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SECTION 5.15: GEOTECHNICAL CHARACTERISATION

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AMENDMENT RECORD			
Rev	Draft	Date	Description
0		April 2015	First Issue.
0a		January 2025	Section updated with PSHA data considering NNR comments.
0b		March 2026	Section updated to align with new format and correct minor editorial issues.

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Executive Summary

The Thyspunt site was first identified as a potential site for establishment of a nuclear installation(s) in the 1980s, and limited geotechnical investigative site work was done near the sea by the Atomic Energy Corporation of South Africa Ltd. Although providing useful background information, this work was not of sufficient detail to compile this Site Safety Report (SSR) and so an additional comprehensive desk study, fieldwork programme, data analysis and ongoing monitoring covering geohydrology was implemented by Eskom Holdings in October 2007. This culminated in the submission of Rev 0 of the Thyspunt SSR (TSSR) in 2015. At the time, the Probabilistic Seismic Hazard Assessment (PSHA) for the site was being undertaken and the data gained to support that study were not included in Rev 0 of **Section 5.15**.

In line with the requirements of the National Nuclear Regulator (NNR), SSRs must be regularly updated and this revision (this report) contains the latest monitoring and investigative data and also addresses NNR comments on Rev 0.

The purpose of Section 5.15 is to present the baseline geotechnical characteristics of the site in order to investigate the acceptability of the site for the safety of a nuclear installation(s). This was done through a combination of the following activities:

- review of available information from the 1980's and the Rev 0 investigative campaigns;
- field investigations supporting the PSHA;
- formulation of a geotechnical/geological model describing the site geotechnical profile;
- Integration with other specialist studies (e.g. the PSHA and **Section 5.11** Geohydrology);
- revising the liquefaction potential assessment using updated methods.

Based on the results and knowledge gained to date, the following key conclusions are drawn (with reference to the site unless otherwise stated):

- Site geotechnical profile (soils):

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- The site features a dune system overlying Table Mountain Group (TMG) rocks dipping southwards of the Skurweberg (at the sea), Goudini, Cederberg and Peninsula Formations (furthest inland).
- The dunes are dominated by cohesionless aeolian soils that are homogenously described as poorly graded with a very high sand size fraction.
- Dune migration has been monitored since 1942, and there is clear evidence that the dunes have stabilised due to alien vegetation growth – risks related to migrating dunes is therefore low if not disturbed at any stage.
- Marine deposits are encountered in an ancient erosion depression in the Goudini/Cederberg rocks and consist, in places, of thinly distributed marine sands underlain by a more dominant pebble/cobble layer sometimes several meters in thickness – the Illustrative Nuclear 1 Footprint is positioned above this cobble layer on the Goudini Formation and dewatering designs will need to cater for this.
- High CaCO_3 concentrations in all likelihood emanating from shells (see below point on groundwater) in the groundwater have resulted in calcification of site soils at the groundwater table level and below where calcrete horizons at various degrees of cementation are common, but not continuous.
- Within isolated looser zones, soils are potentially liquefiable, but the site in general does not present a high liquefaction risk due to the position of the groundwater table beneath zones of low consistency soils.
- Site geotechnical profile (groundwater):
 - Cumulatively, the aeolian sand, marine sand and the pebble/cobble layer form the Algoa Aquifer (an intergranular primary aquifer) underlying the full extent of the site except at the sea; the cobble layer is predominantly present above the Goudini/Cederberg rocks.
 - Groundwater flow within this aquifer is in a south to south-easterly

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direction and is locally influenced south easterly by the orientation of the pebble/cobble horizon in the Goudini/Cederberg depression (orientated northwest-southeast across the site).

- The groundwater table marks the top of the Algoa Aquifer and has high measured CaCO_3 concentrations.
- Site geotechnical profile (rocks):
 - Bedrock at the site consists of a wave cut platform/terrace made up of various sequences in the Table Mountain Group – these sequences dipping at $\sim 50^\circ$ southwards).
 - The Skurweberg Formation dominates at the coast and is the youngest formation in the sequence – moving inland (and northwards), the other sequences at the site consist of the Goudini, Cederberg and Peninsula formations (the oldest).
 - Although all of these sequences exhibit competent rock (albeit fractured with a low but variable rock quality designation), the Goudini and Cederberg formations are marginally less competent than the Skurweberg and Peninsula formations bounding them in the south and north respectively – when the wave cut platform was developed, the Goudini and Cederberg formations appear to have been eroded to a greater extent than the Skurweberg and Peninsula formations, resulting in a depression. The Goudini and Cederberg formations are marginally less competent because they appear to be more jointed, but not necessarily because they may be more weathered as measured density varies very little irrespective of lithology.
 - This Goudini and Cederberg depression houses cobble/marine sand deposits that influence groundwater flow quite considerably on the site as they constitute a preferential flowpath having higher permeability than the surrounding soils above and rocks below.
 - Rock parameters measured vary greatly within and across the various lithologies on the site, and where data density is high (from laboratory test results), most parameters trend towards a rectangular distribution with wide ranges in values – an exception is rock density that varies

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very little irrespective of lithology.

- Discontinuities divide the rock mass into blocks of randomly varying shape and the rock joints are generally tight.
- Rock joint conditions indicate that block failure (sliding, toppling and fall-out) is likely in all manner of excavations depending on their relative orientation to the joints. However, the rock mass exhibits good rock mass ratings and lateral support of any excavations will not be unduly onerous and can be achieved with conventional lateral support methods.

The geotechnical information available for the site to date and presented in this SSR identifies some areas of concern and some 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 site licence.

Certain exclusions are noted, and their implications on uncertainty are discussed, but these exclusions do not impact on this SSR presenting sufficient evidence to adversely affect the suitability of the site. In particular, the location of the Illustrative Nuclear 1 Footprint on the Goudini Formation rocks is noted, as the greatest density of data at the site exists for this formation. The regulatory frameworks/guidelines make provision in the geotechnical characterisation process to close out these exclusions and highlighting them is simply to prevent over-reliance on the data contained in this SSR in implementation planning.

The key findings with regard to the technical suitability of the site are contained below:

- Geotechnical Profile – information gathered to date does not indicate that any of the following are present on the site:
 - previous use of the site (e.g. mining activities);
 - gas pockets, and swelling rocks and shales;
 - zones of weakness or discontinuities in crystalline rocks;
 - indicators of potential cavities and susceptibility to ground collapse in the context of:

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- sinks, sink ponds, caves and caverns;
- sinking streams;
- historical ground subsidence;
- natural bridges;
- rock types such as limestone, dolomite, gypsum, anhydrite, halite, *terra rossa* soils¹, lavas, weakly cemented clastic rocks, coal or ores;
- non-conformities in soluble rocks.
- The following features do occur at the site:
 - surface depressions in the eroded Goudini/Cederberg wave cut platform where cobble deposits are common;
 - random springs at the sea where the groundwater table daylight and in the wetlands to the northeast of the site;
 - neither of these features, however, present any indication that site safety would be compromised and mitigation measures accounting for these features in the design would be unduly onerous.
- Founding Conditions:
 - Design will have to cater for variability in soil consistency – this will not be unduly onerous and a combination of ground improvement measures and employing founding methods pertinent to imposed loads will suffice. Data gathered to close out uncertainties related to site-specific soil profiles, will provide sufficient confidence to design foundations for structures in the soils at the site.
 - Design will have to cater for variability in rock characteristics, but this will not be unduly onerous provided established seismic loading is

¹ A red soil produced by the weathering of limestone

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factored into design and that detailed mapping of the nuclear installation(s) footprint is carried out once bedrock is exposed.

- key design precautions are to:
- factor established seismic loading into design, in particular to understand impedance characteristics of the site soils and weathered rock zones;
- 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 or carrying loads to bedrock with e.g. piled foundations;
- limit the extent of overburden soil in the profile by carefully selecting the footprint position of the safety related structures within the Illustrative Nuclear 1 Footprint – excavation stability, dewatering design and environmental impacts will be simplified/minimised if overburden depths can be reduced;
- take care when spanning lithological boundaries (e.g. with linear structures) and well-developed geotechnical discontinuities (shears, faults);
- optimally align the excavations, excavation crests and tunnels in rock relative to the orientations of bedding strike (270°) and the different joint sets to obviate sliding, toppling and fall-out failure.
- Liquefaction Potential - at earthquake magnitudes and PGA levels derived from the PSHA study, minor portions of the site are susceptible to liquefaction with the following conclusions drawn relating to this:
 - Current analysis indicates that liquefaction potential exists in isolated areas, but in general the site does not exhibit a high liquefaction risk.
 - Poorly consolidated soils in the upper soil horizons will exhibit liquefaction potential should these horizons become saturated in future for whatever reason.

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- Rock Joint Strength
 - Three joint sets dominate at the site, and these may all be significant since intersection of the joint sets may result in block/wedge sliding, toppling or fall-out failure. The stability of the various excavations should be optimised by appropriate orientation relative to the various joint sets.
 - Tunnels, in particular, may pose a risk to the implementation of designs as insufficient information exists to responsibly plan their construction (particularly in the offshore environment). The concern here relates to the extent to which variability in rock strength/hardness/abrasiveness may be encountered along the alignment, making it difficult to quantify/tender/execute designs. This uncertainty will be exacerbated offshore if the rock jointing there is increased or differently orientated.
 - Should alternatives to tunnelling be considered, these should, like tunnelling be subjected to feasibility studies.
- Slope Stability and Earth Structures: Perhaps the single most significant geotechnical characteristic of the site is the extensive, cohesionless overburden overlying the bedrock and the fact that an intergranular aquifer underlies the entire site. The Illustrative Nuclear 1 Footprint is positioned within overburden deposit on the northwest corner suggesting that excavation to bedrock in this area will be ~40 m deep. In addition, the cobble horizon underlies the Illustrative Nuclear 1 Footprint. Soils slopes will need to be battered back at angles in the region of 18° and will need to be dewatered before any confidence in slope stability can be assured. This introduces a number of conceptual design considerations and implications:
 - Dewatering design has to consider the pebble/cobble horizon in the Goudini/Cederberg depression (inserting impermeable barriers through this will be challenging and it is critical to appreciate that this horizon is a preferential groundwater flow path influencing flow direction).
 - Since excavations may be open for extended periods of time (years), groundwater drawdown may have regional environmental impacts in addition to the disposal of the extracted groundwater and of the excavated spoil.

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- Dewatering systems will have to be robust enough to ensure uninterrupted dewatering of the site, particularly during construction.
- Key considerations therefore relating to the suitability of the site are the positioning of the safety related structures within the Illustrative Nuclear 1 Footprint and the optimal placing of the safety related structures closer to the sea (where depth to bedrock is ~15 m).
- Constructed earth structures and improved ground rafts by mechanical compaction will not enable bearing capacities >200 kPa to be obtained. Bearing capacities greater than this can only be achieved with cement stabilisation.
- Auxiliary 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 particularly in linear structures (e.g. pipelines).
- As with auxiliary structures that support the nuclear installation(s), other buried structures are not likely to pose any onerous design challenges from a geotechnical design input point of view – bar issues related to differential founding conditions and differential dynamic loading regimes.

It is therefore concluded that the extensive evidence available to describe the geotechnical profile at the site does not indicate that any geotechnical features pose any disconcerting safety risks to a nuclear installation(s) and that those areas of concern that may arise can be mitigated by design.

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5.15 GEOTECHNICAL CHARACTERISATION

5.15.1 Introduction

The site has passed through a number of screening phases as described in **Chapter 4** (Site Investigation Approach) of this Site Safety Report (SSR). The detailed investigations and the development of an SSR on the engineering geological characteristics and geotechnical profile for the site that are needed to comment on the suitability of the site are presented in this section. The screening information is accordingly supplemented in this section with additional site investigations that include geographical collation of geotechnical information, *in situ* and laboratory testing of the soils and rocks, groundwater level observations and integration with the Probabilistic Seismic Hazard Assessment (PSHA) (Council for Geoscience, 2013). The historical (screening) and new (investigative) 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(s) safety, as prescribed in **Section 5.15** of the Technical Specification (Eskom, 2022).

The investigations were 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.

This section is developed in close collaboration with and complements **Section 5.11** (Geohydrology), **Section 5.13** (Geology) and **Section 5.14** (Seismic Hazard) of this SSR. (Council for Geoscience, 2013) is also a significant contributor to this section, particularly in informing input parameters for the liquefaction potential assessment.

5.15.2 Purpose and Scope

Within the framework of the safety requirements and criteria presented in **Subsection 5.15.3.2** 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;

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- make an in-principle assessment of how the site is likely to respond to loading or external events (demands);
- describe mitigating measures (design principles) that may be considered to ensure site safety in the context of developing 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;
- in-principle, assessment of the soil and rock evolution under induced loads (i.e. when the nuclear installation(s) is physically developed) during the nuclear installation(s) lifetime;
- potential areas of concern and measures for improvement of the foundations and/or founding materials of the nuclear installation;
- monitoring programme/s for geotechnical parameters prior to site development.

Detailed description of the activities carried out during the geotechnical characterisation of the site, including the detailed results achieved, is presented in the appendices of this section. The appendices also provide the quality assurance records and details supporting the statements and conclusions on the properties of rock and soil at the site and reported in 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 supporting site licensing applications.

As described in **Subsection 5.15.3.2**, the geotechnical characterisation of the site follows a four stage approach. This SSR presents a summary of the results of the selection stage and details of the investigations carried out during the

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characterisation stage. The pre-operational and operational stages are future stages, and it should be appreciated that the geotechnical characterisation of the site will evolve through all of the stages. It should also be pointed out that not all of the criteria and requirements in **Subsection 5.15.3.2** are addressed in this SSR. This is primarily due to the fact that certain details of the proposed nuclear installation(s) (e.g. nuclear island footprint location) were unknown at the time of issuing this report.

This section exclusively addresses the selection and characterisation stages of the geotechnical investigation for this SSR and serves no other purpose. It also does not address all of the requirements stipulated in **Subsection 5.15.3.2** as certain details of the proposed nuclear installation 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 and no other aspect or condition that may be considered in, or affect subsequent stages of the project. For example, the information presented in this SSR should not be construed as sufficiently robust to support pricing by vendors.

Additional data gathering will occur (and will be comprehensively addressed) in future investigations (see **Subsection 5.15.3.2**) and present detail that is pertinent to design and implementation of a nuclear installation(s). These exclusions do not impact on this SSR presenting sufficient evidence to make conclusive comment on the site suitability.

5.15.3 Regulatory Framework

5.15.3.1 Relevant Legislation

The legal and regulatory basis is outlined in **Chapter 2** (Legal and Regulatory Basis). 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.

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5.15.3.2 Guidelines on Geotechnical Characterisation Studies

The above regulations are not specific in terms of geotechnical characterisation and so the technical specifications and safety guides listed below were used to shape the approach to the geotechnical characterisation of the site:

- Eskom's Technical Specification for Site Safety Reports (Rev 2). **Section 5.15**: Geotechnical Characterisation_(Eskom, 2022);
- International Atomic Energy Agency (IAEA) Safety Requirements No. NS-G-3.6, Geotechnical Aspects of Site Evaluation and Foundations for Nuclear Installations_(International Atomic Energy Agency, 2004);
- US 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);
- US NRC NUREG-0800: Site Standards Review Plan, Part 2,0: Site Characteristics and Site Parameters (United States Nuclear Regulatory Commission, 2007).

These guidelines relevant to the geotechnical characterisation investigations of the acceptability of a site for development of a nuclear installation(s) require:

- 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 (Department of Energy, 2010);
- monitoring of current and future uncertainties discussed in this SSR (Department of Energy, 2010);
- events (or demands) that potentially can lead to radioactive exposure to be considered as part of the design features of the nuclear installation (s) in future_(Department of Energy, 2010).

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The IAEA requires (e.g. (International Atomic Energy Agency, 2004)) that site characteristics that may affect the safety of the nuclear installation 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. 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 affect the safety of the installation (**Chapter 6**). 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.

The IAEA (International Atomic Energy Agency, 2004) recommends development and implementation of the geotechnical site investigation programme in four stages, namely:

- selection stage, comprising high level candidate site identification;
- characterisation stage (the stage relevant to this SSR) including verification and confirmation activities at sites identified in the selection stage⁴;
- pre-operational stage comprising⁵ continual geotechnical investigations;
- operational stage comprising⁶ monitoring of geotechnical performance.

(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.

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 transport radioactive materials away from the power plant.

⁴ A basis for conducting the characterisation stage is that generalised layouts and imposed loads have been established – this is not the case for this SSR and as such, the characterisation stage is not completed in full in this SSR.

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

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

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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, 2007). 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 and rock 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;
- response of soil and rock to dynamic loading;
- analysis of liquefaction potential;
- earthquake design basis (summary);
- static analyses;
- techniques to improve subsurface conditions.

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NUREG (United States Nuclear Regulatory Commission, 2007), 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).

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) in the USA.

5.15.4 Site Investigation and Data Collection

5.15.4.1 Evaluation of Available Data

The site is situated in the Eastern Cape Province between St. Francis Bay and Oyster Bay. The site was identified in the 1980s as a potential site for developing a nuclear installation(s) and was the subject of various preliminary investigations (International Atomic Energy Agency, 2004) (Atomic Energy Corporation of South Africa Limited, 1988). These historical investigations focused on defining preliminary site suitability from a geotechnical characterisation perspective. The following was reviewed during that stage:

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

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However, the historical investigations did not provide sufficient detail to provide the required level of information and understanding of the site characteristics, justification of the suitability of the site and hence the development of an SSR. A comprehensive investigation programme was therefore initiated in 2008 in which additional data were collated and presented in a previous version of this SSR in 2015 (SRK Consulting, 2015).

Additional data were collected in support of the Probabilistic Seismic Hazard Assessment (PSHA) (Council for Geoscience, 2013), but the PSHA had not reached conclusion and the outcomes thereof were not included in the previous version of this SSR.

This TSSR covers this shortfall.

5.15.4.1.1 Landform – Geomorphology

(a) General description

Comment on Geomorphology draws from a specialist study (Illenberger & Associates, 2010) conducted in support of the Environmental Impact Assessment (EIA) for the Thyspunt site. The site is fronted by a ragged beachfront of rock outcrop, an approximate 40 m transition zone of intermittent groundwater seeps, and a double bench dune system consisting of a lower terrace to a height of about 20 m relative to mean sea level (m msl) and an upper terrace up to 70 m msl. The dune system is densely vegetated (see **Drawing 5.15.1**).

The dune varieties found at Thyspunt are mobile dunefields of the headland-bypass dunefield variety (the Oyster Bay dunefield), and vegetated parabolic dunes and hairpin parabolic dunes. In addition, sidewalls of previously mobile dunefields form long, vegetated dune ridges. Parts of the mobile dunefields have been artificially stabilised with alien vegetation such as Rooikrans. The mobile dunefields are very dynamic.

At Thyspunt groundwater “daylights” in many interdune areas within the Oyster Bay dunefield to form ponds in the interdune areas (also known as dune slacks), where wetlands are often found. The behaviour and flow bypass dunefields, including fairly unique examples of such sidewalls. Overall, the dunefields at Thyspunt have high interpretive value for elucidating coastal dune

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dynamics. Since 1942 the dunefield has become progressively more vegetated, both within the dunefield and along the northern margin. This is caused mostly by various invasive alien Acacia species, predominantly Rooikrans.

(b) Rate of Dune Migration (Mobile Oyster Bay Dunefield)

Dune height varies along the length of the dunefield. Dunes are on average 20 m high in the west, and gradually become smaller towards the east, where the average height is 5 m. Because the volume of sand moving is approximately constant, a larger dune moves slower, and conversely a smaller dune moves faster.

According to aerial photos of the past 50 years, dunes and the leading tongues of dunefields move eastwards at rates of 10 to 30 m/yr. Thus, it would take 1000 -1300 years for a dune to cross the headland.

Characteristics of groundwater and surface water were investigated to help determine the viability, in respect of dune dynamics, of building transmission lines and an access road to Thyspunt from the north, across the Oyster Bay dunefield.

Mobile dune dynamics at Thyspunt were investigated in detail. An access road, transmission lines and a temporary conveyor belt or haul road could potentially be built across the mobile dunes of the Oyster Bay dunefield at Thyspunt.

Topsoil and spoils stockpiles cannot be located on the mobile Oyster Bay dunefield at Thyspunt. Topsoil and spoils stockpiles can be located on the vegetated dunefield at Thyspunt with medium operational impacts.

The geomorphologic conservation value of the headland-bypass dunefields at Thyspunt is high, as they are the only remaining large dunefields of this type that are still active in South Africa. The headland-bypass dunefields at Cape St. Francis are unique on a local, regional and probably global scale. The vegetated dunefield is a classic, almost pristine example of a suite of Holocene and Pleistocene dune ridges with a variety of origins: parabolic dunes, hairpin parabolic dunes, and sidewalls of previously mobile headland.

The average volume or rate of sand movement is 30 m³/m width/year. This rate is not constant: when there is no dune passing a particular point, the rate of

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sand movement is zero, and when a dune is passing a particular point, the rate will increase to typically 4-5 times the average rate.

Figure 5.15.1 is a direct extract from the EIA conducted for the site (Illenberger & Associates, 2010) indicating dune mobility between 1942 and 1985.

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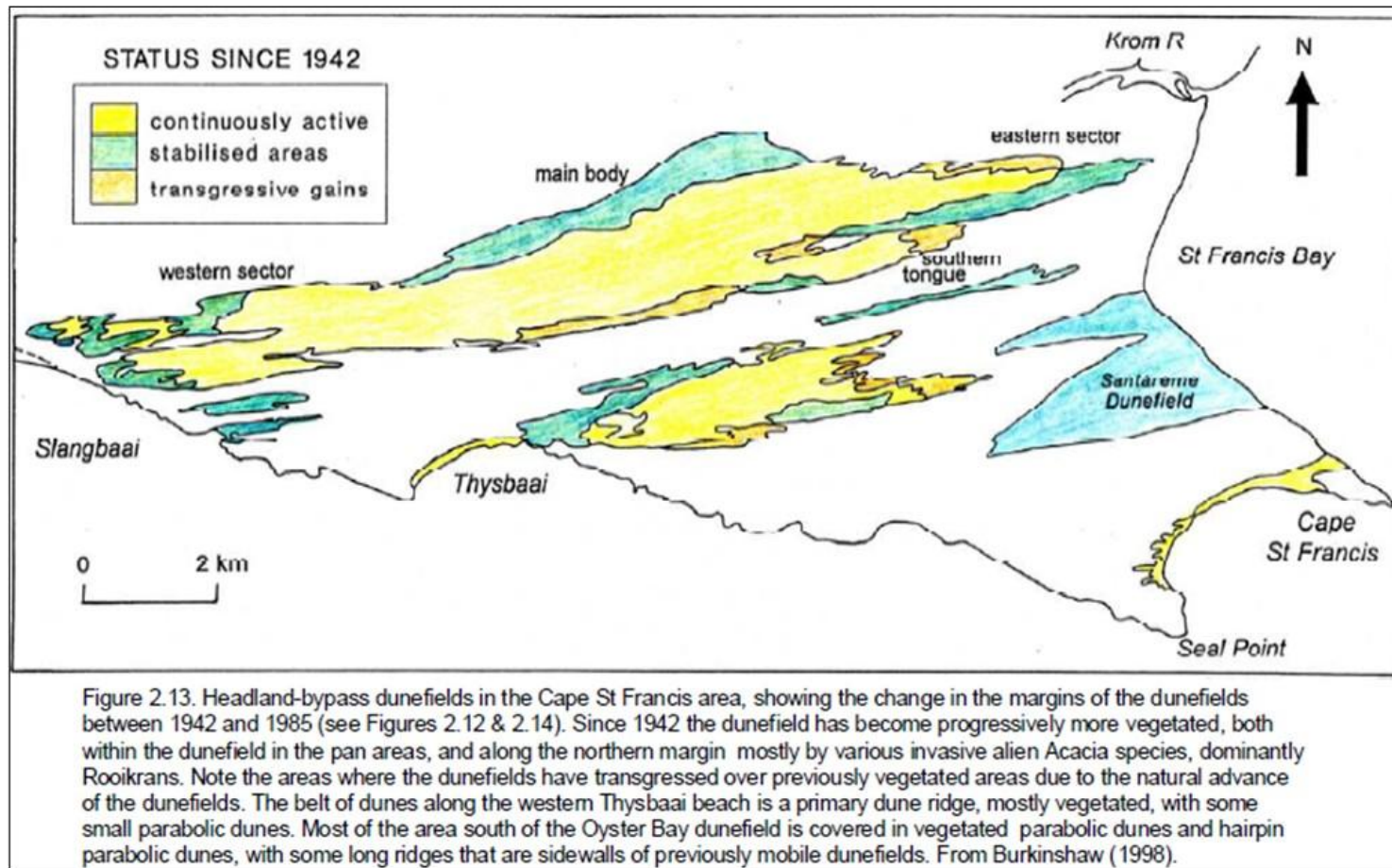


Figure 5.15.1
Description of Mobile Dunes Systems at Thyspunt

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Since dune migration is predominantly in an easterly direction, and since the dunefields immediately surrounding the site have stabilised with alien vegetation since 1942, there is a very low risk of migrating dunes impacting on the safety of the site if not disturbed at any stage.

5.15.4.1.2 Geological conditions

The area surrounding the site is underlain by rocks (sandstones, siltstones and shales) of the Table Mountain Group (TMG) and Bokkeveld Group (Geological Survey of South Africa, 1991) and (Toerien & Hill, 1989). These formations have been folded and faulted due to the movement and thrusting associated with the break-up of Gondwanaland. The folding and thrust faulting appear to have occurred during the convergence of continental plates. This break-up is considered the reason for the formation of small 'rift valleys' that subsequently filled with Cretaceous-age sediments eroded from the adjacent mountains (primarily Peninsula Formation rocks). The westward movement of the Falkland Plate along the Agulhas-Falkland Fracture Zone has resulted in a drag-induced curvature to the eastern ends of the faults and folds that are particularly characteristic of the Eastern Cape (see **Section 5.13**).

The site itself is underlain by quartzitic sandstones and subordinate siltstone of the Skurweberg, Goudini, Cederberg and Peninsula formations of the TMG. These dip at approximately 45° to 50° south and this is due to the location of the site on the southern limb of a major anticline of a fold system (see **Section 5.13**). Outcrops of hard grey quartzitic sandstone with interbedded⁷ black siltstone are visible along the coast. Phyllite also occurs repeatedly within the formation. The bedding strike is sub-parallel to the coastline at ± 300° with the dip direction at ± 210° or north to northwest – south to southeast. Coastal gullies and outcrop ridges are similarly orientated. Small-scale northwest striking gullies also occur where selective erosion of prominent joint sets occurs.

The bedrock is overlain in places by cobble and pebble beds of the basal Alexandria Formation conglomerate averaging 3 m thick at the site, but ranging from 0.15 m to 9 m thick. The cobble and pebble beds are in turn overlain by stabilised sand dunes of the Nanaga Formation with a thickness range of 0 m

⁷ Interbedded: Beds (layers) of rock lying between or alternating with beds of a different kind of rock.

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at the sea to 65 m approximately 800 m from the coast (see **Drawing 5.15.3** and **Drawing 5.15.4** to illustrate).

Several large faults occur to the south, east and west of the site. The Klippunt Fault to the south of the site strikes in a north-northwest – south-southeast direction and forms a large regional normal fault running from the Storms River mouth south-east out into the ocean. Other smaller faults, approximately north-south striking, occur at irregular intervals, for example:

- to the southeast - 1 km from Cape St. Francis, and at De Hoek;
- to the west - at Tony's Bay, at Ashton Bay and near Klipdrif.

A detailed description of the site geology is presented in **Section 5.13** and documentation of geological discontinuities (e.g. faults) is extensively covered in (Council for Geoscience, 2013).

The relatively straightforward exposition of the geological structure enhances the confidence in the recommendations in this SSR on the Geotechnical Characteristics of the Thyspunt Site as described here (and with reference to **Figure 5.15.2 a)** and **b)**):

- Convergence of the continental plates south of the African continent gave rise to a major compressive force perpendicular to the Cape Fold Belt in a direction 30° east accompanied by a minor compressive force in a direction 120° east. The associated principal primitive stresses are denoted by σ_1 and σ_2 .
- The bedding dip corresponds to the anticline of the fold caused by the major compressive force perpendicular to the anticline.
- The strike of the coastal gullies and outcrop ridges corresponds to the fold of the anticline.
- The principal primitive stresses gave rise to conjugate shear failures J_2 and J_3 in directions 350° east and 55° east respectively.
- The comparative magnitudes of the principal primitive stresses gave rise to extensional strain in a direction 120° east and tensional joints J_1 in a

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direction 30° east.

- The conjugate shear failures bisected by extension strain failures represent the classical failure mechanism for rock (extension strain failures are induced in all unconfined compressive strength tests of rock and concrete at an applied stress of about one third of the ultimate compressive strength; it also occurs in triaxial stress conditions but at higher levels of stress; extension strain failure is a property of brittle materials).
- The Klippunt fault and the Smaller Faults referred to, in principle, strike along or are associated with conjugate shear failures J₂.
- The anticlinal fold in addition gave rise to shear failure on the bedding which in turn created preferential zones of weakness that allowed infiltration of groundwater and weathering and erosion of the rock that eventually appeared as gullies and ridges and the Goudini / Cederberg depression mentioned in detail in this SSR. The depression in all likelihood developed in this way and not by weathering of the rock mass because all four formations have practically the same density and strength. The development of the depression was in addition aided by the extensive fracturing of the Goudini and Cederberg formations without detracting from the density and strength of these formations compared to the Skurweberg and Peninsula formations that were equally extensively fractured.

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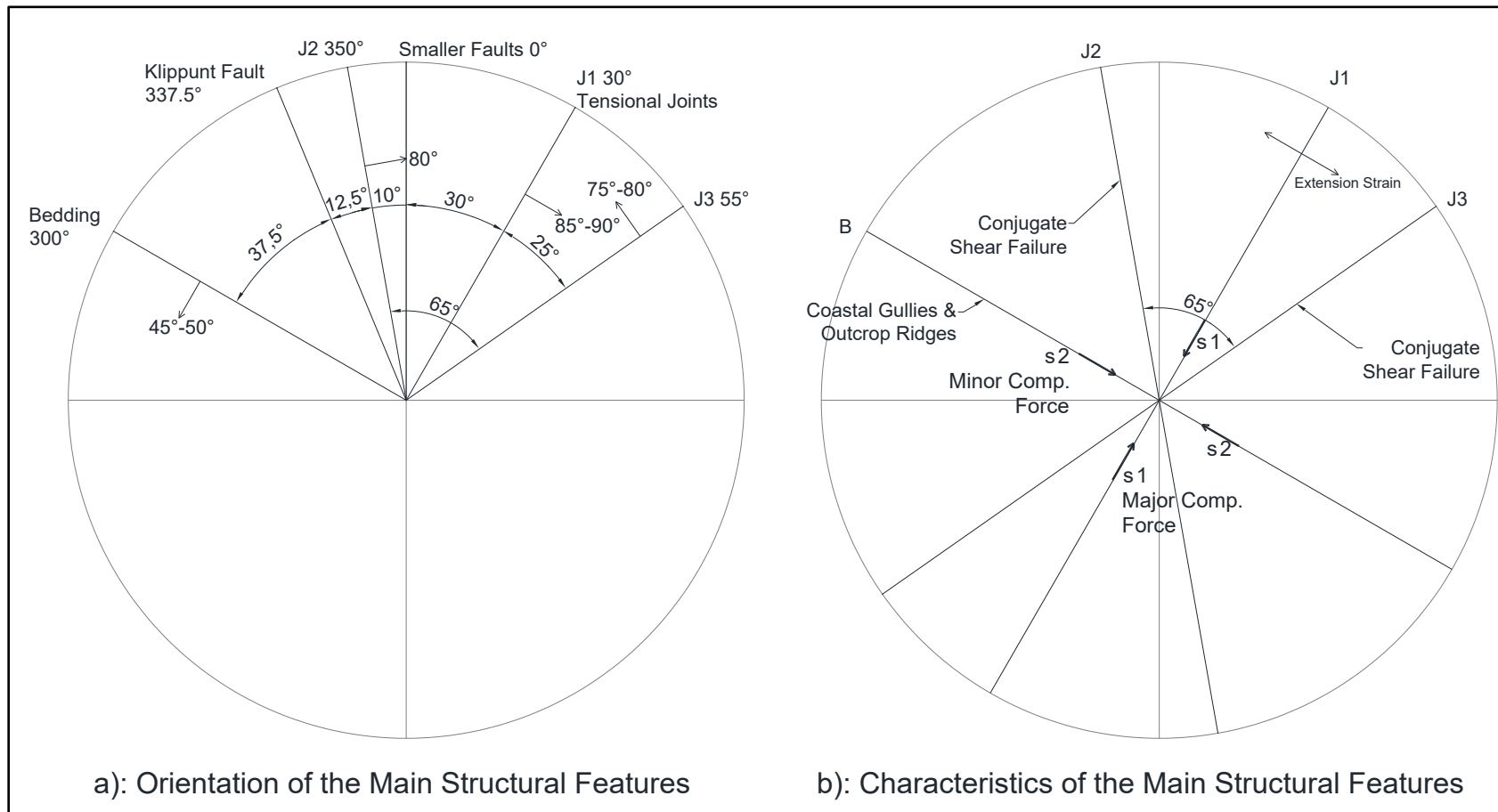


Figure 5.15.2
Thyspunt Geological Structure

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5.15.4.1.3 Groundwater conditions

(a) Physical Conditions

There are two main aquifers⁸ at the site (see **Section 5.11**):

- the 'primary' Algoa Aquifer – an intergranular aquifer;
- the 'secondary' TMG Aquifer – a fractured rock aquifer.

