaviour of local (and regional) groundwater flow and drawdown is understood as this will provide a sound basis for site specific dewatering mitigation measures.

- Variability in rock founding material will be explored in detail (mapped and investigated) when foundation excavations have been successfully implemented. At this stage, a sound basis for deriving mitigation measures will be set and founding material improvement, spanning of weak zones etc. will not present unduly onerous design challenges.
- Avoidance of geological boundaries will not be practical in the Malmesbury Group rocks. Detailed mapping of the exposed bedrock (at foundation excavation stage) will be required to assess the extent of variations in rock parameters between geological boundaries
- The response of safety related structures to earthquake induced dynamic loading will require specialised design considering the outcomes of the PSHA.

5.15.10.2 Integration of Auxiliary Systems

5.15.10.2.1 Subsurface Site Characteristics and Areas of Uncertainty

Primary auxiliary structures such as cooling water intake/outlet structures will need to traverse the site from the sea to the nuclear installation(s). These structures could be linear structures of considerable length (e.g. tunnels or channels). The conceptual integration of these structures with the nuclear island is dealt with in ***Subsection 5.15.10.1*** (in ***Table 5.15.27***) under ‘linear foundations’ and ‘adjoining structures’.

Tunnelling is a special case and is addressed separately in ***Subsection 5.15.10.3***.

5.15.10.2.2 Conceptual Mitigation Measures

Conceptual mitigation measures are similar to those described in ***Subsection 5.15.10.1*** where adjoining structures are discussed.

5.15.10.3 Tunnelling and Rock Slope Stability

5.15.10.3.1 Subsurface Site Characteristics and Areas of Uncertainty

The preceding sections (supported by the appendices) describe the geotechnical profile and highlight the following:

- the fact that the use of tunnels for cooling water supply was not considered a realistic option for the KNPS cooling water systems;
- the soil profile (as summarised in ***Subsection 5.15.10.1***);

- the rock profile (as summarised in ***Subsection 5.15.10.1***);
- the groundwater table (as summarised in ***Subsection 5.15.10.1***);
- the dominance of three major joint sets, their low rock RMR and random occurrences of small fracture spacing is noted – Intersection of these joint sets provides release mechanisms in excavations of all types for sliding, toppling and drop-out failure.
- the high likelihood that water inflow into tunnels will be significant in a highly jointed rock profile overlain by an intergranular aquifer which is in close proximity to the sea;
- knowledge of offshore geotechnical conditions is not relevant to any aspects addressed in ***Section 5.15*** besides tunnelling offshore should this be adopted as a cooling water supply method. Discussion of the offshore geotechnical setting is dealt with in ***Section 5.13*** (Geology) and ***Section 5.14*** (Seismic Hazard) specifically with reference to seismic source characterisation.

Tunnel excavation stability will be governed in general by ‘lack of clamp’ due to anticipated low ambient rock stresses and will manifest in principle as ‘block fall-out’ or ‘running ground’ in the case of soft or sheared zones. This may occur in both the roof and sides of the tunnel. Slabs, blocks and wedges may fall from the roof depending on the orientation of the tunnels relative to the dominant joint sets. Wedges and prisms may fall from the sides depending on the number and orientation of intersecting joint sets. Separation, where this occurs, on sub-horizontal joints together with sub-vertical joints in the sides can give rise to arch failure of the roof. This is not deemed to be a primary concern based on the analyses of the joint conditions carried out, and wherever it may occur could be dealt with by conventional tunnel support. This will need to be re-qualified once offshore geotechnical data becomes available.

Tunnel safety will be closely related to tunnel orientation relative to the dominant joint set strike. Joints present macro and micro joint conditions that will challenge excavation stability and construction methodology should tunnel orientation be within 30° of dominant joint set/bedding strike.

Tunnel construction methodology will be influenced by the inflow of water and by engagement with soft rock or shear zones of considerable width. This, however, will not necessarily pose design constraints to the point where safety is impacted or the site is unsuitable. However, construction methodology will be a critical consideration along with strict quality control and policing of this methodology at construction stage.

The orientation of tunnels will be affected by the undulating bedrock topography. ***Table 5.15.28*** interrogates the tunnelling concept in a similar way to that done for foundations in ***Table 5.15.27***, with similar aims.

The above discussion is in conflict with the outcomes of the KNPS investigations where tunnelling was discounted as a cooling water supply option. Advances in tunnelling technology and the manner in which rock competence is now evaluated requires that this important statement be revised. It remains that the design and construction of tunnels on this site will present challenges particularly related to groundwater inflow into tunnels. However, since most of these challenges can be overcome with conventional tunnel support, the potential for using tunnels for cooling water supply could be revisited. To fully qualify the latter statement, additional information relating to offshore geotechnical conditions will be required prior to confirming that the site is suitable for tunnelling as a cooling water supply option.

Rock slope stability will be impacted largely by the same rock features presented above for tunnelling stability except that rock slopes may be susceptible to toppling failure in addition to the failure mechanisms presented above for tunnels. Toppling slope failure can occur on the sub-vertical joints if slopes are oriented within 30° of the strike of the joints, but these slopes can be supported with conventional lateral support techniques.

Table 5.15.28
Tunnelling Concepts and Geotechnical Data Acquisition Needs

Aspect / Structure	Potential Extent of Structure (Geometry)	Geotechnical Characteristics Affecting the Structure	Reliability of Existing Geotechnical Data	Data Uncertainties	Design Precautions	Construction Difficulties	Potential Operational Impacts	Environmental Impacts
Tunnels[27]	From intake bell in sea floor to the nuclear island. Potentially from coastline to some distance into the sea, spanning lithological boundaries between disparate strata that strike nearly parallel to the coast.	Variability in tunnel head, i.e., depth of overburden, influenced by the geotechnical profile and undulating bedrock topography. Rock joint characteristics and the dominance of three major joint sets with steeply dipping bedding (75˚). Variability in 3D rock characteristics due to weathering (wide distribution of parameters). Groundwater inflow in a highly jointed rock environment overlain by an intergranular aquifer. Dynamic loading regime and intricacies introduced by mixed face conditions, soft ground sections and crossing lithological boundaries and geological discontinuities.	Site data are reliable but lack site specific focus on a local scale (i.e. along proposed tunnel alignment which is not yet known). Data does not appropriately document the lateral variability in rock characteristics over large distances as the site rocks are covered with overburden and not easily accessible. Lack specific identification of secondary faults and geological discontinuities.	Extent of variability (particularly in rock) along liner structure alignment (not yet known) and offshore geotechnical data.	Ground/sea water inflow into tunnel excavations – management or effective dewatering design and disposal of extracted groundwater. Localised response to dynamic loading in varying lithological zones and rock discontinuities – including all manner of failure mechanisms related to seismic wave distortion and localised tension/shear forces at these boundaries that dip at 75° WSW. Foundations required spanning laterally extensive weak zones (e.g. extensive shale / mudstone / siltstone zones). Differential settlement and tolerances. Tunnel orientation to ideally be perpendicular to strike, or if not possible, ideally not within 30° of strike of dominant joint sets. Overcoming tunnel instability due to 'lack of clamp' related to low ambient rock stresses manifesting as 'block fall-out' or 'running ground' in the case of soft or sheared zones. Overcoming slab, block and wedge fallout from the roof related to orientation of the tunnels relevant to the dominant joint sets. Overcoming wedge and prism fallout from the sides depending on the number and orientation of intersecting joint sets. Overcoming separation, on sub-horizontal joints together with sub-vertical joints giving rise to arch failure and rock falls. Managing sand intake around the intake structure. Appropriate design of progressive grouting to advance the tunnel excavation where required. Overcoming circumferential tensile failure in tunnel linings under pumping heads of water.	Ground/sea water ingress management will be a particularly onerous challenge – special tunnelling techniques and skills will be required to keep water out and stabilise tunnels walls and face. Uncertain excavation productivity rates impacted by lithological setting, discontinuities in rock, joint distribution and ground/sea water head, the need for adaptable construction methodology, extent of potential soft ground/fault sections, groundwater inflow on open structures, sub-vertical faults/shear zones parallel to tunnels. Grouting the cavity between rock and tunnel lining.	Plant shutdown and in extreme cases radiation release due to post construction failure of cooling water supply.	Failure of structures leading to temporary loss of cooling water or surface/ground/sea water contamination. Groundwater drawdown effecting regional freshwater ecological environment and groundwater users.

[27] Reference to linear structures and adjoining structures in ***Table T-5.15.25*** also refer to tunnels – ***Table T-5.15.26*** discusses issues relative to tunnels only.

Aspect / Structure	Potential Extent of Structure (Geometry)	Geotechnical Characteristics Affecting the Structure	Reliability of Existing Geotechnical Data	Data Uncertainties	Design Precautions	Construction Difficulties	Potential Operational Impacts	Environmental Impacts
Excavations in rock.	Nuclear installation foundation excavations.	Rock joint characteristics and the dominance of three major joint sets with steeply dipping bedding. Variability in 3D rock characteristics due to weathering (wide distribution of parameters). Dynamic loading regime and seismic wave impedance / amplification when propagating from rock into soil horizons. Groundwater inflow in a highly jointed rock environment overlain by an intergranular aquifer. Dynamic loading regime and intricacies introduced by mixed face conditions and soft ground sections. Differential displacements across secondary faults and geological discontinuities.	Site data are reliable but lack site specific focus on a local scale (i.e. at nuclear installation footprint). Data does not appropriately document the lateral variability in rock characteristics over large distances as the site rocks are covered with overburden and not easily accessible.	Extent of variability (particularly in rock) across foundation excavation (not yet known). Occurrence of secondary faults and geological discontinuities.	Ground/sea water inflow into excavations – management or effective dewatering design and disposal of extracted groundwater. Localised response to dynamic loading at lithological boundaries and rock discontinuities – including all manner of failure mechanisms related to seismic wave distortion and localised tension/shear forces at these boundaries that dip at 75° WSW, Excavations required to span laterally extensive weak zones (e.g. extensive shale / mudstone / siltstone zones). Differential settlement and tolerances. Excavation orientation to ideally be perpendicular to strike, or if not possible, not within 30° of strike of dominant joint sets. Overcoming toppling failure depending on the number and orientation of intersecting joint sets particularly if orientation is within 30° of the steeply dipping bedding.	Ground/sea water ingress management. Uncertain excavation productivity rates impacted by lithological setting, discontinuities in rock, joint distribution and ground/sea water head, the need for adaptable construction methodology, extent of potential soft ground/fault sections, groundwater inflow on open structures, sub-vertical faults/shear zones parallel to tunnels. Excavation of shallow heads of rock on wave-cut platform may require extremely hard ripping or alternatively a great number of closely spaced short blast holes to be drilled and blasted.	None – pre operation	Groundwater drawdown effecting regional freshwater ecological environment and groundwater users.

5.15.10.3.2 Conceptual Mitigation Measures

Closing out the lack of offshore geotechnical data uncertainty is a primary step to conceptualising mitigation measures for the use of tunnels for cooling water supply as it needs to be confirmed whether these offshore conditions are similar (or different) from those encountered onshore.

It will be critical that, if tunnels are used, these are to be orientated perpendicular to the dominant joint sets as far as practically possible. If the nuclear installation layout permits this, tunnel design will be simplified and safety improved. Tunnel sections orientated parallel to strike will have to be specially designed and special precautions taken during construction. Since tunnels are only likely to be potentially used as cooling water intake structures and will, in all likelihood be orientated directly out to sea, it is not likely that this orientation will be impacted by bedding which strikes at 320˚ to 330˚ at least not over long distances.

Any orientation, however, may result in rock falls from tunnel roofs on the sub-vertical joints, released by the subhorizontal joints. The impact that orientation of tunnels relative to the dominant joint set strike (as well as sub-vertical shear zones or faults or major discontinuities) has on tunnel head requirements must, however, be specifically investigated.

Block, slab, prism and wedge fall-out, running ground fallout or other instability problems related to joint separation or other conditions, could be effectively overcome with rockbolts and cable anchors and, if necessary, supplemented with shotcrete which in turn may or may not be mesh or fibre reinforced.

