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Engineering geotechnical model and geotechnical interpretation, intake and outfall tunnels, Alkimos seawater desalination plant, Western Australia

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ABSTRACT: The Alkimos SeaWater Alliance (ASWA) is designing and constructing Stage 1 of the Alkimos Seawater Desalination Plant (ASDP). This paper outlines the development of an engineering geotechnical model (EGM) for a 2.6 km intake tunnel and 4.1 km outfall tunnel. The EGM considers the variable subsurface conditions along the tunnel alignment, with particular focus on the Tamala Limestone, Ascot, and Rockingham Formations through which the tunnel will be constructed. This paper also describes the interpretation of the characteristic geotechnical and hydrogeological parameters used for design, taking into consideration identified geotechnical risks, limited groundbreaking and non-invasive investigations, in situ testing, pumping and injection well tests, laboratory analysis, published correlations, and comparisons with parameters assessed for the onshore works and local projects that encountered similar ground and groundwater conditions.

1 INTRODUCTION

The Alkimos Seawater Desalination Plant (ASDP), located 40 km north of Perth, will deliver 50 GL of drinking water annually in Stage 1, with Stage 2 adding another 50 GL. The nearshore components include the Seawater Intake Tunnel (SIT), Brine Outfall Tunnel (BOT), two Seawater Intake Riser (SIR) structures, and a Brine Linear Outfall Diffuser (BLOD) structure (subject to state environmental approval). This paper focuses on the tunnels. A site layout plan is shown in Figure 1.

Figure 1 Plan layout of Outfall and Intake Tunnel

2 INTAKE AND OUTFALL TUNNELS

The SIT and BOT are 2.6 km and 4.1 km respectively and will be excavated using slurry Tunnel Boring Machines (TBMs) with a 4.185 m excavation diameter. Excavations will start from onshore at the temporary TBM access shaft, progressing seaward. The tunnels will be lined with steel fibre reinforced concrete segments (3.505 m ID, 3.905 m OD). TBM shields will be sacrificed at drive completion. The tunnels are relatively shallow with tunnel crown depths varying from a minimum of approximately 12.3 m (intake tunnel) to a maximum of approximately 46.1m below surface or seabed level (outfall tunnel). For the onshore section of the tunnel, it will be constructed between approximately 4.7 m and 20.6 m below the water table. For the offshore section of the tunnel, it will be constructed below the seabed where water depths below sea level range from 20.6 m to 32.2 m.

3 GEOLOGY

The Alkimos site's surface geology is characterised by the Holocene Safety Bay Sand onshore, forming dunes up to 40 m AHD and 25 m thick, and Marine Sediments offshore, a variable layer of shelly sand and gravel up to 9.7 m thick. Both units are relatively recent formations, with the Safety Bay Sand continuing to develop today and the Marine Sediments subject to ongoing wave and current action.

Underlying these surface layers is the late Pleistocene Tamala Limestone, a widespread formation that creates the Spearwood Dune System and offshore reefs. This variably cemented carbonate rock mass, interlayered with uncemented sand bodies, is found beneath the Safety Bay Sand onshore and the Marine Sediments offshore, sometimes exposed as reefs. The Tertiary Ascot Formation underlies the Tamala Limestone. It consists of carbonate calcareous materials with variable properties, likely representing shallow marine to lagoonal environments. Below this, along the outer part of the Outfall Tunnel alignment lies the informal late Tertiary 'TQ-Sand' unit (possibly the Rockingham Formation), situated between the Ascot Formation and the underlying Cretaceous Osborne Formation.