The TMG bedrock in the region can be classified as a secondary or fractured-rock type aquifer (the TMG Aquifer). Groundwater flow and storage takes place within secondary openings in the rocks formed by joints, faults and fractures. The superficial deposits of the Algoa Group are classified as a primary or intergranular aquifer where groundwater flow and storage take place within the pore spaces between constituent grains. The upper boundary of the aquifer is the water table, and these aquifers are therefore unconfined. The distribution of the aquifers is presented in **Section 5.11**.

The average hydraulic properties of the TMG Aquifer (**Section 5.11**) are:

- transmissivity ranges from 0.2 to 30 m²/d;
- calculated hydraulic conductivity is 0.4 m/d (Nardouw Aquifer) to 0.45 m/day (Peninsula Aquifer);
- calculated flow velocity of 0.42 m/d in the Nardouw Aquifer.

The average hydraulic properties of the Algoa Aquifer as defined in **Section 5.11** are:

- transmissivity ranges from 3 to 430 m²/day, the latter value being from the cobble layer;
- hydraulic conductivity varies from 0.67 to 140 m/day, the latter being from

⁸ Aquifer: A geological formation, which has structures or textures that hold water or permit appreciable water movement through them (Council for Geoscience, 2013). Also defined as the saturated zone of a geological formation beneath the water table, capable of supplying economic and usable volumes of groundwater to borehole(s) and/or springs.

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the cobble layer;

- flow velocity in the Algoa Aquifer (cobble layer and aeolian⁹ sand combined) of 0.86 m/d;
- flow velocity in the cobble layer of 7 m/d – This is relatively high and confirms the highly transmissive nature of the cobble layer.

The integrated efforts of groundwater table monitoring that are presented in **Section 5.11** and this section of this SSR reveal that the groundwater flow direction at the site is in a south to southeasterly direction (see **Section 5.11**) with localised flow direction changes related to the highly transmissive cobble layer (located above the Goudini and Cederberg formations) and its orientation on the site.

The isotropy¹⁰ of transmissivity has not been studied in any detail. When considering dewatering solutions and/or groundwater response to dewatering, both the rate of inflow and dewatering concepts may be affected.

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

(b) Groundwater Chemistry

This section draws from the groundwater quality assessment (**Section 5.11**). Groundwater chemistry from the regional hydrocensus is summarised in **Table 5.15.1**.

⁹ Aeolian: Deposited by wind.

¹⁰ Uniformity in all directions

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Table 5.15.1
Regional Groundwater Chemistry

Borehole Number	Aquifer	Hydro-census	Determinand*												
			Ca	Mg	Na	K	Alkalinity (as CaCO ₃)	Cl	SO ₄	NO ₃ (as N)	F	NH ₄ (as N)	Ortho- Phosphate (as P)	pH**	EC***
Charly Wood	Algoa	Dec-07	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns
		Aug-17	193.8	20.3	227.3	2.1	257	439	179.2	<0.045	0.50	na	<0.02	7.9	220
		Feb-24	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns
Cilliers 2	Algoa +TMG	Dec-07	16.6	31.9	278.7	1.4	217	381	38.3	1.4000	<0.10	0.06	0.03	6.2	147
		Aug-17	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns
		Feb-24	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns
Cilliers 3	TMG	Dec-07	14.0	5.5	43.9	<1.0	40	73	2.3	0.0300	<0.10	0.60	<0.03	8.6	35
		Aug-17	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns
		Feb-24	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns
Gerber 3	TMG	Dec-07	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns
		Aug-17	34	14.3	84	2.6	31	168	61	<0.05	0.30	NA	<0.03	6.1	75
		Feb-24	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns
Gerber 4	TMG	Dec-07	31.2	16.5	78.2	4.4	54	167	34.0	0.0050	<0.10	0.04	<0.03	6.7	71
		Aug-17	9.6	9.0	62.4	1.4	16	186	17.2	0.50	0.30	NA	<0.03	6.0	46
		Feb-24	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns
Grassridge	TMG	Dec-07	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns
		Aug-17	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns
		Feb-24	Done	54.9	367.9	29.9	46	634	121.6	4.77	<0.3	NA	<0.02	6.2	253
GRUN 1	TMG	Dec-07	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns
		Aug-17	53.2	14.8	85.5	5.5	175	154	5.0	0.385	0.40	NA	<0.02	7.7	85
		Feb-24	49.8	12.3	58.9	7.7	194	100	8.1	0.226	<0.30	NA	<0.02	7.0	74

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Borehole Number	Aquifer	Hydro-census	Determinand*												
			Ca	Mg	Na	K	Alkalinity (as CaCO ₃)	Cl	SO ₄	NO ₃ (as N)	F	NH ₄ (as N)	Ortho- Phosphate (as P)	pH**	EC***
GWA_OB2_2017	TMG	Dec-07	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns
		Aug-17	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns
		Feb-24	23.6	29.6	166.6	20.3	132	307	3.7	<0.045	<0.3	NA	<0.02	7.9	129
KKLSP	TMG	Dec-07	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns
		Aug-17	111.8	12.4	62.2	1.8	264	116	20.1	<0.045	0.50	NA	<0.02	8.1	91
		Feb-24	67.3	11.7	112.6	6.4	202	170	42.6	<0.045	<0.3	NA	0.134	7.54	102.8
Langefonteinvelei 1	Algoa	Dec-07	107.2	11.1	49.9	1.5	294	82	6.1	<0.0025	0.16	0.07	<0.03	8.3	78
		Aug-17	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns
		Feb-24	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns
Langefonteinvelei 2	Algoa	Dec-07	137.1	20.9	138.3	2.7	321	224	89.2	<0.0025	0.22	0.10	<0.03	7.5	138
		Aug-17	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns
		Feb-24	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns
MANS (Witbank Huis)	TMG	Dec-07	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns
		Aug-17	12.6	18.0	140.4	1.0	13	260	32.9	0.475	0.30	NA	<0.02	6.7	95
		Feb-24	13.8	23.1	146.3	1.4	26	296	32.4	<0.045	<0.3	NA	<0.02	6.3	114
Milagro 1	TMG	Dec-07	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns
		Aug-17	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns
		Feb-24	12.1	11.7	92.3	6.6	36	150	16.8	<0.045	<0.3	NA	<0.02	6.4	63
Milagro 2	TMG	Dec-07	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns
		Aug-17	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns
		Feb-24	14.6	18.2	119.1	1.4	20	229	23.3	2.35	<0.3	NA	<0.02	6.2	90
Municipal BH 1	TMG	Dec-07	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns
		Aug-17	74.2	13.1	119.1	5.7	184	186	43.4	0.565	0.5	NA	<0.02	7.6	105
		Feb-24	125.1	18.2	117.8	2.8	278	237	38.9	<0.045	<0.3	NA	<0.02	7.7	135

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Borehole Number	Aquifer	Hydro-census	Determinand*												
			Ca	Mg	Na	K	Alkalinity (as CaCO ₃)	Cl	SO ₄	NO ₃ (as N)	F	NH ₄ (as N)	Ortho- Phosphate (as P)	pH**	EC***
Municipal Spring 1	Algoa	Dec-07	103.5	11.8	54.9	1.1	267	105	13.3	<0.0025	0.19	0.04	0.03	8.2	82
		Aug-17	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns
		Feb-24	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns
Oeloff 1	TMG	Dec-07	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns
		Aug-17	17.6	23.5	177.7	1.1	23	340	29.2	0.973	0.40	NA	<0.02	6.6	121
		Feb-24	14.7	20.3	154.4	1.5	36	304	27.0	0.70	<0.3	NA	<0.02	6.4	117
Oeloff 2	TMG	Dec-07	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns
		Aug-17	16.0	26.8	192.8	0.7	16	373	27.1	1.538	0.30	NA	0.10	6.3	131
		Feb-24	20.8	38.3	245.3	1.1	30	459	28.6	1.65	<0.3	NA	<0.02	6.3	172
Oeloff 3	TMG	Dec-07	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns
		Aug-17	80.0	14.3	74.4	0.7	168	144	32.3	0.452	0.50	NA	<0.02	8.0	86
		Feb-24	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns
Oyster Bay Spring 1	Algoa	Dec-07	119.2	20.1	214.9	3.9	255	330	57.9	0.073	0.14	0.04	0.03	8.0	161
		Aug-17	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns
		Feb-24	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns
Oyster Bay Spring 3	Algoa	Dec-07	95.0	14.9	121.9	2.4	230	203	40.0	0.770	0.15	0.05	0.05	8.4	114
		Aug-17	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns
		Feb-24	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns
Oyster Bay Spring 6	Algoa	Dec-07	93.1	14.3	114.3	2.3	206	206	39.6	0.917	0.16	0.04	0.07	8.2	109
		Aug-17	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns
		Feb-24	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns
Oyster Bay Spring 7	Algoa	Dec-07	84.1	12.7	90.5	2.2	163	163	31.8	0.908	0.15	0.07	0.06	8.3	92
		Aug-17	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns
		Feb-24	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns

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Borehole Number	Aquifer	Hydro-census	Determinand*												
			Ca	Mg	Na	K	Alkalinity (as CaCO ₃)	Cl	SO ₄	NO ₃ (as N)	F	NH ₄ (as N)	Ortho- Phosphate (as P)	pH**	EC***
Oyster Bay Spring 8	Algoa	Dec-07	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns
		Aug-17	151.5	49.2	394.7	10.0	232	778	111.8	<0.045	0.50	NA	<0.02	8.1	310
		Feb-24	130.5	35.2	311.4	8.4	276	534	69.2	0.20	<0.3	NA	<0.02	7.9	236
Pebbler	TMG	Dec-07	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns
		Aug-17	40.6	18.4	133.8	4.2	136	227	16.2	0.407	0.50	NA	<0.02	8.2	103
		Feb-24	125.4	17.0	104.4	3.4	304	195	50.9	<0.045	<0.3	NA	<0.02	7.7	127
Pennisands 1	TMG	Dec-07	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns
		Aug-17	16.8	19.7	141.5	1.2	21	269	27	2.490	0.30	NA	<0.02	6.7	100
		Feb-24	22.0	26.8	158.2	4.8	30	370	29.2	11.18	<0.3	NA	<0.02	6.4	150
Pennisands 2A	TMG	Dec-07	10.7	15.6	104.5	1.0	12	181	25.4	4.643	<0.10	0.04	0.06	6.0	63
		Aug-17	14.9	20.0	126	1.5	12	238	21.9	6.560	0.30	NA	0.09	6.1	91
		Feb-24	17.3	24.9	130.8	2.5	28	263	24.4	10.52	<0.3	NA	<0.02	6.3	109
RD1		Dec-07	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns
		Aug-17	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns
		Feb-24	7.9	8.5	68.0	11.0	50	121	16.4	0.20	<0.3	NA	0.04	6.6	57
Strydom 1	TMG	Dec-07	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns
		Aug-17	14.4	17.3	138.3	1.4	27	253	25.8	0.790	0.40	NA	<0.02	6.4	94
		Feb-24	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns
Strydom 3	TMG	Dec-07	3.3	7.6	53.5	<1.0	9	93	19.3	0.040	<0.10	0.04	<0.03	5.9	37
		Aug-17	6.2	12.4	77.6	1.6	4	151	13.1	2.260	0.30	NA	0.09	6.1	56
		Feb-24	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns
Strydom 5	TMG	Dec-07	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns
		Aug-17	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns
		Feb-24	13.0	16.6	130.8	1.6	40	241	24.3	0.16	<0.3	NA	<0.02	6.4	98

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Borehole Number	Aquifer	Hydro-census	Determinand*												
			Ca	Mg	Na	K	Alkalinity (as CaCO ₃)	Cl	SO ₄	NO ₃ (as N)	F	NH ₄ (as N)	Ortho- Phosphate (as P)	pH**	EC***
Strydom 6	TMG	Dec-07	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns
		Aug-17	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns
		Feb-24	11.4	15.1	123.5	1.3	38	223	22.3	<0.045	<0.3	NA	<0.02	6.4	91
Strydom 7	TMG	Dec-07	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns
		Aug-17	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns
		Feb-24	10.1	19.2	130.1	1.2	26	257	23.3	1.36	<0.3	NA	<0.02	6.2	100
Strydom 9	TMG	Dec-07	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns
		Aug-17	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns
		Feb-24	14.4	15.2	154.7	2.0	62	285	9.4	<0.045	<0.3	NA	<0.02	6.7	111
Vulindlela 1	TMG	Dec-07	7.0	10.4	68.2	5.4	18	119	29.9	0.217	<0.10	0.04	<0.03	6.8	51
		Aug-17	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns
		Feb-24	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns
Vulindlela 4	TMG	Dec-07	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns
		Aug-17	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns
		Feb-24	20.6	23.2	187.0	2.3	36	332	36.8	<0.045	<0.3	NA	<0.02	6.3	128
Vulindlela 5	TMG	Dec-07	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns
		Aug-17	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns
		Feb-24	21.4	25.4	176.9	5.5	38	332	35.7	2.65	<0.3	NA	<0.02	6.3	129
Welgelegen 2	TMG	Dec-07	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns
		Aug-17	7.4	11.9	97.4	0.6	11	160	19.7	5.750	0.30	NA	<0.02	6.5	65
		Feb-24	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns
Welgelegen3	TMG	Dec-07	7.7	12.5	96.7	<1.0	15	167	24.5	0.592	<0.10	0.10	<0.03	6.6	66
		Aug-17	7.9	10.7	95.2	0.8	13	162	18.9	2.850	0.30	NA	<0.02	6.3	64
		Feb-24	8.5	13.2	97.2	1.0	28	165	20.0	6.88	<0.3	NA	<0.02	7.4	76

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Borehole Number	Aquifer	Hydro-census	Determinand*												
			Ca	Mg	Na	K	Alkalinity (as CaCO ₃)	Cl	SO ₄	NO ₃ (as N)	F	NH ₄ (as N)	Ortho- Phosphate (as P)	pH**	EC***
Welgelegen 5	TMG	Dec-07	4.1	10.9	83.0	<1.0	10	147	26.4	0.6500	<0.10	0.05	<0.03	6.0	56
		Aug-17	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns
		Feb-24	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns
Welgelegen 6	TMG	Dec-07	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns
		Aug-17	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns
		Feb-24	3.2	7.5	53.2	0.8	18	93	16.0	<0.045	<0.3	NA	<0.02	6.1	42
Bedwell (378 Tuna)	TMG	Dec-07	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns	ns
		Aug-17	146.7	24.6	145.8	3.0	308	293	45.5	1.360	0.40	NA	<0.02	7.7	156
		Feb-24	146.4	24.8	149.4	4.1	350	315	55.7	0.91	<0.3	NA	<0.02	8.0	174
ns ¹¹ = not sampled NA = Not Analysed			* all determinands in mg/ℓ apart from: **pH value at 22°C ***EC measured in mS/m at 25°C												

¹¹ Boreholes were indicated as not sampled (ns) for various reasons e.g., a broken pump, the borehole has collapsed, or it was drilled after a specific hydrocensus.

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The regional groundwater macro-chemistry can be summarised as follows:

- Electrical conductivity (EC) ranges from 35 to 92 mS/m for the TMG Aquifer and 78 to 161 mS/m for the Algoa Aquifer.
- The EC from the spring which supplies water to the village of Oyster Bay measured EC ranges from 116 to 310 mS/m during the 2007, 2017 and 2024 hydrocensuses.
- Very low Fe and Mn concentrations were measured in the TMG Aquifer which is somewhat anomalous for this aquifer.
- The majority of the springs in the area generally have elevated Na and Ca.
- All groundwater samples analysed have a dominant sodium-chloride-sulphate character, which is typical for coastal environments.
- Measured sulphate concentrations were generally low, i.e. <100 mg/l, indicating that groundwater is not likely to be aggressive to concrete.
- Chemical characteristics of groundwater from the different aquifers in the area can be summarised as follows:
 - TMG Aquifer: sodium chloride-type and acidic to neutral pH;
 - Algoa Aquifer: mixed-type with high concentration of $\text{Ca}(\text{HCO}_3)_2$ (in all likelihood originating from extensive shell deposits in the site soils) and neutral to alkaline pH.

The Langelier Saturation Indices representing the Algoa Aquifer vary from 0.40 to 1.04 (0.40 to 0.96 in the TMG Aquifer), indicating that groundwater is likely to cause scaling rather than being corrosive.

Sulfate, which is aggressive to ordinary concrete when present in concentrations >200 mg/l is generally below 100 mg/l apart from a few higher outlier readings in BH THY-RP13 (Algoa Aquifer range from 18 to 34 mg/l and <100 mg/l in the TMG Aquifer) and this risk is therefore considered to be low.

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5.15.4.1.4 Seismic Conditions

Great detail pertaining to the probabilistic seismic hazard assessment (PSHA) for the site can be found in (Council for Geoscience, 2013) in which a Senior Seismic Hazard Analysis Committee (SSHAC) level 3 study was conducted.

Of specific interest to the geotechnical characterisation of the site are the outcomes of the PSHA pertaining to parameters required to conduct a liquefaction potential assessment in accordance with (National Centre for Earthquake Engineering Research, 2001). In essence, although (Council for Geoscience, 2013) specifically does not commit to a peak ground acceleration (PGA) or earthquake magnitude (M), but upper bounds can be derived from the document to be: PGA = 0.31g and M = M_{7.25}.

Other important aspects of the geotechnical character of site (e.g. impedance calculations) are exhaustively covered in (Council for Geoscience, 2013).

5.15.4.1.5 Other Geotechnical Information

Information from previous studies played an important role in the development of the geotechnical characterisation of the site. Of primary importance was the selected use of previous borehole logs and measured *in situ* data emanating from the Atomic Energy Corporation of South Africa Limited (AEC) studies (Atomic Energy Corporation of South Africa Limited, 1986) to (Atomic Energy Corporation of South Africa Limited, 1988) and (Norman, et al., 1987)). The boreholes represented in **Table 5.15.2** were used to inform the investigation planning for the intrusive investigations and the data contained in the borehole logs were used to supplement the geotechnical profile (**Subsection 5.15.5**).

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The pertinent data extracted from these historical boreholes for inclusion in the geotechnical characterisation were the following:

- sand overburden thickness;

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- depth and presence of the cobble layer;
- depth to bedrock and bedrock characteristics;
- rock joint sets as presented in **Table 5.15.3**.

These data (above bullets) were used to supplement the geotechnical profiles shown in **Drawings 5.15.3** and **5.15.4**.

Table 5.15.3
Historical Joint Sets

Joint No.	Strike (°) on Joints	Dip (°)	Comments
J1	030	85-90	Tensional, parallel to major compressive force, consistent strike, quartz veining, well developed
J2	170	80 E	Shear joint set, lesser development, minor variations in strike of between 20° and 30°
J3	055	75 – 80 NW	Shear joint set, well developed

5.15.4.2 Site Investigation for Supplementary Data

5.15.4.2.1 Investigation Planning and Process

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

The geotechnical site characterisation was carried out with reference to a quality data pack (see **Subsection 5.15.9**). 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 5.15.3 shows the geotechnical data gathering approach schematically, indicating the significance of investigation planning and refinement throughout. Refinement of the investigation approach was a feature of the geotechnical characterisation process.

The following refinements/issues should be noted:

- Development of the previous SSR (SRK Consulting, 2015):

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- The first peer review was used to identify existing and/or potential aspects that could make the site unsafe for the purposes of a nuclear installation.
- The second peer review was used to ensure that the timeline, objectives and quality of the geotechnical investigations were observed within the framework of the revised geotechnical process for the site (**Figure 5.15.3**).
- The third peer review was used to ensure that data gathered were optimally presented (in this SSR) to provide the necessary insight into the suitability of the site for development of a nuclear installation – This peer review consisted of ongoing interaction with the peer reviewer over an extended time period.
- This current version of the SSR (**Section 5.15**) required renewed interaction with the peer reviewer, essentially encompassing the following (pertaining to additional data that have contributed to the description of the geotechnical profile of the site in this SSR):
 - The additional boreholes drilled (and *in situ* testing data / laboratory testing data) in support of (Council for Geoscience, 2013) labelled New A to New Z have been used to update the geotechnical profile and the updated geotechnical profile has been peer reviewed – these data were not included in (SRK Consulting, 2015).
 - At the time of writing (SRK Consulting, 2015), seismic loading was not known for the site as the PSHA (Council for Geoscience, 2013) had not been finalised – data emanating from (Council for Geoscience, 2013) has now formed a critical part in updating the liquefaction potential assessment and this updated liquefaction potential assessment has been peer reviewed.

Approach changes to data exploration were made as follows (pre-2013 peer reviewer interaction):

- Intrusive investigation layouts were continually altered to accommodate environmental conservation, optimise data gathering (particularly with reference to the thick overburden soils encountered) and to accommodate

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this SSR timeline.

- The number of data gathering points was altered several times to accommodate additional drilling (number of boreholes and depths), include piezocone (CPTu) and seismic cone penetration tests (to gain an alternative opinion on liquefaction potential), to reduce marginally effective Dynamic Penetrometer Super Heavy (DPSH) data points and to remove shallow intrusive investigation activities (i.e. test pit excavations).
- Difficulties encountered in obtaining undisturbed soil samples on site were noted and appropriate measures instituted to investigate soils *in situ* (e.g. by utilising internationally accepted procedures in analysing Standard Penetration Tests (SPT) to assess liquefaction potential of soils).

Approach changes to analysis of results were made as follows (pre-2013 peer reviewer interaction):

- The highly jointed nature of the site rocks was noted in the course of the investigations and the need to extract detailed information on rock joint conditions from borehole core was introduced into the process through the second peer review.
- SPT results were analysed as a reliable means of assessing liquefaction potential of soils.

The geotechnical characterisation process (SRK Consulting, 2015) was then subjected to a final peer review at the time which brought about a number of features enhancing the section, including:

- placing an emphasis on providing an in principle assessment of the loading (demands) to be expected should a nuclear installation be developed on the site;
- expanding on the need to compare demands against the site capacity, thus forming the basis for developing this SSR (e.g. assessing a range of peak ground accelerations and the sensitivity of liquefaction potential to this range);
- suggestions to increase the value of the extensive database gathered to

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date by improving the presentation of data;

- independent quality assurance in general.

The current peer reviewer interaction therefore considers review of the inclusion of additional data to enhance the description of the geotechnical profile, and does not re-hash the efforts made in developing the previous version of the TSSR (SRK Consulting, 2015).

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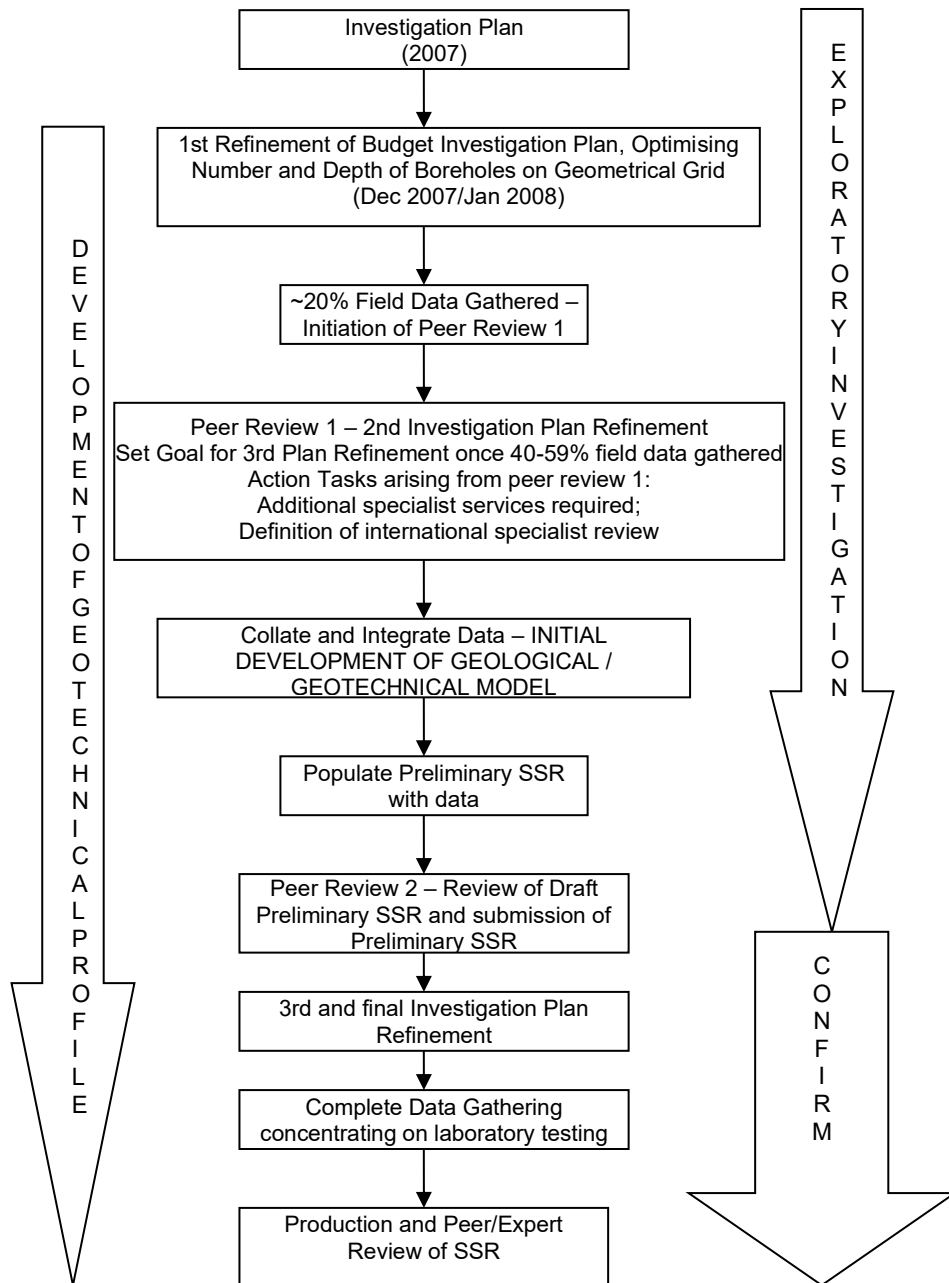


Figure 5.15.3
Geotechnical Process Applied for the Thyspunt Site

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Notes on Figure 5.15.3:

- Supervision and quality control on all aspects of field investigations was a feature of the data collation work carried out on the site.
- Drilling was carried out under the supervision of an experienced drilling foreman.
- Borehole locations were surveyed by a qualified land surveyor to ensure accurate georeferencing of data.
- Standards for drilling sampling and core logging are detailed in Subsection 5.15.4.2
- Sampling was carried out by experienced field technicians.
- Data analysis and reporting on data was carried out by an experienced engineer.

5.15.4.2.2 Intrusive Investigations

Intrusive geotechnical investigations were carried out to supplement the historical data, presented in Subsection 5.15.4.1. These historical data only provided insight into the geotechnical characteristics of the site along a relatively narrow coastal strip (approximately 300 m wide abutting the sea). The intrusive investigations presented in this section are complementary and covered a broader geographical area, with some boreholes being drilled as far inland as 1.2 km from the coast (Drawing 5.15.1).

(a) Spacing (Grid)

Initially, borehole drilling depths and positions were planned on a 220 m x 300 m grid with varying target drilling depths. Table 5.15.4 describes borehole spacing and depths. (International Atomic Energy Agency, 2004) gives general guidelines on borehole spacing and target depths with specific reference to safety-related structures. In general, the following guidelines applied:

- 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:

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- 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.

Details of the nuclear installation and finalised locality of safety-related structures were not specifically known at the time of producing (SRK Consulting, 2015). An illustrative footprint of the nuclear island is, however, now available. Data initially collated (primarily drilling) was planned on a grid system (220 m x 300 m) with varying target drilling depths to describe the broader site geotechnical profile. These data are now supplemented with the drilling campaign executed in support of (Council for Geoscience, 2013), and the position of the Illustrative Nuclear 1 Footprint is considered.

The pre-operational and operational stages of the project will target specific boreholes and borehole depths as set out in (International Atomic Energy Agency, 2004).

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 and;
- the borehole collar elevations with respect to a permanent benchmark.

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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;
- shear and compression wave velocity measurements obtained from compression and shear wave (PS) Suspension Logging;
- per cent recovery of rock core;
- quantity of core not recovered for each core interval or drill run;
- rock quality designation;
- laboratory test results for rocks and soils.

The intended and actual boreholes drilled, the defining target and the actual drilling depths are summarised in **Table 5.15.4**. The table also provides information about the positioning and drilling depths, as well as on any variations from initially planned activities.

(b) Drilling Procedures

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

- All boreholes were vertical.
- All drilling was of N (60 mm – 60 mm core diameter) size, employing the use of diamond or tungsten bits.
- 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

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sand and gravel 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 5.15.5** and the actual programme achieved in **Table 5.15.6**.

(c) *In situ* Testing

Standard Penetration Tests (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 rock/cobbles/bedrock and provide extensive data laterally across the site and with depth. This resulted in recovery of approximately 0.5 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 exceeds 25.

DPSH tests were attempted 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 sand/calcrete horizons.

Eleven piezocone (CPTu) probes were carried out as indicated in **Drawing 5.15.1**.

Finally, PS Suspension Logging was done on the boreholes drilled to support (Council for Geoscience, 2013) and details pertaining to the quality assurance are contained in that reference.

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(d) Sampling

Undisturbed sampling within soil horizons at the site was challenging as sample recovery was inhibited by the cohesionless nature of the soils and by the presence of groundwater. Re-moulded samples representative of the site soils were recovered from a test pit (see **Drawing 5.15.1**) for triaxial laboratory testing. Disturbed soil samples were obtained at regular intervals from borehole core and SPT samples for soil indicator testing and provide an extensive assessment of the soil grading and Atterberg Limits laterally and vertically across the site. These data proved invaluable in conducting the liquefaction potential assessment (see **Subsection 5.15.4.2.4**).

Rock samples were recovered, but sampling was restricted by the highly fractured nature of the material and the difficulties that this introduced in sample integrity during transport to laboratories. It is acknowledged that sampling bias may exist favouring more intact rock core. All rock samples were transported as 'fragile' items. Reasonable representation of the site rock characteristics was obtained through sampling in a high percentage of boreholes and in different geological formations.

(e) 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. 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 storage in cores sheds.

(f) 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 core boxes. Since this SSR is produced in the context of the geotechnical investigation stage of characterisation, no specific borrow or

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construction material samples were targeted. The suitability of these materials will be investigated in the pre-operational and operational stages.

In maintaining pace with the project programme, sample storage time was limited wherever practically possible and transport under the chain of custody expedited.

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*borehole elevation not land surveyed – all remaining boreholes land surveyed and elevations are shown in column marked z

BH= borehole

TB= Borehole ID prefix – these boreholes constituting an initial drilling phase at the site.

NEW = Borehole ID prefix – these boreholes constituting an extended drilling phase at the site.