An additional consideration is the safe design of tunnel lining. Specifically, tunnel lining spanning extended weak zones and/or geotechnical discontinuities may be subjected to differential dynamic loading scenarios. Reflection of shear or compression waves arriving at the geological contact at an angle, and considering that the bedding dips steeply (at 75º), may induce a shear dislocation at the contact in addition to the anticipated (induced) tension loads. Implications of this scenario must be established using the seismic loading regime described in the PSHA and prior to the pre-operational stage to finalise special tunnel lining design at these contacts.

The rock strength and joint conditions will have a direct impact on the rock head[28] of proposed tunnels. Required rock head could be at least three tunnel diameters to maintain tunnel stability in these materials. Offshore geotechnical characteristics will need to be confirmed to assess the impact that tunnel head has on tunnelling feasibility.

Tunnel linings will have to be designed to adequately withstand potential tensions due to the hydraulic pressures of cooling water, particularly if cooling water is pumped. These linings will also have to be designed specially to cross geological transition zones or features (shear zones, faults, discontinuities) in the surrounding rock.

The geological setting of the site (i.e. in the Malmesbury Group rocks that have undergone significant historical deformation), the presence of significant groundwater and the proximity to the sea will make the design and implementation of tunnels demanding. Of particular concern would be tunnelling into the sea where offshore geotechnical characterisation need to be confirmed. All of the above qualifications that trend towards describing tunnels as a potential cooling water supply option need to be revisited once these data become available. Alternative methods to tunnelling (e.g. overland structures) should be considered prior to utilising tunnels. An important mitigation measure therefore is to carry out a detailed feasibility study prior to utilising tunnels.

Rock slopes/cuts can be stabilised with conventional lateral support techniques such as retaining structures, rock bolts, mesh and shotcrete as well as combinations of these.

5.15.10.4 Other Buried and Submerged Structures

5.15.10.4.1 Subsurface Site Characteristics and Areas of Uncertainty

Other buried structures (i.e. buried structures other than tunnels) relate to structures such as pipelines and utilities. These structures are expected to be founded, in general, shallow in the geotechnical profile (i.e. in soil). The design and construction of buried structures will need to take into consideration the following:

- the high liquefaction potential of large areas of the site soils and the presence of the groundwater table within 3 to 5 m from surface;

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[28] Rock head: the vertical thickness of rock material above the tunnel.

- potential difficulties with trench excavations, their stability and excavated materials disposal management;
- knowledge of localised founding conditions;
- the use of site materials for construction.

Table 5.15.29 interrogates the concept of founding/constructing buried structures.

Submerged structures are structures that may be required in the shallow coastal environment and that may be flooded upon completion. It is not known whether such structures would be constructed within marine coffer dams. Integration with ***Section 5.9*** (Oceanography and Coastal Engineering) will be required to assess this.

Table 5.15.29
Buried Structure Concepts and Geotechnical Data Acquisition Needs

Aspect / Structure	Potential Extent of Structure (Geometry)	Geotechnical Characteristics Affecting the Structure	Reliability of Existing Geotechnical Data	Data Uncertainties	Design Precautions	Construction Difficulties	Operational Impacts	Environmental Impacts
Buried structures	Between auxiliary structures / from water sources to nuclear installation(s), between nuclear installation(s) and municipal supplies – in summary, linear structures randomly placed/orientated on site and surrounds – these structures will be founded in soil.	Liquefaction potential and the position of the groundwater table, particularly if these structures are situated near excavated sand slopes - such slopes may be subject to slope failure, at least during construction. Variations in vertical and horizontal consistency of soils and the random presence of calcrete zones near surface. Impedance/amplification of seismic waves propagated from rock into soil (founding) medium. Undulating site surface topography and resulting excavated slope stability. Lithological boundaries and geological discontinuities on which seismically induced strike slip or dip displacement may disrupt crossing linear structures.	Site data are reliable but lack site specific focus on a local scale (i.e. along proposed alignments).	Extent of variability in soil parameters along linear structure alignment (not yet known). Impedance (propagation of seismic waves from rock into soils serving as a founding medium). Occurrence and importance of secondary faults and geological discontinuities sympathetic to major fault and geological features.	High liquefaction potential of soils. Ground improvement methods will not result in bearing capacity exceeding 200 kPa. Differential settlement and tolerances. Proactively cut-off drainage paths of leaking water along linear trench bottoms.	Ground improvement and excavations across undulating landscapes. Soil stabilisation to depth to arrest liquefaction. Material handling to limit environmental disturbance. Embankment stability in deep temporary trenches.	Probably related to nuisance impacts (i.e. random/interrupted supply of services to support buildings). The integrity of ground improvement to arrest liquefaction will need to be monitored on an on-going basis. Water leaks under linear structures may erode trench bed and cause differential settlement.	Materials (soil) handling and excavation footprints.

5.15.10.4.2 Conceptual Mitigation Measures

Founding these structures shallow in the geotechnical profile will require a similar approach to soil improvement as that discussed under linear structures in ***Subsection 5.15.10.1.2***.

5.15.10.5 Earth Structures and their Construction

Earth structures may comprise temporary or permanent cut slopes in soil, built embankment slopes in soil or rubble and dumped rubble breakwaters comprising armour rock and core stone.

These structures may be susceptible to slip circle failure, mass slump failure, liquefaction failure or erosion failure.

5.15.10.5.1 Subsurface Site Characteristics and Areas of Uncertainty

Subsections 5.15.10.1 to ***5.15.10.4*** make repeated reference to excavations that will be required to construct the nuclear installation foundations. These excavations will require stabilisation during construction. In addition, once the locality and finished level of the nuclear installation(s) is fixed, it may become necessary to design remnant slopes that will remain in place (i.e. slopes above the nuclear installation finished level).

The preceding sections (supported by the appendices) describe the site geotechnical profile and refer in particular to the following:

- the groundwater regime and soil/rock profiles referred to in ***Subsection 5.15.7***;
- the shear-strength parameters of soils that are cohesionless and have an internal friction angle in the order of 33°;
- the high liquefaction potential referred to in ***Subsection 5.15.7.5.3***;
- rock joint characteristics referred to in ***Subsection 5.15.7.2***;
- preliminary (pending the PSHA for confirmation) dynamic loading considerations;
- potential cut-slope geometry in foundation excavations which will, on average, be *c.*20 m in height.

Excavation methods, dewatering considerations and disposal of spoil will be key planning considerations. The physical extent of site soils (i.e. thick aeolian and marine deposits) and the presence of groundwater will need to be considered in depth to avoid site safety being compromised during construction.

In the presence of the Sandveld intergranular aquifer, it may not be practical to consider lateral support measures for cut slopes in excess of 15 m high. For this reason, excavations will, in all likelihood, be designed to have shallow embankment angles (in the order of 1:3 vertical to horizontal) to ensure safety during construction. This is a limiting factor on this site for the following reasons:

- Excavation volumes and spoil will be excessive (see ***Subsection 5.15.7.6***) and will need to be considered in the nuclear installation design and SAR.
- Dewatering requirements will need to be robust to counter safety risks related to slope stability, soil liquefaction and reliability of dewatering systems.
- Environmental impacts could be high because of bulk excavations, disposal of spoil and dewatering effects on the regional groundwater table.

5.15.10.5.2 Earthworks Construction

Excavations of the nuclear installation foundations could be carried out either by using conventional earthmoving plant or by slurry pumping taking into consideration the ready supply of groundwater at the site:

- conventional earthmoving plant:
  - Once the groundwater table level is breached (within *c.3*-5 m from surface), conventional earthmoving plant operations will not be possible without prior dewatering.
  - Since the site is underlain by the Sandveld Aquifer, dewatering will require a considerable effort and dewatering design will need to be robust enough to consider open excavations for extended time periods (possibly years).
  - Isolation of the nuclear installation footprint using impermeable groundwater barriers will need to take cognisance of the undulating bedrock topography.

- slurry pumping:
  - Slurry pumping above the groundwater table will need a regular water supply and this could be sourced from the Sandveld Aquifer (i.e. dewatering product).
  - Near surface calcrete horizons encountered in the field investigations could minimise (or even preclude) productivity of slurry pumping near surface.
  - Slurry pumping may have a large environmental impact due to less manageable (onshore) soil disposal areas.

The material generated during excavations could be managed as follows:

- soils:
  - The bulk of excavated soils may be disposed of on or off site. Disposal sites will require adequate planning to ensure controlled impacts on the environment as the disposal sites will be extensive.
  - Some soil material may be utilised at the site, but this will be restricted to general filling as reliable engineering platforms could be difficult to construct with the poorly graded site soils and bearing capacity will not be improved to >200 kPa by mechanical means).

- rocks:
  - Excavations in the Malmesbury rocks (greywacke/hornfels and metamorphosed equivalents) could provide materials for breakwater rubble, general fill or selected fill, but the feasibility of producing sufficient and adequately graded material will need to be investigated further – the bulk of foundation excavations are likely to produce material that is predominantly weathered and probably under-sized for breakwater construction.
  - Successful quarrying (blasting and hauling of specification envelope materials) for breakwater armour rock and core stone protection in the requisite proportions will not be possible at the site and the quarrying feasibility will need to be investigated off site where rock is exposed on surface.
  - Rock durability/strength against wave action will be a consideration.
  - Sources of concrete aggregate will need to be investigated off site.

5.15.10.5.3 Breakwaters

It is possible that some form of defence against sea wave attack will be required (i.e. in the form of breakwaters). Several considerations arise with respect to site materials and their potential use in the construction of breakwaters. These considerations all have the potential to impact on the constructability of breakwaters and include for example:

- rock durability against repeated wave impact;
- suitable volumes of amour rock and core stone sizes (that can be provided by quarrying) in the requisite proportions;
- loss of armour rock due to settlement into the seabed;
- loss of armour rock due to sea current transportation/displacement.

Table 5.15.30 interrogates the concept of earth structures (concentrating on slopes) in a similar way to that done for foundations in ***Table 5.15.30***, with similar aims.

The compatibility of site material for breakwater construction is not critical to the SSR supporting the suitability of the site as such materials could be sourced anywhere. However, constructability of such structures is critical to assessing site suitability. Since these structures were used in the development of the KNPS and are a feature of the KNPS, there is no reason to argue that the site is unsuitable.

Table T-5.15.30
Earth Structures Concepts and Geotechnical Data Acquisition Needs

Aspect / Structure	Potential Extent of Structure (Geometry)	Geotechnical Characteristics Affecting the Structure	Reliability of Existing Geotechnical Data	Data Uncertainties	Design Precautions	Construction Difficulties	Operational Impacts	Environmental Impacts
Temporary and permanent cut slopes	Extensive areas in the case of damped (nuclear island) structures to localised areas for random platform construction. Wide shallow trenches of extensive length for linear and some buried structures thus spanning variable soil founding materials.	Liquefaction potential and the position of the groundwater table. Soil consistency variances with depth and soil shear strength. Impedance/amplification of seismic waves propagated from rock into soil. Position of the excavation. Erodibility (wind and water) of site soils.	Site data are reliable and represents a reliable source of information for bulk earthworks related slope design. More detail will be required to assess localised cut slope scenarios.	Impedance (propagation of seismic waves from rock into soils). Geotechnical setting of localised (and randomly) positioned cut slopes.	Dewatering and liquefaction avoidance. Slope drainage. Erosion control. Minimising environmental degradation.	Mass earthworks (excavation) and groundwater management (dewatering). Spoil disposal. Dust control to minimise risks to the KNPS ventilation systems.	Potential risk posed to safety related structures in the vicinity of poorly designed cut slopes. Long term operation and maintenance of permanent dewatering installations. Dust generation during construction of additional nuclear installation(s) could adversely impact on the KNPS ventilation systems.	Materials (soil) handling – excavation and disposal methods / footprints / receiving environment.
Built slopes and platforms	Dependent on the site layout and position of fill platforms (spatially and finished level).	Liquefaction potential of founding soils and the position of the groundwater table. Engineered soil characteristics and shear strength. Construction/engineering of site soils. Near surface calcrete horizons.	Site data are reliable and represents a reliable source of information for bulk earthworks. Data does not reliably represent localised platform construction scenarios.	Impedance (propagation of seismic waves from rock into soils). Position of fill slopes/platforms. Shear strength of engineered soils. Available quantities of soils that are suitable for stabilisation.	Dewatering and liquefaction avoidance. Slope drainage. Erosion control. Minimising environmental degradation. Soil moisture density relationships and compactibility of fill. Slope stability and the impact of surcharge loading on slope stability.	Soil moisture density relationships and compactibility of fill.	Potential risk posed to safety related structures in the vicinity of poorly designed/constructed cut slopes.	Platform footprint impacts on the environment as well as materials handling impacts (e.g. temporary stockpiling).