4 HYDROGEOLOGY

Historically, groundwater levels onshore in the area typically ranged between RL 0.7 m AHD and RL 1.0 m AHD. Tidal ranges of approximately 0.1 m to 0.8 m have been observed in near-coastal groundwater. Away from the coast, seasonal groundwater fluctuations along the tunnel alignment typically range from RL 0.6 m AHD (seasonal low) to RL 0.9-1.0 m AHD (seasonal high). For the design of the onshore tunnel sections, recommend groundwater levels, considering a 1 in 100 Year ARI event and accounting for potential climate influences based on published state government data range from a minimum of 0.5 m AHD to 0.25m AHD to a maximum of 1.65m AHD to 2.3 m AHD.

5 SEA LEVEL

The offshore mean sea level (MSL) at Alkimos is 0 m AHD, with a Lowest Astronomical Tide (LAT) of -0.51 m AHD and Highest Astronomical Tide (HAT) of 0.63 m AHD. Sea levels are used in tunnel design for assessing groundwater pressures on the tunnel lining.

6 GEOTECHNICAL INVESTIGATIONS

6.1 *Geotechnical Risks*

Key risks for TBM tunnels given the existing ground and groundwater conditions at the ASDP included variable earth pressure coefficients, karsts, highly permeable zones, mixed face tunnelling conditions and ground settlements within the onshore section of the tunnel.

6.2 *Fieldworks*

6.2.1 *Historical Investigations*

ASWA conducted a gap analysis to assess what additional onshore and nearshore geotechnical investigations were needed to supplement historical data for detailed design. Existing data included 24 boreholes, 11 Cone Penetration Tests (CPTs). The historical boreholes were drilled to depths of 5.9 m to 46.3 using a variety of methods conventional drilling using Jack-Up Barge (JUB), diver operated drilling rig and Portable Remote Operated Drill (PROD) for drilling and CPTs. The geophysical surveys consisted of 8 km^2 Multibeam Echo Sounder (MBES), 8 km^2 Sidescan Sonar (SSS), 105 km of seismic reflection, 54 km of seismic refraction, 30 km of electrical resistivity, and 112 km of magnetometer, and geotechnical soil, rock, geochemical and chemical laboratory testing.

6.2.2 *Detailed Investigations*

Detailed investigations aimed to address geotechnical risks and data gaps to improve understanding of ground and groundwater conditions. These investigations comprised 6 onshore boreholes, 9 nearshore boreholes, 12 groundwater monitoring wells, 5 pumping wells, 6 injection tests, groundwater tests, and overwater geophysical surveys (approximately 0.75 km^2 MBES, 30.8 km continuous marine seismic refraction (CMSR), 32.5 km multi-channel seismic reflection (MCSR), and 37.6 km Pinger SBP).

The onshore boreholes were drilled using a Hydropower Scout V track-mounted drilling rig. Boreholes terminated at depths ranging from 43.73 m to 55.50 m below ground level. The borehole drilling utilised PQ triple barrel coring equipment (PQ3). Boreholes were logged in accordance with AS1726:2017 with SPTs carried out in accordance with AS1289.6.3.1-2004. The nearshore geotechnical investigation used an Edson 3000 rig mounted on a four-legged JUB. Boreholes were drilled (fully cased) to depths ranging from 26 m to 30 m below the seabed using PQ3 coring equipment. The groundwater boreholes were drilled using a Hydropower Scout V track mounted drill rig. Boreholes were terminated at depths ranging from 30.0 m to 40.5 m below ground level. The boreholes were drilled using mud rotary equipment to advance them.

6.3 *Laboratory Testing*

Comprehensive laboratory testing was conducted as part of the historical and detailed investigations. Testing included soil classification tests, density tests, permeability tests, chemical tests

(ASS Testing), rock tests (point load strength index, Uniaxial Compressive Strength (UCS), Cerchar abrasion, Brazilian indirect tensile strength) and advanced analyses (palynology, X-Ray Diffractometer, Scanning Electron Microscope). These tests aimed to refine geological and geotechnical models and derive site-specific characteristic design parameters.