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Table 5.15.5
Other Intrusive Geotechnical Investigations Carried Out

Activity	Purpose	Relevant Standard/Method
Standard Penetration Tests (SPT)	Explore <i>in situ</i> 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)
PS Suspension Logging	Vs and Vp profiling	See (Council for Geoscience, 2013)
Dynamic Penetrometer Super Heavy Tests (DPSH)	Explore <i>in situ</i> 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)
Cone Penetrometer with Porewater Pressure (piezocone) Measurement Tests (CPTu)	<i>In situ</i> static liquefaction potential assessment and <i>in situ</i> soil consistency	In accordance with international reference test procedure laid down in proceedings of First international Symposium on Penetration Testing (ISOPT), 1988 (Campanella & Robertson, 1988)
Seismic Cone Penetrometer Measurement Tests	<i>In situ</i> liquefaction potential assessment and <i>in situ</i> soil shear wave velocity measurement	No international standard is yet available, but is in process of being drafted
Excavation of Test Pits	Provide samples for laboratory static liquefaction potential assessment	–
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

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Exploratory Method	ID	Coordinates			Depth		Remarks
		X	Y	Z	Planned	Actual	
DPSH	TDP31	-3784610.5	-26785.9	N/A	Refusal/Bedrock	N/A	Omitted due to consistent shallow
DPSH	TDP32	-3784693.9	-26582.4	N/A	Refusal/Bedrock	N/A	Omitted due to consistent shallow
DPSH	TDP33	-3784054.3	-28405.3	N/A	Refusal/Bedrock	N/A	Omitted - inaccessible dune area
DPSH	TDP34	-3784503.23	-28195.5	18.562	Refusal/Bedrock	N/A	Omitted - inaccessible dune area
DPSH	TDP35	-3784220.91	-27948.4	44.63	Refusal/Bedrock	7.5	Presumed refusal on
DPSH	TDP36	-3784252.4	-27776.0	N/A	Refusal/Bedrock	N/A	Omitted due to consistent shallow
DPSH	TDP37	-3784343.7	-27153.1	N/A	Refusal/Bedrock	N/A	Omitted due to consistent shallow
DPSH	TDP38	-3784423.68	-26950.79	24.051	Refusal/Bedrock	4.4	Presumed refusal on
DPSH	TDP39	-3784510.7	-26746.1	N/A	Refusal/Bedrock	N/A	Omitted due to consistent shallow
DPSH	TDP40	-3784595.67	-26543.85	17.997	Refusal/Bedrock	7.4	Presumed refusal on
DPSH	TDP41	-3784036.0	-28284.3	N/A	Refusal/Bedrock	N/A	Omitted - inaccessible dune area
DPSH	TDP42	-3784194.26	-27870.25	58.59	Refusal/Bedrock	6.2	Presumed refusal on
DPSH	TDP43	-3784349.84	-27029.66	28.216	Refusal/Bedrock	5.3	Presumed refusal on
DPSH	TDP44	-3784502.5	-26624.5	18.562	Refusal/Bedrock	4.5	Presumed refusal on
DPSH	TDP45	-3784204.6	-28106.8	N/A	Refusal/Bedrock	N/A	Omitted - inaccessible dune area
DPSH	TDP46	-3784515.03	-26868.38	18.368	Refusal/Bedrock	5.6	Presumed refusal on
DPSH	TDP47	-3784709.28	-26977.76	16.717	Refusal/Bedrock	6.4	Probable refusal on
DPSH	TDP48	-3784412.26	-28170.63	18.048	Refusal/Bedrock	6.6	Probable refusal on
Piezocones (CPTu)	PR1	-3784318.2	-28598.7	18.7	Refusal/Bedrock	6.8	Presumed refusal on
Piezocones (CPTu)	PR2	-3784312.4	-28138.7	29.7	Refusal/Bedrock	5.5	Presumed refusal on
Piezocones (CPTu)	PR3	-3784520.9	-27742.5	14.1	Refusal/Bedrock	7.2	Probable refusal on
Piezocones (CPTu)	PR4	-3784550.8	-27349.9	22.5	Refusal/Bedrock	8.9	Presumed refusal on
Piezocones (CPTu)	PR5	-3784623.1	-26906.4	24.7	Refusal/Bedrock	12.2	Presumed refusal on
Piezocones (CPTu)	PR6	-3784040.2	-26429.9	27.4	Refusal/Bedrock	4.5	Presumed refusal on
Piezocones (CPTu)	PR7	-3784351.2	-27272.9	29.2	Refusal/Bedrock	12.9	Presumed refusal on
Piezocones (CPTu)	PR8	-3784413.8	-26822.6	24.1	Refusal/Bedrock	4.1	Presumed refusal on
Piezocones (CPTu)	PR9	-3784481.4	-26378.6	18	Refusal/Bedrock	15.3	Presumed refusal on
Piezocones (CPTu)	PR10	-3783970.3	-27050.6	46	Refusal/Bedrock	3.5	Presumed refusal on
Piezocones (CPTu)	PR11	-3784018.5	-26665.4	39	Refusal/Bedrock	12.6	Presumed refusal on
Seismic Probes	PR1	-3784318.2	-28598.7	18.7	Refusal/Bedrock	3.7	Presumed refusal on
Seismic Probes	PR4	-3784550.8	-27349.9	22.5	Refusal/Bedrock	3.8	Presumed refusal on
Seismic Probes	PR5	-3784623.1	-26906.4	24.7	Refusal/Bedrock	9.2	Presumed refusal on
Seismic Probes	PR6	-3784040.2	-26429.9	27.4	Refusal/Bedrock	3.2	Presumed refusal on
Seismic Probes	PR7	-3784351.2	-27272.9	29.2	Refusal/Bedrock	5.2	Presumed refusal on
Seismic Probes	PR10	-3783970.3	-27050.6	46	Refusal/Bedrock	2.2	Presumed refusal on

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Notes:

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

Probable refusal indicates a higher likelihood that 'presumed refusal' as other data (e.g. adjacent boreholes) lend credibility to the presence of bedrock/cobbles/dense sand.

5.15.4.2.3 Laboratory Investigations

To confirm field investigation data collected, a laboratory testing programme for soils and rocks was implemented. It was developed in compliance with the relevant international standards (United States Nuclear Regulatory Commission, 2003) and (American Society for Testing and Materials International, 2008), (British Standards Institution, 1990), (International Society for Rock Mechanics, 1979), and (International Society for Rock Mechanics, 2007). Soil and rock samples obtained from the intrusive investigations were subjected to a laboratory testing programme based on the parameters recommended in **Table 5.15.7** and the resulting testing programme is detailed in **Table 5.15.8**. This section covers the specifications used for the laboratory investigations and the methods by which they were carried out.

The scope and objectives of the programme, as well as the procedures applied for sampling, storage, equipment, handling of test specimens and analysis of results are presented in this section.

The purpose of the laboratory testing programme was to identify and classify soils and rock from and around 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. 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. The physical properties of soil and rock were determined through laboratory testing.

Sample representativeness proved to be a challenge at the site particularly within the site rocks that are highly fractured. Prescribed laboratory testing methods (**Table 5.15.8**) require rock samples at least 200 mm in length. The sampling process was affected by the available lengths of intact core (see **Appendix 5.15.C**) and this resulted in sampling of less frequently jointed core

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sticks. This may have biased the results to represent stronger rather than weaker rock.

The tests performed were:

- classification tests,
- engineering properties tests.

Classification tests and the evaluation of engineering properties of soil and rock are generally governed by the regime of parameters and tests presented in **Table 5.15.7** (United States Nuclear Regulatory Commission, 2003)

Table 5.15.7
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
Specified Gravity	Grain Size Analysis	Triaxial Compression
Erodibility	Compaction	Slake Durability
Carbonates and Sulphates content*	Salt content*	Specific Gravity
Oedometric, Young modulus*		

* (Eskom, 2022)

The tests undertaken in this SSR investigation were based on the limited knowledge and understanding of the design of the planned nuclear installation(s). In the characterisation/verification stage of the geotechnical investigation, it is generally assumed that broadly generalised layouts and loads have been established (**Subsection 5.15.3.2**). However, no specific

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structure layouts or loading information was available at the time of developing this SSR.

The target parameters in **Table 5.15.7** were gathered through intrusive investigation methods (**Subsection 5.15.4.2**) and laboratory investigations generally aligned with the programme shown in **Table 5.15.8** and further expanded on in **Subsection 5.15.4.3**.

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Table 5.15.8
Laboratory Testing Programme

Laboratory Test	Purpose	Standard/method
Soil		
Soil Characterisation Testing	Characterisation of the site soils including full hydrometer analysis	ASTM D422 (International Atomic Energy Agency, 2004)
	Characterisation of the site soils	BS1377: Part 2: 1990 9.2 (British Standards Institution, 1990)
Atterberg Limits	Standard determination of Atterberg Limits (Liquid Limit, Plastic Limit and Plasticity Index)	TMH1 A2, A3 and A4 (Committee of State Road Authorities, 1986a)
	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)
Triaxial Testing	Measurement of critical state soil parameters (λ , κ , M)	Project specific triaxial testing specification (see Appendix 5.15.G)
Permeability Testing in Triaxial Apparatus	Measurement of site soil permeability to supplement the assessment presented in Section 5.11 of this SSR	Project specific triaxial testing specification (see Appendix 5.15.G)
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)

5.15.4.2.4 Liquefaction Potential Assessment

Analysis of the field investigation and laboratory testing programme results indicated a presence of soils with low fines content that are saturated and hence could exhibit a liquefaction potential. This is primarily supported by the fact that

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confined strata consisting of non-cohesive soils of variable consistency were identified below the groundwater table by SPT tests.

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

During the intrusive investigations, 1 080 SPT tests were conducted on the site soils. These data provide a good geographical representation of the nature of the site soils. Based on the methodologies recommended in NCEER (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 $(N_1)_{60}$ blow counts were used to calculate the Cyclic Resistance Ratio (CRR)¹² for each SPT test. The CRR represents the capacity of the site soils to withstand liquefaction under certain loads (demands) introduced by some form of trigger (in this instance, ground shaking due to seismic activity).

The Cyclic Stress Ratio (CSR)¹³, representing loads from ground shaking (or demand) were calculated for a Peak Ground Acceleration (PGA) value of 0.31 g and for an earthquake magnitude of $M_{7.2}$ (Council for Geoscience, 2013).

¹² As a function of corrected SPT N value and hypothetical earthquake magnitudes.

¹³ As a function of hypothetical PGA and the stress environment in which the SPT tests were carried out.

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Assessing the CSR against the CRR enables a comparison of the site soils capacity to withstand liquefaction under of the anticipated dynamic loads resulting from earthquakes.

A prerequisite for liquefaction to occur in soils is that the soils must be saturated (i.e. be located below the groundwater table). The details of proposed foundations for the nuclear installations are not yet known and proposed construction methodology is also not known. Since construction methodology could involve implementing subsurface hydraulic barriers to support dewatering design, there may be scenarios developing during construction that alter the groundwater table (upwards) in certain areas on the site (e.g. up-gradient of hydraulic barriers, the groundwater table may rise). For this reason, the liquefaction potential assessment carried out for this SSR presents assumes that liquefaction could in fact occur in both currently saturated and unsaturated soils that may become saturated.

To test the robustness of the analyses presented in **Subsection 5.15.4.3.4**, alternative methods for assessing liquefaction potential were explored in accordance with (National Centre for Earthquake Engineering Research, 2001). These alternatives assessed the liquefaction potential from SPT tests and shear wave velocity measurements.

5.15.4.2.5 Surface Investigations

Surface investigations were limited to a single test pit excavated to obtain samples for triaxial laboratory testing. The excessive thickness of the sand overburden at the site precluded any extension of surface investigations at the site as drilling data were preferred to explore the characteristics of the extensive overburden to depth.

Limiting surface investigations in lieu of drilling does not impact on this SSR assessment of the site suitability, but rather enhances this SSR assessment by ensuring a better understanding of the soil characteristics at the site.

5.15.4.2.6 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:

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- 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 ongoing 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.

The site currently does not have any human induced conditions that could impact on site safety in the geotechnical context with specific reference to the above and are not expected to be imposed during the life of the proposed nuclear installation(s). This statement is supported by the currently undeveloped nature of the site and the mandatory exclusion zones that are implicit in the development of nuclear sites for the future.

5.15.4.2.7 Summary of Current and Future Investigations

(a) Current Investigations (Characterisation Stage)

A geotechnical characterisation programme with a geographical focus on the site was launched by Eskom Holdings (further referred to as Eskom) in October 2007 to supplement the historical investigations carried out at selection stage. This programme, focussed on the verification and, where possible, the confirmation stages and included:

- review of available information and in particular reports and maps of the AEC (Atomic Energy Corporation of South Africa Limited, 1986) to (Atomic Energy Corporation of South Africa Limited, 1988);
- intrusive field investigations including:
 - drilling of geotechnical core boreholes;
 - SPT tests in soils;

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- PS Suspension Logging to measure Vs and Vp;
- point load testing of rocks;
- DPSH penetration tests;
- excavation of a test pit.
- interpretation of field information in the form of detailed borehole logs and rock joint condition assessments;
- liquefaction¹⁴ potential investigations that comprised of:
 - analysis of SPT tests and Vs measurements in site soils;
 - seismic cone penetration field tests;
 - laboratory tests.
- laboratory testing of selected soil and rock samples, including:
 - soil characterisation tests with grading analyses, hydrometer and Atterberg Limits¹⁵ analyses;
 - rock uniaxial compressive strength (UCS) testing with measurement of Young's Modulus (E) and Poisson's ratio (ν) and rock bulk and dry density.
- monitoring of groundwater levels and integration with monitoring results presented in **Section 5.11**;
- analysis of data obtained from field and laboratory sources;

¹⁴ 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.

¹⁵ 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.

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- review of the above processes by an independent peer reviewer.

(b) Pre-operational Stage

Investigations will continue from the start of the construction until the start of the operation of the proposed nuclear installation(s). The aim is to complete and refine the assessment of the site characteristics with new and up-to-date data obtained during the excavation and construction of the nuclear installation(s) foundations (International Atomic Energy Agency, 2004).

(c) Operational Stage

During the operation of the nuclear installation, the settlement of structures and water table levels will be measured for the foundations (International Atomic Energy Agency, 2004). These data will be compared with predictions to enable an updated safety assessment for the nuclear installation to be made. The type of parameters, the frequency of their measurements and all related activities will be addressed in the maintenance programme of the nuclear installation foundations (International Atomic Energy Agency, 2004). An example would be the monitoring of the integrity of cement stabilised soil rafts should these be employed in the foundation layerworks as was done at the Koeberg Nuclear Power Station.

5.15.4.3 Data Presentation and Interpretation

5.15.4.3.1 Results of Intrusive Investigations

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

- borehole logs in **Appendix 5.15.B**;
- borehole core photographs in **Appendix 5.15.C**;
- joint condition logs in **Appendix 5.15.D**;
- SPT test result plots in **Appendix 5.15.E** and individual results in **Appendix 5.15.B**;

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- DPSH test result plots in **Appendix 5.15.F**;
- CPTu and seismic cone penetration test results in **Appendix 5.15G**;
- Point Load test results in **Appendix 5.15.B**.

(a) Soil Profile

The geographical distribution of the soils at the site can be summarised as follows (see also **Drawings 5.15.1**, **5.15.2**, **5.15.3** and **5.15.4**):

- At the sea, the sand thickness approaches zero as evidenced by the extensive rock outcrop along the coastline (except for Thysbaai where beach deposits are evident).
- Further inland, there is a steep topographical rise into an environmentally sensitive coastal dune system in the west (where drilling access was not possible due to high environmental damage and general access risks) and the sand is inferred to be up to 35 m thick within 400 m from the coastline.
- Pedogenic zones are encountered frequently, possibly manifesting in a zone of fluctuating groundwater table as indicated on the cross sections – it is noted from **Section 5.11** that the Algoa Aquifer has high concentrations of $\text{Ca}(\text{HCO}_3)_2$ (in all likelihood originating from the extensive shell deposits in the site soils) that may contribute to the development of calcrete.
- Gravel/pebble/cobble deposits ¹⁶are encountered in wave-cut depressions in the Goudini and Cederberg rocks in particular, but occasionally in the Skurweberg rocks.
- general observations:
 - The wave-cut platform level dips gently towards the east in the direction of Thysbaai.
 - The depression is a shallow trough postulated to extend to the west

¹⁶ Referred to generally as 'the cobble layer' in this SSR and in **Section 5.11**

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(above the Goudini Formation) in the inaccessible dune area.

- Groundwater generally flows to the south, southeast, this flow direction being influenced by the pebble/cobble layer predominantly in the Goudini depression.
- The two benches in the dune system undulate in surface elevation in the east– west direction.

(i) *In Situ* Density

Borehole samples from the drilling programme were logged and colour, consistency (guided by SPT tests), soil type (guided by laboratory testing, see **Subsection 5.15.4.2.3**), structure and origin were documented (see **Appendices 5.15.B** and **5.15.C**). The site soils are dominated by aeolian sand deposits but in places, are underlain by marine sands and a cobble horizon resting on a wave cut bedrock terrace with pedogenic horizons randomly distributed but trending to a zone where the groundwater table fluctuates.

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Uncorrected SPT plots representative of the site (from (SRK Consulting, 2015)) are presented in **Appendix 5.15.E** (where they are superimposed on the position of the groundwater table extracted from the measurements summarised in **Table 5.15.11**). Individual SPT results are shown on the borehole logs in **Appendix 5.15.B**. A marked increase in N value (to refusal) is observed in the vicinity of the groundwater table. This is evidence of calcrete horizons forming within a zone where groundwater fluctuation has presumably occurred. The high CaCO₃ concentrations in a shell rich environment (see **Subsection 5.15.4.1.3**) support this interpretation.

Of significance is the fact that below the groundwater table there is a trend noted where a reduction in SPT N value occurs (e.g. values often measured below SPT N = 20), indicating loose/looser zone/s where, by inference, calcrete is not present¹⁷. Thereafter no obvious pattern is observed showing an increase in soil consistency with depth as would normally be expected. Instead, random zones of competent material and intermittent less competent material are evident. DPSH tests refused in the calcrete and/or very dense soil horizons and were marginally effective in the geotechnical investigation. The DPSH plots do, however, clearly show an increase in soil consistency from surface towards the groundwater table. Detailed plots are presented in **Appendix 5.15.F**.

Refinement of the geotechnical characterisation process was strongly influenced by the variability in site soil consistency – particularly in soil horizons below the groundwater table. Representative sampling proved practically impossible without employing extreme techniques such as *in situ* freezing of soil. SPT tests and Vs profiling therefore became the primary source of characterisation data backed up by index tests of disturbed samples. An *in situ* liquefaction potential assessment was commissioned in the investigative process (see **Subsection 5.15.4.2.4**) to interrogate the use of SPT data for liquefaction potential assessments and included CPTu tests, seismic probe tests. PS suspension logging was later carried out to support (Council for Geoscience, 2013).

¹⁷ Due to the practical difficulties of drilling in cohesionless soils beneath the groundwater table at depth, a caveat is that some SPT tests may have been conducted in sand that had fallen into boreholes, potentially giving spurious results.

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(ii) Shear Wave Velocity

Shear wave velocities in the site soils are depicted in **Drawing 5.15.6** and **Drawing 5.15.7**.

(b) Rock Profile

Borehole core was logged (**Appendix 5.15.B**), photographed (**Appendix 5.15.C**) and subjected to point load testing to calibrate physical description 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 process 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 an undulating wave-cut platform. Quartzitic sandstones of the Skurweberg Formation occur at the sea underlain by a Skurweberg/Goudini quartzitic sandstone transition, followed by the Goudini Formation quartzitic sandstone. The sequence then continues inland, with the Cedarberg Formation and ultimately the Peninsula Formation (see **Section 5.13**) – the Skurweberg is the youngest formation in the sequence and the Peninsula Formation the oldest. Geotechnical core drilling was performed within all of these formations (but with data density highest in the Skurweberg and Goudini formations) and the geotechnical characteristics, as described below, dominate.

(i) Wave-cut Platform

The wave-cut platform has the following physical characteristics as presented in **Drawings 5.15.1, 5.15.2, 5.15.3** and **5.15.4**:

- in the western portion of the site:
 - Within the Skurweberg, the platform is between 10 and 15 m msl at the coast and up to 500 m inland.
 - Thereafter, an environmentally sensitive dune system ruled out any drilling and no confirmation of the wave-cut terrace level is available.
 - To overcome this data gap, data were interpolated from the

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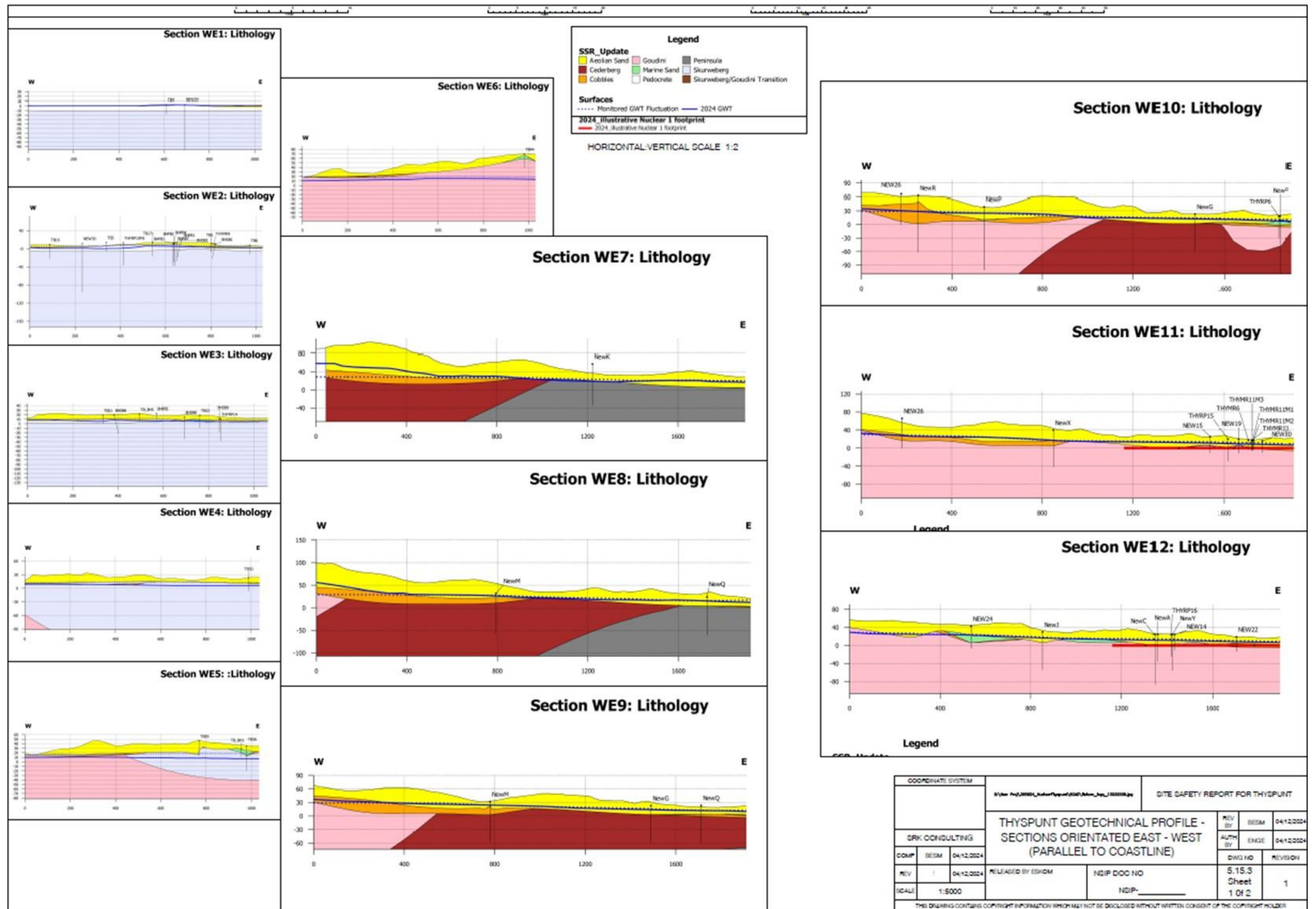
geohydrological and geotechnical investigations to estimate these platform levels – the interpolated results propose that the platform remains at approximately 10 to 15 m msl. However, it is postulated that there may be a continuation of the depression in the Goudini rocks as indicated in **Drawing 5.15.2**.

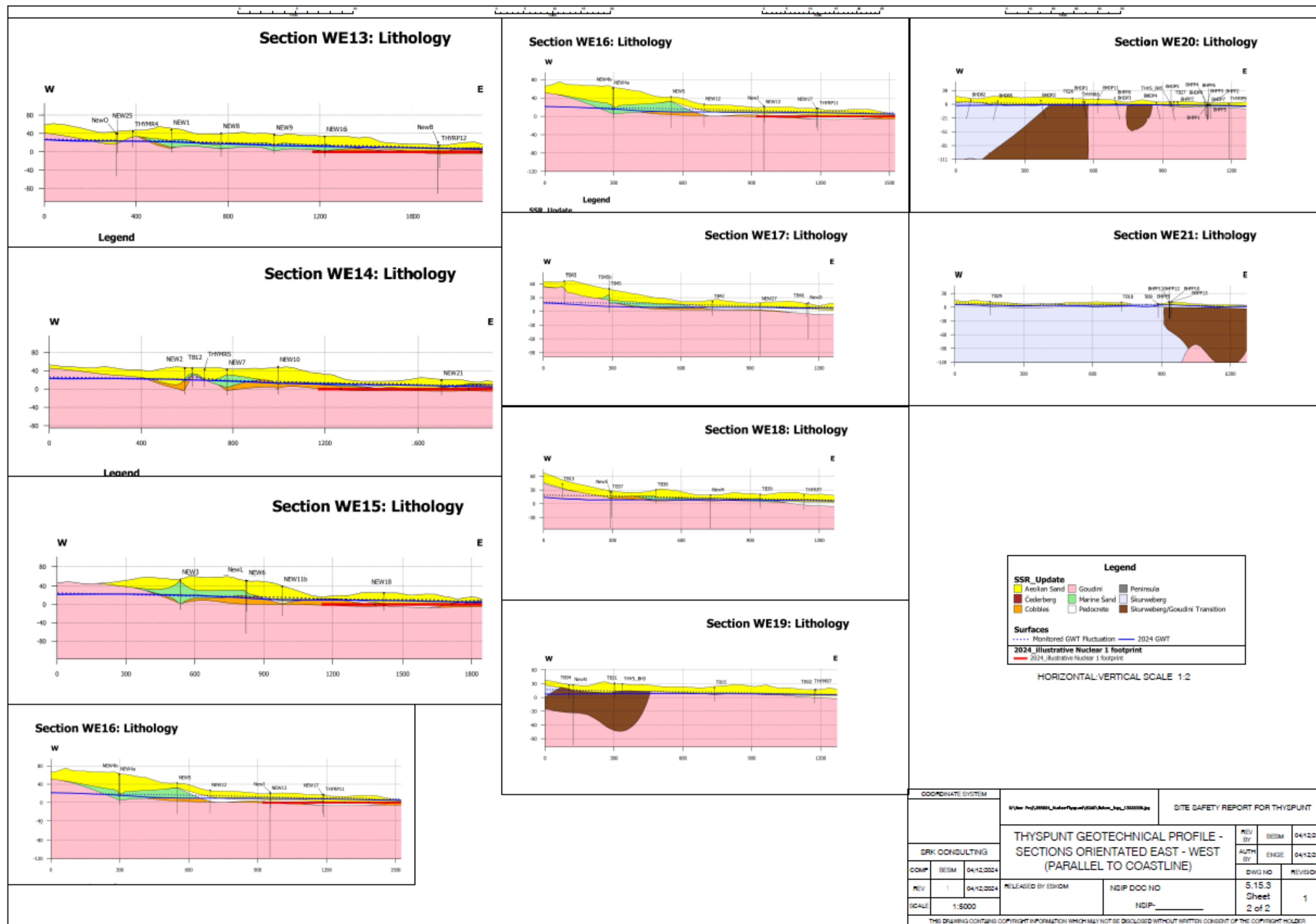
- Random dips/depressions in the platform terrace level carry marine deposits consisting of pebbles and cobbles in a sand matrix.
- The undulating wave cut terrace elevation is an important feature of the site with respect to any proposed tunnelling design.
- in the central portion of the site:
 - In the Skurweberg rocks, the platform level is situated at just above mean sea level to +5 m msl within a thin coastal strip (500 – 700 m wide).
 - It then dips down to mean sea level (or just below at -3 m msl) within the Goudini rocks to about 900 m inland – This depression carries a pebble/cobble horizon with a sand matrix.
 - A gradual increase in bedrock elevation then occurs as the profile extends into the Peninsula rocks – In this zone, platform levels are unconfirmed but interpolated between geotechnical and geohydrological borehole logs, the platform elevation rises to at least +20 m msl (see **Section 5.11**).
 - The undulating wave-cut terrace elevation is an important feature of the site with respect to any proposed tunnelling design.
- in the east:
 - Within the Skurweberg Formation, the level of the platform is at or just below mean sea level (+5 m msl to -2 m msl) with a few depressions carrying a pebble/cobble layer.
 - Thereafter, and moving into the Goudini rocks, the platform level decreases to below mean sea level, the lowest point along N/S11 being

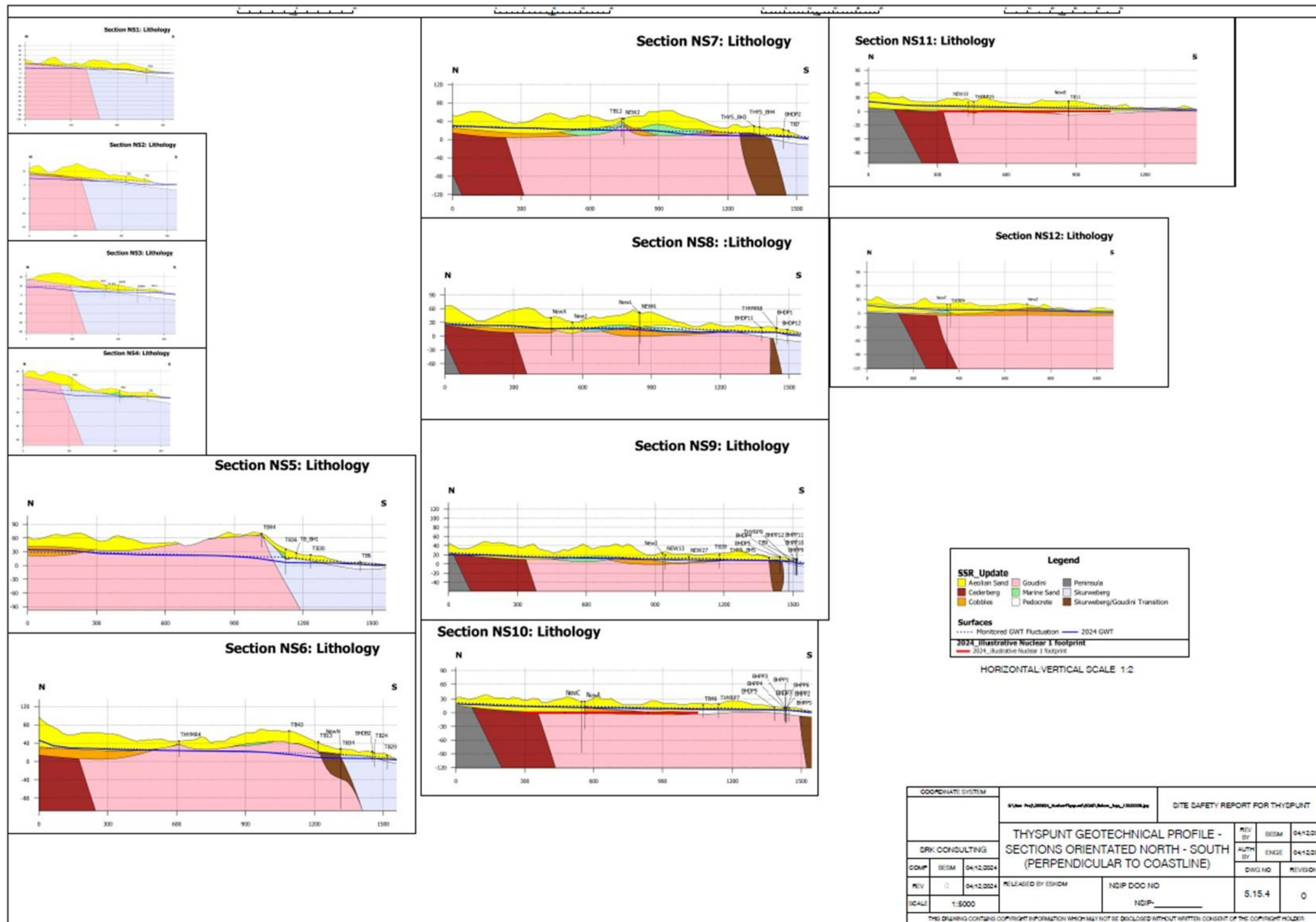
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at -6 m msl.

- Further inland, the wave-cut terrace gradually rises towards the Cederberg Formation, continuing this trend into the Peninsula Formation where confirmed bedrock levels are as elevated as 24 m msl. It appears as if this rise in bedrock level may extend further inland within Peninsula rocks.
- Particularly in the Goudini and Cederberg formations, depressions exist that host a highly transmissive cobble horizon – this horizon is established enough to influence groundwater flow directions on the site.
- The undulating wave cut terrace elevation is an important feature of the site with respect to any proposed tunnelling design.







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(ii) Skurweberg-Goudini Contact

More detailed regional delineation of the site geology and the contact between the Skurweberg and Goudini formations is contained in **Section 5.13**. Coastal mapping (see **Section 5.13**), historical drilling (Atomic Energy Corporation of South Africa Limited, 1986) to (Atomic Energy Corporation of South Africa Limited, 1988) and (Norman, et al., 1987) and the geotechnical drilling programme for this SSR all indicate that the Skurweberg/Goudini contact could be described as a deposition transition zone (see **Drawing 5.15.3**). The transition is not of sufficient extent (horizontally) to have been specifically targeted in the intrusive field investigations or laboratory testing programme (**Subsection 5.15.4.2.3**).

(iii) Point Load Index

Summary statistics of point load test results carried out on geotechnical core are shown in **Table 5.15.9**. Results are expressed as UCS calculated from point load index, (PLI). Point load tests were carried out during the fieldwork programme and the results are presented in **Appendix 5.15.B**. Point load tests are an unreliable means of assessing rock strength and they were carried out to calibrate visual description of the site rocks only.

Table 5.15.9
Statistics of Point Load Test Results – Summary

(a)Skurweberg and Goudini PLS Statistics

Parameter	Measured Parallel to Bedding (MPa)		Measured Perpendicular to Core Axis and at 60 ° to Bedding (MPa)	
	Skurweberg	Goudini	Skurweberg	Goudini
Sample ¹⁸	233.0	105.0	78.0	79.0
Range	413.5	272.2	284.0	306.3
Minimum	13.6	0.0	22.2	0.0
Maximum	427.1	272.2	306.3	306.3

¹⁸ Number of tests carried out.

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(b) Remainder of the TMG Formations and Overall Statistics

Parameter	Skurweberg	Skurweberg / Goudini Transition	Goudini	Cederberg	Peninsula
Sample	365.0	16.0	532.0	35.0	61.0
Range	417.4	243.1	347.2	333.3	291.7
Minimum	9.7	13.9	0.0	13.9	55.6
Maximum	427.1	256.9	347.2	347.2	347.2

Point load tests were carried out to assist in describing the rocks in the borehole logs (**Appendix 5.15 B**). Amongst the dominant joints shown in **Table 5.15.3**, there are also the bedding joints. In accordance with testing procedures (American Society for Testing and Materials International, 2008), point load tests were carried out perpendicular to the core axis and at 60° to the axis and parallel to bedding planes and the results in **Table 5.15.10** differentiate as such where sufficient data warrant (i.e. in the data rich Skurweberg and Goudini Formations). The highly jointed nature of the site rocks is a feature that certainly impacts the site rock geotechnical characteristics.

Comments on the point load test results are:

- As the point load tests were primarily used to accurately describe the rocks in the logs (**Appendix 5.15.B**), the results carried out perpendicular and parallel to the bedding planes do not reveal any notable trend bar indicating a wide range in results typical of sedimentary deposits.
- Test results trend to a rectangular distribution (see **Figure 5.15.4**) which is to be expected in sedimentary rocks as the depositional and tectonic environment influences rock strength.
- The point load test results in the Skurweberg and Peninsula rocks envelop the other rocks.
- Besides informing physical descriptions and indicating potential anisotropy¹⁹ (**Appendix 5.15.B**), the point load test results lend no other

¹⁹ Anisotropy is the property of being directionally dependent.