Aspect / Structure	Potential Extent of Structure (Geometry)	Geotechnical Characteristics Affecting the Structure	Reliability of Existing Geotechnical Data	Data Uncertainties	Design Precautions	Construction Difficulties	Operational Impacts	Environmental Impacts
Breakwaters	Coastal structure protecting marine engineering structures, systems or components	Rock durability and specification envelope production in variable material. Shallow marine geotechnical conditions (an unknown).	Site data are reliable and represents a reliable source of information in the site region context. Data does not reliably represent localised characteristics at proposed quarry sites. No shallow marine geotechnical data exists.	Shallow marine geotechnical conditions and founding of breakwaters. Sustainable quarry conditions and sites which will be off-site.	Rock durability against repeated wave impact; Suitable volumes of amour rock and core stone sizes (that can be provided by quarrying); Loss of amour rock due to settlement into the seabed. Loss of amour rock due to sea current transportation / displacement.	Aggressive wave environment. Variable quarrying conditions even when investigated on a local scale.	Damage to marine structures, systems or components.	No direct impacts.

5.15.10.5.4 Conceptual Mitigation Measures

Mitigation measures for earthworks construction include the following:

- liquefaction potential and dewatering:
  - Liquefaction can only take place in saturated soils and robust dewatering design will mitigate this risk. In addition, there is no risk to foundation failure of the nuclear installation(s) related to liquefaction if the nuclear installation(s) will be founded on bedrock / cement stabilised raft.
  - Dewatering is therefore a key activity associated with the nuclear installation foundation construction and it is important to highlight a number of engineering principles that are essential for successful dewatering and reduction of safety risks:
    - Dewatering systems must be durable as excavations will be open for a long period of time (several years as presented in ***Chapter 3*** (Overview of Planned Activities at the Site) of this SSR).
    - Extended drawdown profiles will be desirable to remove groundwater from excavation slopes and/or from behind lateral support systems thereby limiting slope (including liquefaction induced) failure risks related to open excavations.
    - Since the site is underlain by the Sandveld Aquifer, it may be necessary to isolate the excavation footprint from the aquifer by installing a subsurface grout curtain, but the design of this will need to take cognisance of the undulating wave cut platform.
    - To improve long term durability of dewatering systems, an accessible drainage gallery that can be regularly maintained should be considered.

- construction methodology and environmental conservation:
  - Detailed consideration of excavation stability will be required. It is likely that shallow embankment angles (in the order of <1:3 vertical to horizontal or approximately 18°) will be required to ensure safety during construction.
  - The bulk earthworks methodology could consider slurry pumping operations as slurry pumping will assist in initiating groundwater table drawdown.

- Regardless of the earthmoving method employed, high environmental damage risks may occur as an average 20 m of overburden exists across the site – alternative siting of the nuclear installation(s) is thus not a feasible mitigating measure.

5.15.10.6 Liquefaction Potential

5.15.10.6.1 Subsurface Site Characteristics and Areas of Uncertainty

An important consideration is that liquefaction potential is high in areas across the site (***Subsection 5.15.6.6***). Design of the nuclear installation(s) are challenged by this as was the case with the development of the KNPS. Nullifying safety risks due to liquefaction potential of soils can be addressed with similar mitigations as were employed at the KNPS (e.g. constructing cement stabilised rafts under structure foundations to carry loads to bedrock).

Table T-5.15.31 interrogates the impact that highly liquefiable soil may have on the conceptual design of the nuclear installation(s).

Table T-5.15.31
Liquefaction Potential Impacts on the Concept Design of the Nuclear Installation(s) and Geotechnical Data Acquisition Needs

Aspect	Potential Extent of Aspect (Geometry)	Geotechnical Characteristics Affecting the Conceptual Design	Reliability of Existing Geotechnical Data	Data Uncertainties	Design Precautions	Construction Difficulties	Operational Impacts	Environmental Impacts
Liquefaction failure in founding materials	Vast tracts of the site where soils are saturated or may become saturated.	Soil consistency, Vs and the position of the groundwater table. Impedance/amplification of seismic waves propagated from rock into soil. Seismic response spectra. Maximum principle effective stress due to overburden or imposed loading (aggravates liquefaction potential)	Soil characterisation data sufficiently reliable to establish consistently high risks across the site.	Impedance (propagation of seismic waves from rock into soils) that will be closed out by using the seismic loading defined in the PSHA.	Large tracts of the site present high liquefaction risks. Long term dewatering requirements will need to be robust and capable of maintaining groundwater table drawdown for extended periods (years). Soil improvement will be required if soil rafts are required to assist in attaining finished design levels and/or to assist in controlling impedance/amplification of seismic waves – standard (mechanical) improvement measures will not suffice and cement stabilised improvement measures will have to be considered.	Access to bedrock as a founding platform for soil rafts and the associated slope stability and dewatering issues. Conforming to the profile of the wave cut platform with groundwater isolation structures (e.g. impermeable curtains)	Monitoring of soil raft integrity will be an operational requirement. Monitoring of groundwater levels and dewatering efficacy will be an operational requirement.	Groundwater drawdown effecting regional freshwater ecological environment and groundwater users. Material handling and disposal
Liquefaction failure in cut slopes	Vast tracts of the site where soils are saturated or may become saturated.	Soil consistency, Vs and the position of the groundwater table. Impedance/amplification of seismic waves propagated from rock into soil. Seismic response spectra. Position of the excavation relative to high risk areas, the groundwater table and distance from the sea.	Soil characterisation data sufficiently reliable to establish consistently high risks across the site.	Impedance (propagation of seismic waves from rock into soils) that will be closed out by using the seismic loading defined in the PSHA.	Large tracts of the site presents high liquefaction risks in slope soils. Long term dewatering requirements will need to be robust and capable of maintaining groundwater table drawdown for extended periods (years). Erosion control, slope drainage and access to the excavation.	Conforming to the profile of the wave cut platform with groundwater isolation structures (e.g. impermeable curtains). Constructing access ramps to the excavation in soils with variable consistency. Erosion control, slope drainage and access to the excavation.	Monitoring of slope integrity will be an operational requirement should remnant slopes remain. Monitoring of groundwater levels and dewatering efficacy will be an operational requirement.	Groundwater drawdown effecting regional freshwater ecological environment and groundwater users. Material handling and disposal

5.15.10.6.2 Conceptual Mitigation Measures

The SSR investigations show that liquefaction potential is high across the site (***Subsection 5.15.6.6***). Considering the risks associated with liquefaction failure, mitigation against liquefaction failure will be a cornerstone of the nuclear installation design as was the case with development of the KNPS.

Dewatering and soil stabilisation will remove any risks related to liquefaction failure. It is the robustness of dewatering design, however, which is a more significant challenge as cement stabilisation has proved successful with development of the KNPS. It must be remembered that dewatering of the site for construction of a nuclear installation(s) carries a number of risks, namely:

- potential risks to the integrity of existing foundations on the site (i.e. the KNPS);
- high risks to the environment related to drawdown effects (e.g. adverse impact on groundwater use boreholes) and possible sea water intrusion;
- a prerequisite that dewatering systems perform through the bulk of the construction period (years) and potentially through the life of the nuclear installation(s) (site specific requirements potentially dictating this, e.g. if remnant slopes could undergo liquefaction failure);
- soil stabilisation will require on-going monitoring through the lifetime of the nuclear installation(s) to confirm integrity of stabilised soils.

The design engineers will be required to balance these risks and a number of in-principle suggestions can be made as follows:

- Liquefaction failure risks exist across the site and the methods employed for addressing these risks at the KNPS will be valid anywhere on the site as the KNPS site has a similar geotechnical profile to the remainder of the site.
- Isolation of the nuclear installation footprint from groundwater intrusion may be challenging due to the undulating nature of the wave cut platform and the potential difficulties that this could introduce, e.g., in installing cut-off systems.

The primary mitigation measure to reduce liquefaction failure risks is to ensure robust dewatering systems and carefully design soil improvement measures.

5.15.10.7 Construction in Rock

5.15.10.7.1 Subsurface Site Characteristics and Areas of Uncertainty

The preceding sections (supported by the appendices) describe the site rock profile and highlight the following:

- the groundwater regime and soil/rock profiles as described in ***Subsection 5.15.7***;
- the rock profile as described in ***Subsection 5.15.7.2***;
- rock joint characteristics, as described in ***Subsection 5.15.7.2.3***.

The dominant joint sets have been commented on in this section and the impact that these intersecting joints will have on excavations is noted in ***Subsection 5.15.7.2.4***. Other excavations, however, will be required in rock and similar stability concerns (as in tunnels) will be raised related to excavation stability.

It is pertinent to comment on issues that will have an impact on construction programming as programming risks may impact certain construction activities such as the time period in which dewatering systems will be required to function. Excavations in rock could result in extended construction programmes for the following reasons:

- The site rocks exhibit zones that are hard and potentially abrasive (greywacke and hornfels in particular – see ***Appendix 5.15.B***) and the toughness of these rocks will have an impact on production rates and therefore could impact on excavation programming.
- These rocks will be highly abrasive on all manner of equipment and will impact significantly on wear and tear of all equipment (e.g. tunnel boring machines; conventional plant with tracks and tyres, quarrying plant) – this may have a further impact on programming.

Inadequate planning related to this could stress dewatering designs should dewatering be required for time periods considerably longer than originally anticipated.

Table 5.15.32 interrogates the impact that construction in rock may have on the conceptual design of the nuclear installation(s).

Table T-5.15.32
Construction in Rocks and Geotechnical Data Acquisition Needs

Aspect	Potential Extent of Aspect (Geometry)	Geotechnical Characteristics Affecting the Conceptual Design	Reliability of Existing Geotechnical Data	Data Uncertainties	Design Precautions	Construction Difficulties	Operational Impacts	Environmental Impacts
Cut slope stability	Under damped (nuclear island) structures. Positioned practically anywhere on the site.	Groundwater table. Rock strength and deformability parameters; Rock joint characteristics defining block shapes, sizes and orientations and freedom of movement. Position/orientation of the excavation. Seismic response spectra.	Site data are reliable but lack site specific focus on a local scale (i.e. at footprint). Data does not appropriately document the lateral variability in rock characteristics as the site rocks are covered with overburden and not easily accessible.	Position and orientation of excavations. Water ingress into excavations from rock joints.	Dewatering in all likelihood will affect drawdown in the soils horizon only – rock joints are likely to remain saturated until excavations in rock are completed and ‘groundwater dissipates’ into excavations. Inflow rates into rock excavations are an unknown (flow rates and volumes) and management of groundwater may become an issue. Assisted rock slope drainage may be required to speed up dewatering of rock joints and factor up slope stability. Design and installation of rock wells, especially if artesian conditions in rock exist, may be necessary. Cuts in excess of 40° in mudstone/siltstone/shale and 45˚ in greywacke and hornfels will require lateral support.	Groundwater ingress management and associated environmental impacts (disposal of water). Rock spoil disposal.	None – rock excavations will be backfilled.	Materials (rock spoil and groundwater) disposal.

5.15.10.7.2 Conceptual Mitigation Measures

Rock excavations can be stabilised using conventional lateral support techniques such as rock bolts/anchors, concrete retaining walls or cutback systems (benching). Rock bolts at approximately 1 m spacing incorporating mesh and/or fibre reinforced shotcrete with rock bolts as lateral restraints will provide adequate restraint for most excavations. Regularly spaced rock anchors with soldier and whaler beams may be required in vertically sided temporary or permanent excavations.

The influence that the toughness and abrasiveness that the site rocks have on construction cost and programme must be factored into design and construction programming to mitigate risks to construction quality and hence safety.