7 ENGINEERING GEOLOGICAL MODEL

7.1 *Development of Model*

The EGM incorporated data from various sources throughout the project's lifecycle, including reinterpretation of information from historical investigations. It was structured by dividing observed materials into Units sharing similar characteristics and distinguishable from adjacent Units. While Units generally aligned with geological stratigraphy, some were merged or split based on engineering behaviour similarities or differences and their relevance to the project. The resulting model (see Table 1), based on intrusive investigations, in situ and laboratory testing, approximates actual site conditions. Use of the geophysical data obtained from CMSR, and MCSR to inform the EGM was limited due to issues with the resolution and reliability of the results, however it was correlated with the borehole data where possible. In particular, where strong limestone outcropped at the seabed (reef structures) or was found at shallow depth, the resolution of the seismic refraction and reflection with depth was impeded. The lesson learnt is that the benefit of conducting geophysics in such nearshore environments should be carefully considered in terms of the usefulness and limitations of the data that can be retrieved at depth.

Table 1. Developed Engineering Geological Model for the nearshore tunnel alignments

Project Unit	Project Unit Name	Approximate Geological Age	Project Sub-Unit	Description
2	Marine Sediments	Recent to Quaternary	2c	Coarse grained (i.e., materials of a non-cohesive nature) soils
5	Tamala Limestone	Quaternary	5a	Fresh to Moderately Leached limestone / carbonate sandstone in the Unit
			5b	Moderately to Highly Leached limestone / carbonate sandstone in the Unit
			5c	Extremely Leached Limestone to Residual Soils. Possibly a range of relic paleosols, solution feature infill, or carbonate depleted layers within the Tamala Limestone
6	Ascot Formation	Tertiary	6a	Variably cemented silty sands with variable quantities of authogenic nodules (recovered as gravels during investigation). Local layers cemented to rock generally varying from very low to medium strength. Typically varying from type D1 duricrust to type D3 duricrust according to AS1726:2017. Sub-Unit 6a consists of mostly rock strength and cemented materials
			6b	Sub-Unit 6b consisting mostly of soil strength materials but with some cemented layers
			6c	Unconsolidated sand layers within the Ascot Formation
			6d	Silt / Siltstone layers within the Ascot Formation
8	Osborne Formation	Cretaceous	8a	Variably cemented glauconitic sandstones and fine-grained materials. Including very low strength rocks, claystones, and sandstones
			8b	Fine grained (i.e., materials of a cohesive nature) soils
			8c	Coarse grained (i.e., materials of a non-cohesive nature) soils
9*	TQ Sands	Tertiary	9a	Marine sands underlying the Ascot Formation

*It should be noted that Unit 9 (TQ Sands) is numerically out of sequence due to project phasing from onshore to nearshore. Chronologically Unit 9 should sit above Unit 8.

Extracts from the geological sections for both the intake and outfall tunnel alignments are presented in Figure 2 (outfall tunnel) and Figure 3 (intake tunnel) with the vertical tunnel alignment shown in red.

7.2 *Complexity*

The project site's complexity stems from variable properties of Tamala Limestone and Ascot Formation in the tunnel alignment. These carbonate units show diverse characteristics due to carbonate enrichment and leaching processes. This variability was captured in the EGM by dividing these units into sub-units. Tamala Limestone is a cemented sand dune sequence with carbonate variations and karstic features. The Ascot Formation, more advanced at Alkimos than at Perth Airport, represents a transition between carbonate soil and rock. Poor sample recovery during drilling, especially in the Ascot Formation, led to uncertainties in determining cementation and Rock Quality Designation (RQD) values. The EGM aimed to provide a reasonable interpretation of materials likely encountered, considering these complexities.