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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 test being carried out on samples less frequently jointed and may have biased the results to represent stronger rather than weaker rock.

In summary, the site bedrock surface consists of an undulating wave-cut platform variably positioned with respect to mean sea level and dipping off into a depression in the Goudini and Cederberg rocks which are marginally less competent (because they appear to be core jointed) (**Figure 5.15.4**). This depression contains marine pebble/cobble deposits. A rise in elevation of the wave-cut platform then occurs either side of the Goudini rocks in the Skurweberg (to the south) and Peninsula (to the north) rocks. There is evidence that the geological contact between the Skurweberg and Goudini formations is a transition zone also indicating lower UCS values.

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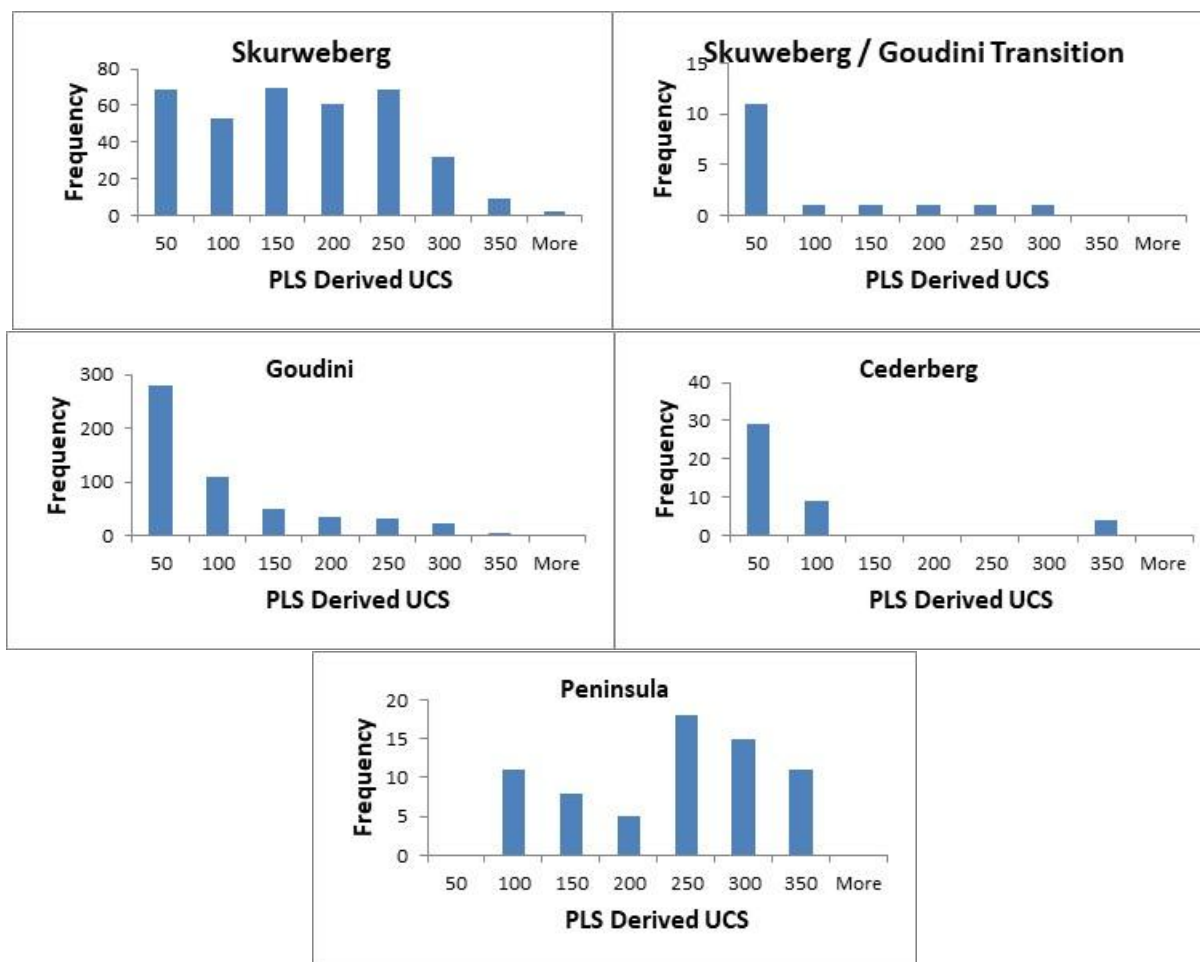


Figure 5.15.4: Point Load Derived UCS Histograms

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(iv) Rock Joint Condition

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 5.15.3**) recognised this and a greater emphasis was placed on more detailed description of the rock joint conditions. Although peer review guidance was in support of (SRK Consulting, 2015), this approach was also adopted in the additional drilling done in support of (Council for Geoscience, 2013).

Boreholes were re-logged with the aim of describing joint conditions in greater detail and these logs are included in **Appendix 5.15.D**. A total of 2 961 joints were described in the Skurweberg Formation rocks and 5 054 in the Goudini Formation rocks culminating in the bulk of the rock joint assessment database. In contrast, 1 609 and 586 joints were assessed in the Cederberg and Peninsula Formations respectively.

Table 5.15.10 summarises the joint conditions from the voluminous information contained in **Appendix 5.15.D**. **Table 5.15.10** represents statistics on the entire data set contained in **Appendix 5.15.D**.

Figure 5.15.5 shows a histogram of the fracture spacing and **Figure 5.15.6** shows a histogram of the rock quality designation (RQD). Once again, these figures represent 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 including:

- carrying out a statistical analysis on the fracture spacing data after grouping these data into the individual joint sets and into rock type.
- weighting the measurements to cater for the fact that, e.g. a ten metre zone of no joints cannot be treated the same as e.g. a five metre zone with five joints per metre – Weighted averages were then calculated using the length of the individual geotechnical intervals logged (see **Appendix 5.15.D**), divided by the total length of all the intervals for that rock type.

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Based on the weighted joint spacing for each logged set, the blocks that will form as a result of the intersecting joints will be approximately 170 mm long by 170 mm wide by 1 200 mm thick in the Skurweberg Formation and 160 mm wide by 160 mm long by 1 000 mm thick in the Goudini Formation. The fracture spacing was corrected for borehole sample bias prior to the calculation of these weighted averages.

Utilising these data, (Laubscher, 1990) was then used to calculate rock mass ratings (RMRs) in the Skurweberg and Goudini Formation rocks as these rocks underlie the Illustrative Nuclear 1 Footprint (dominated by the Goudini Rocks). Means were calculated based on weighting each 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 5.15.11**.

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Table 5.15.10
Summary of Rock Joint Conditions

Parameter	Skurweberg Formation			Goudini Formation			Cederberg Formation			Peninsula Formation			Remarks
	Min	Ave.	Max	Min	Ave.	Max	Min	Ave.	Max	Min	Ave.	Max	
Fracture Frequency (F/m)	0	6	21	0	3	40	0	6	18	0	3	13	Fracture spacing denoting the linear distance between joints
RQD (%)	0	34	96	0	14	104	0	7	65	0	22	80	Rock quality designation = % intact core > 100 mm in length in a Geotech. Interval/drill run
Recovered Core													
% solids	92%			78%			73%			94%			'Solids' denoting recovered core that is still in an intact cylindrical state
% matrix	8%			22%			27%			6%			'Matrix' denoting recovered core consisting of broken rock fragments
Joint Distribution													
	1 sub-hor	2 Bed	3 sub-vert	1 sub-hor	2 Bed	3 sub-vert	1 sub-hor	2 Bed	3 sub-ver	1 sub-hor	2 Bed	3 sub-vert	
Number Logged	428	1622	911	1611	2221	1222	374	1224	11	59	185	49	The physical number of each joint set logged
Percentage (%)	14%	55%	31%	32%	44%	24%	23%	76%	1%	20%	63%	17%	The % that each joint set constitutes of the total number of joints logged
Joint Surface Condition													
Macro (%)	1 sub-hor	2 Bed	3 sub-vert	1 sub-hor	2 Bed	3 sub-vert	1 sub-hor	2 Bed	3 sub-vert	1 sub-hor	2 Bed	3 sub-vert	Describing the macro joint condition and denoting this as a % of the total number of joints logged
Straight	43%	36%	7%	11%	27%	4%	5%	20%	0%	36%	23%	8%	
Slight undulating	23%	38%	41%	57%	40%	24%	86%	40%	82%	19%	58%	29%	
Curved	14%	21%	9%	16%	23%	15%	6%	40%	18%	7%	13%	49%	
Uni-directional, wavy	0%	0%	0%	0%	0%	0%	0%	0%	0%	0%	0%	0%	
Multi-directional wavy	18%	5%	43%	16%	9%	56%	3%	0%	0%	39%	5%	14%	
Micro (%)	1 sub-hor	2 Bed	3 sub-vert	1 sub-hor	2 Bed	3 sub-vert	1 sub-hor	2 Bed	3 sub-vert	1 sub-hor	2 Bed	3 sub-vert	Describing the micro joint condition and denoting this as a % of the total number of joints logged
Polished	0%	0%	0%	4%	3%	1%	0%	8%	0%	3%	0%	0%	
Smooth Planar	25%	32%	6%	10%	24%	4%	5%	13%	0%	41%	20%	8%	
Rough Planar	18%	2%	1%	8%	5%	0%	0%	2%	0%	0%	21%	0%	
Slickensided Undulating	0%	2%	0%	9%	6%	2%	73%	30%	9%	5%	0%	0%	
Smooth Undulating	16%	47%	16%	27%	43%	28%	13%	42%	91%	12%	33%	59%	
Rough Undulating	20%	12%	34%	18%	6%	10%	0%	0%	0%	0%	21%	18%	
Slickensided Stepped	0%	0%	0%	2%	2%	1%	5%	3%	0%	0%	0%	0%	
Smooth Stepped	2%	3%	7%	9%	4%	14%	1%	0%	0%	39%	3%	14%	
Rough Stepped/Irregular	16%	2%	36%	10%	4%	40%	3%	0%	0%	0%	2%	0%	
Joint Infill (%)	1 sub-hor	2 Bed	3 sub-vert	1 sub-hor	2 Bed	3 sub-vert	1 sub-hor	2 Bed	3 sub-vert	1 sub-hor	2 Bed	3 sub-vert	Describing the joint infill type and denoting this as a % of the total number of joints logged
Irregular	0%	0%	0%	0%	0%	0%	0%	0%	0%	0%	0%	0%	
Irregular	0%	0%	0%	0%	0%	0%	0%	0%	0%	0%	0%	0%	
Soft, Sheared Fine	9%	37%	9%	11%	21%	5%	0%	0%	0%	0%	3%	0%	
Soft, Sheared Medium	0%	0%	0%	0%	0%	0%	0%	0%	0%	0%	0%	0%	
Soft, Sheared Coarse	0%	0%	0%	0%	0%	0%	0%	19%	0%	0%	0%	0%	
Non-Softening, Sheared Fine	6%	1%	3%	5%	2%	0%	58%	0%	0%	24%	7%	4%	
Non-Softening, Sheared Medium	0%	0%	0%	0%	0%	0%	0%	0%	0%	0%	0%	0%	
Non-Softening, Sheared Coarse	0%	0%	1%	0%	0%	0%	0%	12%	0%	0%	0%	0%	
Non-Softening, Sheared Staining	45%	43%	26%	17%	28%	32%	5%	69%	73%	5%	52%	51%	

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Parameter	Skurweberg Formation			Goudini Formation			Cederberg Formation			Peninsula Formation			Remarks
None	38%	19%	61%	66%	49%	63%	37%	0%	27%	71%	38%	45%	
Wall Alteration (%)	1 sub-hor	2 Bed	3 sub-vert	1 sub-hor	2 Bed	3 sub-vert	1 sub-hor	2 Bed	3 sub-vert	1 sub-hor	2 Bed	3 sub-vert	Describing the joint wall alteration and denoting this as a % of the total number of joints logged
Wall = rock hardness	99%	98%	100%	100%	98%	100%	100%	100%	100%	100%	100%	100%	
Wall >rock hardness	0%	0%	0%	0%	0%	0%	0%	0%	0%	0%	0%	0%	
Wall < rock hardness	0%	2%	0%	0%	1%	0%	0%	0%	0%	0%	0%	0%	

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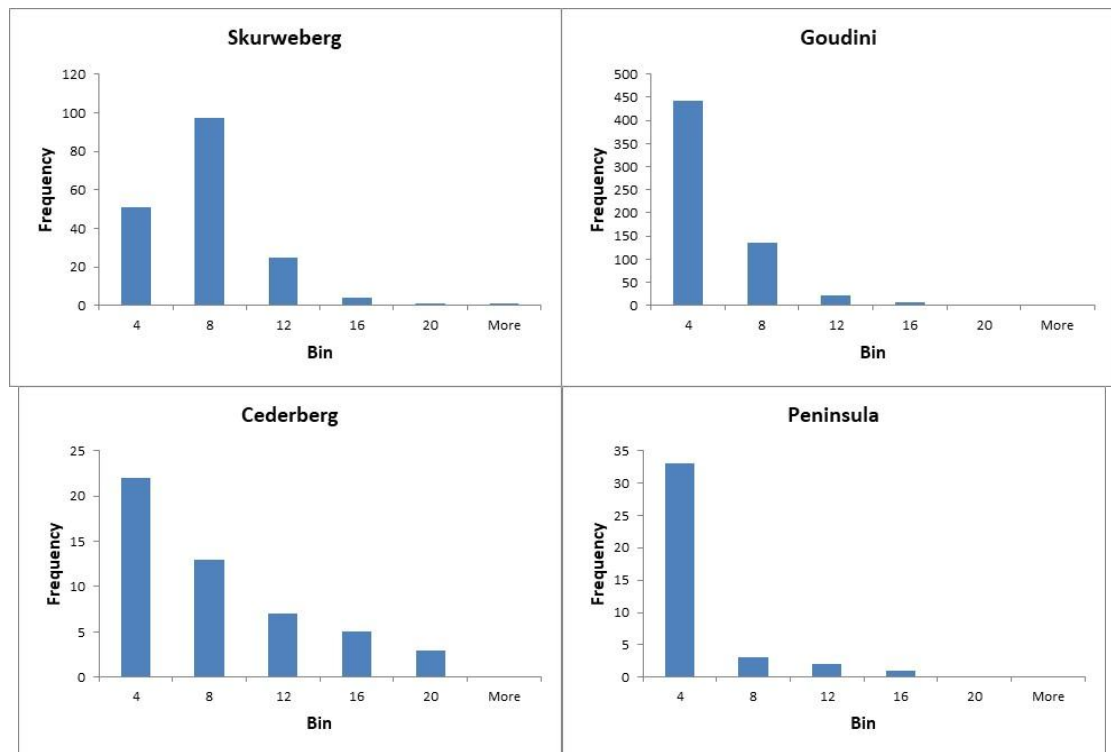


Figure 5.15.5
Fracture Frequency (f/m)

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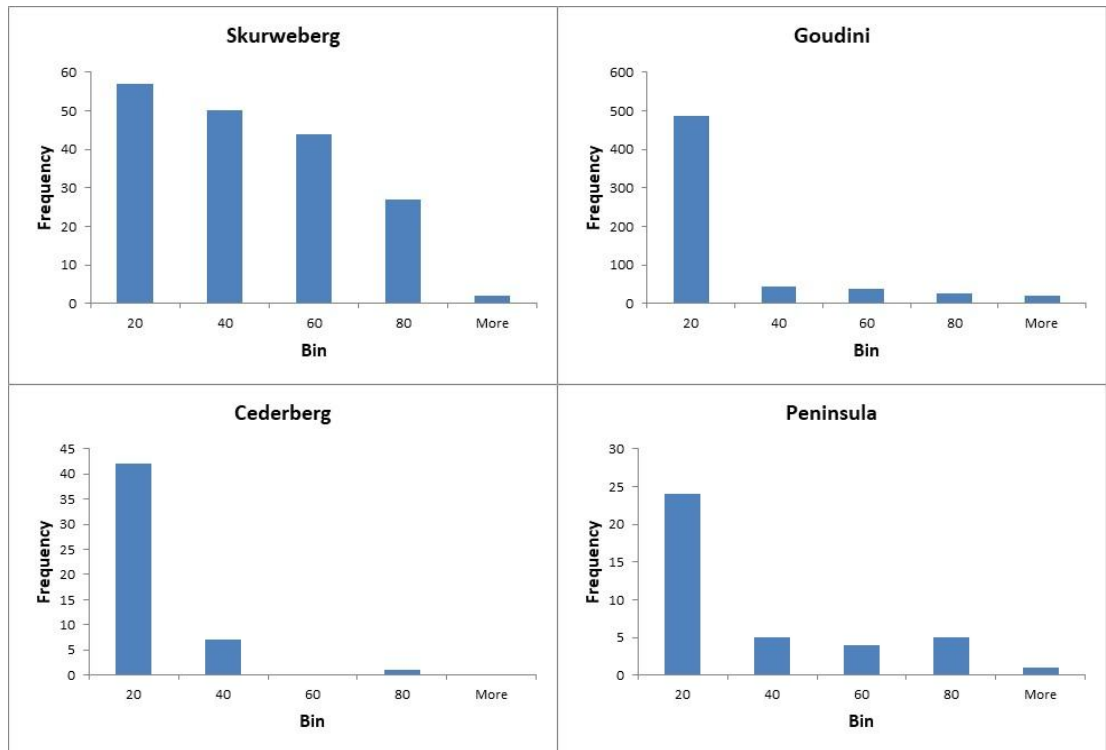


Figure 5.15.6
Rock Quality Designation

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Table 5.15.11
Rock Mass Ratings

	RMR ₂₀	Class	Blast	Weath- ering	Orien- tation	Stress	MRMR ₂₁	Class	Support Required
Goudini									
Min	25	4B	0.94	0.96	0.8	0.97	18	5A	Bolts at approximately 1 m spacing incorporating mesh and/or fibre reinforced shotcrete with rock bolts as lateral restraints
Max	80	2A	0.94	0.96	0.8	0.97	56	3A	Local rock bolts at joint intersections
Mean	33	4A	0.94	0.96	0.8	0.97	23	4B	Bolts at approximately 1 m spacing incorporating mesh and/or fibre reinforced shotcrete with rock bolts as lateral restraints
Standard Deviation	26								
Skurweberg									
Min	27	4B	0.94	0.96	0.8	0.97	19	5A	Bolts at approximately 1 m spacing incorporating mesh and/or fibre reinforced shotcrete with rock bolts as lateral restraints
Max	97	1A	0.94	0.96	0.8	0.97	68	2B	None
Mean	34	4A	0.94	0.96	0.8	0.97	24	4B	Bolts at approximately 1 m spacing incorporating mesh and/or fibre reinforced shotcrete with rock bolts as lateral restraints
Standard Deviation	27								

From the detailed rock joint assessment above, the following is noted:

- The site rocks have a generally low RQD, but display a wide range in values in conformance with the material variability stressed in this SSR – this is primarily linked to the extent of fracturing measured.
- Although displaying a variable (but low) RQD, the rock mass is expected to exhibit stability when considering the RMR which is good – this is primarily due to joint condition, with a general lack of joint infill, a high variability in macro and micro joint geometry and joint walls displaying competence equivalent to the rock hardness.

²⁰ Rock Mass Rating

²¹ Mining Rock Mass Rating

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- The Illustrative Nuclear 1 Footprint (see **Drawing 5.15.1**) is located on the Goudini formation rocks, that rock formation that carries by far the highest sample of measured data at the site and this bodes well for the conclusions drawn in this SSR related to safety as there is high confidence in the data underneath the Illustrative Nuclear 1 Footprint.
- As depicted in **Table 5.15.11**, lateral support (where required) can be achieved by conventional means Rock Slope and Tunnel Stability.

A site characteristic of significance is the well-developed joint sets contributing to the low (but variable) RQD. There is little doubt that any excavations in rock on this site where the dominant joints daylight into cut slopes will require lateral support. These measures will however not be unduly onerous to design.

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

- Resulting blocks reveal that any excavation can fail by sliding along the bedding plane if a rock face is oriented within 30° of the strike of the bedding – 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 if slopes are oriented within 30° of the strike of the joints.
- Fall-out failure can occur from tunnel roofs on the sub-vertical joints, released by bedding or the sub horizontal joints – this failure mechanism is often referred to as ‘lack of clamp’.

These failure mechanisms can be contained using rock support techniques as suggested in **Table 5.15.11**. Failure is more likely to occur on bedding rather than the sub-vertical joints, due to the bedding dip, high prevalence of smooth joint conditions on bedding and the generally rougher joint conditions on the sub-vertical joints.

The adjusted rock mass ratings shown in in **Table 5.15.11** were calculated by reducing the RMR values for 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) values can be used for estimating support requirements as indicated.

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These are indicative values for conceptual design only and individual excavations would require a dedicated design based on orientation and size of excavations.

Empirical estimates (Laubscher, 1990) of allowable slope angles determined from the average adjusted rock mass rating indicate that rock slopes in all of the rock formations present on site should not be excavated at angles steeper than 40° for slopes up to 50 m in height. This is an indicative slope angle for conceptual design purposes only and individual excavations would require a detailed design based on orientation and height of the slope. Steeper slope angles would be possible utilising lateral support.

(v) Rock Joint Characteristics and Tunnelling

Potential Failure Modes in Tunnels Striking North-South or East-West

The potential failure modes in tunnels along and perpendicular to the strike of the bedding are illustrated in **Figures 5.15.7 a)** through **5.15.7 f)**. The bedding is assumed to represent a fourth joint set. Rock reinforcement comprises fully grouted face plated rock bolts of conventional length. Surface support alternatively comprise rock bolted mesh, mesh reinforced shotcrete, unreinforced shotcrete or shotcrete reinforced with either mesh or steel fibre. The thickness of the shotcrete may vary in terms of conditions.

With reference to **Figures 5.15.7 a)** through **5.15.7 f)**, this detailed evaluation of the potential modes of failure in tunnels should bolster confidence in the recommendations in the SSR:

- **Figure 5.15.7 a)** shows the cross-section of a tunnel along bedding strike with the following observations:
 - Three joint sets divide the rock into regular blocks.
 - The back is clamped by the major NS tectonic compressive force.
 - Block slide may occur in the sidewalls.
 - Conventional rock reinforcement and surface support should be adequate.

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- Smooth-wall blasting will promote overall tunnel stability.
- The length of advance and the installation of support will be determined by the competence of excavated rock walls.
- The Klippunt and smaller faults strike diagonally across the tunnel and will not affect the stability of its back or sidewalls unless of considerable width.
- **Figure 5.15.7 b)** shows the longitudinal section of a tunnel along bedding strike with the following observations:
 - Four joint sets dissect the rock into thin wedges and feather edges.
 - The back is subject to open tensional joints crossing the tunnel.
 - Depending on block size this may give rise to instability.
 - The face may be subject to ravelling of small blocks.
 - Conventional rock reinforcement and surface support will ensure adequate stability but may require to be installed at close spacing at to the face.
 - Smooth-wall blasting will promote overall tunnel stability.
 - The length of advance and the installation of support will be determined by the competence of excavated rock walls.
 - The Klippunt and smaller faults strike diagonally across the tunnel and will not affect the stability of its back or sidewalls unless of considerable width.
- **Figure 5.15.7 c)** shows the plan view of a tunnel along the bedding strike with the following observations:
 - Four joint sets intersect the rock into thin wedges and feather edges.
 - The sidewalls are subject to open tensional joints crossing the tunnel.
 - Depending on block size this may give rise to instability.

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- Conventional rock reinforcement and surface support will ensure adequate stability but may require to be installed closely to the face.
 - Smooth-wall blasting will promote overall tunnel stability.
 - The length of advance and the installation of support will be determined by the competence of excavated rock walls.
 - The Klippunt and smaller faults strike diagonally across the tunnel and will not affect the stability of its back or sidewalls unless of considerable width.
- **Figure 5.15.7 d)** shows the cross-section of a tunnel perpendicular to the bedding strike with the following observations:
 - Four joint sets divide the rock into regularly shaped block.
 - The back is subject to open tensional joints parallel to the tunnel.
 - Block fall from the back will likely occur.
 - Sidewall instability is unlikely.
 - Conventional rock reinforcement and surface support should be adequate.
 - Smooth-wall blasting will promote overall tunnel stability.
 - The length of advance and the installation of support will be determined by the competence of excavated rock walls.
 - The Klippunt and smaller faults strike diagonally across the tunnel and will not affect the stability of its back or sidewalls unless of considerable width.
 - **Figure 5.15.7 e)** shows the longitudinal section of a tunnel perpendicular to the bedding strike with the following observations:
 - Three joint sets divide the rock into multiple thin wedges and feather edges.

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- Together with the lack of clamp across the tunnel this may cause significant block falls from the back and block sliding from the face depending on the direction of face advance.
 - Conventional rock reinforcement and surface support will ensure adequate stability but may require to be installed closely to the face.
 - Smooth-wall blasting will promote overall tunnel stability.
 - The length of advance and the installation of support will be determined by the competence of excavated rock walls.
 - The Klippunt and smaller faults strike diagonally across the tunnel and will not affect the stability of its back or sidewalls unless of considerable width.
- **Figure 5.15.7 f)** shows the plan view of a tunnel perpendicular to the bedding strike with the following observations:
 - Four joint sets divide the rock into regularly shaped blocks.
 - The back and sidewalls are subject to open tensional joints parallel to the tunnel.
 - Instability of the face is unlikely.
 - Conventional rock reinforcement and surface support should be adequate.
 - Smooth-wall blasting will promote overall tunnel stability.
 - The length of advance and the installation of support will be determined by the competence of excavated rock walls.
 - The Klippunt and smaller faults strike diagonally across the tunnel and will not affect the stability of its back or sidewalls unless of considerable width.

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Potential Failure Modes in Tunnels Striking Along and Across Faults

The stability of tunnels striking along faults will be adversely affected by the weak rock on the faults if it coincides with the faults depending upon the width of the fault.

The stability of tunnels striking across faults will not be affected by the weak rock on the faults unless the faults are of considerable width.

Potential Failure Modes of Rock Embankments Striking North-South or East-West

The potential failure modes of rock embankments along and perpendicular to the strike of the bedding are discussed.

This detailed evaluation of the potential modes of failure of rock embankments should bolster confidence in the recommendations in the SSR. Several scenarios are addressed as follows:

- An inclined south sidewall of an embankment along the bedding strike. It is similar in geological structure compared to the south sidewall of the tunnel in **Figure 5.15.7 a)** and compared to the north face of a tunnel in **Figure 5.15.7 e)**:
 - Three joint sets divide the rock into regularly shaped blocks.
 - The potentially open tensional joints are perpendicular to the embankment, and will allow infiltration of surface runoff and aggravate the looseness of the rock mass.
 - Excavation of the embankment will subject the rock mass to considerable stress relief on relaxing the major primitive compressive stresses.
 - Conventional rock reinforcement and surface support should be adequate for shallow embankments.
 - The height of the blast face and the installation of the support will be determined by the competence of the excavated face.
 - For deeper embankments, longer fully grouted cable anchors may be

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required in addition to conventional rock bolts. Surface support will in all instances be required and deeper embankments may have to be benched to reduce the overall inclination.

- Smooth-wall blasting will promote overall embankment stability.
- The Klippunt and smaller faults strike diagonally across the tunnel and will not affect the stability of its back or sidewalls unless of considerable width.
- An inclined north sidewall of an embankment along the bedding strike. It is similar in geological structure compared to the north sidewall of the tunnel in **Figure 5.15.7 a)** and compared to the south face of a tunnel in **Figure 5.15.7 e)**:
 - Three joint sets divide the rock into regularly shaped blocks.
 - The potentially open tensional joints are perpendicular to the embankment, and will allow infiltration of surface runoff and aggravate the looseness of the rock mass.
 - Excavation of the embankment will subject the rock mass to considerable stress relief on relaxing the major primitive compressive stresses.
 - Conventional rock reinforcement and surface support should be adequate for shallow embankments.
 - The height of the blast face and the installation of the support will be determined by the competence of the excavated face.
 - For deeper embankments, longer fully grouted cable anchors may be required in addition to conventional rock bolts. Surface support will in all instances be required and deeper embankments may have to be benched to reduce the overall inclination.
 - Smooth-wall blasting will promote overall embankment stability.
 - The Klippunt and smaller faults strike diagonally across the tunnel and will not affect the stability of its back or sidewalls unless of considerable width.

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- An inclined west sidewall of an embankment perpendicular to the bedding strike. It is similar in geological structure compared to the west face of the tunnel in **Figure 5.15.7 d)** and compared to the east face of a tunnel in **Figure 5.15.7 b)**:
 - Four joint sets divide the rock into regularly shaped blocks.
 - The potentially open tensional joints are parallel to the embankment, and will allow infiltration of surface runoff and aggravate the looseness of the rock mass.
 - Excavation of the embankment will subject the rock mass to considerable stress relief on relaxing the major primitive compressive stresses. This may promote toppling failure of blocks depending upon the inclination of the embankment.
 - Conventional rock reinforcement and surface support should be adequate for shallow embankments.
 - The height of the blast face and the installation of the support will be determined by the competence of the excavated face.
 - For deeper embankments, longer fully grouted cable anchors may be required in addition to conventional rock bolts. Surface support will in all instances be required and deeper embankments may have to be benched to reduce the overall inclination.
 - Smooth-wall blasting will promote overall embankment stability.
 - The Klippunt and smaller faults strike diagonally across the tunnel and will not affect the stability of its back or sidewalls unless of considerable width.
- An inclined east sidewall of an embankment perpendicular to the bedding strike. It is similar in geological structure compared to the east sidewall of the tunnel in **Figure 5.15.7 d)** and compared to the west face of a tunnel in **Figure 5.15.7 b)**:
 - Four joint sets divide the rock into regularly shaped blocks.
 - The potentially open tensional joints are parallel to the embankment,

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and will allow infiltration of surface runoff and aggravate the looseness of the rock mass.

- Excavation of the embankment will subject the rock mass to considerable stress relief on relaxing the major primitive compressive stresses. This may promote toppling failure of blocks depending upon the inclination of the embankment.
- Conventional rock reinforcement and surface support should be adequate for shallow embankments.
- The height of the blast face and the installation of the support will be determined by the competence of the excavated face.
- For deeper embankments, longer fully grouted cable anchors may be required in addition to conventional rock bolts. Surface support will in all instances be required and deeper embankments may have to be benched to reduce the overall inclination.
- Smooth-wall blasting will promote overall embankment stability.
- The Klippunt and smaller faults strike diagonally across the tunnel and will not affect the stability of its back or sidewalls unless of considerable width.

Potential Failure Modes of Rock Embankments Striking Along and Across Faults

The stability of embankments striking along faults will be adversely affected by the weak rock on the faults if it coincides closely with the faults.

The stability of embankments striking across faults will not be affected by the weak rock on the faults unless the faults are of considerable width.

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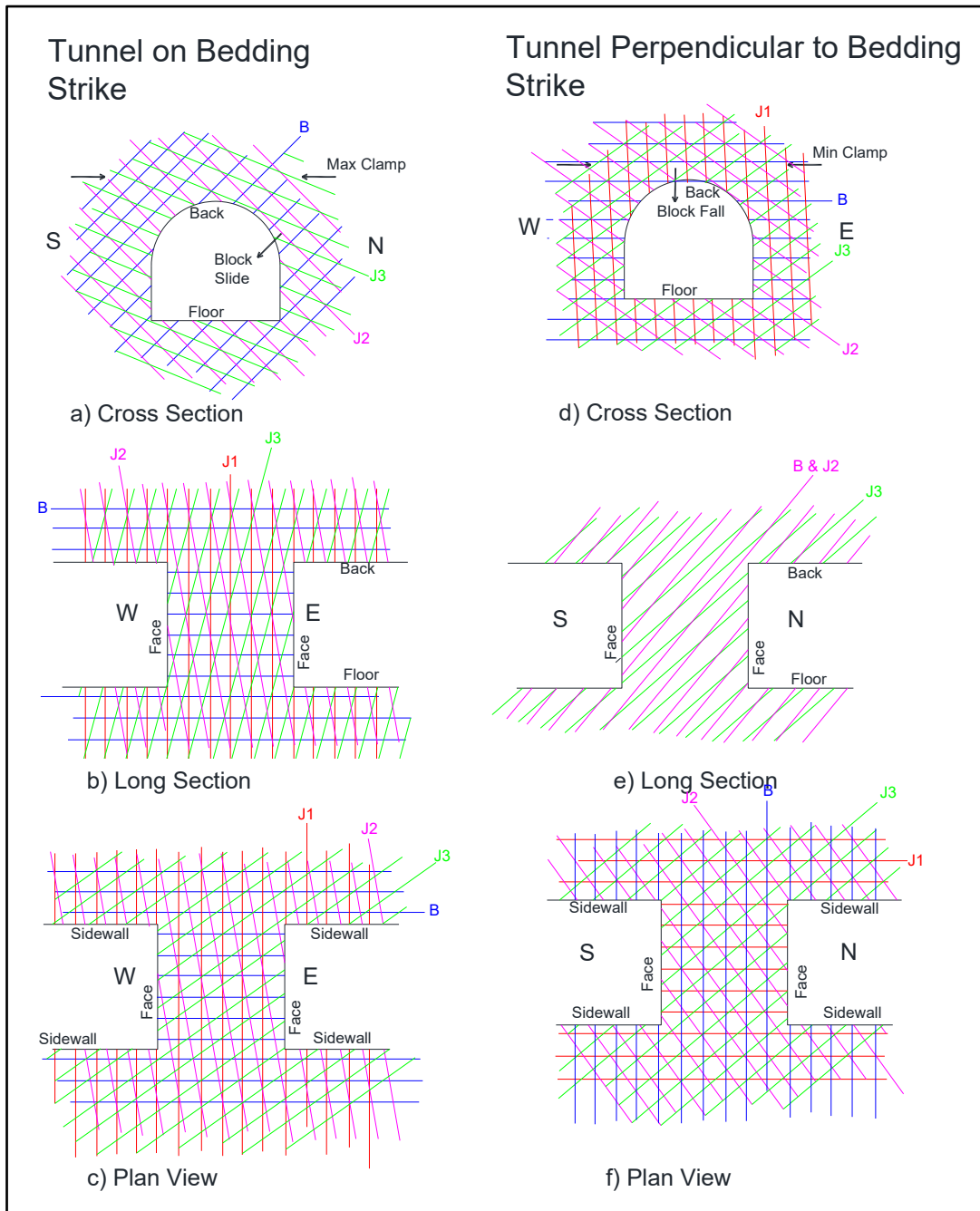


Figure 5.15.7
Tunnelling Failure Modes

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5.15.4.3.2 Results of Groundwater Level Investigations

Groundwater levels are monitored in all boreholes listed in **Table 5.15.4** and in the geohydrological boreholes (see **Section 5.11**). **Section 5.11** presents the results in detail pertaining to groundwater level monitoring and **Drawing 5.15.3** and **Drawing 5.15.4** illustrate the fluctuating groundwater table measurements.