The undulating nature of the wave-cut platform will require that shallow heads of rock will need to be removed in order to provide an even construction surface. Such shallow excavation may require extremely hard ripping in places or may require a great number of closely spaced short blast holes to be drilled and blasted. Either process will be costly and time consuming and must be factored into design and programming. Potential risks of blast induced vibrations on the KNPS will need to be investigated.

5.15.11 Management of Uncertainties

5.15.11.1 Uncertainties

This section summarises the uncertainties related to the site geotechnical profile as it is currently described and proposes methods in managing these uncertainties. These uncertainties are:

- the impact that the final positioning and layout of the nuclear island may have on safe operation of the KNPS and appropriate design of any future nuclear installation(s);
- lack of offshore geotechnical data pertaining to tunnelling;
- understanding of localised variability in founding under the nuclear island;
- understanding of the occurrence and significance (or importance) of secondary faults and geological discontinuities (that are smaller than the major faults, but still large enough to cause severe disruption if subject to differential settlement or strike slip);

- abrasiveness of site rock and the impact that this could have on costs and construction programming, in turn impacting on construction quality and hence safety;

- offshore geotechnical conditions, both shallow (for shallow marine structures such as intake basins) and deep (in the event that tunnelling is considered as an option for cooling water intake).

- It is noted that a newer version of NUREG 0800 ((United States Nuclear Regulatory Commission, 2007) exists. This will be updated in this section along with the outcomes of Sections 5.13 and 5.14 as the revisions to chapter 2 in NUREG 0800 cut across the geotechnical / seismic / geological (5.15/5.14/5.13) disciplines.

5.15.11.2 Approach to Management of Uncertainties

The approach to managing uncertainties hinges around reducing data uncertainty and integrating these new data with existing data to supplement the characterisation. The continual geotechnical characterisation of the site needs to reduce data uncertainty prior to the pre-operational stage and the most significant uncertainties related to data at the site are:

- lack of offshore geotechnical data should tunnelling be considered.

Closing these data gaps will require additional intrusive investigations under the following framework of activities:

- offshore drilling:
  - Depending on the extent offshore to which geotechnical data is required, boreholes will either be drilled from jack-up platforms and/or barges mobilised in the sea or directional drilling from land will be done to explore the target zones.
  - Geotechnical data mirroring that data presented in ***Appendix 5.15.B*** will be gathered in each borehole.

The above additional information, integrated with the PSHA will reduce data uncertainties and increase confidence.

5.15.11.3 Monitoring

Continuous monitoring of geotechnical parameters will improve the understanding of the geotechnical profile and assist in the approach to management of uncertainties. Groundwater level fluctuations have been monitored throughout the geotechnical characterisation stage of the project. It is recommended that this monitoring is maintained throughout the pre-operational and operational stages at which point appropriate expansion of monitored parameters may be considered to gauge dewatering effects, for example.

Additional monitoring (on a continuous monthly basis) that may be required in pre-operational and operational stages of the project may include monitoring of:

- structure settlement and or lateral movement utilising extensometers and inclinometers;
- lateral movement in structures (e.g. retaining structures or linear structures);
- the efficacy of dewatering systems and regional environmental impacts of dewatering through groundwater table monitoring;
- the integrity of soil improvement works particularly cement stabilised rafts engineered to remove liquefaction risks and that are in close proximity to the sea (and sulfate-rich environments during dewatering when inflow from the sea is possible) and may be saturated or periodically saturated;
- soil/rock slopes employing techniques such as inclinometers, piezometers and pore pressure/movement alert levels.

5.15.11.4 Implications of Uncertainties

Sufficient information is available to confirm the suitability of the site and the presence of the KNPS on the site lends many valuable lessons learnt. Further information will be gathered for the design and construction of any future nuclear installation(s) (as described in ***Subsection 5.15.11.2***) and certain parameters may be monitored on an on-going basis to confirm assumptions and confirm the adequacy of construction and operational programmes and controls.

5.15.12 Management System

The geotechnical investigations performed for this SSR entailed the following:

- desk study;
- site investigation;

- interpretation;
- data analysis and reporting;
- monitoring.

A quality assurance programme was established to control the effectiveness of the execution of these investigations, the data analysis and the formulation of conclusions on the site acceptability. This conforms to the overall management system for this SSR (***Chapter 10***, Management System), regulations (Department of Energy, 2010) and international guidelines (***Subsection 5.15.3***) and relevant Eskom classification procedures. The geotechnical evaluation of the site has been determined as Safety Class C, and in terms of the procedure, compliance with an ISO 9001 or equivalent system was implemented.

The activities carried out as part of the evaluation of the site and the results achieved are presented in detail in appendices to this section. These appendices provide the back-up for the data presented in the section. They present a clear and auditable trail showing how key decisions were made and conclusions reached. The information presented in the appendices includes:

- ***Appendix 5.15.B*** – Borehole Logs;
- ***Appendix 5.15.C*** – Borehole Core Photographs;
- ***Appendix 5.15.D*** – Rock Joint Logs;
- ***Appendix 5.15.E*** – SPT Test Results;
- ***Appendix 5.15.F*** – DPSH Test Results;
- ***Appendix 5.15.G*** – Soil Laboratory Test Results;
- ***Appendix 5.15.H***– Rock Laboratory Test Results;
- ***Appendix 5.15.I*** – Geophysical Investigation Results;
- ***Appendix 5.15.J*** – Quality data pack.

Prior to the start of the site investigation described herein, the following documents (***Appendix 5.15.J***) were compiled by the consultant and approved by Eskom to assist in quality assurance and ensure that site work was carried out safely:

- Project Quality Plan (subsequently changed to the Integrated Management System);
- Approved suppliers lists;
- Method Statement;
- Risk Assessment;
- Health, Safety and Environmental Management Plan.

The characterisation of geotechnical site parameters and their evaluation do not always lend themselves to direct verification by inspections or tests that can be precisely defined. Interaction with the peer reviewer occurred regularly in the 2008 field data gathering process and input was made on improving data gathering aspects of the site characterisation. This was carried out by a suitably qualified, independent and experienced professional.

The activities that have been carried out with their respective links to other SSR sections/chapters and quality control requirements are presented in ***Table 5.15.33***.

Table T-5.15.33
Summary of Activities, Links and Quality Requirements

Activity	Chapter Links		Quality Requirements
	Inputs	Outputs	
Intrusive Investigation Data Point Siting	***Sections 5.11***, ***5.13***; and ***5.14***. Integrative discussion with the Geohydrology, Geology and Seismic teams to optimise data gathering through intrusive investigations.	Optimal presentation of data to enhance the geotechnical profile that is influenced by geohydrological, geological and seismic characteristics.	Table showing rationale for number, position, depth, data point position changes and reasons for changes. Peer review of siting.
Drilling / Testing	***Sections 5.11***, ***5.13***, and ***5.14.*** Geological information was used as key input into the site geotechnical profile and the need for supplementing historical data with SSR intrusive investigations.	***Sections 5.11***, ***5.13*** and ***5.14***. Lithological logs and other geological information will be used as key input into the Geohydrology, Geology and Seismic Characterisation sections.	Risk assessment; Method Statement; Health, Safety and Environmental Management Plan; Peer review of data gathering approaches.
Laboratory Analysis	***Section 5.11***. Groundwater quality data was used as input to the Geotechnical Characterisation ***Section 5.15***	***Sections 5.15*** Concerns/uncertainties related to adverse/aggressive subsurface geochemical environments to be documented.	Use of approved suppliers; Robust laboratory testing programme; Certificate of accreditation for selected laboratories; Internationally recognised testing standards.
Development of the geotechnical profile	***Sections 5.11*** and ***5.13***. Integrated data from the Geohydrological and Geology sections was used to construct the 3D geotechnical profile.	***Sections 5.15*** An integrated geotechnical profile showing an accurate depiction of the groundwater conditions, geology and geotechnical characteristics of the site	Accurate land survey of data positions in WGS 84 (boreholes, test pits, probing positions).
Monitoring	***Section 5.11.*** and ***5.2*** Groundwater levels from the Geohydrology section were used to supplement data gathered in the geotechnical characterisation and vice versa.	***Sections 5.11***, ***5.15*** Data gathered in both technical disciplines define the groundwater table.	Monitoring protocol.

A regulatory compliance table (***Table T-5.15.34***) is given below to indicate where the relevant issues have been dealt with in the section.

Table T-5.15.34
Regulatory Compliance Matrix

Act/Regulation	Regulation	Issue	Section where covered
Regulations (Department of Energy, 2010)	3 (2) (a) and 5 (3)	Defining the submission of a site safety report and the site characterisation content thereof.	***5.15.10***
Regulations (Department of Energy, 2010)	4 (5)	Accounting for natural phenomena and potential man-made hazards.	***5.15.10***

5.15.13 Conclusions

The geotechnical information available for the site to date and presented in this SSR identifies a number of areas of concern and technical uncertainty. Provided mitigation is considered and the continual investigation of uncertainties pursued, sufficient geotechnical information is contained in the geotechnical profile to support Eskom's application for a Nuclear Installation Licence (NISL).

Some uncertainties are noted, and the implications of uncertainty discussed (***Subsection 5.15.11.4***). However, these do not impact on this SSR presenting sufficient evidence to make conclusions on the site suitability. The guidelines (***Subsection 5.15.4***) make provision in the geotechnical characterisation process to mitigate these uncertainties.

The key findings with regard to the technical suitability of the site are contained below. A description of the main geotechnical characteristics that could affect site suitability and safety of the KNPS and/or any future nuclear installation(s) are presented.

5.15.13.1 Geotechnical Profile

- soils:

- The site features a dune system overlying marine sand deposits (together averaging *c.*21 m thick across the investigated site), in turn overlying Malmesbury Group rocks consisting of greywacke and interbedded shale, mudstone and siltstone and metamorphosed equivalents with sporadic occurrences of other sedimentary rocks.
- The soils are dominated by cohesionless aeolian and marine deposited soils that are homogenously described as poorly graded with a very high sand size fraction, but some fines (<5 per cent) present.
- Soil consistency increases marginally with depth, but there are many occurrences within the boreholes drilled where consistency increases only gradually and medium dense (at best) consistency is reached at the base of the soil profile.
- In the presence of the Sandveld Aquifer (see below conclusions on groundwater), large portions of the site are classified as having a high liquefaction potential.
- Uncertainties:
    - Particular soil and groundwater conditions under any proposed nuclear installation(s) will require more detailed localised investigation to assess localised founding conditions and soil conditions that could give rise to excessive dust generation as this may impact on the safe operation of the KNPS ventilation systems during future construction activities.
    - No uncertainties were encountered that require any previous conclusions drawn on the KNPS to be revisited.

- groundwater:

    - Cumulatively, the aeolian and marine sand form the Sandveld Aquifer, an intergranular aquifer underlying the full extent of the site.
    - The groundwater table marks the top of the Sandveld Aquifer and is located *c.*5 m below ground level.
    - Groundwater flow within the Sandveld Aquifer is perpendicular to the coastline.
    - Groundwater in the Sandveld Aquifer is generally of a sodium chloride type but younger groundwater in the vicinity of the site tends towards calcium bicarbonate but magnesium sulfate and magnesium chloride character groundwater is also encountered.
    - The Sandveld Aquifer has a neutral to alkaline pH.

- Underlying the Sandveld Aquifer is the Malmesbury Aquifer which contains sodium chloride type groundwater and has acidic to neutral pH.
- Groundwater is not anticipated to present aggressiveness risks to concrete, but corrosion risk to steel is high but can be mitigated through controlling concrete permeability and with commercially available reinforcing coatings.
- Uncertainties:
    - Dewatering at any future nuclear installation site could impact on the safety of the KNPS foundations should drawdown of the groundwater table extend to the KNPS footprint.
    - Groundwater characterisation has been carried out across the whole site and no uncertainties related to data gaps.