7.3 *Tamala Limestone*

Tamala Limestone in the project area exhibits wide-ranging engineering properties, from well-cemented, high-strength rock to very weakly cemented, low-strength rock. These variations are associated with post-depositional processes like cementation and dissolution. The EGM divides Tamala Limestone into three Sub-Units, Fresh to Moderately Leached Limestone; Moderately to Highly Leached Limestone; Extremely Leached Limestone, and Void infill. Geotechnical conditions vary significantly throughout the project area, with material types ranging from loose sand to variable strength rock, including high-strength calcrete or 'limestone caprock'. This variability occurs over short lateral and vertical distances due to differences in cementation, solution weathering, and the presence of features like pinnacles and subvertical solution cavities.

7.4 *Ascot Formation*

Underlying the Tamala Limestone and overlying Osborne Formation, Ascot Formation shows highly variable properties from soil to rock. Predominantly carbonate calcareous materials, it varies due to dissolution and re-precipitation processes. Authigenic cementation resulted in a spectrum of uncemented soil to fully cemented carbonate rock. Sample recovery challenges led to uncertainties in material properties across the site.

7.5 *TQ Sand*

'TQ-Sand', not formally recognised in the Perth Basin, is a typically very dense, quartz sand unit between the Ascot and Osborne Formation along the outer Outfall Tunnel alignment. Presumed to be late Tertiary deposits, possibly Rockingham Formation, though previously thought not to extend north of Perth. The Unit is dominated by coarse grained soil materials. Approximately 70.5 % of the sampled material was soil, with 29.5 % of core loss also noted, which implies a higher soil fraction.