The groundwater level monitoring results demonstrate the following:

- The groundwater table consistently emerges from below ground to above ground at the sea, supporting a series of coastal seeps/wetlands (see **Section 5.11** and **Section 5.3**, Ecology). Boreholes TB1, TB4 and TB10, in particular, reveal shallow groundwater and in the case of TB10, this is artesian²².
- There is a gradual rise in groundwater table elevation when moving inland from the sea (in the dune area, see **Drawing 5.15.3**) where the depth from surface to groundwater varies significantly (from zero m in dune valleys to over 40 m under dune peaks).
- Groundwater levels have fluctuated over the monitoring period as indicated in **Drawing 5.15.3** and **Drawing 5.15.4**.
- Approximately 1.5 km inland and to the north-east of Thyspunt, the groundwater table again emerges from below ground to above ground in low lying areas giving rise to another series of wetlands, termed 'dune-slack wetlands' (see also **Section 5.3** and **Section 5.11**).
- Geohydrological borehole THYRP1M1 situated approximately five km inland from the sea has a groundwater table level of 143 m msl whilst boreholes drilled at the sea have an average elevation of approximately 5 m msl.
- Groundwater flow direction is in a south, south-easterly direction and not perpendicular to the coastline (**Section 5.11**) – particular note should be

²² Artesian: A confined aquifer containing groundwater that will flow upwards through a well without the need for pumping.

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taken of the influence that the highly transmissive cobble layer situated above the Goudini/Cederberg rocks has as this tends to locally influence flow direction more towards the east or towards Thysbaai (see localised groundwater table contour plot in **Drawing 5.15.5**).

In summary, the thickness of the aeolian soils at the site varies from zero m (at the sea) to 65 m (in the dune area). There is a general increase in consistency from surface down to the position of the groundwater table where calcrete zones are present, but randomly distributed. Underlying this (and the position of the groundwater table), is a thick deposit of Aeolian sands (dominant) and marine sand/pebbles/cobbles (less dominant but uniformly distributed above the Goudini bedrock depression) of highly variable consistency. Groundwater flow direction is in a south to south-easterly direction. It is not known whether calcrete zones are continuous and thus not known whether a perched water table is present above this zone, but this is considered unlikely as there is no evidence in the form of surface water features (outside of dune slack wetlands) to support this.

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5.15.4.3.3 Results of Laboratory Investigations

Laboratory test results are presented to confirm the geotechnical characteristics described in **Subsection 5.15.4.2** as follows:

- results of soil triaxial and permeability tests in **Table 5.15.14**;
- results of soil characterisation tests (including Atterberg Limits) in **Appendix 5.15.H**;
- results of rock Uniaxial Compressive Strength (UCS), Young's Modulus and Poisson's Ratio tests in **Appendix 5.15.I**.

(a) 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.H**). Samples were taken and analysed to provide a three-dimensional data set representative of the site soils.

Characterisation of the site soils was the primary goal for this SSR. The laboratory test results confirmed homogeneity of the site soils as they are described in **Appendix 5.15.B**.

Table 5.15.12 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') and as such, Atterberg Limits were not targeted.

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Table 5.15.12
Summary Statistics of Laboratory Soil Characterisation Test Results

	Grading			
	Clay Size Range (% Passing)	Silt Size Range (% Passing)	Sand Size Range (% Passing)	Gravel Size Range (% Passing)
Sample	139	139	156	156
Mean	4.00	3.00	93.00	0
Median	3.00	2.00	94.00	0
Standard Deviation	2.24	2.17	4.34	2.38
Range	12.00	11.00	28.00	20.00
Minimum	0	0	70.00	0
Maximum	12.00	11.00	98.00	20.00

The site soils are:

- homogenous across the site and dominated by sand (of aeolian origin primarily) with some fines occasionally present in the form of silt and clay;
- non-plastic and poorly graded;
- classified as silty sands (SM), poorly graded sands (SP) or a combination (SM-SP) in the Unified Soil Classification system.

Figure 5.15.8 shows the grading curves for all samples analysed.

The homogeneity of the site soils is a feature of **Figure 5.15.8**.

Soil samples were analysed using two different methods for determining the Liquid Limit (LL) – (TMH1 – (Committee of State Road Authorities, 1986a) and (BS1377 – (British Standards Institution, 1990)). The multiple analysis approach was intended to investigate the influence of calcrete encountered at the site on the LL of selected samples. In tests carried out under the TMH1 specifications, LL was not measured because of the non plastic nature of the site soils. However, under the BS1377 specification, LL in the region of 30 per cent was measured.

The differences in these test results could be explained as follows:

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- The TMH1 standards require mandatory drying of samples at 105 – 110 °C which often has an effect on reducing the LL and/or PI (Plasticity Index) of samples.
- The angularity of sand impacts on the LL (Hogegentogler, et al., 1937).
- LL of coarse (silt sized calcite) is impacted by the presence of CaCO₃ with a linear increase in LL with increase in CaCO₃ content (Netterberg, 1969).

It appears that the LL results may have exposed a relationship between LL and CaCO₃ content in the site soils. This relationship could be linked to the high concentrations of CaCO₃ in groundwater (in all likelihood originating from the shell-rich site soils).

Triaxial testing results of representative soil samples are summarised in **Table 5.15.14.**

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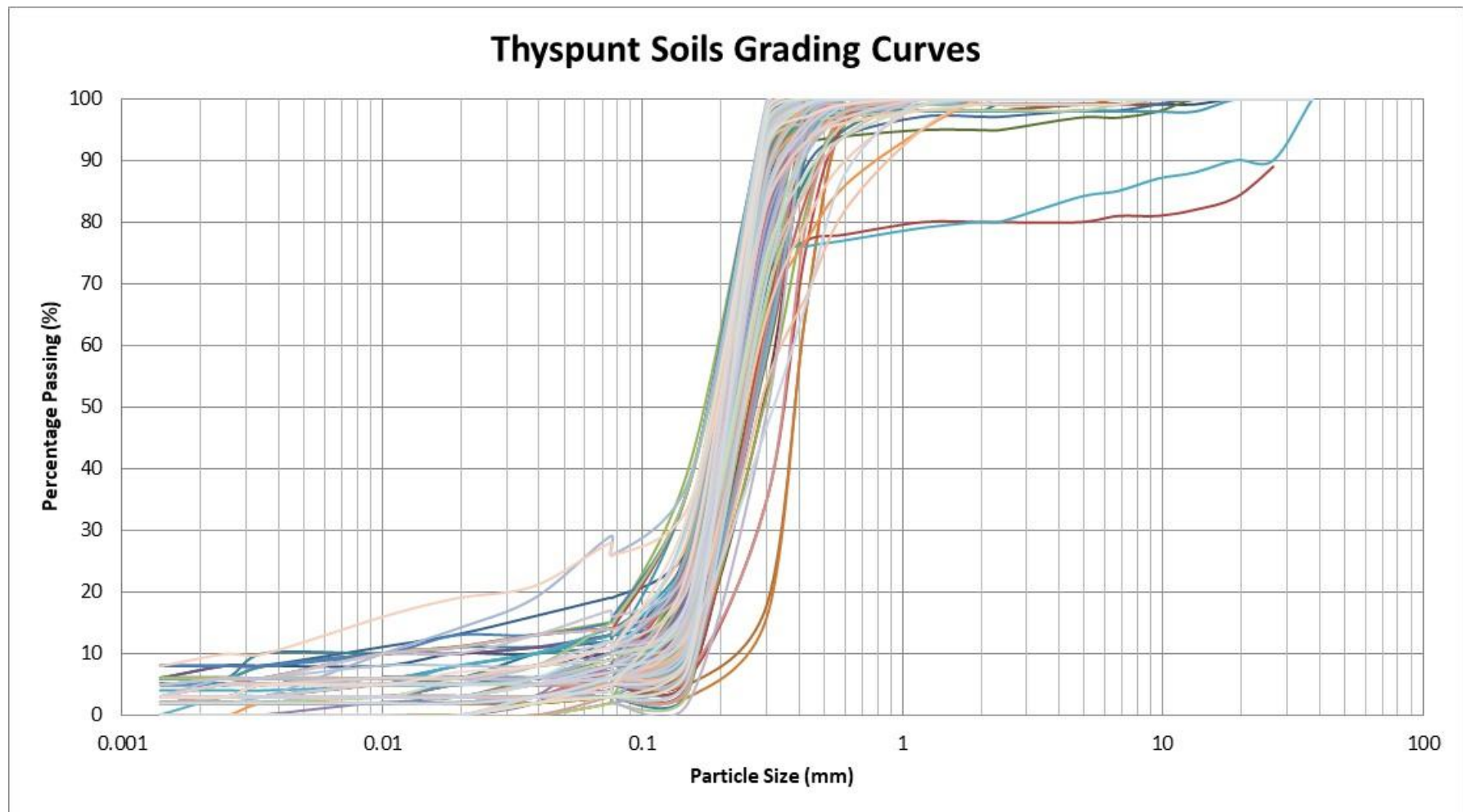


Figure 5.15.8
Grading Curves for Thyspunt Soils

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Table 5.15.13
Results of Laboratory Soils Triaxial Test - Summary

Parameter	Result
Internal angle of friction (Φ)	30°
Critical stress ratio (M)	1.2
Slope of critical state line (λ_{10})	0.027
Soil permeability	1.8×10^{-3} to 1.9×10^{-3} cm/s

The measured soil permeability varies from that quoted in **Subsection 5.15.4.1.3** (i.e. 6.6 m/day or 3×10^{-3} cm/s), but falls within the same range. It should however, be noted that the calculated permeability results (**Subsection 5.15.4.1.3**) represent the permeability of the intergranular aquifer (overburden sand and cobble horizon combined) – where the permeability of the cobble horizon has been established as very high and this influences the overall permeability of the intergranular aquifer. It is, in all probability, for this reason that the permeability measured in **Table 5.15.13**, which is only representative of the soil and not the cobble horizon, is lower.

Assessing the soil friction angle using the voluminous SPT database for the site in accordance with (Clayton, 1993), friction angles calculated range from 27° to 40°, with a standard deviation of 6°.

(b) Soil density

Because of the difficulty in obtaining undisturbed soil samples (see **Subsection 5.15.4.2.2**); the laboratory testing programme did not target the *in situ* density of site soils. 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. (Cubrinovski & Ishihara, 2001) was used as a basis for this assessment and **Table 5.15.14** shows the results.

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Table 5.15.14
***In situ* Soil Density Ranges**

Parameter	Result
D ₅₀ ²³ of site soils	0.215 ± 0.021
Void ratio range(e _{max} – e _{min})	0.511 ± 0.025
Relative Density (D _r) as a ratio	1.09 ± 0.25
Density ranges (kN/m ³)	Very loose γ _d = 13.68 γ _{sat} = 18.33
	Loose γ _d = 14.67 γ _{sat} = 18.94
	Medium dense γ _d = 15.81 γ _{sat} = 19.65
	Dense γ _d = 17.14 γ _{sat} = 20.48
	Very dense γ _d = 18.72 γ _{sat} = 21.46

Assessing the consistency of the soils based on the voluminous SPT test results against the ranges provided in **Table 5.15.14** and **Table 5.15.15** indicates the SPT guided *in situ* soil density for the site with depth.

Table 5.15.15
***In situ* Soil Density Assessment Results**

Depth Down Profile (m)	Dry Density (kN/m ³)	Saturated Density (kN/m ³)
<5	16.5	20.1
5-10	17.6	20.8
10-15	17.8	20.9
15-20	18.0	21.0
20-25	18.1	21.1
25-30	18.0	21.0
30-35	18.3	21.2
35-40	18.1	21.1
40-45	18.2	21.1
>45	18.3	21.2

The homogeneity of the site soil particle size distribution is again exposed by the consistency in the D₅₀ (or average grain size) of the soils. From the

²³ D₅₀ represents that grain size corresponding to 50% of the soil weight passing a sieve with an opening = D₅₀, sometimes referred to as the average grain size.

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calculated *in situ* density ranges, the following is noted:

- Soils in the medium dense (or less) range may be challenging to found in. In reality, the bearing capacity of medium dense (or less) soils may be variable introducing differential bearing capacity concerns.
- Soils with consistency of dense to very dense will, on the other hand, present founding environments in which variability is considerably reduced, reducing risks of differential settlement and foundation design simplified as a result.

(c) Soil Slope Stability and Foundation Conditions

The soils at the site are homogenous and dominated by aeolian sand deposits. They are poorly graded, classifying as SM, SP or SM-SP. They are non-plastic, but appear to exhibit a linear relationship between LL and CaCO₃ content. The soils are cohesionless and have an internal angle of friction ranging between 27° and 40° with zero cohesion. The uniformity/homogeneity of the site soils has been established through field and laboratory tests.

The cohesionless nature of the soils, the thick deposits encountered at the site and the presence of an intergranular aquifer raises stability of any constructed or excavated soil slopes on the site as a critical and environmentally far reaching consideration. In order to maintain a factor of safety of 3 for slopes (International Atomic Energy Agency, 2004) it is not conservative to assume that slopes may be designed at slope angles in the region of 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 and prevent daylighting of the phreatic surface at slope faces which would result in sloughing failure due to the lack of cohesion in these materials. Such sloughing failure may eventually lead to undercutting and more catastrophic slope failure.

At the proposed depth of excavation (i.e. to bedrock) a shallow slope angle of 18° will result in an extensive excavation footprint that will impact considerably on the environment. Dewatering will be a mandatory requirement for slope stabilisation and this will have an impact on the drawdown of groundwater.

Founding of conventional structures (i.e. non safety related structures) is an important consideration as the longevity of these structures is important in

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supporting the nuclear installation(s). The design engineers will aim to limit foundation instability and designs will need to consider the following:

- All conventional foundations (e.g. pad foundations, strip foundations) must be founded on at least 'dense' material (see **Appendix 5.15.B**) to minimise settlement risks – dense horizons are at depth in the dune fields thus potentially requiring piled foundations depending on the imposed loading of individual structures or structural components.
- Since large portions of the site are covered by sand dunes, ground improvement measures may thus be required for conventional foundations.

(d) Rock Strength, Deformability and Density

This section presents the results from the rock laboratory tests for the site. The measurement of rock properties and the results of laboratory testing are presented in this section, but should be read in conjunction with the Vs profile presented in **Drawing 5.15.6** and **Drawing 5.15.7**.

The influence of discontinuities on test results was anticipated at the outset of the programme and rock samples analysed were therefore photographed before and after testing to try and visually gauge this influence (see **Appendix 5.15.I**).

A statistical summary of the rock laboratory test results is presented in **Table 5.15.16** and the detailed results, along with pre and post-failure photographs, in **Appendix 5.15.I**.

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Table 5.15.16
Summary Statistics of Rock Laboratory Tests

Parameter	UCS	E	v	Bulk Density	Dry Density
	(MPa)	(GPa)		(kg/m ³)	(kg/m ³)
Skurweberg					
Sample	74	73	73	74	74
Mean	109	92.3	0.185	2617	2612
Standard Deviation	61.2	65.7	0.104	38	43
Range	250	545	0.487	260	290
Minimum	3.82	0.53	0.0134	2450	2410
Maximum	254	546	0.5	2710	2700
Goudini					
Sample	207	202	202	207	207
Mean	30	25.3	0.335	2566	2545
Standard Deviation	35.0	30.8	0.159	136	149
Range	196	186	1.102	1050	1080
Minimum	0.314	0.358	0.008	2020	1980
Maximum	196	186	1.11	3070	3060
Cederberg					
Sample	12	11	11	12	12
Mean	6	19.7	0.395	2683	2655
Standard Deviation	3.7	16.9	0.138	101	110
Range	13	50	0.400	240	300
Minimum	1.49	3.32	0.099	2530	2460
Maximum	14.1	53.2	0.5	2770	2760
Peninsula					
Sample	15	15	15	15	15
Mean	121	76.4	0.246	2671	2664
Standard Deviation	79.4	20.3	0.098	136	138
Range	262	63	0.319	650	660
Minimum	17.4	45.3	0.106	2480	2470
Maximum	279	108	0.425	3130	3130

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The summary statistics presented in **Table 5.15.16** are graphically presented in **Figure 5.15.9** and **Figure 5.15.10**. The results are rectangularly distributed which is common in sedimentary rocks that are characterised with varying geological layers.

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

- general:
 - The test results are, in principle, rectangularly distributed and this confirms the variability in the site rock parameters – this variability will have to be factored into the design and design specific geotechnical investigations.
 - The pre-operational and operational stages of the geotechnical characterisation investigation will need to explore variability with respect to specific safety-related structures and the works affected must be designed to meet the range of values.
 - 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 samples taken from less frequently jointed core and may have biased the results to represent stronger rather than weaker rock.
- uniaxial compressive strength (UCS):
 - The results have a rectangular distribution in principle, although this statement is more confidently reported for the Skurweberg and Goudini rocks in which a much greater number of tests were done than in the Cederberg and Peninsula rocks.
 - The above bullet point considered, it is noted that the Goudini and Cederberg rocks present lower UCS values and this may well have contributed to their advanced erosion when the wave cut platform developed (leaving a cobble filled depression as reported on extensively in this SSR).
- Young's Modulus (E):

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- The distribution is once again rectangular in principle, typical of sedimentary rocks and highlights variability.
- There is a similar drop off in E for the Goudini and Cederberg rocks to that noted in the UCS values.
- Poisson's Ratio (ν):
 - The distribution of results is again rectangular in principle.
 - It must be noted that spurious values were measured returning values > 0.5 as can be seen in **Appendix 5.15.1**. The jointed/fractured nature of the rocks undoubtedly influenced this and 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 Ratio²⁴ 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.
 - Poisson's ratio values in the range of 0.15 to 0.25 are generally accepted as a reliable estimate of values for elastic, un-fractured TMG rock and it is noted that the apparently more competent Skurweberg and Peninsula rocks have means in this range, but the apparently less competent Goudini and Cederberg rocks have means higher than this range.
- bulk and dry density:
 - The measured bulk and dry densities of all site rocks are very similar, indicating the one set of parameters measured that does not display high variability.

²⁴ Values of Poisson's ratio are expected to range from 0.25 – 0.5 and recorded values in excess thereof are unusual. There is precedent for the occurrence of high values documented in work by Barton and Bandis (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.

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In summary, the laboratory analyses confirm that the site rocks display a high variability in parameters (apart from density) typical of sedimentary rocks because sediments deposited alternate and weather to varying extents. 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. At this stage in the development of the site geotechnical description, and considering the position of the Illustrative Nuclear 1 Footprint on the Goudini rocks, a vast amount of data exist – analysis of these data do not suggest any particular challenges related to rock stability apart from dealing with variability.

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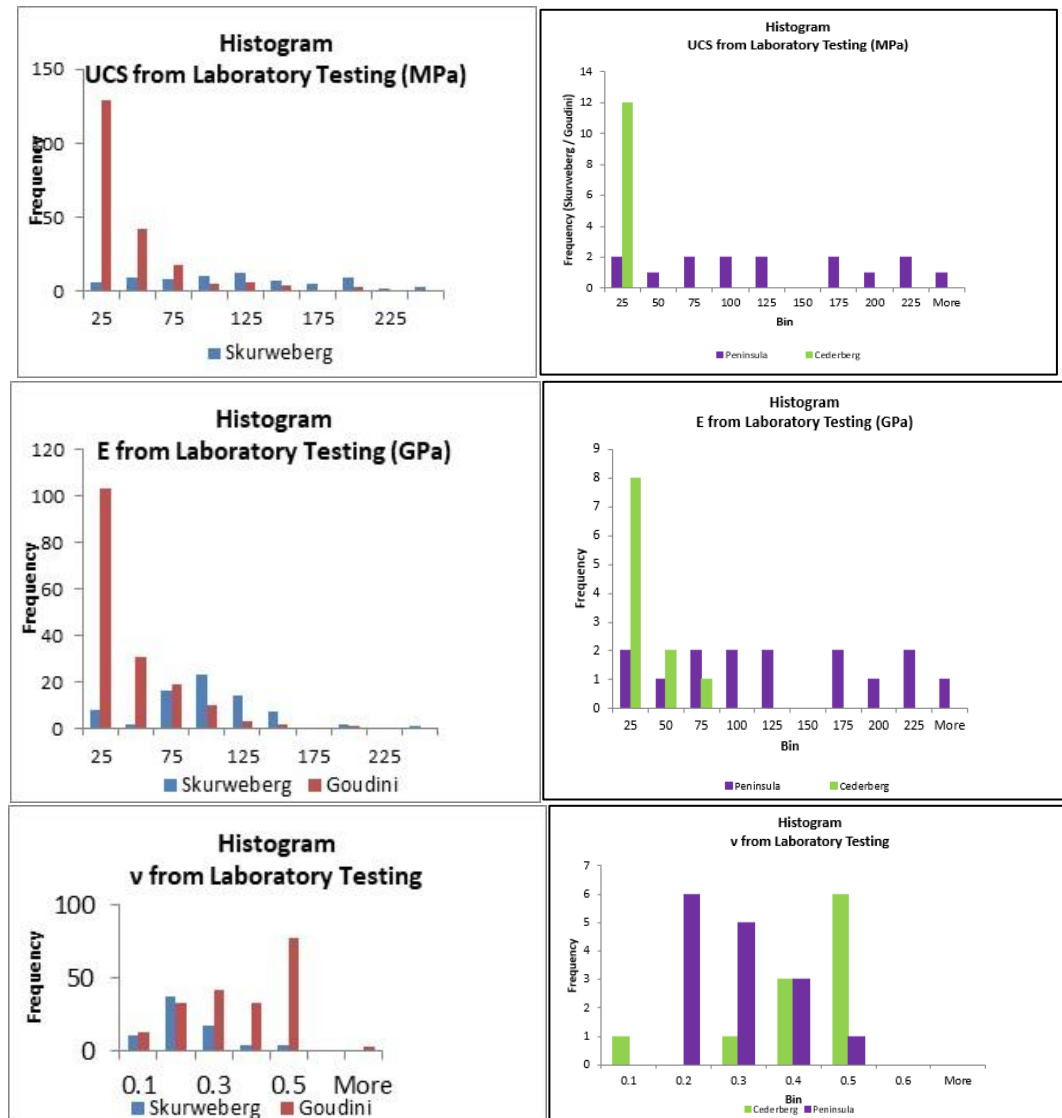


Figure 5.15.9
UCS, Young's Modulus and Poisson's Ratio from Laboratory Tests

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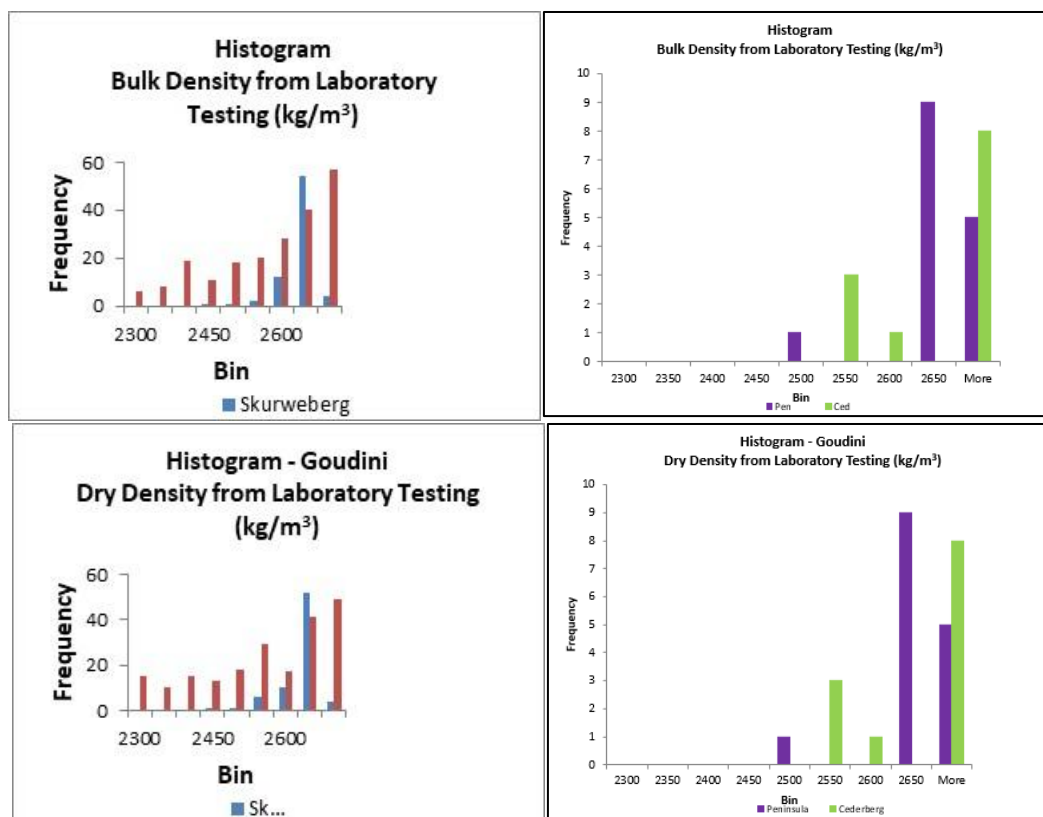


Figure 5.15.10
Bulk and Dry Density from Laboratory Tests

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5.15.4.3.4 Results of the Liquefaction Potential Assessment

The liquefaction potential assessment was carried out as follows:

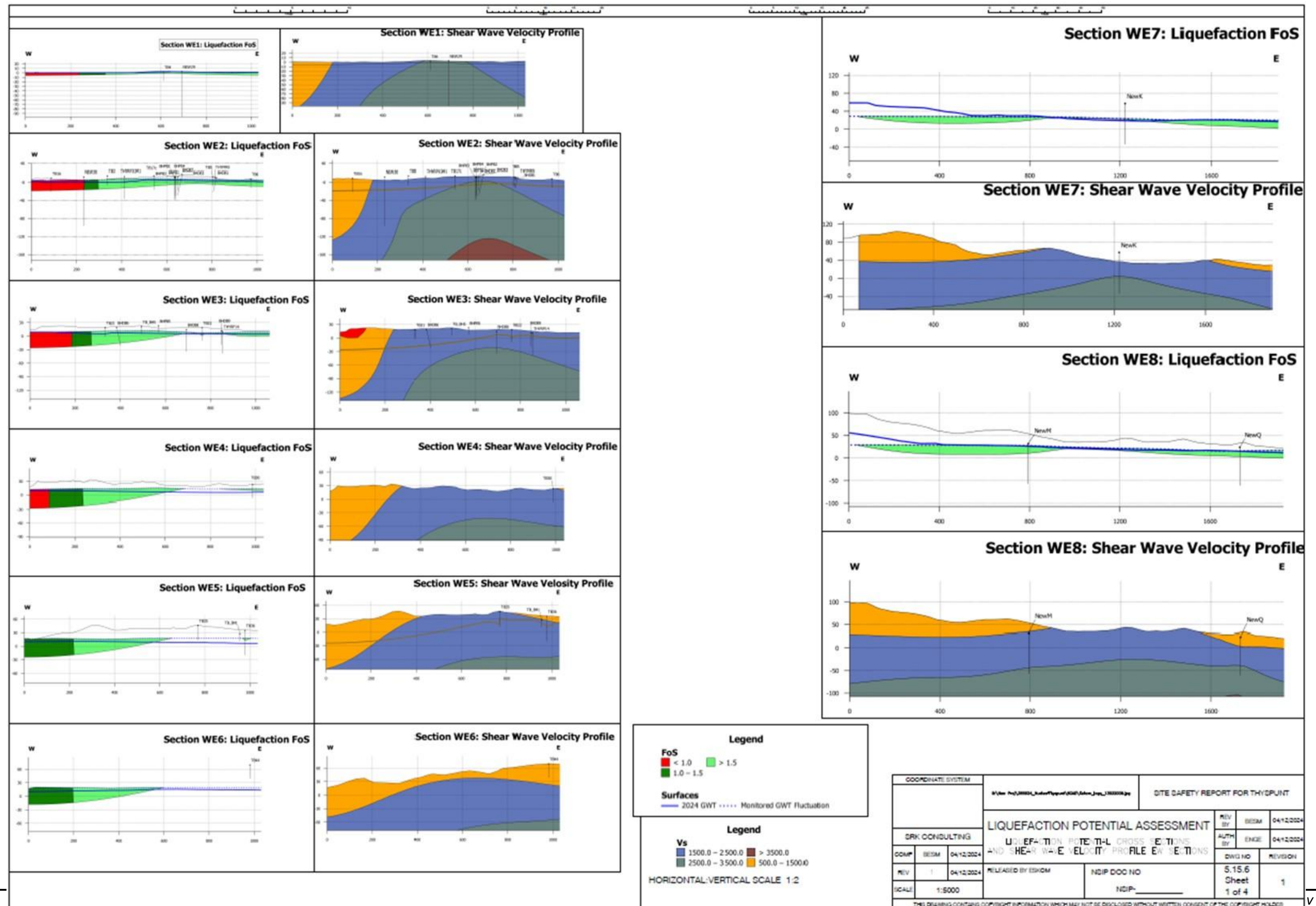
- The triggering mechanism considered was for a seismic event of $M_{7.25}$ resulting in a PGA of 0.31g derived from the PSHA (Council for Geoscience, 2013).
- Soils located under the top extent of the fluctuating groundwater table (**Section 5.11**), i.e. saturated soils, were assessed for their liquefaction potential.
- SPT and Vs data were both used to assess liquefaction potential using methods described in (National Centre for Earthquake Engineering Research, 2001).

Drawing 5.15.6 and **Drawing 5.15.7** display the results of the liquefaction potential assessment graphically. The following is noted:

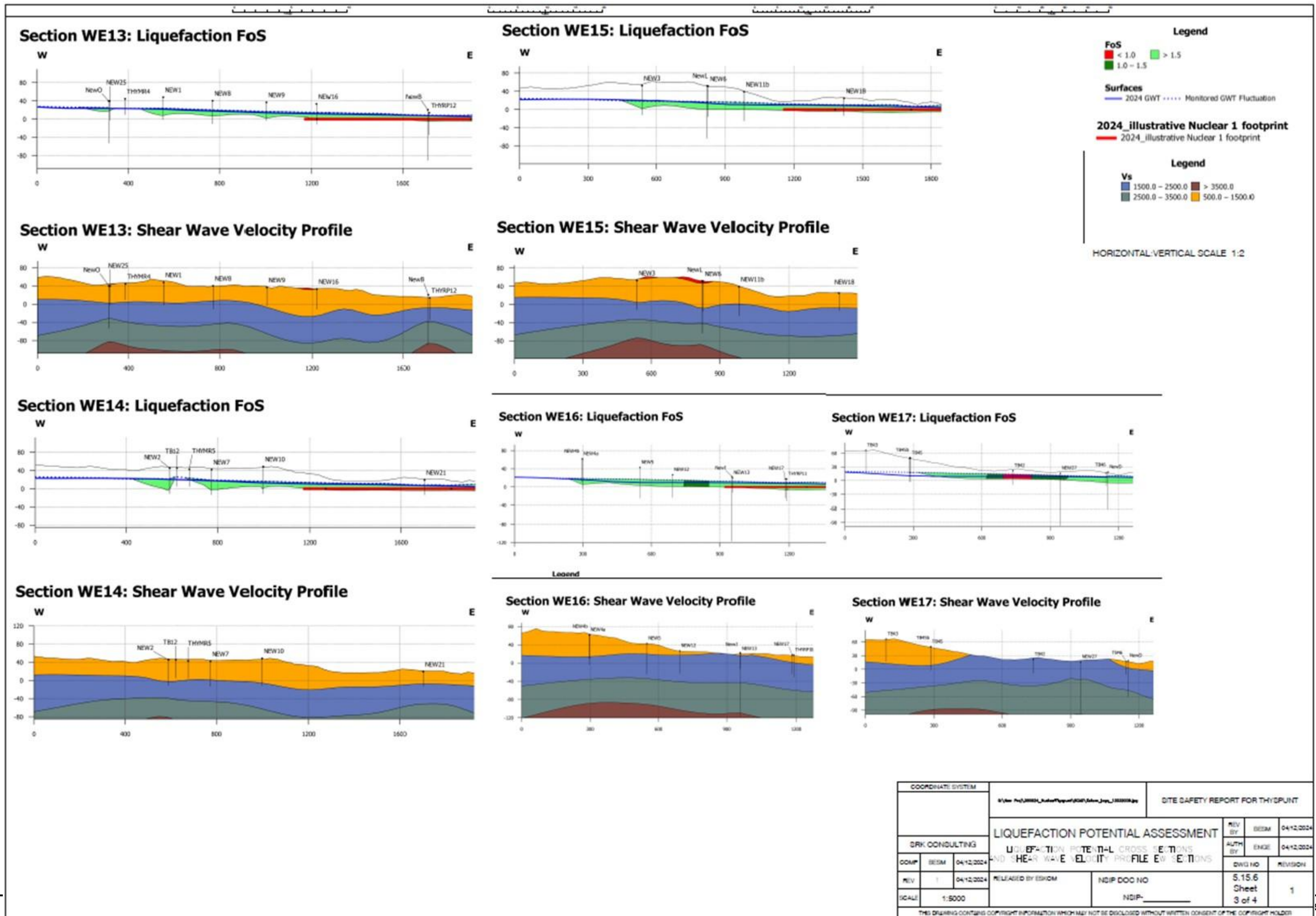
- Isolated zones on the site indicate a high liquefaction potential, but in general, the site soils are of high consistency, and a vast majority of these soils are located above the groundwater table.
- The Vs profile indicates consistently high Vs values in the upper (soil) horizons apart from several low Vs zones at surface and above the groundwater table.