- rocks:

    - The Malmesbury Group rocks are dominated by greywacke with interbedded shale, mudstones and siltstone and metamorphosed equivalents with bedding strike at between 320˚ and 330° and dipping at approximately 75° west-southwest.
    - Minor occurrences of hornfels, meta-greywacke and vuggy quartz occur in the sedimentary sequence.
    - The greywacke, hornfels and meta-greywackes are more competent than the shale, mudstone and siltstones which are more prone to weathering.
    - This varying weathering profile has shaped the bedrock into an undulating wave cut platform with the average bedrock elevation at -10.1m msl.
    - Excavatibility of rock and abrasiveness and wear on equipment was not specifically investigated in this study, but could be a critical consideration in construction.
    - Rock strength and deformability parameters present wide distributions in the site rocks with 1.2<UCS (MPa) <195, 0.2<E (GPa) <204, 0.02< ν <0.7, 1 900<Bulk Dens (kg/m^3) <2 770 and 1 840< Dry Dens (kg/m^3)<2 2920.
    - The dominance of three major joint sets impacts rock quality (best presented as RMR), but 42 per cent<RMR<86 per cent translates to rock quality described as ‘fair to good’. Intersection of these joint sets provides release mechanisms in any manner of excavations for sliding, toppling, face fall-out (ravelling) and rock fall failure.
    - Uncertainties:

- Once the positions of any future nuclear installation(s) are finalised, additional geotechnical investigations will be required through both intrusive (drilling) investigations and through detailed mapping of the exposed bedrock surface once foundation excavations are carried out as was done with the KNPS.
- The probable existence of a similar rock profile to that described in this SSR will be confirmed at this stage.
- Offshore geotechnical data are lacking and this is an uncertainty that will require mitigation should the future nuclear installation(s) require extensive offshore construction activities such as cooling water tunnel intakes and/or intake basins. Dedicated investigation programmes will be required to mitigate this data gap.

5.15.13.2 Presence of Specific Geotechnical Conditions

Information gathered to date does not indicate that any of the following are present on the site (this is, however to be confirmed under localised footprints in the pre-operational and operational geotechnical investigations):

- previous use of the site (e.g. mining activities);
- gas pockets, and swelling rocks;
- zones of weakness or discontinuities in crystalline rocks, which were not specifically targeted in investigations, but were also not encountered in any of the boreholes drilled;
- indicators of potential cavities and susceptibility to ground collapse in the context of:
  - sinks, sink ponds, caves and caverns;
  - sinking streams;
  - historical ground subsidence;
  - natural bridges;
  - surface depressions;
  - springs;

- rock types such as limestone, dolomite, gypsum, anhydrite, halite, terra rossa soils[29], lavas, weakly cemented clastic rocks, coal or ores;
- non-conformities in soluble rocks.

Shale was encountered in the geotechnical profile and the extent of this under localised footprints must be confirmed in the pre-operational and operational geotechnical investigations

- Uncertainties:

  - Once the positions of any future nuclear installation(s) are finalised, additional geotechnical investigations will be required through both intrusive (drilling) investigations and through detailed mapping of the exposed bedrock surface once foundation excavations are carried out as was done with the KNPS.
  - The probable lack of any of the abovementioned specific geotechnical conditions will be confirmed at this stage.

5.15.13.3 Founding Conditions

In the context of the geotechnical profile summarised above the following conclusions can be drawn on the founding conditions of the site. These conclusions are presented in the context of the potential impact on safety of the nuclear installation(s):

- Design will have to cater for variability in soil consistency and with the fact that soils are poorly consolidated to depth. This will not be unduly onerous and a combination of ground improvement measures, founding methods pertinent to imposed loads and data gathered to reduce uncertainties related to site-specific (i.e. localised under any proposed nuclear installation foundations) soil profiles, will provide sufficient confidence to design nuclear installation component foundations in the soils at the site. Site soils improvement by mechanical means will not yield bearing capacities > 200 kPa.

- The confirmed high liquefaction potential of the site soils, in the presence of the Sandveld Aquifer, is a critical design driver at this site and dewatering will be a feature of any future construction activities.

[29] A red soil produced by the weathering of limestone

- Design will have to cater for variability in rock characteristics. This will not be unduly onerous provided that detailed mapping of the nuclear installation footprint is carried out once bedrock is exposed and collection of Vs data is addressed.

- key design precautions are to:

  - factor the outcome of detailed mapping of the exposed bedrock at foundation excavation stage into designs;
  - factor the outcomes of future offshore geotechnical investigations into design of any marine structures such as cooling water tunnel intakes and/or basins;
  - factor the outcomes of the finalised PSHA into design and to gain an understanding of impedance characteristics of the site soils and weathered rock zones as well as to confirm the actual extent of liquefiable soils on the site;
  - employ ground improvement techniques responsibly noting that bearing capacity will not be improved to >200 kPa by mechanical means - Bearing capacity in excess of this can only be achieved with cement stabilisation, importation of good quality founding materials or carrying loads directly to bedrock.
  - avoid spanning or siting the footprint of any future nuclear installation(s) across extended weak zones (e.g. in the mudstone/siltstone/shale) and well developed geotechnical discontinuities (shears / faults) – not all of which are identified yet, but will be identified when foundation excavations expose the site bedrock;
  - align excavations in rock and tunnels as near to perpendicular to the bedding strike (at approximately 325°) wherever practically possible, and prepare for special stability designs should such orientations be within 30° of bedding strike.

5.15.13.4 Liquefaction Potential

The site is underlain by an intergranular aquifer and soils are poorly consolidated throughout the soil profile. At the PGA (0.4g, $M_{6.5}$ event), a high liquefaction potential exists in large areas of the site with the following conclusions drawn relating to this:

- Liquefaction potential is high across vast tracts of the site.

- Liquefaction under KNPS is not a risk.

- Mitigation of liquefaction risks has successfully been employed at the site in the construction of KNPS.
- Because of the occurrence of liquefiable soils, this will not have a marked impact on the siting of the nuclear installation footprint as liquefaction risks appear to be geographically widespread.
- Development of the KNPS site faced similar liquefaction potential challenges and it is reasonable to say that methods used in development of the KNPS will mitigate this risk elsewhere on this site.

5.15.13.5 Rock Joint Strength

- Three joint sets dominate at the site, and these all carry significance as intersection of these joint sets may result in any manner of sliding, toppling face fall-out or rock fall failure. Orientation of excavation crests (and tunnels) within 30° of the strike (330˚ for the dominant bedding) of these joint sets will exacerbate stability concerns.
- Tunnels in particular may pose a risk to the implementation of designs as insufficient information exists to responsibly plan their construction offshore. The extent to which variability in rock strength/hardness/abrasiveness will be encountered along the alignment will make it challenging to quantify/tender/execute designs. This is a particular concern in the offshore area where the geotechnical conditions have not been investigated.
- Should alternatives to tunnelling exist, these should be, along with tunnelling, subjected to feasibility studies.

5.15.13.6 Slope Stability and Earth Structures

Perhaps the single most significant geotechnical characteristic of the site is the average *c.*21 m thick unconsolidated overburden overlying the bedrock and the fact that an intergranular aquifer exists within this overburden. Irrespective of the positioning the nuclear installation(s) footprint, slope stability of foundation excavations will remain a challenge at the site. The soils will need to be battered back at angles < 18° and will need to be devoid of groundwater before any confidence in slope stability can be argued.

This introduces a number of conceptual design considerations and implications:

- The proximity of any future nuclear installation(s) to the KNPS is an important factor as dewatering, encroachment of the excavations and construction generated dust could all impact on the safe operation of the KNPS.

- The excavation footprint and disposal of produced spoil (stockpile footprints, impact on the receiving environment) will be a design driver.

- Dewatering design has to consider the undulating nature of the wave cut platform), the fact that excavations will be open for extended time periods (years) and that groundwater drawdown may have regional environmental impacts as will disposal of extracted groundwater.

- Dewatering systems will have to be robust enough to ensure constant dewatering of the site.

Constructed earth structures and improved ground rafts will, through mechanical compaction, not reach a bearing capacity >200 kPa. Bearing capacities required in excess of this can only be achieved with cement stabilisation and this will require composite design goals encompassing the need to reduce liquefaction risks. This approach was successfully undertaken in the development of the KNPS

5.15.13.7 Auxiliary Structures

Structures supporting the nuclear installation(s) are not likely to pose any onerous design challenges from a geotechnical design input viewpoint. Cognisance of differential founding conditions and differential dynamic loading regimes will, however, be a key consideration, as will the ceiling bearing capacity of *c.*200 kPa be for founding considerations.

5.15.13.8 Other Buried Structures

As with auxiliary structures, supporting buried structures are not likely to pose any onerous designs challenges from a geotechnical design input viewpoint – bar issues related to differential founding conditions and differential dynamic loading regimes.

5.15.14 Concluding Remarks on Site Suitability (Safety)

No information was forthcoming to challenge the previously motivated suitability of the KNPS site. On the contrary, monitored performance of the cement stabilised rafter under KNPS indicates very good performance that screens liquefaction risks out.

Sufficient information exists to argue that this whole site is suitable for construction of any future nuclear installation(s). Geotechnical conditions are similar to the north of the KNPS as were previously described for the development of the KNPS. Related design, construction and operational challenges will be in the same orders of magnitude elsewhere on the site as they were in the development of the KNPS.

Management of uncertainties is essential prior to considering any further development of this site. In particular, confirmation of offshore geotechnical data is required. Similarly, focussed/detailed data gathering beneath any proposed future nuclear installation development is required as was the strategy for development of the KNPS.

From a safety perspective, sufficient detail exists to suggest that development of nuclear installation(s) on this site will not present safety related challenges that cannot be mitigated by sound engineering. This SSR highlights those aspects of geotechnical design that may present safety risks and rational engineering mitigations, many of which are proven for this site as they were applied to the development of the KNPS.

5.15.16 References

1. American Society for Testing and Materials International, 2007. *Standard Test Method for Particle-Size Analysis of Soils. ASTM D422.* West Conshohocken: s.n.

2. American Society for Testing and Materials International, 2008. *Standard Test Method for Determination of the Point Load Strength Index of Rock and Application to Rock Strength Classifications, Active standard developed by sub-committee ASTM D5731 – 08.* West Conshohocken: s.n.

3. American Society for Testing and Materials International, 2018. *Standard Test Method For Specific Gravity And Absorption of Rock For Erosion Control.* West Conshohocken: s.n.

4. Barton, N. & Bandis, S., 1982. *Effects of Block Size on the Shear Behaviour of Jointed Rock. Keynote Lecture 23rd US Symposium on Rock Mechanics.* Berkeley, s.n.

5. Brink, A., 1981. *Engineering Geology of Southern Africa, Volume 2.* Johannesburg: s.n.

6. Brink, A., 1985. *Engineering Geology of Southern Africa, Volume 4.* Johannesburg: s.n.

7. British Standards Institution, 1990. *Methods of test for soils for civil engineering purposes. Classification tests. BS1377.* London: s.n.

8. Campanella, R. G. & Robertson, P. K., 1988. *Reference test procedure laid down in the proceedings of the First International Symposium on Penetration Testing (1SOPT).* Vanvouver: s.n.

9. Clayton, C. R. I., 1993. *The Standard Penetration Test (SPT): Methods and Use,* Surrey: CIRIA.

10. Committee of State Road Authorities, 1986. *Standard Methods of Testing Road Construction Materials. TMH1 A2, A3 and A4.* Pretoria: s.n.

11. Cubrinovski, M. & Ishihara, K., 2001. *Correlation between Penetration Resistance and Relative Density of Sandy Soils. 15th International Conference on Soil Mechanics and Geotechnical Engineering.* Istanbul: s.n.

12. Department of Energy, 2010. *R.927: The Regulations on Licensing of Sites for New Nuclear Intsallations.* Pretoria: Department of Energy.

13. Eskom, 2006. *Koeberg Nuclear Power Station Site Safety Report – Geology and Seismology. Rev 3.* Johannesburg: s.n.

14. Eskom, 2010. *Technical Specification for Site Safety Reports. NSIP01388 Rev 0.* Johannesburg: Eskom.

15. Eskom, 2017. *Long Term Monitoring of Soil Cement Sub-foundation,* Cape Town: Eskom.

16. Eskom, 2020. *Nuclear Engineering Life of Plant - Soil-Cement Sub Foundation (HMI),* Cape Town: Eskom.

17. Eskom, 2021. *Koeberg Seismic Re-evaluation Strategy, 240 - 160677773.* Cape Town: Eskom.

18. Hogegentogler, C. A. et al., 1937. *Engineering Properties of Soil.* 5 ed. New York: s.n.

19. Illenberger, 2010. *ENVIRONMENTAL IMPACT ASSESSMENT FOR THE PROPOSED NUCLEAR POWER STATION ('NUCLEAR 1') AND ASSOCIATED INFRASTRUCTURE: Dune Geomorphology Impact Assessment,* s.l.: s.n.

20. International Atomic Energy Agency, 2004. *Geotechnical Aspects of Site Evaluation and Foundations for Nuclear Power Plants. Safety Standard Series, Safety Guide No. NS G 3.6.* Vienna: IAEA.

21. International Society for Rock Mechanics, 1979. *Suggested Method for Determining Uniaxial Compressive Strength and Deformability of Rock materials.* Lisbon: s.n.

22. Laubscher, D. H., 1990. *A geomechanics classification system for the rating of rock mass in mine design. Journal of the South African Institute of Mining and Metallurgy, Vol. 90.* Johannesburg: South African Institute of Mining and Metallurgy.

23. National Centre for Earthquake Engineering Research, 2001. Update of the Simplified Procedure for Assessing Soil Liquefaction Potential. *JOURNAL OF GEOTECHNICAL AND GEOENVIRONMENTAL ENGINEERING.*

24. National Nuclear Regulator, 2006. *RD-0016: Requirements for*

Authorisation Submissions Involving Computer Software and Evaluation Models for Safety Calculations, Centurion: NNR.

25. National Nuclear Regulator, 2012. *PP-0014: Consideration for External Events for New Nuclear Installations,* Centurion: NNR.

26. National Nuclear Regulator, 2016. *Regulatory Guide RG-0011: Interim Guidance on the Siting of Nuclear Facilities.,* Centurion: NNR.

27. Stamatakos, J. & Watson-Lamprey, J., 2024. *Duynefontyn SSHAC EL-2 PSHA,* s.l.: Eskom.

28. South African National Standard, 2014b. *SANS 3001-AG2: Civil engineering test methods: Determination of the average least dimension of aggregates by direct measurement.* 1 ed. Pretoria: SABS Standards Division.

29. South African National Standard, 2014c. *SANS 3001-AG20: Civil engineering test methods: Determination of the bulk density, apparent density and water absorption of aggregate particles retained on the 5 mm sieve for road construction materials.* 1.1 ed. Pretoria: SABS Standards Division.

30. South African National Standard, 2014d. *SANS 3001-AG21: Civil engineering test methods: Determination of the bulk density, apparent density and water absorption of aggregate particles passing the 5 mm sieve for road construction materials.* 1.1 ed. Pretoria: SABS Standards Division.

31. South African National Standard, 2014. *SANS 3001-AG1: Civil engineering test methods: Particle size analysis of aggregates.* 1.2 ed. Pretoria: SABS Standards Division.

32. South African National Standard, 2015. *ANS 3001-AG4: Civil engineering test methods: Determination of the flakiness index of coarse aggregate.* 1.2 ed. Pretoria: SABS Standards Division.

33. South African National Standard, 2015b. *SANS 3001-AG5: Civil engineering test methods: Sand equivalent value of fine aggregates.* 1.1 ed. Pretoria: SABS Standards Division.

34. South African National Standard, 2019. *SANS 3001-AG10: Civil engineering test methods: ACV (aggregate crushing value) and 10 % FACT (fines aggregate crushing test) values of coarse aggregates.* 1 ed. Pretoria: SABS Standards Division.

35. United States Nuclear Regulatory Commission, 2003. *Laboratory Investigations of Soils and Rocks for Engineering Analysis and Design of Nuclear Power Plants. Regulatory Guide 1.138. Rev 2.* Washington DC: s.n.

36. United States Nuclear Regulatory Commission, 2003. *Site Investigations for Nuclear Power Plants. Regulatory Guide 1.132. Rev 2.* Washington DC: s.n.

37. United States Nuclear Regulatory Commission, 2007. *NUREG-0800, Section 2.4.12 Groundwater. Rev 3.,* Washington D.C.: United States Nuclear Regulatory Commission.

APPENDICES

Appendix A-5.15.A
Aspects of Work Requiring Geotechnical Design (International Atomic Energy Agency, 2004)

Aspect	Mechanism of Failure/Areas of Concern	Capacity and Demand Parameters					Design Considerations	(Conceptual) Assessment of the Site Capacity and Impacts on Site Safety
		Required	Available Parameters		Outstanding Parameters			
			Parameter	Where Found in SSR	Parameter	Programme for obtaining		
SLOPE STABILITY								
soil slope / embankment stability (excavated and constructed slopes)	erosion failure	soil indicator parameters	grading and Atterberg limits	***Appendices A-5.15.B***, ***A-5.15.C*** and ***A-5.15.G*** ***Subsection 5.15.7***	• plant layout, geometry, founding levels and excavation levels.	Prior to the pre-operational stage.	• integration with slip circle failure slope design (below) • critical soil slope angle derived from design rainfall and material properties (assessment of stream power of sheet flow) • slope protection measures and longevity of slope protection measures • surface water management requirements	Homogenous site soils will allow the design engineer to overcome problems and design is not anticipated to be unduly onerous, except when designing dewatering systems where achieving longevity in these systems may present challenges of both a technical and environmental impact nature. **The site is considered licensable as standard civil engineering design can mitigate erosion failure risks to safety.**
		geotechnical profile	descriptive soil and rock profile	***Appendices 5.15.B*** to ***5.15.I*** ***Subsections 5.15.7***				
		groundwater regime	groundwater table profile	***Section 5.11*** ***Subsections 5.15.7.3***				
		hydrology and hydraulics parameters (design rainfall depths, etc.)	specific meteorological, hydrological and hydraulic assessments for the site	***Sections 5.8*** and ***5.10***				
	classic slip circle slope failure	soil indicator parameters	grading and Atterberg limits	***Appendices A-5.15.B***, ***A-5.15.C*** and ***A-5.15.G*** ***Subsection 5.15.7***	• plant layout, geometry, founding levels and excavation levels.	Prior to the pre-operational stage.	• integration with erosion critical slope angle assessment (above) • conventional slope stability design assessing driving and resisting loads; • dewatering requirements and maintenance of dewatering systems • surface water management requirements	Homogenous site soils will allow the design engineer to overcome problems and design is not anticipated to be unduly onerous, except when designing dewatering systems where achieving longevity in these systems may present challenges of both a technical and environmental impact nature. Robust dewatering design will be a primary mitigation. **The site is considered licensable as standard civil engineering design can mitigate slip circle slope failure risks to safety. The latter statement does not pertain specifically to slopes constructed close to the KNPS.**
		soil shear strength	friction angle and cohesion	***Subsection 5.15.7.4.1***				
		groundwater regime	groundwater table profile	***Section 5.11*** ***Subsection 5.15.7.3***				
		geotechnical profile	descriptive soil and rock profile	***Appendices 5.15.B*** to ***5.15.I*** ***Subsections 5.15.7***				
		surface water management		***Sections 5.8*** and ***5.10***				
	liquefaction potential of soils impacting slope stability	SPT test results and Vs profile of the site soils	SPT N values	***Appendices 5.15.B*** and ***A-5.15.E*** ***Subsection 5.15.6.6***	• final plant layout, geometry, founding levels, excavation levels and confirmed imposed loads. • detailed laboratory testing of undisturbed soil samples at the chosen nuclear installation footprint.	Prior to the pre-operational stage.	• integration with erosion critical slope angle and slip circle failure slope design assessment (above) • dewatering requirements and maintenance of dewatering systems • ground improvement measures • foundation design in safety-related structures founded on soils • location of the nuclear installation footprint	Liquefaction potential is high and will be finally confirmed when the nuclear installation footprint is fixed. With these data available, the design engineer can quantify design against liquefaction failure. The design of dewatering systems may present challenges of both a technical and environmental impact nature. There is no evidence to suggest that liquefaction risks cannot be mitigated as was done in the development of KNPS. **The site is licensable as generic designs (such as the KNPS) are able to cater for response spectra. The site is therefore licensable on condition that the seismic response spectra (defined by the PSHA) are considered in final designs.**
		groundwater regime	groundwater table profile	***Section 5.11*** ***Subsections 5.15.7.3***				
		seismic response spectra	PGA and earthquake magnitude	***Section 5.14*** and (Eskom, 2021)				

Aspect	Mechanism of Failure/Areas of Concern	Capacity and Demand Parameters					Design Considerations	(Conceptual) Assessment of the Site Capacity and Impacts on Site Safety
		Required	Available Parameters		Outstanding Parameters			
			Parameter	Where Found in SSR	Parameter	Programme for obtaining		
rock slope / embankment stability	toppling failure	rock indicator parameters	Core recovery, RQD, fracture frequency, UCS, Young's modulus and Poissons ratio, joint conditions	***Appendices 5.15.B*** to ***5.15.C*** and ***5.15.H*** ***Subsection 5.15.7.5.2***	• plant layout, geometry, founding levels, excavation levels and confirmed imposed loads	Prior to the pre-operational stage.	• imposed loads • excavated rock slope angle • dewatering requirements and maintenance of dewatering systems • conventional lateral support requirements • calculated joint shear strength parameters (subjective assessment) and rock mass behaviour	The design engineer will need to orientate structures carefully to maximise the inherent strength in the site rocks and not introduce inherent failure mechanisms related to intersecting rock joints. Conventional lateral support systems will be required - **the site is licensable as conventional design[30] can mitigate design challenges and ensure site safety.**
		assessment of joint conditions	Joint orientation / surface conditions / frequency	***Appendices 5.15.B*** to ***5.15.D*** ***Subsection 5.15.7.2.3***				
		geotechnical profile	descriptive soil and rock profile	***Appendices 5.15.B*** to ***5.15.I*** ***Subsections 5.15.7***				
		groundwater regime	groundwater table profile	***Section 5.11*** ***Subsections 5.15.7.3***				
	wedge failure	rock indicator parameters	Core recovery, RQD, fracture frequency, UCS, Young's modulus and Poissons ratio	***Appendices 5.15.B*** to ***5.15.D*** ***Subsection 5.15.7.2.3***	• plant layout, geometry, founding levels, excavation levels and confirmed imposed loads	Prior to the pre-operational stage	• imposed loads • excavated rock slope angle • dewatering requirements and maintenance of dewatering systems • conventional lateral support requirements • calculated joint shear strength parameters (subjective assessment) and rock mass behaviour	The design engineer will need to orientate structures carefully to maximise the inherent strength in the site rocks and not introduce inherent failure mechanisms related to intersecting rock joints. Conventional lateral support systems will be required - **the site is licensable as conventional design can mitigate design challenges and ensure site safety.**
		assessment of joint conditions	Joint orientation / surface conditions / frequency	***Appendices 5.15.B*** to ***5.15.D*** ***Subsection 5.15.7.2.3***				
		geotechnical profile	descriptive soil and rock profile	***Appendices 5.15.B*** to ***5.15.I*** ***Subsections 5.15.7***				
		groundwater regime	groundwater table profile	***Section 5.11*** ***Subsections 5.15.7.3***				
	slip circle failure in soft rock	shear strength of weathered rock		***Appendices 5.15.B*** to ***5.15.J***	• plant layout, geometry, founding levels, excavation levels and confirmed imposed loads • friction angle and cohesion of weathered rock in exposures.	Prior to the pre-operational stage	• imposed loads from overburden soils • excavated rock slope angle • dewatering requirements and maintenance of dewatering systems • conventional lateral support requirements • calculated joint shear strength parameters (subjective assessment) and rock mass behaviour	Conventional lateral support systems will be required, however design needs to take cognisance of shear strength parameters of weathered rock material. **Provided all design parameters are gathered, the site is licensable as conventional design can mitigate design challenges and ensure site safety.**
		geotechnical profile	descriptive soil and rock profile	***Appendices 5.15.B*** to ***5.15.I*** ***Subsections 5.15.7***				
		groundwater regime	groundwater table profile	***Section 5.11*** ***Subsection 5.15.7.3***				
FOUNDATIONS								
		soil indicator parameters	grading and Atterberg limits	***Appendices A-5.15.B***, ***A-5.15.C*** and ***A-5.15.G***		Prior to the pre-operational stage	• integration with other foundations	

[30] Conventional design refers to standard design practices that have a well-documented history of success.