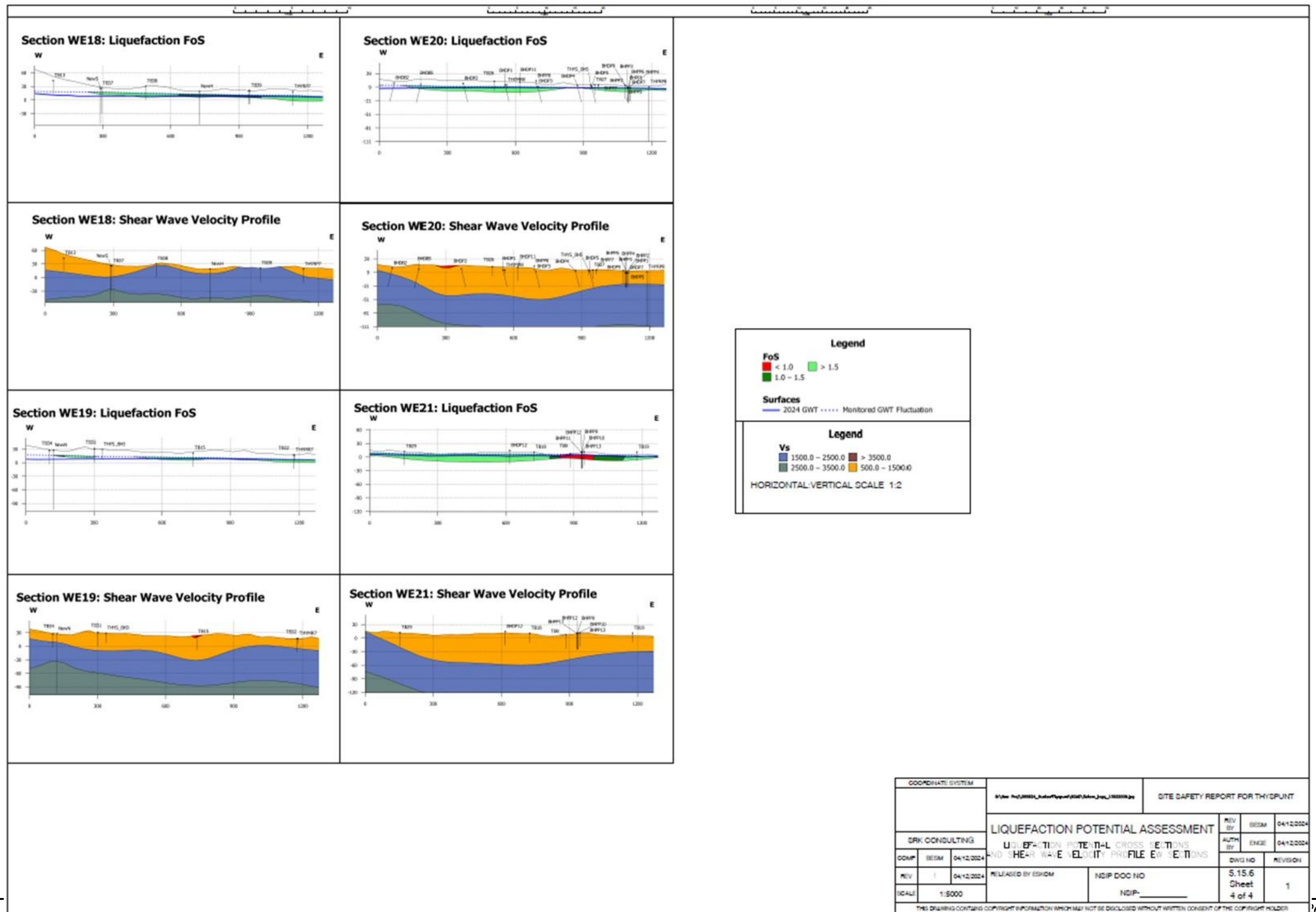
In conclusion:

- The SPT and Vs data at the site are extensive. Analysis of these data utilising methods in (National Centre for Earthquake Engineering Research, 2001) and considering only those soils that are saturated suggests that liquefaction risks at the site are low.
- However, should for some unforeseen reason, soils in the upper horizons at the site become saturated (e.g. if a seepage cutoff wall is installed creating elevated groundwater levels on the up-gradient side) it is likely that liquefaction risks will dramatically increase.

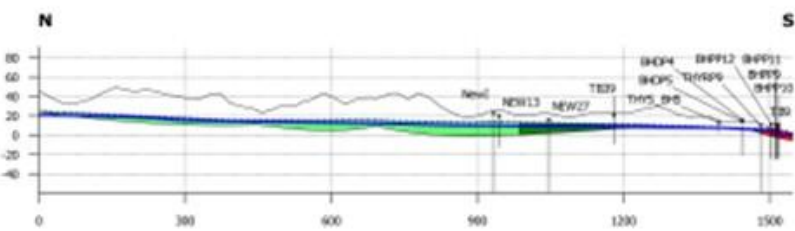




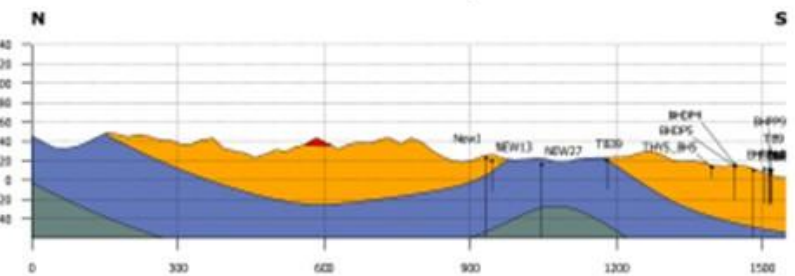




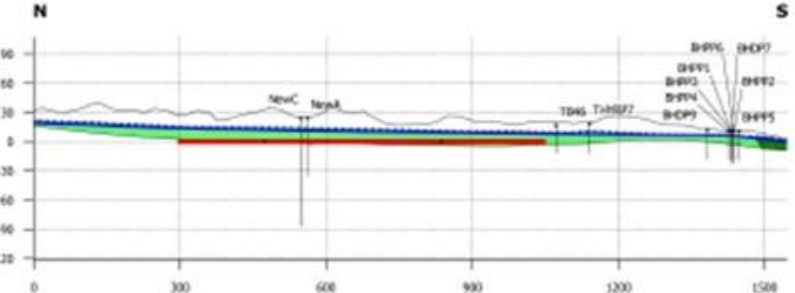
Section NS9: Liquefaction FoS



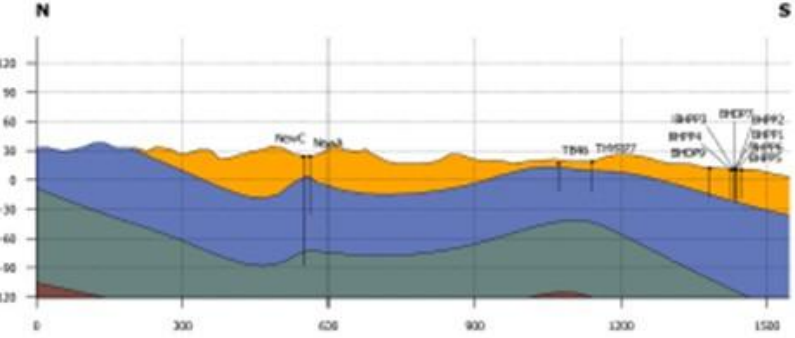
Section NS9: Shear Wave Velocity Profile



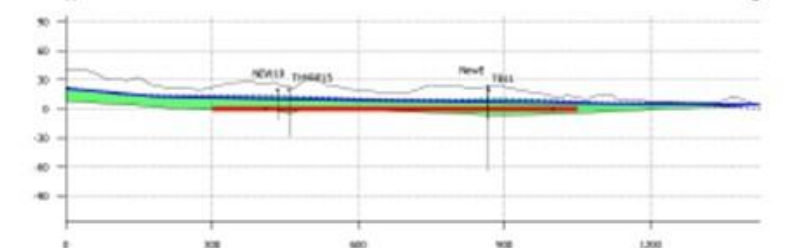
Section NS10: Liquefaction FoS



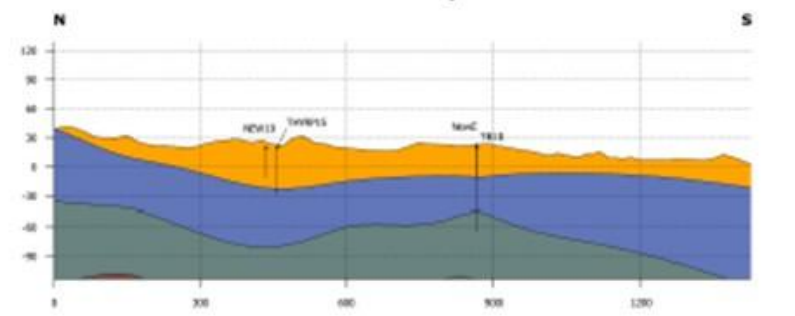
Section NS10: Shear Wave Velocity Profile



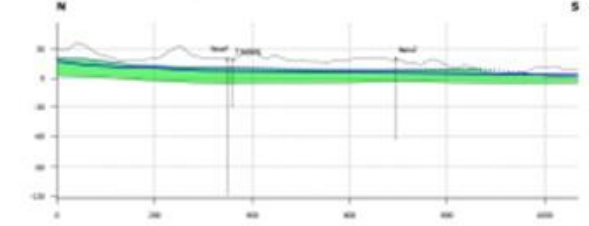
Section NS11: Liquefaction FoS



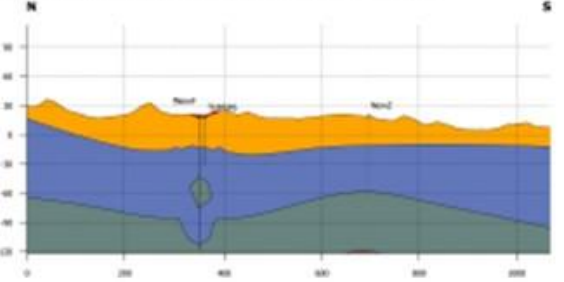
Section NS11: Shear Wave Velocity Profile



Section NS12: Liquefaction FoS



Section NS12: Shear Wave Velocity Profile



Legend

FoS

- < 1.0
- 1.0 - 1.5
- > 1.5

Surfaces

- 2024 GWT
- Monitored GWT Fluctuation

2024_illustrative Nuclear 1 footprint

- 2024_illustrative Nuclear 1 footprint

Legend

Vs

- 1500.0 - 2500.0
- 2500.0 - 3500.0
- 3500.0 - 5000.0
- 500.0 - 1500.0

HORIZONTAL:VERTICAL SCALE 1:2

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DRK CONSULTING		LIQUEFACTION POTENTIAL ASSESSMENT	
COMP	ESDM	04/12/2024	REV BY
REV	1	04/12/2024	ESDM
SCALE	1:5000	RELEASED BY	ESDM
NDIP DOC NO		NDIP	5.15.7
NDIP		Sheet	2 of 2
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Alternative Checks

The first alternative check of the liquefaction potential at the site was carried out by Rust Geotechnical Engineers (SRK Consulting, 2015).

The conclusions of this study are best described in **Figure 5.15.11** The independent assessment highlighted the following:

- Areas in the vicinity of boreholes NEW1, TB12, TB42, TB15 and CPT probe PR3 (south of TB30) were areas in which potentially liquefiable soils were encountered.
- CPT probes PR7 (close to TB37) and PR9 (close to NEW17) highlighted cases where soils were described as ‘borderline liquefiable’, or not conclusively proven to be liquefiable.
- Boreholes NEW20, NEW10, NEW18, TB39, TB26, TB45 and TB43 were highlighted as areas with ‘suspect liquefiable’ soils or soils having the potential to be liquefiable.

Comparing these results with those presented above, there is a correlation in results in the sense that the spatial distribution of liquefaction potential is similar.

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Figure 5.15.11
Results of Assessment of Static Liquefaction Potential (after Rust *et al*)

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5.15.4.4 Cultural and Environmental Considerations

(International Atomic Energy Agency, 2004) highlights protection of cultural resources, such as archaeological sites and artefacts. These issues are dealt with in the environmental impact assessment (EIA) for the proposed nuclear installation(s).

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, if positioned in areas of thick overburden, 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. (International Atomic Energy Agency, 2004) highlights risks related to placement of fill into wetlands. The site has several wetlands (**Section 5.3** and **Section 5.11**) and management of excavated material (spoil) should take this into account.

To place these environmental risks in context, **Drawing 5.15.8** considers the excavation that would be required to take the Illustrative Nuclear 1 Footprint down to a bedrock founding level assuming that excavation slopes will be battered back to angles in the region of 18° for stability reasons (see **Subsection 5.15.4.3.3**). The resulting environmental impacts can be described as follows:

- The volume of cut assuming the entire Illustrative Nuclear 1 Footprint is taken down to bedrock is 6.8 million m³.
- Management of this extent of spoil will be challenging as considerable footprints elsewhere on site (or off site) will be disturbed to appropriately manage topsoil and general cut material – it needs to be borne in mind that the excavated material will bulk up to a considerably larger volume (at least initially).

The cumulative impacts of site disturbance from excavations/spoil stockpiles and groundwater table drawdown during dewatering (**Section 5.11**) will, with

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little doubt, result in significant environmental damage at this site if not carefully planned and designed.

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5.15.5 Site Geotechnical Profile for the Thyspunt Site

5.15.5.1 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 5.15.B** and **Drawings 5.15.1, 5.15.2, 5.15.3** and **5.15.4**);
- the physical properties of soil and rock and values used for classification (**Appendices 5.15.B, 5.15.C** and **5.15.D**);
- shear wave velocities in the site soils (**Subsection 5.15.4.3.1**) – Shear wave velocities in the site rocks are documented in (Council for Geoscience, 2013) and shown in **Drawing 5.15.6** and **5.15.7**;
- mechanical properties of site materials obtained by *in situ* or laboratory tests (**Subsections 5.15.4.2** and **5.15.4.2.3**);
- characteristics of the groundwater table (**Drawing 5.15.5** and **Subsection 5.15.4.1.3**);
- 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 zero to 65 m (in vertical extent) aeolian dominated soil deposits in the western and central portion and these deposits taper in vertical thickness towards the sea and to the east.

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- The aeolian soils are homogenous in grading and are poorly graded with a very high sand size fraction as demonstrated by the laboratory test results.
- Underlying the aeolian deposits, and confined to ancient erosion depressions in the Goudini (predominantly) and Cederberg rocks, are thinly distributed marine sands and a more dominant pebble/cobble layer.
- The aeolian soils and marine deposits (soils and cobbles) together constitute a primary intergranular aquifer the Algoa Aquifer. The groundwater table marks the top of the Algoa Aquifer and high CaCO_3 (in all likelihood originating from the shell-rich site soils) concentrations measured in the groundwater have resulted in calcification of site soils at the groundwater table level where highly competent calcrete horizons are common.
- Groundwater flow within this aquifer is in a south to southeasterly direction and is influenced locally by the orientation of the erosional depression in the Goudini and Cederberg rocks and the pebble/cobble horizon deposited there.
- Soil consistency is variable either side of the groundwater table/calcrete zone and is very dense at this level but becomes loose (in places) to very dense below the water table – the upper surface horizons display the lowest consistency as evidenced by the SPT test and Vs measurements done.
- The generally high soils consistency limits liquefaction potential to small zones on the site that are of looser consistency and located below the groundwater table – should the upper soil horizons become saturated for whatever reason, these soils are likely to increase the site liquefaction potential considerably.
- Mechanical ground improvement in these soils does not normally produce bearing capacities in excess of 200 kPa at best – bearing capacities required in excess of this will only be achieved with cement stabilisation improvement measures.

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- site rocks:
 - Bedrock at the site consists of a wave cut platform/terrace made up of various sequences in the Table Mountain Group.
 - The Skurweberg Formation dominates at the coast and is the oldest formation in the sequence – moving inland (and northwards), the other sequences at the site consist of the Goudini, Cederberg and Peninsula formations.
 - Although all of these sequences exhibit competent rock (albeit fractured with a low but variable RQD), the Goudini and Cederberg formations are marginally less competent than the Skurweberg and Peninsula formations bounding them in the south and north respectively as they appear to be more jointed. When the wave cut platform was developed, the Goudini and Cederberg Formations appear to have been eroded to a greater extent than the Skurweberg and Peninsula Formations, resulting in a depression.
 - This Goudini and Cederberg depression houses considerable cobble/marine sand deposits that influence groundwater flow quite considerably on the site as they constitute a preferential flowpath having higher permeability than the surrounding soils above and rocks below.
 - Rock parameters measured vary greatly within and across the various lithologies on the site, and where data density is high (from laboratory test results), most parameters trend towards a rectangular distribution with wide ranges in values – an exception is rock density that varies very little irrespective of lithology.
 - Rock joint conditions indicate that block failure (sliding, toppling) is likely in all manner of excavations oriented within 30° of bedding strike, but that the rock mass exhibits a good RMR and lateral support of any excavations will not be unduly onerous and can be achieved with conventional lateral support methods.

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5.15.5.2 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 and shales;
- 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;
 - springs²⁵;
 - rock types such as limestone, dolomite, gypsum, anhydrite, halite, *terra rossa* soils²⁶, lavas, weakly cemented clastic rocks, coal or ores;
 - non-conformities in soluble rocks.

However, information gathered to date through intrusive investigations carried out for this SSR do not indicate that any of these features are present on the site apart from springs at the sea (which are present due to the phreatic surface

²⁵ There are several springs at the sea where the groundwater table intersects surface, and isolated springs in the wetlands some distance to the northeast, but these are all distant of the Illustrative Nuclear 1 Footprint.

²⁶ A red soil produced by the weathering of limestone.

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daylighting in the intergranular aquifer). The pre-operational and operational stages of the geotechnical investigation will need to specifically investigate the occurrence of these features on localised foundation footprints, but as indicated, there is no evidence to suggest that any unduly onerous challenges may arise due to any of these features (individually or cumulatively considered).

5.15.5.3 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;
- highly jointed nature of site rocks;
- bias in rock sampling due to highly fractured nature of rock forcing sample selection from better core;
- localised variability in soil consistency below groundwater table;
- undulation in wave-cut platform;
- position of the Skurweberg-Goudini contact;
- bias in point load strength towards stronger rock due to fractured nature of rock;
- high variability in geotechnical properties of the rock due to wide rectangular distribution;
- orientation of cut slopes in rock relative to bedding strike;
- highly transmissive/permeable cobble layer with localised groundwater flow towards the south east;
- potential perched water tables;
- deep cohesionless sand overburden together with an inter-granular aquifer giving rise to very extensive excavations/dewatering requirements and

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potentially major environmental impacts – disposal of extensive volumes of excavated sand potentially causing major environmental impact;

- differential movement across rock structures and lithological units and soil-rock interfaces.

These concerns are raised to ensure that they are considered in design, and do not present unduly onerous constraints on the design and will be mitigated broadly as follows:

- Concerns related to the thick soil overburden and characterisation thereof:
 - Safety related structures will be founded on bedrock, which is the most robust response to dealing with the thick overburden as far as nuclear safety is concerned.
 - Founding on bedrock will require robust dewatering designs: the characterisation of the soils in this Section and of the groundwater regimes in **Section 5.11** present a sound basis to carry out detailed dewatering designs and reliable dewatering systems can be designed and implemented using this background information.
 - It remains (as alluded to elsewhere in this Section) that interception / cut-off of groundwater flow in the cobble layer may pose practical problems at construction stage, but the philosophy of excavating to bedrock to found safety related structures presents an opportunity to excavate through the cobble layer and implement dewatering infrastructure (probably in the form of large subsurface drains).
- Concerns related to site rocks:
 - The jointed nature of the rock and any discontinuities that may exist under the Illustrative Nuclear 1 Footprint will be exposed and mapped in detail when the foundation excavations are made. This detailed mapping will allow final design responses to any potential challenges exposed when analysing the mapping data. Considering the data at hand as documented in this Section, such designs will not be unduly onerous.

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- Orientation of excavations in rock (e.g. tunnels) will be explored in detail once the positions of such excavations are known. At this stage, additional targeted drilling benefitting from knowledge of structure layouts will provide sufficient information to design slope stabilisation mitigations using conventional means (e.g. anchors / rock bolts / shotcrete and mesh).
- The Illustrative Nuclear 1 Footprint (as currently reported) does not span over geological contacts and risks related to this are already mitigated for safety related structures. Adjoining structures and/or linear structures founded in or excavated through rock will be designed as per the above bullet point and such designs will not be unduly onerous once targeted data is obtained for each of these structures.

These mitigation measures are dealt with in the context of various proposed structures in **Sub-section 5.15.6** and **Sub-section 5.15.7**.

5.15.6 Aspects of Works Requiring Geotechnical Design

This SSR is positioned within the characterisation stage, but since plant layouts and imposed loads are not known, this SSR does not satisfy all of the specifications related to this stage of investigation. This section does, however, provide a clear understanding of the geotechnical setting of the site. In the absence of plant-specific design information, a rigorous assessment of the suitability of the site for construction of a nuclear installation(s) is not possible as the site characteristics (capacity) cannot be measured against the proposed loads (demands) in every context.

This makes preliminary evaluation²⁷ of the sufficiency of the site capacities (in terms of the relevant geotechnical characteristics identified in this SSR) in meeting the prospective loads/demands of the proposed nuclear installation(s) the appropriate proximate approach in assessing suitability. A systematic presentation of aspects of work requiring geotechnical design is summarised below and presented in detail in **Appendix 5.15.A**. Where necessary, subjective engineering judgment is relied upon in this appendix to assess the ability of the various aspects of work to appropriately withstand the prospective

²⁷ Preliminary evaluation based on site data but not necessarily representative of a specific foundation footprint and design load.

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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 site characterisation activities undertaken in the characterisation stage of the site.

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

- soil slope stability design mitigating:
 - erosion failure;
 - classic slip-circle failure;
 - liquefaction failure;
 - piping failure.
- rock slope stability design mitigating:
 - toppling failure;
 - wedge failure;
 - planar failure;
 - fall-out 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

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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.
 - other structure design mitigating:
 - pipeline failure;
 - cyclic hydrodynamic loading on breakwaters.
 - construction planning mitigating:
 - environmental impacts related to bulk excavations, dewatering of bulk excavations and groundwater table drawdown.

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:

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- The nuclear installation island and turbine hall will be founded on or in competent bedrock.
- Founding materials must be of such a nature that design of foundations can be simplified, uncertainty reduced and risks to safety thus mitigated through design.
- Alternatively, variability in founding materials must be sufficiently understood such that foundation design can be simplified, uncertainty removed and risks to safety mitigated through design.
- excavations:
 - Excavations through soil overburden and rock are required to reach founding levels (particularly in the Illustrative Nuclear 1 Footprint) and the site geotechnical profile should pose a low risk to excavation stability provided that appropriate excavation designs are carried out and consider dewatering.
 - Alternatively, the geotechnical profile (including the groundwater regime) must be sufficiently understood such that excavation design can be simplified, uncertainty reduced and risks to safety mitigated through design.
 - Excavation design must take cognisance of environmental conservation without risking site safety.
- site locality and access to cooling water:
 - The position of the Illustrative Nuclear 1 Footprint has been provided and is located close to the sea at Thysbaai – the footprint provided is, however large (measuring ~520 m x 480 m) and it is not known whether all structures within this footprint will be founded on rock.
 - The cooling water intake could be via a conventional cooling water intake system or via a tunnel intake, both approaches requiring careful design and planning.
- auxiliary works (infrastructure supporting the nuclear installation(s)) will

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need to support the safety of the nuclear installation(s).

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

Appendix 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 pre-empts design criteria to overcome these mechanisms of failure. Lastly, **Appendix 5.15.A** makes an in-principle assessment of the site capacity related directly to the potential demands (loads) 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 5.15.A**) to form a precursor to future design (or a design concept). Since the site is under scrutiny from regulatory authorities as a potential site for a 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. Subsequent phases of the geotechnical investigations (see **Subsection 5.15.3.2**) will concentrate on design and the geotechnical parameters that are required to support detailed design and ultimately to confirm detailed design.

Highlighting uncertainties will assist in the robust planning of the pre-operational and operational phase geotechnical investigations (see **Subsection 5.15.3.2**) as screening 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 questioning the geotechnical suitability of the site. Acknowledging that the plant layout will be known at that stage, the pre-operational and operational phase geotechnical investigations must be carried out with the aim of closing out uncertainty and informing design.

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

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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 in principle assessment in **Appendix 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 acceptability of the site.

5.15.7.1.1 Founding Conditions

5.15.7.1.2 Subsurface Site Characteristics and Areas of Uncertainty

The main foundations of the proposed nuclear installation (the nuclear island) will be placed in bedrock within the Illustrative Nuclear 1 Footprint and this may give rise to temporary/permanent excavations of depth in sand that may be subject to slope failure particularly since the Goudini/Cederberg cobble layer will transmit considerable groundwater to this area. 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 which is variable with an apparent increase in consistency at (or in the vicinity of) the groundwater table and thereafter randomly increasing and/or decreasing immediately below the groundwater table and generally showing a steady increase to depth with some exceptions – Randomly distributed calcrete horizons present floating islands of higher consistency in the profile and there is some evidence to suggest that a fluctuating groundwater table may have had an influence on the formation of calcrete horizons.
- depth to bedrock which varies from near (or at) surface adjacent to the sea to varying depths in the dune area, peaking at approximately 65 m depth below dune crests, with a notable erosion depression in the Goudini/Cederberg formation rocks – This depression hosts a cobble / pebble layer that acts as a preferential groundwater flowpath.
- the presence of an intergranular aquifer, the top of which is the groundwater table, with strong groundwater flow towards the sea and locally influenced

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by the highly transmissive cobble deposits found in the Goudini/Cederberg Formation depression – this locally diverting flow in a south easterly direction under the site.

- lateral and vertical variability in bedrock founding materials, noting the following:
 - the depositional sequence of argillaceous zones documented in the boring logs contributes to variability;
 - in cases, the weathering profile within site rocks, noting variability between successive strata deposits with extremely hard/unweathered rock deposits alternating randomly with slightly weathered rock and highly weathered rock (sometime argillaceous) in no particular pattern or sequence;
 - the distribution of competent and incompetent materials, which appears to be random, highlighting variability, and this no more evident than in laboratory test results which consistently trend to a rectangular distribution – a feature of the sedimentary rocks.
- the need for founding material improvement measures in medium dense (or less) soils.

The following characteristics of site soils are highlighted:

- site soils are granular and the potential for heave/expansion is thus absent.
- settlement of structures founded in soils can be limited by not founding in medium dense (or less) soils and by soil improvement measures – Targeting medium dense horizons carries a risk of differential settlement as zones of more competent material (e.g. pedogenic zones) will be encountered.

At present, the primary consideration for the nuclear installation(s) foundation safety remains the variability in the geotechnical profile. The site rock properties vary throughout and across lithological boundaries and soil consistency varies vertically and laterally. The Illustrative Nuclear 1 Footprint is located on the Goudini sequence and does not straddle lithological boundaries, but there is a

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significant database compiled in this lithological unit to demonstrate that variability in rock quality must always be considered.

This variability creates a challenge for the nuclear installation(s) design. Therefore, differential rock bearing capacity²⁸ 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(s) foundations considering that future geotechnical investigations will be carried out to close out uncertainty under localised safety sensitive structures. Once footprints within the Illustrative Nuclear 1 Footprint for safety related structures are selected, this variability can be investigated in detail on a localised scale to assess variability that will immediately affect founding. The geotechnical profile described in this SSR will prove invaluable in planning the final stages of data gathering in this context.

Considering the above discussion, **Table 5.15.17** 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 geotechnical profile as well as the confidence in this profile as presented in this SSR;
- the impact that the geotechnical profile may have on the design of conceptual structures;
- data gaps (uncertainties) that require closing out in future geotechnical investigations – data that are 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

²⁸ Bearing Capacity: The carrying capacity of soil/rock materials as an indicator of what loads can safely be placed on such materials.

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works requiring geotechnical design.

The aim of **Table 5.15.17** is to present a preliminary 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 safety stance) from a geotechnical characterisation viewpoint (see **Subsection 5.15.3.2**).

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Table 5.15.17
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	Environmental Impacts
Small, conventional spread and strip foundations (founding in soils).	Localised and ideally shallow founding in the geotechnical profile. Adjacent foundations potentially spanning very dense/dense (e.g. calcrete) zones and medium dense to loose non-cemented zones. If situated near the sea, potentially spanning lithological boundaries in site rocks.	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. Founding in medium dense aeolian 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, Lateral and vertical variations in rock founding conditions. Dynamic loading_(Council for Geoscience, 2013)	Site data are reliable and describes the homogeneity of soils (particle size distribution) across the site as well as the variability of consistency down the profile. Data highlights variability in rock founding conditions. Data lacks site specific focus on a local scale. Detailed PSHA is available to inform design.	Location of safety related structures within the Illustrative Nuclear 1 Footprint 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 liquefaction of soils. Differential settlement may occur – soil improvement measures will be required even for lightly loaded structures, but bearing capacity in excess of 200 kPa will not be attainable. Soil improvement will be optimal if soil is removed and replaced in engineered layers not exceeding 200 mm thick and reliable soil rafts should be in the region of 2 m thick. Founding within/near the groundwater table may introduce seasonal responses in founding soil consistency. Founding to be specified in dense soils or improved soil horizons. Design bearing loads > 200 kPa in soils will require piling. Spanning lithological boundaries with adjacent footings in rock is to be avoided.	Effective 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. Foundation excavations deeper than 1.5 m will require shoring – the presence of groundwater will exacerbate excavation instability. Foundations/blinding concrete will need to be poured immediately after excavating foundation preparation and approval.	No impacts provided correct founding methods are selected and ground improvement measures are carried out correctly.	Groundwater drawdown effecting localised freshwater ecological environment Excavation footprints and cumulative impact of temporary stockpile footprints (e.g. in soil improvement works for larger platforms)
Adjoining small, conventional spread and strip foundations (founding in soils) to monolithic structures (founded in rock)	Transition zones between monolithic structure foundations and adjoining structure foundations (i.e. on a localised scale). Founding depths may vary between monolithic	Liquefaction potential in founding materials and the groundwater table characteristics. Differentials resulting from different founding regimes (rock and soil/improved soil) – introducing both differential bearing capacity and differential dynamic loading regimes. Monolithic structure foundation	Site data are reliable but lacks site specific focus on a local scale.	Location of safety related structures within the Illustrative Nuclear 1 Footprint and site specific vertical extent of compressible soils. Site specific vertical variability in soil consistency needs to	Dewatering to allow access to founding materials and to remove liquefaction potential of soils. 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	Effective dewatering and disposal of extracted groundwater. Over-excavation of monolithic structure footprints requiring engineered backfill in close proximity to monolithic structures. Construction programme /	No 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.	Groundwater drawdown effecting localised freshwater ecological environment Excavation footprints and cumulative impact of temporary stockpile footprints

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Aspect / Structure	Potential Extent of Structure (Geometry)	Geotechnical Characteristics Affecting Founding	Reliability of Existing Geotechnical Data	Data Uncertainties	Design Precautions	Construction Difficulties	Operational Impacts	Environmental Impacts
	structures (founded in rock) and adjoining structures (founded in soil)	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).		be investigated for specific structure interfaces.	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.	sequencing may be impacted by design and/or localised geotechnical conditions.		
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. Lateral (and vertical) variability in rocks. 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 lacks site specific focus on a local scale (relevant to both soils and rocks).	Location of safety related structures within the Illustrative Nuclear 1 Footprint 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 or to soft rock/bedrock horizons where bearing capacity will surpass that of soils.	None – if correct founding methods are employed (e.g. piles)	No impacts provided correct founding methods are selected.	No obvious impacts unless dewatering is required
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). Structures could be founded in soils or rocks depending on location of footprint on the site.	Liquefaction potential of slope excavations and the groundwater table characteristics. Consolidation of loose soils triggered by plant vibrations. Variations in vertical consistency of soils and the random presence of cemented / calcretised zones. Soils will not have bearing capacity >200 kPa even with mechanical ground improvements – desired bearing capacity in excess of this will require cement stabilisation. Variability in vertical and	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.	Location of safety related structures within the Illustrative Nuclear 1 Footprint and site-specific vertical extent of compressible soils. Site specific vertical variability in soil consistency and 3D variability in rock characteristics needs to be investigated for individual structures.	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 – if not, settlement could render certain plant components temporarily inoperable.	Groundwater drawdown effecting localised freshwater ecological environment. Excavation footprints and cumulative impact of temporary stockpile footprints.