Aspect	Mechanism of Failure/Areas of Concern	Capacity and Demand Parameters					Design Considerations	(Conceptual) Assessment of the Site Capacity and Impacts on Site Safety
		Required	Available Parameters		Outstanding Parameters			
			Parameter	Where Found in SSR	Parameter	Programme for obtaining		
small spread and strip foundations in soils	bearing failure, sliding failure, liquefaction failure, settlement and differential settlement			***Subsection 5.15.7***	• plant layout, geometry, founding levels, excavation levels and confirmed imposed loads;		• dewatering requirements and maintenance of dewatering systems • ground improvement measures • construction sequencing • seismic response • settlement tolerance • position of footings relevant to nuclear island • trigger mechanisms for liquefaction failure • site soil capacity with respect to trigger mechanisms	Liquefaction potential is high and will be finally confirmed when the nuclear installation footprint is fixed. With these data available, the design engineer can quantify design against liquefaction failure. There is no evidence to suggest that liquefaction risks cannot be mitigated as was done in the development of KNPS. Founding in medium dense soils will not be possible on the site as differential settlement risks will be high in this material. **Generic designs (such as was done for the KNPS) were able to cater for current response spectra. The site is therefore licensable on condition that the seismic response spectra (as defined in the PSHA) are considered in final designs.**
		soil shear strength parameters	friction angle and cohesion	***Subsection 5.15.7.5.1***				
		geotechnical profile	descriptive soil and rock profile	***Appendices 5.15.B*** to ***5.15.I*** ***Subsections 5.15.7***				
		SPT test results for liquefaction assessment	SPT N values	***Appendices 5.15.B*** and ***A-5.15.E*** ***Subsection 5.15.6.6***				
		groundwater regime	groundwater table profile	***Section 5.11*** ***Subsections 5.15.7.3***				
adjoining spread footings to monolithic structures	differential settlement	soil indicator parameters	grading and Atterberg limits	***Appendices A-5.15.B***, ***A-5.15.C*** and ***A-5.15.G*** ***Subsection 5.15.7***	• plant layout, geometry, founding levels, excavation levels and imposed loads; • Vs and PGA in different geological formations • seismic response spectra • detailed laboratory testing of undisturbed soil samples at the chosen nuclear installation footprint.	Prior to the pre-operational stage	• integration with other foundations • dewatering requirements and maintenance of dewatering systems • ground improvement measures • construction sequencing • differential movement between foundations founded in rock / foundations founded in soil and damped/undamped foundations • settlement tolerance • position of footings relevant to monolithic structures • trigger mechanisms for liquefaction failure • site soil capacity with respect to trigger mechanisms	Liquefaction potential is high and will be finally confirmed when the nuclear installation footprint is fixed. With these data available, the design engineer can quantify design against liquefaction failure. There is no evidence to suggest that liquefaction risks cannot be mitigated as was done in the development of KNPS. Founding in medium dense soils will not be possible on the site as differential settlement risks will be high in this material. **Generic designs (such as was done for the KNPS) were able to cater for current response spectra (as defined in the KNPS PSHA). The site is therefore licensable on condition that the seismic response spectra (as defined in the PSHA) are considered in final designs.**
		soil shear strength parameters	friction angle and cohesion	***Subsection 5.15.7.5.1***				
		geotechnical profile	descriptive soil and rock profile	***Appendices 5.15.B*** to ***5.15.I*** ***Subsections 5.15.7***				
		groundwater regime	groundwater table profile	***Section 5.11*** ***Subsections 5.15.7.3***				
		SPT test results for liquefaction assessment	SPT N values	***Appendices 5.15.B*** and ***A-5.15.E*** ***Subsection 5.15.6.6***				
	differential dynamic loading	Vs and PGA in different geological formations (rocks and soils)	PGA and earthquake magnitude	***Section 5.14***				
towers and stacks	bearing failure, sliding failure, settlement and differential settlement	soil indicator parameters	grading and Atterberg limits	***Appendices A-5.15.B***, ***A-5.15.C*** and ***A-5.15.G*** ***Subsection 5.15.7***	• plant layout, geometry, founding levels, excavation levels and imposed loads; • detailed laboratory testing of undisturbed soil samples at the chosen site.	Prior to the pre-operational stage	• integration with other foundations • dewatering requirements and maintenance of dewatering systems • ground improvement measures • construction sequencing • seismic response • settlement tolerance • trigger mechanisms for liquefaction failure	Liquefaction potential is high and will be finally confirmed when the nuclear installation footprint is fixed. With these data available, the design engineer can quantify design against liquefaction failure. There is no evidence to suggest that liquefaction risks cannot be mitigated as was done in the development of KNPS.
		soil shear strength parameters	friction angle and cohesion	***Subsection 5.15.7.5.1***				
		geotechnical profile	descriptive soil and rock profile	***Appendices 5.15.B*** to ***5.15.I*** ***Subsections 5.15.7***				
		groundwater regime	groundwater table profile	***Section 5.11*** ***Subsections 5.15.7.3***				

Aspect	Mechanism of Failure/Areas of Concern	Capacity and Demand Parameters					Design Considerations	(Conceptual) Assessment of the Site Capacity and Impacts on Site Safety
		Required	Available Parameters		Outstanding Parameters			
			Parameter	Where Found in SSR	Parameter	Programme for obtaining		
		SPT test results for liquefaction assessment	SPT N values	***Appendices 5.15.B*** and ***A-5.15.E*** ***Subsection 5.15.6.6***			• site soil capacity with respect to trigger mechanisms	**Generic designs (such as was done for the KNPS) were able to cater for current response spectra (as defined in the KNPS PSHA). The site is therefore licensable on condition that the seismic response spectra (as defined in the PSHA) are considered in final designs**
		rock indicator parameters	Core recovery, RQD, fracture frequency, UCS, Young's modulus and Poissons ratio	***Appendices 5.15.B*** to ***5.15.C*** and ***5.15.H*** ***Subsection 5.15.7.5.2***				
heavy vibratory machinery and settlement sensitive structures	bearing failure, sliding failure, settlement and differential settlement	soil indicator parameters	grading and Atterberg limits	***Appendices A-5.15.B***, ***A-5.15.C*** and ***A-5.15.G*** ***Subsection 5.15.7***	• plant layout, geometry, founding levels, excavation levels and imposed loads; • detailed laboratory testing of undisturbed soil samples at the chosen nuclear installation footprint.	Prior to the pre-operational stage	• integration with other foundations • dewatering requirements and maintenance of dewatering systems • ground improvement measures • construction sequencing • seismic response • settlement tolerance • trigger mechanisms for liquefaction failure • site soil capacity with respect to trigger mechanisms	Liquefaction potential is high and will be finally confirmed when the nuclear installation footprint is fixed. With these data available, the design engineer can quantify design against liquefaction failure. There is no evidence to suggest that liquefaction risks cannot be mitigated as was done in the development of KNPS. **Generic designs (such as was done for the KNPS) were able to cater for current response spectra (as defined in the KNPS PSHA). The site is therefore licensable on condition that the seismic response spectra (as defined in the PSHA) are considered in final designs**
		soil shear strength parameters	friction angle and cohesion	***Subsection 5.15.7.4.1***				
		geotechnical profile	descriptive soil and rock profile	***Appendices 5.15.B*** to ***5.15.I*** ***Subsections 5.15.7***				
		SPT test results for liquefaction assessment	SPT N values	***Appendices 5.15.B*** and ***A-5.15.E*** ***Subsection 5.15.6.6***				
		groundwater regime	groundwater table profile	***Section 5.11*** ***Subsections 5.15.7.3***				
		rock indicator parameters	Core recovery, RQD, fracture frequency, UCS, Young's modulus and Poissons ratio	***Appendices 5.15.B*** to ***5.15.C*** and ***A-5.15.H*** ***Subsection 5.15.6.4***				
damped foundations (the nuclear island)	bearing failure, sliding failure, settlement and differential settlement	rock indicator parameters	Core recovery, RQD, fracture frequency, UCS, Young's modulus and Poissons ratio	***Appendices 5.15.B*** to ***5.15.C*** and ***5.15.H*** ***Subsection 5.15.7.5.2***	• plant layout, geometry, founding levels, excavation levels and imposed loads; • secondary faults and geological discontinuities.	Prior to the pre-operational stage	• integration with other foundations • dewatering requirements and maintenance of dewatering systems • ground improvement measures • construction sequencing • seismic response • settlement tolerance	With confirmed seismic response spectra included in final design, the design engineer can overcome problems and design, although challenging is not anticipated to be impossible. Designs straddling geological formations and/or shear/fault zones will carry high levels of uncertainty and will impose unacceptable risks on foundation failure. **The site is licensable on condition that the updated seismic response spectra (as defined in the PSHA) are considered in final designs to ensure site safety and that foundations are positioned such that lithological boundaries and/or shear/fault zones are not spanned. Detailed mapping of the foundation excavation will be required at this stage as was done for the KNPS.**
		assessment of joint conditions	Joint orientation / surface conditions / frequency	***Appendices 5.15.B*** to ***5.15.D*** ***Subsection 5.15.7.2.3***				
		geotechnical profile	descriptive soil and rock profile	***Appendices 5.15.B*** to ***5.15.I*** ***Subsections 5.15.7***				
	differential dynamic loading and implications of uncertainty to all manner of failure mechanisms	Differential Vs and PGA	PGA and earthquake magnitude	***Section 5.14***				

Aspect	Mechanism of Failure/Areas of Concern	Capacity and Demand Parameters					Design Considerations	(Conceptual) Assessment of the Site Capacity and Impacts on Site Safety
		Required	Available Parameters		Outstanding Parameters			
			Parameter	Where Found in SSR	Parameter	Programme for obtaining		
	seismic wave distortion	seismic response spectra	PGA and earthquake magnitude	***Section 5.14***				
	tension/shear failure over lithological boundaries, shear zones and faults	geotechnical profile	descriptive soil and rock profile	***Appendices 5.15.B*** to ***5.15.I*** ***Subsections 5.15.7***				
mass foundations	bearing failure, sliding failure, settlement and differential settlement	rock indicator parameters	Core recovery, RQD, fracture frequency, UCS, Young's modulus and Poissons ratio	***Appendices 5.15.B*** to ***5.15.C*** and ***5.15.H*** ***Subsection 5.15.7.5.2***	• plant layout, geometry, founding levels, excavation levels and imposed loads; • secondary faults and geological discontinuities.	Prior to the pre-operational stage	• integration with other foundations • dewatering requirements and maintenance of dewatering systems • ground improvement measures • construction sequencing • seismic response • settlement tolerance	With confirmed seismic response spectra included in final design, the design engineer can overcome problems and design, although challenging is not anticipated to be impossible. Designs straddling geological formations and/or shear/fault zones will carry high levels of uncertainty and will impose unacceptable risks on foundation failure. **The site is licensable on condition that the updated seismic response spectra (as defined in the PSHA) are considered in final designs to ensure site safety and that foundations are positioned such that lithological boundaries and/or shear/fault zones are not spanned. Detailed mapping of the foundation excavation will be required at this stage as was done for the KNPS.**
		assessment of joint conditions	Joint orientation / surface conditions / frequency	***Appendices 5.15.B*** to ***5.15.D*** ***Subsection 5.15.7.2.3***				
		geotechnical profile	descriptive soil and rock profile	***Appendices 5.15.B*** to ***5.15.I*** ***Subsections 5.15.7***				
	differential dynamic loading and implications of uncertainty to all manner of failure mechanisms	Differential Vs and PGA	PGA and earthquake magnitude	***Section 5.14***				
	seismic wave distortion	seismic response spectra	PGA and earthquake magnitude	***Section 5.14***				
	tension/shear failure over lithological boundaries, shear zones and faults	geotechnical profile	descriptive soil and rock profile	***Appendices 5.15.B*** to ***5.15.I*** ***Subsections 5.15.7***				
COOLING WATER INTAKE/OUTLET STRUCTURES								
Tunnels, canals and shafts	traversing lithological boundaries, soft/shear/fault zones	geotechnical profile	descriptive soil and rock profile	***Appendices 5.15.B*** to ***5.15.I*** ***Subsections 5.15.7***	• plant layout, geometry, founding levels, excavation levels and imposed loads; • secondary faults and geological discontinuities. • Confirmed geotechnical profile in the offshore environment.	Prior to the pre-operational stage	• integration with other construction elements • dewatering requirements and maintenance of dewatering systems • ground improvement measures • construction sequencing • seismic response	With finalised seismic response spectra included in final design, the design engineer can overcome problems and design, although challenging is not anticipated to be impossible. Designs straddling geological formations and/or shear/fault zones will carry high levels of uncertainty and will impose unacceptable risks on foundation failure.
	water inflow during construction	assessment of joint conditions	Joint orientation / surface conditions / frequency	***Appendices 5.15.B*** to ***5.15.D*** ***Subsection 5.15.7.2.3***				