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Aspect / Structure	Potential Extent of Structure (Geometry)	Geotechnical Characteristics Affecting Founding	Reliability of Existing Geotechnical Data	Data Uncertainties	Design Precautions	Construction Difficulties	Operational Impacts	Environmental Impacts
		horizontal rock characteristics due to varying degrees of weathering (rectangular distribution of parameters) 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).						
Linear structures (e.g. cooling water canals) Tunnels are treated separately in <u>Table 5.15.18</u>	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). Potentially on an extensive linear scale traversing lithological boundaries	Liquefaction potential and the position of the groundwater table. Variations in vertical consistency of soils and the random presence of cemented / calcretised zones may introduce localised bearing capacity variability and/or excavation difficulties. Variability in 3D rock characteristics due to weathering profile (rectangular distribution of parameters). Dynamic loading regime 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 form bedrock (with high V_s) into soils with V_s orders of magnitude lower.	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.	Location of safety related structures within the Illustrative Nuclear 1 Footprint and site-specific vertical extent of compressible soils. Extent of variability along linear structure alignment (not yet known)	Foundations required spanning laterally extensive weak zones and or lithological boundaries. Differential settlement and tolerances. Differential dynamic loading (across lithological boundaries) and impedance considerations for structures founded in soil. Earthworks requirements through undulating dune surface topography and impacts on dewatering, design against liquefaction failure and slope stability. Ground improvement measures will not increase in bearing capacity above 200 kPa by mechanical means.	Earthworks and materials handling to limit environmental impacts. Dewatering over extended distances.	Plant shutdown due to post construction failure of (e.g.) cooling water supply.	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.
Damped foundations	Under the nuclear island and relatively local (i.e. non linear). Potentially situated in areas of high overburden thickness (inland) or low to zero overburden	Liquefaction potential and the position of the groundwater table impacting on slope stability. Variable 3D rock characteristics due to weathering profile and/or geology (rectangular distribution of parameters) – bearing capacity may vary from low strength (in highly weathered material) to high strength in unweathered	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	Detailed mapping of bedrock underlying foundations – to be carried out after foundation excavations to bedrock are completed. Exact position of the proposed safety	Slope stability of foundation excavation cuts and environmental impacts due to cutback angles and dewatering requirements. Differential dynamic response in different geology and across lithological boundaries must be avoided – lithological boundaries must not be spanned unless specifically designed for.	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	Plant shutdown due to post construction failure.	Groundwater drawdown effecting regional freshwater ecological environment and groundwater users. Cumulative impact of excavation and spoil footprints. Impacts on the receiving environment of

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Aspect / Structure	Potential Extent of Structure (Geometry)	Geotechnical Characteristics Affecting Founding	Reliability of Existing Geotechnical Data	Data Uncertainties	Design Precautions	Construction Difficulties	Operational Impacts	Environmental Impacts
	thickness (near the sea). Structures will be founded in bedrock and on competent (slightly / unweathered rock)	material). Overburden profile and necessary foundation excavations (influence on project programme, excavation feasibility / design / construction and environmental consequence) 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). Seismic response spectra.	with overburden and not easily accessible.	related structures' footprint within the Illustrative Nuclear 1 Footprint.	Integration of damped structure foundations with auxiliary / supporting structure foundations and issues relating to differential settlement and dynamic loading. Damping/impedance within foundation soils. Tension/shear failure over lithological boundaries influencing integration of auxiliary structures. Stability of spoil stockpiles.	backfilling with e.g. mass concrete/cement stabilised soil. Construction programme / sequencing may be impacted by design and/or localised geotechnical conditions.		potentially large scale spoil disposal if not stockpiled.

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5.15.7.1.3 Conceptual Mitigation Measures

This report presents sufficient information to make an in-principle assessment that the site capacity with respect to founding materials will not require unduly onerous designs. In addition, more detailed investigations in later phases of the geotechnical investigation will not be unduly onerous and will benefit greatly from the information in this SSR. The repeated reference to variability of site founding materials cannot go unchecked. During the pre-operational and operational stages, it will be necessary to finalise designs to take account of site specific conditions and variability in conditions at the final nuclear installation footprint where additional site specific investigations will be required to, but not necessarily limited to, provide data related to the uncertainties listed in **Table 5.15.17**.

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 derived from these investigations or can be relocated if unfavourable conditions are encountered.

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 5.15.17**. These are:

- small, conventional spread and strip foundations (founding in soils):
 - Liquefaction failure of founding materials carries a risk in certain areas on the site and dewatering of these materials will reduce the probability of liquefaction failure to zero. Dewatering systems will, however, be required to maintain the drawdown achieved during construction for extended time periods.
 - Soil improvement measures to reduce variability are likely to provide

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solutions for many shallow founding concerns. 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 <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.
- More heavily loaded structures will require piling.
- adjoining small, conventional spread and strip foundations (founding in soils) to monolithic structures (founded in rock and/or on stabilised soil rafts):
 - 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 the likelihood of differential movement.
- towers and stacks:
 - If situated in thick overburden areas, these structures should be piled as any attempts at soil improvement may result in large excavations with associated slope stability concerns, dewatering concerns and associated environmental impacts.
 - If founded on rock, avoidance of geological boundaries and geotechnical discontinuities (shears, faults) is a non-negotiable

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mitigation measure.

- The response of safety related structures to earthquake induced dynamic loading will require specialised design consideration in accordance with (Council for Geoscience, 2013).
- structure containing heavy vibratory machinery and/or settlement sensitive structures:
 - Soil improvement measures are not advised if founded in areas of thick overburden deposits – these structures should be piled or relocated to areas where founding on rock is possible.
 - If founded on rock, avoidance of geological boundaries and geotechnical discontinuities (shears, faults) is a non-negotiable mitigation measure.
 - The response of safety related structures to earthquake induced dynamic loading will require specialised design consideration in accordance with (Council for Geoscience, 2013).
- linear structures:
 - When founded in soil, and depending on the load these structures will impose on founding materials, 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.
 - When founded in rock, excavations along the extent of the structure must be completed, bedrock mapped, and an assessment of the weathering profile conducted to ascertain whether extended weak zones exist. Once again, weak zones must be assessed in context of design loads and either founded directly on weathered rock/soft rock if the bearing capacity is assessed to be sufficient or weathered material should be removed if not and replaced with cement stabilised soil

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and/or mass concrete.

- If structures traverse areas where deep excavations in soil are required (e.g. traversing the dune area), slope stability issues and environmental impacts will be primary design drivers – soil slopes should be battered back to 1 vertical in 3 horizontal after dewatering and management of excavated spoil given due attention.
- The response of safety related structures to earthquake induced dynamic loading will require specialised design consideration in accordance with (Council for Geoscience, 2013) e.g. elastically supported beams be considered in design.
- damped foundations:
 - These foundations will be in rock and the stability of excavated slopes in soils to reach bedrock carries a high risk.
 - Such slopes may consist of soil, rock, or composite slopes. Soil slopes should be battered back to 1 vertical in 3 horizontal after dewatering, and rock slopes running within 30° of bedding strike (270°) 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. Dewatering design will have many design drivers depending on the site specific conditions 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 expose bedrock. 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.

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- Avoidance of geological boundaries and geotechnical discontinuities (shears, faults) is a non-negotiable mitigation measure – in addition, the existence of small shear zones along bedding planes resulting from Permo-Triassic-age compression (folding and thrusting) may, upon detailed mapping, be more prevalent than currently understood and this should be a characteristic that should draw particular attention when mapping the excavations.
- The response of safety related structures to earthquake induced dynamic loading will require specialised design consideration in accordance with (Council for Geoscience, 2013).

5.15.7.2 Integration of Auxiliary Systems

5.15.7.2.1 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. 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 5.15.7.1.1** (in **Table 5.15.17**) under ‘linear foundations’ and ‘adjoining structures’.

Tunnelling is a special case and is addressed separately in **Subsection 5.15.7.3**.

5.15.7.2.2 Conceptual Mitigation Measures

Conceptual mitigation measures are similar to those described in **Subsection 5.15.7.1.1** where adjoining structures are discussed.

5.15.7.3 Tunnelling and Rock Slope Stability

5.15.7.3.1 Subsurface Site Characteristics and Areas of Uncertainty

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

- the soil profile (see summary in **Subsection 5.15.7.1.1**);

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- the rock profile (see summary in **Subsection 5.15.7.1.1**);
- the groundwater table see summary in **Subsection 5.15.7.1.1**;
- the dominance of three major joint sets, their low rock RMR and small fracture spacing is noted – Intersection of these joint sets provides release mechanisms in any manner of excavations for sliding, toppling and rock fall failure.
- Tunnels and other excavations in rock will require careful consideration pertaining to the orientation of cuts relative to the major joints sets (and bedding) as described in some detail in **Subsection 5.15.4.3.1**.
- 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.

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 relevant 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, but it cannot be totally discounted due to the variability in rock joint conditions.

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 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. However, construction methodology will be a critical consideration and may impact on costs and overall project programme.

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The orientation of tunnels will be affected by the undulating bedrock topography, particularly if orientated perpendicular to the sea (i.e. approximately perpendicular to the bedding strike) and the established Goudini/Cederberg bedrock depression described in **Subsection 5.15.4.3**. **Table 5.15.18** interrogates the tunnelling concept in a similar way to that done for foundations in **Table 5.15.17**, with similar aims.

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.

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Table 5.15.18
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 ²⁹	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 parallel to the coast.	Variability in tunnel head influenced by the geotechnical profile and undulating bedrock topography (especially the Goudini depression). Rock joint characteristics and the dominance of three major joint sets. Variability in 3D rock characteristics due to weathering (rectangular 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.	Site data are reliable but lacks 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.	Extent of variability (particularly in rock) along linear 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 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 50° S, Foundations required spanning laterally extensive weak zones. Differential settlement and tolerances. Tunnel orientation to ideally be perpendicular to strike, or if not possible, not within 30° of strike of dominant joint sets noting that special designs and construction provisions will be required where this is not possible. 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	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 Grouting the cavity between rock and tunnel lining.	Plant shutdown 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.

²⁹ Reference to linear structures and adjoining structures in [Table 5.15.22](#) also refer to tunnels – [Table 5.15.23](#) discusses issues relative to tunnels only.

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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
					intersecting joint sets. Overcoming separation, on sub-horizontal joints together with sub-vertical joints giving rise to arch failure. Rock falls can occur from tunnel roofs on the sub-vertical joints, released by the sub horizontal joints. Managing sand intake around the intake structure. Appropriate design of progressive grouting to advance the tunnel excavation where required. Specific failure modes as identified in <u>Subsection 5.15.4.3.1</u>			
Excavations in rock	nuclear installation foundation excavations	Rock joint characteristics and the dominance of three major joint sets. Variability in 3D rock characteristics due to weathering (rectangular 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.	Site data are reliable but lacks 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).	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 50° S, Excavations required to span laterally extensive weak 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 noting that special designs and construction provisions will be required where this is not	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	None – pre operation	Groundwater drawdown effecting regional freshwater ecological environment and groundwater users.

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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
					possible. Overcoming toppling failure depending on the number and orientation of intersecting joint sets. Specific failure modes as identified in <u>Subsection 5.15.4.3.1</u>			

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5.15.7.3.2 Conceptual Mitigation Measures

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. Any orientation, however, may result in rock falls from tunnel roofs on the sub-vertical joints, released by the sub horizontal 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, could be effectively overcome with rockbolts and cable anchors and, if necessary, supplemented with shotcrete which in turn may or may not be mesh reinforced.

An additional consideration is the safe design of tunnel lining. Specifically, tunnel lining at geological boundaries and geotechnical discontinuities may be subjected to differential dynamic loading scenarios. Shear or compression waves arriving at the geological contact at an angle, and considering that the bedding dips steeply (at 50°), reflection of these waves may induce a shear dislocation at the contact in addition to the anticipated (induced) tension loads. Implications of this scenario are to take cognisance of (Council for Geoscience, 2013).

The rock strength and joint conditions will have a direct impact on the rock head³⁰ of proposed tunnels. Required rock head could be at least 3 tunnel diameters to maintain tunnel stability in these materials.

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

³⁰ Rock head: the vertical thickness of rock material above the tunnel.

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the surrounding rock.

Cognisance of seismic loading described in (Council for Geoscience, 2013) will be necessary to improving confidence and ensure site safety. This is the primary mitigation measure to be instituted prior to finalising design.

The geological setting of the site (i.e. in the TMG that has undergone significant historical deformation), the presence of significant groundwater and the proximity to the sea will make the design and implementation of tunnels demanding and costly. Of particular concern would be tunnelling into the sea where no reliable geotechnical characterisation exists. 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 considering tunnels as an option.

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.

5.15.7.4 Other Buried Structures and Submerged Structures

5.15.7.4.1 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) except for near the sea where rock outcrops. The design and construction of buried structures will need to take into consideration the following:

- trench excavations, their stability and excavated materials management;
- groundwater management near the sea;
- knowledge of localised founding conditions;
- use of site materials for construction.

Table 5.15.19 interrogates the concept of founding/constructing buried structures.

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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 could be constructed within marine coffer dams in the wave loading environment at the site and this is considered out of the scope of this report. This should be investigated in detail at an appropriate stage and should such structures be required.

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Table 5.15.19
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, between nuclear installation and municipal supplies – in summary, linear structures randomly placed/orientated on site and surrounds – these structures may be founded in soil or rock.	Liquefaction potential and the position of the groundwater table, particularly if these structures are placed on top of liquefiable soils sand or if they are situated near excavated sand slopes containing liquefiable soils - such slopes may be subject to slope failure, at least during construction. Variations in vertical consistency of soils and the random presence of calcrete zones. 3D variability in rock characteristics. Impedance/amplification of seismic waves propagated from rock into soil (founding) medium. Undulating site surface topography and resulting excavated slope stability.	Site data are reliable but lacks site specific focus on a local scale (i.e. along proposed alignments). Data does not appropriately document the 3D variability in rock characteristics over large distances as the site rocks are covered with overburden and not easily accessible.	Extent of variability along linear structure alignment (not yet known). Effect of geological contacts/transitions on site seismic response	Structures required to span laterally extensive weak/relatively poorer geotechnical zones and/or lithological boundaries. Ground improvement methods will not result in bearing capacity exceeding 200 kPa. Differential settlement and tolerances.	Ground improvement and excavations across undulating landscapes. Material handling to limit environmental disturbance.	Probably related to nuisance impacts (i.e. random/interrupted supply of services to support buildings).	Materials (soil) handling and excavation footprints.

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5.15.7.4.2 Conceptual Mitigation Measures

Founding these structures shallow in the geotechnical profile will require a similar approach to soil improvement as that discussed in **Subsection 5.15.7.1.1**.

5.15.7.5 Earth Structures and their Construction

Earth structures may comprise 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, liquefaction failure or erosion failure. In addition, the groundwater table daylighting on cuts slopes may result in sloughing failure, potentially leading to undercutting and ultimately slope failure.

5.15.7.5.1 Subsurface Site Characteristics and Areas of Uncertainty

Subsection 5.15.7.1.1 to **5.15.7.4** make repeated reference to excavations that will be required to construct the nuclear installation(s) foundations. These excavations will require stabilisation during construction. In addition, once 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 5.15.7.1.1**;
- the shear-strength parameters of soils that are cohesionless and have an internal friction angle of the order of 30°;
- rock joint characteristics referred to in **Subsection 5.15.7.1.1** and the orientation of excavation cuts relative to bedding strike in particular;
- (Council for Geoscience, 2013) dealing with dynamic loading;

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- potential cut-slope geometry in foundation excavations which could range from insignificant cuts to cuts over 65 m in vertical extent.

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

It is not practical to consider lateral support measures for cut slopes in excess of 20 m high. For this reason, excavations will, in all likelihood, be designed to have shallow embankment angles (in the order of 1 vertical to 3 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 5.15.4.4**) and will need to be considered in the nuclear installation design and SAR.
- Dewatering requirements will be robust and considerable to counter safety risks related to lateral loads and liquefaction potential.

Environmental impacts could be high because of bulk excavations, disposal of spoil and dewatering effects on the regional groundwater table depending on where excavations are made.

5.15.7.5.2 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:
 - No extraordinary problems with excavations above the groundwater table are envisaged, but once the groundwater table level is breached, conventional earthmoving plant operations will not be possible without prior dewatering.
 - Since the site is underlain by the Algoa Aquifer, dewatering will require a considerable effort and dewatering design will need to be robust

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enough to consider open excavations for extended time periods (years).

- Isolation of the nuclear installation(s) footprint using impermeable groundwater barriers will need to take cognisance of the cobble deposits should the nuclear installation footprint be located in the region of these deposits – penetrating this horizon will be a challenge and innovative construction methods may be required to successfully place a barrier through the cobble deposits.
- slurry pumping:
 - Slurry pumping above the groundwater table will need a regular water supply and this could be sourced from the Algoa Aquifer (i.e. dewatering product).
 - Thick calcrete horizons encountered in the field investigations could minimise productivity of slurry pumping and the technical feasibility of slurry pumping should be explored in subsequent geotechnical investigations.
 - Isolation of the nuclear installation footprint using impermeable groundwater barriers will need to take cognisance of the cobble deposits.
 - 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, increasing in significance relative to the thickness of overburden at any chosen excavation site.
 - Some soil material may be utilised at the site, but this will be restricted

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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 without cement stabilisation.

- pebble/cobble layer:
 - If the nuclear installation(s) is located on/near these deposits, removal of the rounded gravels pebbles and cobbles will be necessary – at the same time, this material could be a useful source of construction material (i.e. aggregate, rockfill, engineered fill, rip-rap³¹).
 - If the nuclear installation(s) is located elsewhere, the pebble/cobble layer may not be encountered, but it will have an influence on dewatering design.
- rocks:
 - Excavations in site rocks 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 may produce material that is predominantly weathered.
 - Successful quarrying (blasting and hauling of specification envelope materials) for breakwater armour rock and core stone protection will be challenging at the site and the quarrying feasibility will need to be investigated (e.g. to avoid quarrying in shale or siltstone and/or to demonstrate acceptable rock size distribution produced by conventional quarrying activities in highly and variably fractured rock).
 - Rock durability/strength against wave action may also be a consideration.
 - Sources of concrete aggregate at the site will require selective quarrying to target better quality materials – use of TMG rocks as an aggregate source will require appropriate assessment of alkali silica

³¹ Rock used to armor shorelines, streambeds, bridge abutments, pilings and other shoreline structures against scour or water erosion.

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reaction (ASR) potential.

- Selective quarrying will be difficult considering the extensive overburden on the site and the associated environmental concerns that may be involved in exploration and production from quarries – quarries may need to be located off site in outcropping areas.
- Suitable quarries for durable breakwater rock of requisite size distribution may not be available close by.

5.15.7.5.3 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 armour rock and core stone sizes (that can be provided by quarrying);
- loss of armour rock due to settlement into the seabed;
- loss of armour rock due to sea current transportation/displacement.

Table 5.15.20 interrogates the concept of earth structures (concentrating on slopes) in a similar way to that done for foundations in **Table 5.15.17**, with similar aims.

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Table 5.15.20
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
Cut slopes	Extensive areas in the case of damped (nuclear island) structures to localised areas for random platform construction. Linear arrangement for linear and some buried structures thus spanning variable soil and/or rock 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. Erodability (wind and water) of site soils.	Site data are reliable and represents a reliable source of information for bulk earthworks related slope design. Data does not reliably represent localised cut slope scenarios.	Impedance (propagation of seismic waves from rock into soils). Detailed dewatering impacts on groundwater drawdown (related to site specific geographical areas, proximity to the sea etc). 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.	Potential risk posed to safety related structures in the vicinity of poorly designed cut slopes.	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 and the position of the groundwater table. Engineered soil characteristics and shear strength. Construction/engineering of site soils. 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.	Position of fill slopes/platforms. Shear strength of engineered soils.	Dewatering and liquefaction avoidance. Slope drainage. Erosion control. Minimising environmental degradation. Soil moisture density relationships and compactability of fill. Slope stability and the impact of surcharge loading on slope stability.	Meeting compaction specification in high silt content fill and the impact of moisture content fluctuations (exacerbated by the availability of construction water and rainfall). Soil moisture density relationships and compactability 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)
Breakwaters	Coastal structure protecting marine engineering structures,	Rock durability and specification envelope production in variable	Site data are reliable and represents a reliable source of information in	Shallow marine geotechnical conditions. Quarry conditions and	Rock durability against repeated wave impact; Suitable volumes of	Aggressive wave environment. Variable quarrying conditions even when investigated on a	Damage to marine structures, systems or components.	No direct impacts.

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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
	systems or components	material. Rock durability. Shallow marine geotechnical conditions (an unknown).	the site region context. Data do not reliably represent localised characteristics at proposed quarry sites. No shallow marine geotechnical data exist	sites.	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	local scale		

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5.15.7.5.4 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 even though the site does not demonstrate a particular propensity to be marred with liquefaction failures. In addition, there is no risk to foundation failure of the nuclear installation related to liquefaction if the nuclear installation will be founded on bedrock and/or on cement stabilised rafts.
 - Dewatering is therefore the 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**).
 - 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 Algoa 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 cobble deposits if situated above the Goudini/Cederberg rocks.
 - 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:

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- Detailed consideration of excavation stability will be required. It is probable that shallow embankment angles (in the order of 1 vertical to 3 horizontal ~ 18°) will be required to ensure safety during construction.
- The bulk earthworks methodology should consider a combination of conventional earthmoving operation and slurry pumping operations.
- Conventional earthmoving methods should be employed until the operation of mechanical plant is no longer optimal (i.e. below the groundwater table and/or if extensive pedogenic horizons are to be removed).
- Slurry pumping should then be an effective means of earthmoving, as well as initiating groundwater table drawdown.
- Regardless of the earthmoving method employed, high environmental damage risks will result if the nuclear installation(s) is located in thick overburden (dunes).
- Locating the nuclear installation(s) close to the sea (in areas of lesser overburden thickness) is the only mitigating measure in this regard that has a high probability of success.

5.15.7.6 Liquefaction Potential

5.15.7.6.1 Subsurface Site Characteristics and Areas of Uncertainty

As is noted **Subsection 5.15.7.1.1** to **5.15.7.5**, liquefaction potential of soils does not manifest as a great risk at this site provided that upper spoil horizons do not become saturated in future (for whatever reason). Such liquefaction scenarios would impact on a number of the conceptual structures discussed.

Table 5.15.21 interrogates the impact that potentially liquefiable soil may have on the conceptual design of the nuclear installation.

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Table 5.15.21
Liquefaction Potential Impacts on the Concept Design of the Nuclear Installation 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	Site areas described that have high liquefaction potential (<i>in situ</i> soils liquefaction potential) as well as other areas that may be susceptible pending the outcome of parameters gained from the SHA.	Soil consistency 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. Maximum principle effective stress due to overburden or imposed loading (aggravates liquefaction potential)	Soil characterisation data sufficiently reliable to demarcate high and low risk areas as well as reliably sketch the overburden profile on the site. Seismic loading has been confirmed by the PSHA and in general, liquefaction potential is low for the site.	Site specific soil profiles and depth to bedrock.	High risk areas to be avoided in footprint selection process. 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. The influence of the cobble layer above the Goudini/Cederberg Formation rocks on dewatering design and construction of groundwater isolation structures.	Access to bedrock as a founding platform for soil rafts and the associated slope stability and dewatering issues. Penetrating the inaccessible cobble layer (generally at depths in excess of 20m) 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	Site areas described that have high liquefaction potential (<i>in situ</i> soils liquefaction potential) as well as other areas that may be susceptible pending the outcome of parameters gained from the SHA.	Soil consistency 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 demarcate high and low risk areas as well as reliably sketch the overburden profile on the site. Seismic loading has been confirmed by the PSHA and in general, liquefaction potential is low for the site.	Site specific soil profiles and depth to bedrock.	High risk areas to be avoided in footprint selection process. Long term dewatering requirements will need to be robust and capable of maintaining groundwater table drawdown for extended periods (years). The influence of the cobble layer above the Goudini Formation rocks on dewatering design and construction of groundwater isolation structures. Erosion control, slope drainage and access to the excavation.	Penetrating the inaccessible cobble layer (generally at depths in excess of 20m) with groundwater isolation structures (e.g. impermeable curtains). Constructing access ramps to the excavation in soils ranging from loose to dense consistency. Disposal of well-developed calcrete material. 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

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5.15.7.6.2 Conceptual Mitigation Measures

The SSR investigations suggest that liquefaction potential is generally low at the site (**Subsection 5.15.4.2.3**). However, considering the risks associated with liquefaction failure, it is not short sighted to also suggest that liquefaction may be considered a risk by the design engineers in the event that the upper soil horizons become saturated in future for whatever reason. In such scenarios, mitigation against liquefaction failure will be a key design consideration.

Dewatering and/or preventing the upper soil horizons from becoming saturated will remove any risks related to liquefaction failure. It is the robustness of dewatering design, however, which is a more significant challenge. It must be remembered that dewatering of the site for construction of a nuclear installation(s) carries a number of risks, namely:

- high risks to the environment related to drawdown effects (e.g. adverse impact on groundwater use boreholes) and possible sea water intrusion – this is dealt with in detail in **Section 5.11**;
- 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).

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

- Dewatering is complicated since the nuclear installation(s) footprint (within the Illustrative Nuclear 1 Footprint) is located above the cobble deposits in the Goudini/Cederberg depression for a number of reasons:
 - Isolation of the nuclear installation(s) footprint from groundwater intrusion will be difficult as isolation barriers will need to extend through the cobbles and achieving this will introduce any number of construction problems.
 - Placing an isolation barrier on the upgradient side of the Illustrative Nuclear 1 Footprint could lead to groundwater mounding in the

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upgradient zone – with an increased potential of saturating upper soils horizons.

- The cobble layer undoubtedly is a large scale preferential groundwater flow path meaning that greater volumes of groundwater will need to be managed and pumping rates will be high.
- Linked to the above point is the risk that dewatering may impact more regionally if the cobble layer needs to be purged of groundwater thereby increasing the risk of environmental impacts – this is dealt with in detail in **Section 5.11**.
- The long term management/stabilisation of remnant slopes must be understood once the founding locations and levels of safety related structures are fixed.

The primary mitigation measure to reduce liquefaction failure risks at this stage in the geotechnical investigation is therefore to reduce the extent of overburden saturation.

5.15.7.7 Construction in Rock

5.15.7.7.1 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 5.15.7.1.1**;
- the rock profile as described in **Subsection 5.15.7.1.1**;
- dynamic loading regimes as detailed in (Council for Geoscience, 2013);
- rock joint characteristics, as described in **Subsection 5.15.7.1.1**.

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 5.15.7.3**. Other excavations, however, will be required in rock and

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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 costs and programming. The contractor will be forced to factor this into tender rates and programmes and if sufficient emphasis is not placed on this important issue, impacts on site safety could result for the following reasons:

- The site rocks exhibit zones that are hard and abrasive (**Appendix 5.15.B**) and the toughness of these rocks will have an impact on production rates and therefore could impact on programming and costs. Inadequate planning related to this could place construction quality and hence safety at risk.
- 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, etc) – this will have an impact on construction cost and programming.
- Since the site rocks are, to some extent weathered near the bedrock surface, attaining founding levels will require variable excavations in rock. The primary discontinuities in the site rocks are associated with three dominant joint sets, and excavations in bedrock are, for economical reasons, likely to be at steep angles (in excess of 40°). As indicated in **Subsection 5.15.4.3** and considering that saturated conditions will be encountered, rock slope stabilisation may be required.
- This will not be an unduly onerous design issue, but cognisance of the dewatering design and excavation slope angle will be key design drivers.

Table 5.15.22 interrogates the impact that construction in rock may have on the conceptual design of the nuclear installation. Tunnels are a special case and are dealt with in **Subsection 5.15.7.3**.

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Table 5.15.22
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 in either the Skurweberg or Goudini rocks, but not straddling the contact.	Groundwater table. Rock strength and deformability parameters; Rock joint characteristics defining block shapes, sizes and orientations and freedom of movement Position of the excavation. Seismic response spectra.	Site data are reliable but lack site specific focus on a local scale (i.e. at footprint of safety related structures). 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 (TMG Aquifer) and the cobble layer	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° 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.

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5.15.7.7.2 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). The normally high friction angle of TMG rocks (outside of argillaceous zones) will make stabilising measures effective and not unduly onerous or costly. 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 has on construction cost and programme must be highlighted to the contractor and the contractor must provide a detailed risk assessment to prove that these risks have been mitigated in the construction methodology and have been allowed for in contractual rates.

5.15.8 Management of Uncertainties

5.15.8.1 Uncertainties

This section summarises the uncertainties related to the site geotechnical profile relevant to this stage in the geotechnical investigation and proposes methods in managing these uncertainties. These uncertainties are:

- the impact that the positioning of the nuclear island within the Illustrative Nuclear 1 Footprint may have on safety;
- understanding of the localised site liquefaction potential and dewatering requirements should isolation barriers be used to effect dewatering;
- confirmation of any differential seismic loading at lithological contacts – in particular, the effect of the geological contact on site response will be critical for linear structures (e.g. tunnels);
- understanding of localised variability in founding rocks once the positioning of the nuclear island within the Illustrative Nuclear 1 Footprint is fixed;

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- abrasiveness of site rock;
- shallow marine geotechnical conditions that remain un-investigated.

5.15.8.2 Approach to Management of Uncertainties

The approach to managing uncertainties hinges around better understanding of the localised site geotechnical profile with respect groundwater conditions and dewatering effects in the presence of the cobble layer, local seismic response spectra and founding variability in rock and soil. In addition, the geotechnical characterisation of the site will develop with time and once the positioning of the nuclear island within the Illustrative Nuclear 1 Footprint is known, completion of the geotechnical characterisation stage, the pre-operational and operational stages (see **Subsection 5.15.3**) will be required to address uncertainties.

Inherent in this approach will be to:

- monitor certain physical conditions and/or parameters to consistently improve confidence;
- use the data presented in this section to plan subsequent investigative phases in which localised geotechnical data (i.e. data relevant to the immediate nuclear installation(s) footprint) are gathered.

Designs will need to consider seismic loading regimes as described in (Council for Geoscience, 2013).

5.15.8.3 Monitoring Geotechnical Parameters

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 and it is recommended that this is maintained throughout the pre-operational and operational stages at which point appropriate expansion of monitored parameters may be considered. Additional monitoring (on a continuous monthly basis) that may be required in pre-operational and operational stages of the project may include monitoring of:

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- structure settlement;
- lateral movement in structures (e.g. retaining structures or linear structures);
- the efficacy of dewatering systems and regional environmental impacts of dewatering;
- the integrity of soil improvement works, particularly cement stabilised rafts, that are in close proximity to the sea (and sulphate rich environments) and may be saturated or periodically saturated;
- soil/rock slopes employing techniques such as inclinometers, piezometers, iterative slope stability analyses and pore pressure/movement alert levels.

5.15.9 Management System

The geotechnical investigations performed for this SSR entailed the following:

- desk study;
- site investigations;
- cross discipline integration with **Section 5.11**, **Section 5.13** (and the associated PSHA) and **Section 5.14** in particular;
- 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), Siting 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, i.e. not important to nuclear

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safety, 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 quality assurance records for key decisions and methodologies used and 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** – Critical State Triaxial Soil Testing Specification;
- **Appendix 5.15.H** – Soil Laboratory Test Results;
- **Appendix 5.15.I** – Rock Laboratory Test Results.

Prior to the start of the site investigation described herein, the following documents 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;
- Method Statement;
- Risk Assessment;
- Health, Safety and Environmental Management Plan.

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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 and therefore a peer review process was followed. 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 5.15.23**.

Table 5.15.23
Summary of Activities, Links and Quality Requirements

Activity	Chapter/Section Links		Quality Requirements
	Inputs	Outputs	
Intrusive Investigation Data Point Siting	<u>Sections 5.11, 5.13; and 5.14</u> and (Council for Geoscience, 2013). Integrative discussion with the Geohydrology, Geology and Seismic teams to optimise data gathering through intrusive investigations	Optimal presentation of data in <u>Section 5.15</u> 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	<u>Sections 5.11, 5.13, and 5.14.</u> Geological information will be used as key input into the site geotechnical profile and the need for supplementing historical data with SSR intrusive investigations.	<u>Sections 5.11, 5.13 and 5.14.</u> 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

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Laboratory Analysis	<u>Section 5.11.</u> Groundwater quality data used as input to the Geotechnical Characterization <u>Section 5.15</u>	<u>Sections 5.15</u> 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 Detailed description of project specific testing specifications Peer review
Development of the geotechnical profile	<u>Sections 5.11 and 5.13.</u> Integrated data from the Geohydrological and Geology sections used to construct the 3D geotechnical profile.	<u>Sections 5.15</u> 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) Peer review
Monitoring	<u>Section 5.11.</u> Groundwater levels from the Geohydrology section will be used to supplement data gathered in the geotechnical characterization and vice versa.	<u>Sections 5.11, 5.15</u> Data gathered in both technical disciplines will define the groundwater table.	Monitoring protocol

A regulatory compliance table (**Table 5.15.24**) is given below to indicate where the relevant issues have been dealt with in the section.

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Table 5.15.24
Regulatory Compliance Matrix

Act/Regulation	Regulation	Issue	Section where covered
Siting 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	<u>5.15.7</u>
Regulations (Department of Energy, 2010)	4 (5)	Accounting for natural phenomena and potential man-made hazards.	<u>5.15.7</u>
Regulatory Guide RG-0011	7.2.1	Seismic and Geological Considerations	<u>5.15.4, 5.15.7</u>
	7.2.4	Geotechnical Hazards	<u>5.15.4, 5.15.5, 5.15.7</u>

5.15.10 Conclusions

The geotechnical information available for the site to date and presented in this SSR identifies a number of areas of concern and some 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 site licence.

Certain exclusions (see **Subsection 5.15.3.2**) are noted, and their implications on uncertainty discussed (**Subsection 5.15.8**). However, these exclusions do not impact on this SSR presenting sufficient evidence to make conclusions on the site suitability. In particular, the greatest density of data exists for Goudini

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Formation rocks underlying the Illustrative Nuclear 1 Footprint, and conclusions drawn in this Section pertaining to safety related structures are therefore drawn with confidence due to the reliability of data available. The regulatory frameworks/guidelines (**Subsection 5.15.3.2**) make provision in the geotechnical characterisation process to close out these exclusions (as noted in **Subsection 5.15.3.2**). The highlighting of these exclusions is simply to prevent over-reliance on the data contained in this SSR in implementation planning.

The key findings with regard to the technical suitability of the site are contained below:

5.15.10.1 Geotechnical Profile

- soils:
 - The site features a dune system overlying TMG rocks of the Skurweberg, Goudini, Cederberg and Peninsula Formations.
 - The dunes are dominated by cohesionless aeolian soils that are homogenously described as poorly graded with a very high sand size fraction.
 - Dune migration has been monitored since 1942 and there is clear evidence that the dunes have stabilised due to alien vegetation growth – risks related to migrating dunes are therefore low.
 - Marine deposits are encountered in an ancient erosion depression in the Goudini/Cederberg rocks and consist, in places, of thinly distributed marine sands underlain by a more dominant pebble/cobble layer sometimes many metres in thickness – the Illustrative Nuclear 1 Footprint is positioned above this cobble layer and dewatering designs will need to cater for this.
 - High CaCO₃ concentrations (in all likelihood originating from the shell-rich site soils) in the groundwater have resulted in calcification of site soils at the groundwater table level where highly competent calcrete horizons are common, but not continuous.

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- Within isolated looser zones, soils are potentially liquefiable, but the site in general does not present a high liquefaction risk due to the position of the groundwater table beneath zones of low consistency soils.
- groundwater:
 - Cumulatively, the aeolian sand, marine sand and the pebble/cobble layer form the Algoa Aquifer (an intergranular primary aquifer) underlying the full extent of the site except at the sea and the cobble layer only present above the Goudini/Cederberg rocks.
 - The groundwater table marks the top of the Algoa Aquifer and has high measured CaCO_3 concentrations (in all likelihood originating from the shell-rich site soils).
 - Groundwater flow within this aquifer is in a south to southeasterly direction and is locally influenced southeasterly by the orientation of the pebble/cobble horizon in the Goudini/Cederberg depression (orientated northwest-southeast across the site) – the Illustrative Nuclear 1 Footprint is positioned above this cobble layer and dewatering designs will need to cater for this.
- rocks:
 - Bedrock at the site consists of a wave cut platform/terrace made up of various sequences in the Table Mountain Group.
 - The Skurweberg Formation dominates at the coast and is the oldest formation in the sequence – moving inland (and northwards), the other sequences at the site consist of the Goudini, Cederberg and Peninsula formations.
 - Although all of these sequences exhibit competent rock (albeit fractured with a low but variable RQD), the Goudini and Cederberg formations are marginally less competent than the Skurweberg and Peninsula formations bounding them in the south and north respectively. When the wave cut platform was developed, the Goudini and Cederberg formations appear to have been eroded to a greater

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extent than the Skurweberg and Peninsula formations, resulting in a depression. The Goudini and Cederberg Formations are marginally less competent because they appear to be more jointed but not because they may be more weathered which they are not because rock density varies very little irrespective of lithology, refer second paragraph following.

- This Goudini and Cederberg depression houses considerable cobble/marine sand deposits that influence groundwater flow quite considerably on the site as they constitute a preferential flowpath having higher permeability than the surrounding soils above and rocks below.
- Rock parameters measured vary greatly within and across the various lithologies on the site, and where data density is high (from laboratory test results), most parameters trend towards a rectangular distribution with wide ranges in values – an exception is rock density that varies very little irrespective of lithology.
- Discontinuities divide the rock mass into blocks of randomly varying shape. The rock joints are generally tight.
- Rock joint conditions indicate that block failure (sliding, toppling and fall-out) is likely in all manner of excavations depending on their relative orientation to the joints. However, the rock mass exhibits a good RMR and lateral support of any excavations will not be unduly onerous and can be achieved with conventional lateral support methods.

Information gathered to date does not indicate that any of the following are present on the site:

- previous use of the site (e.g. mining activities);
- gas pockets, and swelling rocks and shales;
- zones of weakness or discontinuities in crystalline rocks;
- indicators of potential cavities and susceptibility to ground collapse in the context of:

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- sinks, sink ponds, caves and caverns;
- sinking streams;
- historical ground subsidence;
- natural bridges;
- rock types such as limestone, dolomite, gypsum, anhydrite, halite, *terra rossa* soils³², lavas, weakly cemented clastic rocks, coal or ores;
- non-conformities in soluble rocks.
- The following features do occur at the site:
 - surface depressions in the eroded Goudini/Cederberg wave cut platform where cobbles are common;
 - random springs at the sea where the groundwater table daylights and in the wetlands to the northeast of the site;
 - neither of these features, however, present any indication that site safety could be compromised and mitigation measures accounting for these features in the design are not unduly onerous.

5.15.10.2 Founding Conditions

In the context of the geotechnical profile summarised above the following conclusions can be drawn on the founding conditions of the site:

- Design will have to cater for variability in soil consistency – this will not be unduly onerous and a combination of ground improvement measures, founding methods pertinent to imposed loads and data gathered to close out uncertainties related to site-specific soil profiles, will provide sufficient confidence to design structure foundations in the soils at the site.

³² A red soil produced by the weathering of limestone

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- Design will have to cater for variability in rock characteristics but this will not be unduly onerous provided input from (Council for Geoscience, 2013) is factored into design and that detailed mapping of the nuclear installation(s) footprint is carried out once bedrock is exposed.
- key design precautions are to:
 - factor seismic loading, described in detail in (Council for Geoscience, 2013) into design, in particular to understand impedance characteristics of the site soils and weathered rock zones.
 - 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 or carrying loads to bedrock with e.g. piled foundations.
 - limit the extent of overburden soil in the profile by carefully selecting the footprint position of the safety related structures within the Illustrative Nuclear 1 Footprint – Excavation stability, dewatering design and environmental impacts will be simplified/minimised if overburden depths could be reduced.
 - take care when spanning lithological boundaries (e.g. with linear structures) and well developed geotechnical discontinuities (shears, faults).
 - optimally align the excavations, excavation crests and tunnels in rock relative to the orientations of the bedding and the different joint sets to obviate sliding, toppling and fall-out failure.

5.15.10.3 Liquefaction Potential

At earthquake magnitudes and PGA levels derived from (Council for Geoscience, 2013), minor portions of the site are susceptible to liquefaction with the following conclusions drawn relating to this:

- Current analysis indicates that liquefaction potential exists in isolated areas but in general the site does not exhibit a high liquefaction risk.

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- Poorly consolidated soils in the upper soil horizons will exhibit liquefaction potential should these horizons become saturated in future for whatever reason.

5.15.10.4 Rock Joint Strength

- Three joint sets dominate at the site, and these may all be significant since intersection of the joint sets may result in sliding, toppling or fall-out failure. The stability of the various excavations should be optimised by appropriate orientation relative to the various joint sets.
- Tunnels in particular may pose a risk to the implementation of designs as insufficient information exists to responsibly plan their construction (particularly in the offshore environment). The concern here relates to the extent to which variability in rock strength/hardness/abrasiveness may be encountered along the alignment, making it difficult to quantify/tender/execute designs. This uncertainty will be exacerbated offshore if the rock jointing there is worse or differently oriented.
- Should alternatives to tunnelling be considered, these should like with tunnelling, be subjected to feasibility studies.

5.15.10.5 Slope Stability and Earth Structures

Perhaps the single most significant geotechnical characteristic of the site is the extensive, cohesionless overburden overlying the bedrock and the fact that an intergranular aquifer underlies the entire site. The Illustrative Nuclear 1 Footprint is positioned within overburden deposit on the northwest corner, suggesting that excavation to bedrock in this area will be ~40 m deep. In addition, the cobble horizon underlies the Illustrative Nuclear 1 Footprint. Soils slopes will need to be battered back at angles in the region of 18° and will need to be dewatered before any confidence in slope stability can be assured.

This introduces a number of conceptual design considerations and implications:

- Dewatering design has to consider the pebble/cobble horizon in the Goudini/Cederberg depression (inserting impermeable barriers through this as well as the fact that this horizon is a preferential groundwater flow path influencing flow direction).

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- Since excavations may be open for extended periods of time (years) groundwater drawdown may have regional environmental impacts in addition to the disposal of the extracted groundwater (refer to **Section 5.11** that deals with this challenge in detail) and of the excavated spoil.
- Dewatering systems will have to be robust enough to ensure uninterrupted dewatering of the site particularly during construction.

Key considerations therefore relating to the suitability of the site are the positioning of the safety related structures within the Illustrative Nuclear 1 Footprint and the optimal placing of the safety related structures closer to the sea (where depth to bedrock is ~15 m).

Constructed earth structures and improved ground rafts will by mechanical compaction will not enable bearing capacities >200 kPa to be obtained. Bearing capacities greater than this can only be achieved with cement stabilisation.

5.15.10.6 Auxiliary Structures

Structures supporting the nuclear installation 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 particularly in linear structures (e.g. pipelines).

5.15.10.7 Other Buried Structures

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

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Appendix 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	(In-principle) 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	<u>Appendices 5.15.B, 5.15.C</u> and <u>5.15.H Subsection 5.15.4.2.3</u>	plant layout within Illustrative footprint, geometry, founding levels, excavation levels and imposed loads	pre-operational and operational stages	- 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. <u>The site is licensable as standard civil engineering design can mitigate design challenges and ensure site safety.</u>
		geotechnical profile	descriptive soil and rock profile	<u>Appendices 5.15.B</u> to <u>5.15.J Subsections 5.15.4.3</u> and <u>5.15.5</u>				
		groundwater regime	groundwater table profile	<u>Section 5.11 Subsections 5.15.4.1.3</u> and <u>5.15.5</u>				
		hydrology and hydraulics parameters (design rainfall depths, etc)	specific meteorological, hydrological and hydraulic assessments for the site	<u>Sections 5.8</u> and <u>5.10</u>				
	classic slip circle slope failure	soil indicator parameters	grading and Atterberg limits	<u>Appendices 5.15.B, 5.15.C</u> and <u>5.15.H Subsection 5.15.4.2.3</u>	plant layout within Illustrative footprint, geometry, founding levels, excavation levels and imposed loads	pre-operational and operational stages	- 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. <u>The site is licensable as standard civil engineering design can mitigate design challenges and ensure site safety.</u>
		soil shear strength	friction angle and cohesion	<u>Subsection 5.15.4.3.3</u>				
		groundwater regime	groundwater table profile	<u>Section 5.11 Subsections 5.15.4.1.3</u> and <u>5.15.5</u>				
		geotechnical profile	descriptive soil and rock profile	<u>Appendices 5.15.B</u> to <u>5.15.J Subsections 5.15.4.3</u> and <u>5.15.5</u>				
		surface water management		<u>Section 5.10</u>				
	liquefaction potential of soils impacting slope stability	SPT test results	SPT N values Vs profile	<u>Appendices 5.15.B</u> and <u>5.15.E Subsection 5.15.4.2.4</u>	- plant layout within Illustrative footprint, geometry, founding levels, excavation levels and imposed loads - detailed laboratory testing of undisturbed soil samples at the chosen nuclear installation footprint	pre- operational and operational stages	- 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 low. The design of dewatering systems may present challenges of both a technical and environmental impact nature <u>The site is licensable on condition that the confirmed seismic response spectra are considered in final designs.</u>
		groundwater regime	groundwater table profile	<u>Section 5.11 Subsections 5.15.4.1.3</u> and <u>5.15.5</u>				
		seismic response spectra	PGA and expected earthquake magnitude	<u>Section 5.14, Subsection 5.15.4.1.4</u> and (Council for Geoscience, 2013)				

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Aspect	Mechanism of Failure/Areas of Concern	Capacity and Demand Parameters				Design Considerations	(In-principle) 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 Poisson's ratio, joint conditions	<u>Appendices 5.15.B to 5.15.C</u> and <u>5.15.I Subsection 5.15.4.2.2</u>	plant layout within Illustrative footprint, geometry, founding levels, excavation levels and imposed loads	pre-operational and operational stages	- 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 - <u>the site is licensable as conventional design³³ can mitigate design challenges and ensure site safety.</u>
		assessment of joint conditions	Joint orientation / surface conditions / frequency	<u>Appendices 5.15.B to 5.15.D Subsection 5.15.4.3.1</u>				
		geotechnical profile	descriptive soil and rock profile	<u>Appendices 5.15.B to 5.15.J Subsections 5.15.4.3 and 5.15.5</u>				
		groundwater regime	groundwater table profile	<u>Section 5.11 Subsections 5.15.4.1.3</u> and 5.15.5				
	wedge failure	rock indicator parameters	Core recovery, RQD, fracture frequency, UCS, Young's modulus and Poisson's ratio	<u>Appendices 5.15.B to 5.15.C</u> and <u>A-5.15.I Subsection 5.15.4.2.2</u>	plant layout within Illustrative footprint, geometry, founding levels, excavation levels and imposed loads	pre-operational and operational stages	- 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 - <u>the site is licensable as conventional design can mitigate design challenges and ensure site safety.</u>
		assessment of joint conditions	Joint orientation / surface conditions / frequency	<u>Appendices 5.15.B to 5.15.D Subsection 5.15.4.3.1</u>				
		geotechnical profile	descriptive soil and rock profile	<u>Appendices 5.15.B to 5.15.J Subsections 5.15.4.3 and 5.15.5</u>				
		groundwater regime	groundwater table profile	<u>Section 5.11 Subsections 5.15.4.1.3</u> and <u>5.15.5</u>				
	slip circle failure in soft rock	shear strength of weathered rock		<u>Appendices 5.15.B to 5.15.J</u>	- plant layout within Illustrative footprint, geometry, founding levels, excavation levels and imposed loads - friction angle and cohesion of weathered rock in exposures	pre-operational and operational stages	- 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. <u>Provided all design parameters are gathered, the site is licensable as conventional design can mitigate design challenges and ensure site safety.</u>
		geotechnical profile	descriptive soil and rock profile	<u>Appendices 5.15.B to 5.15.J Subsections 5.15.4.3 and 5.15.5</u>				
		groundwater regime	groundwater table profile	<u>Section 5.11 Subsections 5.15.4.3</u> and <u>5.15.5</u>				
FOUNDATIONS								
		soil indicator parameters	grading and Atterberg limits	<u>Appendices 5.15.B, 5.15.C</u> and <u>5.15.H Subsection 5.15.4.2.3</u>			integration with other foundations	Liquefaction potential is low.

³³ Conventional design refers to standard design practices that have a well-documented history of success.

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Aspect	Mechanism of Failure/Areas of Concern	Capacity and Demand Parameters				Design Considerations	(In-principle) 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	soil shear strength parameters	friction angle and cohesion	<u>Subsection 5.15.4.3.3</u>	- plant layout within Illustrative footprint, geometry, founding levels, excavation levels and imposed loads; - detailed laboratory testing of undisturbed soil samples at the chosen nuclear installation footprint	pre- operational and operational stages	- 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	Conventional foundations present little design challenge should liquefaction potential not be considered a risk. Founding in medium dense soils will not be possible on the site as differential settlement risks will be high in this material <u>The site is licensable on condition that the confirmed seismic response spectra are considered in final designs.</u>
		geotechnical profile	descriptive soil and rock profile	<u>Appendices 5.15.B to 5.15.J</u> <u>Subsections 5.15.4.3 and 5.15.5</u>				
		SPT test results for liquefaction assessment	SPT N values	<u>Appendices 5.15.B and 5.15.E</u> <u>Subsection 5.15.4.2.4</u>				
		groundwater regime	groundwater table profile	<u>Section 5.11</u> <u>Subsections 5.15.4.3 and 5.15.5</u>				
adjoining spread footings to monolithic structures	differential settlement	soil indicator parameters	grading and Atterberg limits	<u>Appendices 5.15.B, 5.15.C and 5.15.H</u> <u>Subsection 5.15.4.3</u>	- plant layout within Illustrative footprint, geometry, founding levels, excavation levels and imposed loads; - detailed laboratory testing of undisturbed soil samples at the chosen nuclear installation footprint	pre- operational and operational stages	- 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 low. Conventional foundation interface designs will present little challenge should liquefaction potential not be considered a risk and if structures are founded to avoid differential dynamic loading. <u>The site is licensable on condition that the confirmed seismic response spectra are considered in final designs.</u>
		soil shear strength parameters	friction angle and cohesion	<u>Subsection 5.15.4.3.3</u>				
		geotechnical profile	descriptive soil and rock profile	<u>Appendices 5.15.B to 5.15.J</u> <u>Subsections 5.15.4.3 and 5.15.5</u>				
		groundwater regime	groundwater table profile	<u>Section 5.11</u> <u>Subsections 5.15.4.1.3 and 5.15.5</u>				
		SPT test results for liquefaction assessment	SPT N values	<u>Appendices 5.15.B and 5.15.E</u> <u>Subsection 5.15.4.1.4</u>				
	differential dynamic loading	Vs and PGA in different geological formations (rocks and soils)	PGA and expected earthquake magnitude	<u>Section 5.14, Subsection 5.15.4.1</u>				
towers and stacks	bearing failure, sliding failure, settlement and differential settlement	soil indicator parameters	grading and Atterberg limits	<u>Appendices 5.15.B, 5.15.C and 5.15.H</u> <u>Subsection 5.15.4.2.3</u>	- plant layout within Illustrative footprint, geometry, founding levels, excavation levels and imposed loads; - detailed laboratory testing of undisturbed soil samples at the chosen nuclear installation footprint	pre- operational and operational stages	- 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 low. Conventional foundation designs will present little challenge should liquefaction potential not be considered a risk and if structures are founded to avoid differential dynamic loading. <u>The site is licensable on condition that the confirmed seismic response spectra are considered in final designs.</u>
		soil shear strength parameters	friction angle and cohesion	<u>Subsection 5.15.4.3.3</u>				
		geotechnical profile	descriptive soil and rock profile	<u>Appendices 5.15.B to 5.15.J</u> <u>Subsections 5.15.4.3 and 5.15.5</u>				
		groundwater regime	groundwater table profile	<u>Section 5.11</u> <u>Subsections 5.15.4.1.3 and 5.15.5</u>				

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Aspect	Mechanism of Failure/Areas of Concern	Capacity and Demand Parameters				Design Considerations	(In-principle) 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	<u>Appendices 5.15.B</u> and <u>5.15.E</u> <u>Subsection 5.15.4.2.4</u>			site soil capacity with respect to trigger mechanisms	
		rock indicator parameters	Core recovery, RQD, fracture frequency, UCS, Young's modulus and Poisson's ratio	<u>Appendices 5.15.B</u> to <u>5.15.C</u> and <u>5.15.I</u> <u>Subsection 5.15.4.2.2</u>				
heavy vibratory machinery and settlement sensitive structures	bearing failure, sliding failure, settlement and differential settlement	soil indicator parameters	grading and Atterberg limits	<u>Appendices 5.15.B</u> , <u>5.15.C</u> and <u>5.15.H</u> <u>Subsection 5.15.4.2.3</u>	- plant layout within Illustrative footprint, geometry, founding levels, excavation levels and imposed loads; - detailed laboratory testing of undisturbed soil samples at the chosen nuclear installation footprint	pre- operational and operational stages	- 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 low. Conventional foundation designs will present little challenge should liquefaction potential not be considered a risk and if structures are founded to avoid differential dynamic loading. <u>The site is licensable on condition that the confirmed seismic response spectra are considered in final designs.</u>
		soil shear strength parameters	friction angle and cohesion	<u>Subsection 5.15.4.3</u>				
		geotechnical profile	descriptive soil and rock profile	<u>Appendices 5.15.B</u> to <u>5.15.J</u> <u>Subsections 5.15.4.3</u> and <u>5.15.5</u>				
		SPT test results for liquefaction assessment	SPT N values	<u>Appendices 5.15.B</u> and <u>5.15.E</u> <u>Subsection 5.15.4.2.4</u>				
		groundwater regime	groundwater table profile	<u>Section 5.11</u> <u>Subsections 5.15.4.1.3</u> and <u>5.15.5</u>				
		rock indicator parameters	Core recovery, RQD, fracture frequency, UCS, Young's modulus and Poisson's ratio	<u>Appendices 5.15.B</u> to <u>5.15.C</u> and <u>5.15.I</u> <u>Subsection 5.15.4.2.2</u>				
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 Poisson's ratio	<u>Appendices 5.15.B</u> to <u>5.15.C</u> and <u>5.15.I</u> <u>Subsection 5.15.4.2.2</u>	plant layout within Illustrative footprint, geometry, founding levels, excavation levels and imposed loads; confirmed Vs and seismic response spectra	pre- operational and operational stages	- integration with other foundations - dewatering requirements and maintenance of dewatering systems - ground improvement measures - construction sequencing - seismic response - settlement tolerance	Final design to consider PSHA outcomes (seismic loading) - 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.
		assessment of joint conditions	Joint orientation / surface conditions / frequency	<u>Appendices 5.15.B</u> to <u>5.15.D</u> <u>Subsection 5.15.4.3.1</u>				
		geotechnical profile	descriptive soil and rock profile	<u>Appendices 5.15.B</u> to <u>5.15.J</u> <u>Subsections 5.15.4.3</u> and <u>5.15.5</u>				

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Aspect	Mechanism of Failure/Areas of Concern	Capacity and Demand Parameters				Design Considerations	(In-principle) Assessment of the Site Capacity and Impacts on Site Safety	
		Required	Available Parameters		Outstanding Parameters			
			Parameter	Where Found in SSR	Parameter			Programme for obtaining
	differential dynamic loading and implications of uncertainty to all manner of failure mechanisms	Differential Vs and PGA	PGA and expected earthquake magnitude	<u>Section 5.14, Subsection 5.15.4.1</u>			<u>The site is licensable on condition that the confirmed seismic response spectra 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.</u>	
	seismic wave distortion	seismic response spectra	PGA and expected earthquake magnitude	<u>Section 5.14, Subsection 5.15.4.1</u>				
	tension/shear failure over lithological boundaries, shear zones and faults	geotechnical profile	descriptive soil and rock profile	<u>Appendices 5.15.B to 5.15.J Subsections 5.15.4.3 and 5.15.5</u>				
mass foundations	bearing failure, sliding failure, settlement and differential settlement	rock indicator parameters	Core recovery, RQD, fracture frequency, UCS, Young’s modulus and Poisson’s ratio	<u>Appendices 5.15.B to 5.15.C and 5.15.I Subsection 5.15.4.2.2</u>	plant layout within Illustrative footprint, geometry, founding levels, excavation levels and imposed loads;	pre- operational and operational stages	- integration with other foundations - dewatering requirements and maintenance of dewatering systems - ground improvement measures - construction sequencing - seismic response - settlement tolerance	Final design to consider PSHA outcomes (seismic loading) - 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. <u>The site is licensable on condition that the confirmed seismic response spectra 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.</u>
		assessment of joint conditions	Joint orientation / surface conditions / frequency	<u>Appendices 5.15.B to 5.15.D Subsection 5.15.4.3.1</u>				
		geotechnical profile	descriptive soil and rock profile	<u>Appendices 5.15.B to 5.15.J Subsections 5.15.4.3 and 5.15.5</u>				
	differential dynamic loading and implications of uncertainty to all manner of failure mechanisms	Differential Vs and PGA	PGA and expected earthquake magnitude	<u>Section 5.14, Subsection 5.15.4.1</u>				
	seismic wave distortion	seismic response spectra	PGA and expected earthquake magnitude	<u>Section 5.14, Subsection 5.15.4.1</u>				
	tension/shear failure over lithological boundaries, shear zones and faults	geotechnical profile	descriptive soil and rock profile	<u>Appendices 5.15.B to 5.15.J Subsections 5.15.4.3 and 5.15.5</u>				
COOLING WATER INTAKE/OUTLET STRUCTURES								

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Aspect	Mechanism of Failure/Areas of Concern	Capacity and Demand Parameters				Design Considerations	(In-principle) Assessment of the Site Capacity and Impacts on Site Safety	
		Required	Available Parameters		Outstanding Parameters			
			Parameter	Where Found in SSR	Parameter			Programme for obtaining
Tunnels, canals and shafts	traversing lithological boundaries, soft/shear/fault zones	geotechnical profile	descriptive soil and rock profile	<u>Appendices 5.15.B to 5.15.J Subsections 5.15.4.3 and 5.15.5</u>	plant layout within Illustrative footprint, geometry, founding levels, excavation levels and imposed loads;	pre- operational and operational stages	- integration with other construction elements - dewatering requirements and maintenance of dewatering systems - ground improvement measures - construction sequencing - seismic response - variability of dynamic response over lithological boundaries - calculated joint shear strength parameters (subjective assessment) and rock mass behaviour - settlement tolerance	Final design to consider PSHA outcomes (seismic loading) - 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. <u>The site is licensable on condition that the confirmed seismic response spectra 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.</u>
	water inflow during construction	assessment of joint conditions	Joint orientation / surface conditions / frequency	<u>Appendices 5.15.B to 5.15.D Subsection 5.15.4.3.1</u>				
		groundwater regime	groundwater table profile	<u>Section 5.11 Subsections 5.15.4.1.3 and 5.15.5</u>				
	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 expected earthquake magnitude	<u>Section 5.14, Subsection 5.15.4.1</u>				
		geotechnical profile	descriptive soil and rock profile	<u>Appendices 5.15.B to 5.15.J Subsections 5.15.4.3 and 5.15.5</u>				
	block failure into tunnel excavations	rock indicator parameters	Core recovery, RQD, fracture frequency, UCS, Young’s modulus and Poisson’s ratio	<u>Appendices 5.15.B to 5.15.C and 5.15.I Subsection 5.15.4.2.2</u>				
		assessment of joint conditions	Joint orientation / surface conditions / frequency	<u>Appendices 5.15.B to 5.15.D Subsection 5.15.4.3.1</u>				
	Squeezing failure related to rock mass controlled behaviour as opposed to structure controlled	Differential Vs and PGA seismic response spectra	PGA and expected earthquake magnitude	<u>Section 5.14, Subsection 5.15.4.1</u>				
		geotechnical profile	descriptive soil and rock profile	<u>Appendices 5.15.B to 5.15.J Subsections 5.15.4.3 and 5.15.5</u>				
		assessment of joint conditions	Joint orientation / surface conditions / frequency	<u>Appendices 5.15.B to 5.15.D Subsection 5.15.4.3.1</u>				

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		Required	Available Parameters		Outstanding Parameters			
			Parameter	Where Found in SSR	Parameter			Programme for obtaining
		rock indicator parameters	Core recovery, RQD, fracture frequency, UCS, Young's modulus and Poisson's ratio	<u>Appendices 5.15.B to 5.15.C</u> and <u>5.15.I Subsection 5.15.4.2.2</u>				
	dislocation at bends	Differential Vs and PGA seismic response spectra	PGA and expected earthquake magnitude	<u>Section 5.14, Subsection 5.15.4.1</u>				
interfaces between tunnels and intake/NPP structures	differential settlement	rock indicator parameters	Core recovery, RQD, fracture frequency, UCS, Young's modulus and Poisson's ratio	<u>Appendices 5.15.B to 5.15.C</u> and <u>5.15.I Subsection 5.15.4.2.2</u>	plant layout within Illustrative footprint, geometry, founding levels, excavation levels and imposed loads;	pre- operational and operational stages	- integration with all structures - dewatering requirements and maintenance of dewatering systems - ground improvement measures - construction sequencing - seismic response - settlement tolerance	Final design to consider PSHA outcomes (seismic loading) - 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. <u>The site is licensable on condition that the confirmed seismic response spectra are considered in final designs to ensure site safety and that interfaces are positioned such that lithological boundaries and/or shear/fault zones are not spanned.</u>
		assessment of joint conditions	Joint orientation / surface conditions / frequency	<u>Appendices 5.15.B to 5.15.D Subsection 5.15.4.3.1</u>				
		geotechnical profile	descriptive soil and rock profile	<u>Appendices 5.15.B to 5.15.J Subsections 5.15.4.3</u> and <u>5.15.5</u>				
		soil indicator parameters	grading and Atterberg limits	<u>Appendices 5.15.B, 5.15.C</u> and <u>5.15.H Subsection 5.15.4.2.3</u>				
	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 expected earthquake magnitude	<u>Section 5.14, Subsection 5.15.4.1</u>				
		geotechnical profile	descriptive soil and rock profile	<u>Appendices 5.15.B to 5.15.J Subsections 5.15.4.3</u> and <u>5.15.5</u>				
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 Poisson's ratio	<u>Appendices 5.15.B to 5.15.C</u> and <u>5.15.I Subsection 5.15.4.2.2</u>	plant layout within Illustrative footprint, geometry, founding levels, excavation levels and imposed loads;	pre- operational and operational stages	- integration with between structural elements - dewatering requirements and maintenance of dewatering systems - ground improvement measures	Liquefaction potential is low. Conventional pipeline founding designs will present little challenge should liquefaction potential not be considered a risk and if structures are founded to avoid differential dynamic loading.

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		Required	Available Parameters		Outstanding Parameters			
			Parameter	Where Found in SSR	Parameter			Programme for obtaining
		soil indicator parameters	grading and Atterberg limits	<u>Appendices 5.15.B, 5.15.C</u> and <u>5.15.H Subsection 5.15.4.2.3</u>	detailed laboratory testing of undisturbed soil samples at the chosen nuclear installation footprint		- construction sequencing - seismic response - settlement tolerance - trigger mechanisms for liquefaction failure - site soil capacity with respect to trigger mechanisms	<u>The site is licensable on condition that the confirmed seismic response spectra are considered in final designs.</u>
		geotechnical profile	descriptive soil and rock profile	<u>Appendices 5.15.B</u> to <u>5.15.J Subsections 5.15.4.3</u> and <u>5.15.5</u>				
		groundwater regime	groundwater table profile	<u>Section 5.11 Subsections 5.15.4.1.3</u> and <u>5.15.5</u>				
		SPT test results for liquefaction assessment	SPT N values	<u>Appendices 5.15.B</u> and <u>5.15.E</u>				
breakwaters	construction material production failure (from quarries) related to blasting production	rock indicator parameters	Core recovery, RQD, fracture frequency, UCS, Young's modulus and Poisson's ratio	<u>Appendices 5.15.B</u> to <u>5.15.C</u> and <u>5.15.I Subsection 5.15.4.2.2</u>	- construction programme, plant layout geometry - quarry feasibility studies	pre- operational and operational stages	- 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 is not likely to be unduly onerous. <u>The site is licensable on condition that the appropriate quarry feasibility investigations are undertaken.</u>
		assessment of joint conditions	Joint orientation / surface conditions / frequency	<u>Appendices 5.15.B</u> to <u>5.15.D Subsection 5.15.4.3.1</u>				
	erosion failure	marine hydrodynamic environment	peak wave loads	<u>Section 5.9</u>				
floor slabs	Settlement and differential settlement	soil indicator parameters	grading and Atterberg limits	<u>Appendices 5.15.B, 5.15.C</u> and <u>5.15.H Subsection 5.15.4.2.3</u>	- founding levels and imposed loads;	pre- operational and operational stages	- soil improvement - settlement tolerance	<u>Conventional design will mitigate design challenges and the site is licensable.</u>
reservoirs, tanks and dams	Settlement and differential settlement	soil indicator parameters	grading and Atterberg limits	<u>Appendices 5.15.B, 5.15.C</u> and <u>5.15.H Subsection 5.15.4.2.3</u>	plant layout within Illustrative footprint, geometry, founding levels and imposed loads	pre- operational and operational stages	- Loads - Size - Strength - settlement tolerance	<u>Conventional design will mitigate design challenges and the site is licensable.</u>
		geotechnical profile	descriptive soil and rock profile	<u>Appendices 5.15.B</u> to <u>5.15.J Subsections 5.15.4.3</u> and <u>5.15.5</u>				
	slope stability of earth structures	(see slope stability above)						
CONSTRUCTION DESIGN								
excavation of the construction terrace		plant layout, geometry, founding levels, excavation levels and imposed loads			plant layout within Illustrative footprint, geometry, founding levels and imposed loads	pre- operational and operational stages	- mass haul design - spoil footprint optimisation - risk assessment of spoil stockpile failure to safety related structures	Conventional design will need to mitigate sufficiently against environmental damage within the Illustrative footprint. <u>Site licensability is severely impacted by potential threats to the environment with regards to large scale foundation excavations.</u>

Aspect	Mechanism of Failure/Areas of Concern	Capacity and Demand Parameters					Design Considerations	(In-principle) Assessment of the Site Capacity and Impacts on Site Safety
		Required	Available Parameters		Outstanding Parameters			
			Parameter	Where Found in SSR	Parameter	Programme for obtaining		
	extent of soil excavation, disposal of spoil (excavated soil) and dewatering causing unacceptable environmental damage	geotechnical profile	descriptive soil and rock profile	<u>Appendices 5.15.B</u> to <u>5.15.J</u> <u>Subsections 5.15.4.3</u> and <u>5.15.5</u>	shear strength parameters and groundwater regime of re-worked spoil			
	Slope failure of spoil stockpiles	shear strength parameters and groundwater regime of re-worked spoil						

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The following data are provided in electronic format:

Appendix 5.15.B

Borehole Logs

Appendix 5.15.C

Borehole Core Photographs

Appendix 5.15.D

Rock Joint Condition Logs

Appendix 5.15.E

Results of the Standard Penetration Tests (SPT)

Appendix 5.15.F

Result of the Dynamic Penetrometer Super Heavy (DPSH) Tests

Appendix 5.15.G

Critical State Triaxial Testing Project Specification

Appendix 5.15.H

Soil Laboratory Test Results

Appendix 5.15.I

Rock Laboratory Test Results