Aspect	Mechanism of Failure/Areas of Concern	Capacity and Demand Parameters					Design Considerations	(Conceptual) Assessment of the Site Capacity and Impacts on Site Safety
		Required	Available Parameters		Outstanding Parameters			
			Parameter	Where Found in SSR	Parameter	Programme for obtaining		
		groundwater regime	groundwater table profile	***Section 5.11*** ***Subsections 5.15.7.3***			• variability of dynamic response over lithological boundaries • calculated joint shear strength parameters (subjective assessment) and rock mass behaviour • settlement tolerance	**The site is licensable on condition that the updated seismic response spectra (as defined in the PSHA) and updated offshore geotechnical profiles are considered in final designs to ensure site safety and that foundations are positioned such that lithological boundaries and/or shear/fault zones are not spanned.**
	differential dynamic loading and implications of uncertainty to all manner of failure mechanisms including seismic wave distortion and tension/shear failure over lithological boundaries, shear zones and faults	Differential Vs and PGA seismic response spectra	PGA and earthquake magnitude	***Section 5.14***				
		geotechnical profile	descriptive soil and rock profile	***Appendices 5.15.B*** to ***5.15.I*** ***Subsections 5.15.7***				
	block failure into tunnel excavations	rock indicator parameters	Core recovery, RQD, fracture frequency, UCS, Young's modulus and Poissons ratio	***Appendices 5.15.B*** to ***5.15.C*** and ***5.15.H*** ***Subsection 5.15.7.5.2***				
		assessment of joint conditions	Joint orientation / surface conditions / frequency	***Appendices 5.15.B*** to ***5.15.D*** ***Subsection 5.15.7.2.3***				
	Squeezing failure related to rock mass controlled behaviour as opposed to structure controlled	Differential Vs and PGA seismic response spectra	PGA and earthquake magnitude	***Section 5.14***				
		geotechnical profile	descriptive soil and rock profile	***Appendices 5.15.B*** to ***5.15.I*** ***Subsections 5.15.7***				
		assessment of joint conditions	Joint orientation / surface conditions / frequency	***Appendices 5.15.B*** to ***5.15.D*** ***Subsection 5.15.7.2.3***				
		rock indicator parameters	Core recovery, RQD, fracture frequency, UCS, Young's modulus and Poissons ratio	***Appendices 5.15.B*** to ***5.15.C*** and ***5.15.H*** ***Subsection 5.15.7.5.2***				
	dislocation at bends	Differential Vs and PGA seismic response spectra	PGA and earthquake magnitude	***Section 5.14***				

Aspect	Mechanism of Failure/Areas of Concern	Capacity and Demand Parameters					Design Considerations	(Conceptual) Assessment of the Site Capacity and Impacts on Site Safety
		Required	Available Parameters		Outstanding Parameters			
			Parameter	Where Found in SSR	Parameter	Programme for obtaining		
interfaces between tunnels and intake/ nuclear installation structures	differential settlement	rock indicator parameters	Core recovery, RQD, fracture frequency, UCS, Young's modulus and Poissons ratio	***Appendices 5.15.B*** to ***5.15.C*** and ***5.15.H*** ***Subsection 5.15.7.5.2***	• plant layout, geometry, founding levels, excavation levels and imposed loads; • secondary faults and geological discontinuities.	Prior to the pre-operational stage	• integration with all structures • dewatering requirements and maintenance of dewatering systems • ground improvement measures • construction sequencing • seismic response • settlement tolerance	With finalised seismic response spectra included in final design, the design engineer can overcome problems and design, although challenging is not anticipated to be impossible. Designs straddling weak rock zones and/or shear/fault zones may carry high levels of uncertainty and may impose unacceptable risks on foundation failure, but this can only be determined at a later date. **The site is licensable on condition that the confirmed seismic response spectra (as defined in the PSHA) are considered in final designs to ensure site safety and that foundations are positioned such that weak rock zones and/or shear/fault zones are not spanned or suitably negotiated.**
		assessment of joint conditions	Joint orientation / surface conditions / frequency	***Appendices 5.15.B*** to ***5.15.D*** ***Subsection 5.15.7.2.3***				
		geotechnical profile	descriptive soil and rock profile	***Appendices 5.15.B*** to ***5.15.I*** ***Subsections 5.15.7***				
		soil indicator parameters	grading and Atterberg limits	***Appendices A-5.15.B***, ***A-5.15.C*** and ***A-5.15.G*** ***Subsection 5.15.7***				
	differential dynamic loading and implications of uncertainty to all manner of failure mechanisms including seismic wave distortion and tension/shear failure over lithological boundaries, shear zones and faults	Differential Vs and PGA seismic response spectra	PGA and earthquake magnitude	***Section 5.14***				
		geotechnical profile	descriptive soil and rock profile	***Appendices 5.15.B*** to ***5.15.I*** ***Subsections 5.15.7***				
MISCELLANEOUS STRUCTURES								
pipelines	failure of surface pipelines in trenches, large diameter pipelines at depth	rock indicator parameters	Core recovery, RQD, fracture frequency, UCS, Young's modulus and Poissons ratio	***Appendices 5.15.B*** to ***5.15.C*** and ***5.15.H*** ***Subsection 5.15.7.5.2***	• plant layout, geometry, founding levels, excavation levels and imposed loads; • detailed laboratory testing of undisturbed soil samples at the chosen nuclear installation footprint • secondary faults and geological discontinuities.	Prior to the pre-operational stage	• integration with between structural elements • dewatering requirements and maintenance of dewatering systems • ground improvement measures • construction sequencing • seismic response • settlement tolerance • trigger mechanisms for liquefaction failure • site soil capacity with respect to trigger mechanisms	Liquefaction potential is high and will be finally confirmed when the nuclear installation footprint is fixed. With these data available, the design engineer can quantify design against liquefaction failure. There is no evidence to suggest that liquefaction risks cannot be mitigated as was done in the development of KNPS. Conventional pipeline founding designs will present little challenge should liquefaction potential not be considered a risk or be quantified in the specific area and if structures are founded to avoid differential dynamic loading. **Generic designs (such as was done for the KNPS) were able to cater for current response spectra (i.e. response spectra relevant to the KNPS PSHA). The site is therefore licensable on condition that the updated seismic response spectra (as defined in the PSHA) are considered in final designs.**
		soil indicator parameters	grading and Atterberg limits	***Appendices A-5.15.B***, ***A-5.15.C*** and ***A-5.15.G*** ***Subsection 5.15.7***				
		geotechnical profile	descriptive soil and rock profile	***Appendices 5.15.B*** to ***5.15.I*** ***Subsections 5.15.7***				
		groundwater regime	groundwater table profile	***Section 5.11*** ***Subsections 5.15.7.3***				
		SPT test results for liquefaction assessment	SPT N values	***Appendices 5.15.B*** and ***A-5.15.E*** ***Subsection 5.15.6.6***				

Aspect	Mechanism of Failure/Areas of Concern	Capacity and Demand Parameters					Design Considerations	(Conceptual) Assessment of the Site Capacity and Impacts on Site Safety
		Required	Available Parameters		Outstanding Parameters			
			Parameter	Where Found in SSR	Parameter	Programme for obtaining		
breakwaters	construction material production failure (from quarries) related to blasting production	rock indicator parameters	Core recovery, RQD, fracture frequency, UCS, Young's modulus and Poissons ratio	***Appendices 5.15.B*** to ***5.15.C*** and ***5.15.H*** ***Subsection 5.15.7.5.2***	• construction programme, plant layout geometry • quarry feasibility studies.	Prior to the pre-operational stage	• rock protection works materials size envelope • production plan • blasting design • loading, hauling and placing design	Provided adequate information pertaining to dedicated production quarry feasibility is gained, the design engineers can overcome problems and design will not be unduly onerous. **The site is licensable on condition that the appropriate quarry feasibility investigations are undertaken.**
		assessment of joint conditions	Joint orientation / surface conditions / frequency	***Appendices 5.15.B*** to ***5.15.D*** ***Subsection 5.15.7.2.3***				
	erosion failure	marine hydrodynamic environment	peak wave loads	***Section 5.9***				
floor slabs	Settlement and differential settlement	soil indicator parameters	grading and Atterberg limits	***Appendices A-5.15.B***, ***A-5.15.C*** and ***A-5.15.G*** ***Subsection 5.15.7***	• layouts, localised founding levels and imposed loads.	Prior to the pre-operational stage	• soil improvement • settlement tolerance	**Conventional design will mitigate design challenges and safety impacts are low – the site is therefore licensable.**
reservoirs, tanks and dams	Settlement and differential settlement	soil indicator parameters	grading and Atterberg limits	***Appendices A-5.15.B***, ***A-5.15.C*** and ***A-5.15.G*** ***Subsection 5.15.7***	• layouts, geometry, founding levels and imposed loads.	Prior to the pre-operational stage	• Loads • Size • Strength • settlement tolerance	**Conventional design will mitigate design challenges and safety impacts are low – the site is therefore licensable.**
		geotechnical profile	descriptive soil and rock profile	***Appendices 5.15.B*** to ***5.15.I*** ***Subsections 5.15.7***				
	slope stability of earth structures	(see slope stability above)						
CONSTRUCTION DESIGN								
excavation of the construction terrace	extent of soil excavation, disposal of spoil (excavated soil) and dewatering causing unacceptable environmental damage	plant layout, geometry, founding levels, excavation levels and imposed loads			• plant layout, geometry, founding levels and imposed loads • shear strength parameters and groundwater regime of re-worked spoil.	Prior to the pre-operational stage	• mass haul design • spoil footprint optimisation • risk assessment of spoil stockpile failure to safety related structures	Conventional design will not mitigate sufficiently against environmental damage should the nuclear installation(s) be located near to the KNPS. Locating the plant in such areas will introduce high failure consequence risks related to excavated slope stability, dewatering and the effects of dewatering on the KNPS foundations. **Site licensability is impacted by potential risks to the safe operation of the KNPS.** **Locating future nuclear installation(s) remotely from the KNPS introduces less design challenge and the potential impacts referred to in the previous paragraphs diminish.**
		geotechnical profile	descriptive soil and rock profile	***Appendices 5.15.B*** to ***5.15.I*** ***Subsections 5.15.7***				
	Slope failure of spoil stockpiles	shear strength parameters and groundwater regime of re-worked spoil						

Appendix A-5.15.B
Borehole Logs

Due to the large volume of this appendix, the information is provided in electronic format.

Appendix A-5.15.C
Borehole Core Photographs

Due to the large volume of this appendix, the information is provided in electronic format.

Appendix A-5.15.D
Rock Joint Condition Logs

Due to the large volume of this appendix, the information is provided in electronic format.

Appendix A-5.15.E
Results of the Standard Penetration Tests (SPT)

Due to the large volume of this appendix, the information is provided in electronic format.

Appendix A-5.15.F
Result of the Dynamic Penetrometer Super Heavy (DPSH) Tests

Due to the large volume of this appendix, the information is provided in electronic format.

Appendix A-5.15.G
Soil Laboratory Test Results

Due to the large volume of this appendix, the information is provided in electronic format.

Appendix A-5.15.H
Rock Laboratory Test Results

Due to the large volume of this appendix, the information is provided in electronic format.

Appendix A-5.15.I
Geophysical Investigation Results

Due to the large volume of this appendix, the information is provided in electronic format.

Appendix A-5.15.J
Quality Data Pack

Due to the large volume of this appendix, the information is provided in electronic format.