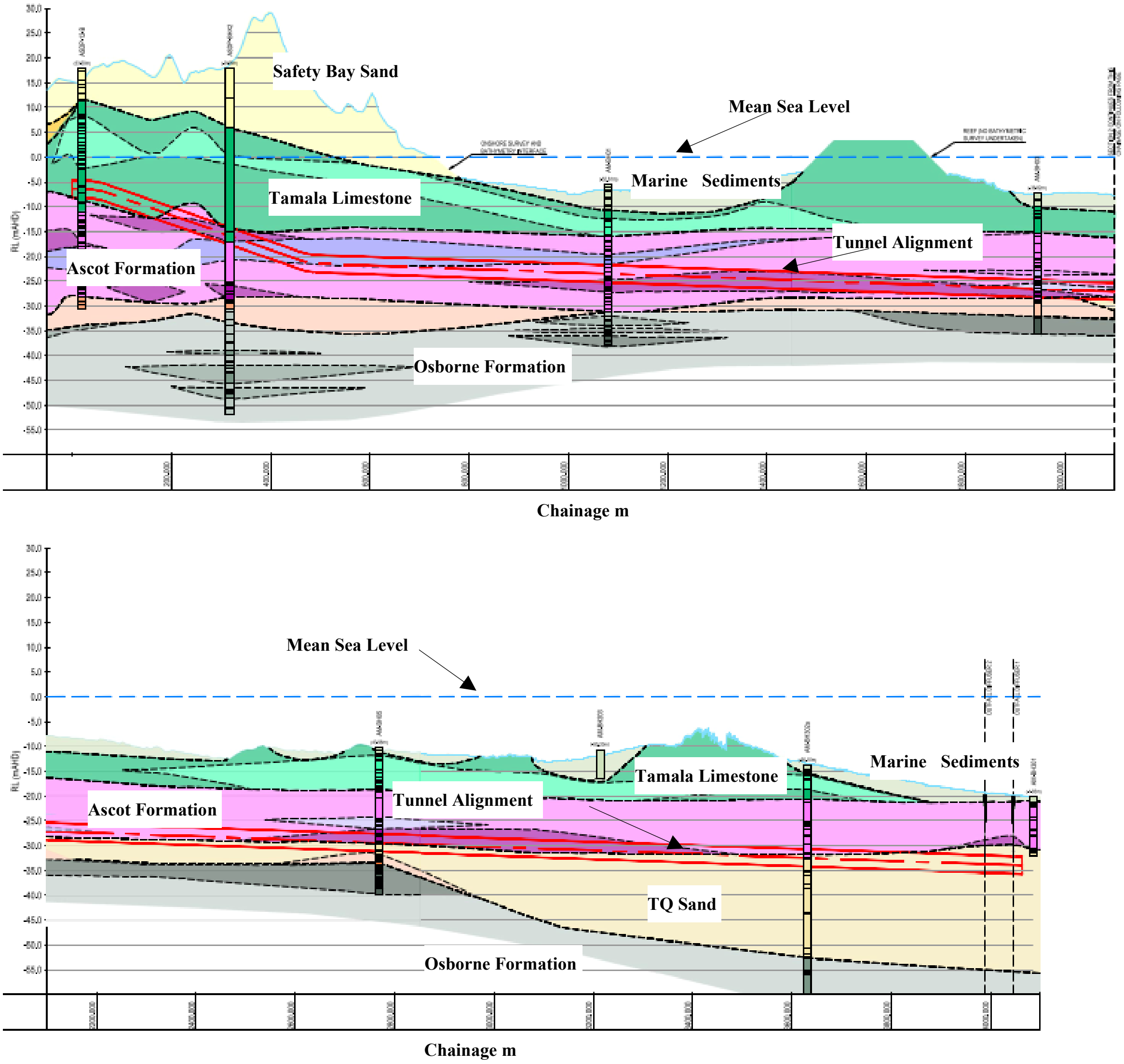

Figure 2. Outfall tunnel geological section

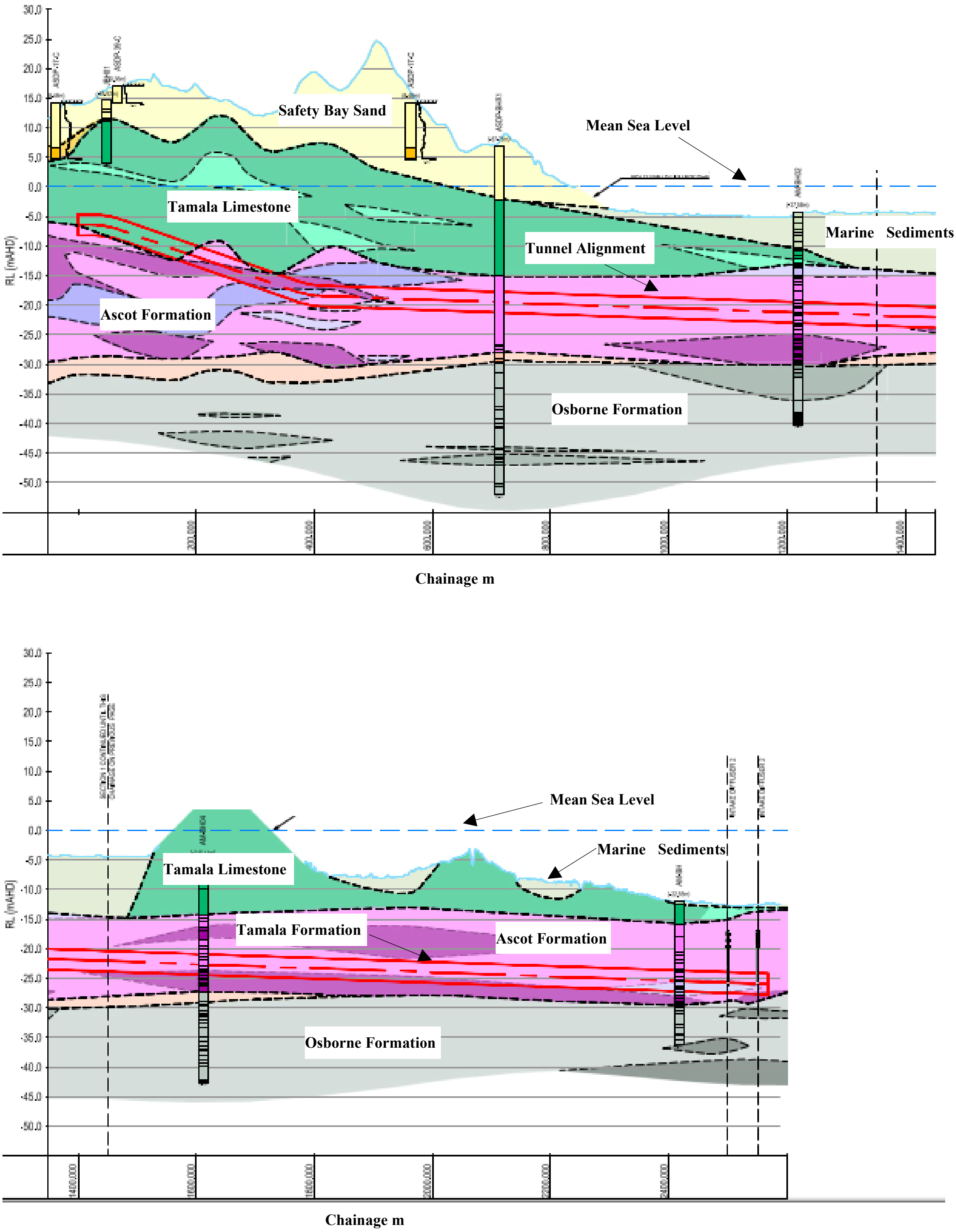

Figure 3: Intake tunnel geological section

8 INTERPRETATION

8.1 *Geotechnical Parameters*

Material properties for nearshore geotechnical units were derived from borehole logs, CPT traces, geophysical survey data, and in situ and laboratory test results. Rock mass parameters considered rock mass structure, discontinuities, and Geological Strength Index (GSI). Nearshore and onshore parameters for the same units were compared.

Effective friction angles for coarse-grained soils were estimated using SPT N values (Schmertmann, 1975) and CPT tip resistance data (Robertson, 2012). For fine-grained soils, correlations with plasticity index were employed (BS8002; Kenney, 1959; Parry, 1960; Zhu and Yin, 2000; Sorensen, 2013). Young's modulus was estimated from SPT N values (Decourt, 1995) CPT data for soils (Robertson, 2012) and published correlations for rock (Tomlinson, 2001; Deere and Miller, 1966; Hoek & Diederichs, 2006). Stress history and earth pressure coefficients were derived using CPT and DMT data (Kulhawy and Mayne, 1990).

8.2 *Hydrogeological Parameters*

Vertical and horizontal permeability were assessed for relevant geotechnical units using Particle Size Distribution test data (Hazen, 1911), falling head permeability tests (Cedergren, 1977) onshore monitoring wells, and onshore pumping and injection well test results. Interpreted parameters were compared to Alkimos Wastewater Treatment Plant data, local and regional published data, and estimates from nearby Perth-based projects.

The aquifer pumping test results were interpreted based on two conceptualisations: unconfined (Neuman, 1974) and leaky confined (Hantush, 1960) aquifers. The high permeability of the aquifers caused a rapid response in nearby observation bores, potentially indicating a partially confined response. While the Tamala Limestone is primarily considered an unconfined water table aquifer, the lower part of the limestone exhibited confined behavior due to the pumping well screen placement and possible layering. The unconfined model was therefore deemed more appropriate based on the absence of significant low permeability layers and better curve matching the test data. The Neuman curve more closely followed the observed data slope compared to the Hantush curve, which deviated in the latter half of the final step of the pumping test.

The injection test results were analysed using Aqtesolv. The unconfined (Neuman) solution with 20% specific yield best represented the test results. For injection tests conducted in wells with screens in the Tamala Limestone, results generally aligned with pumping test results, except for a single well which showed lower hydraulic conductivity (100 m/day vs. 175 m/day). Two injection well tests yielded slightly higher results, while one injection test matched the pumping tests. The injection tests produced a narrower hydraulic conductivity range of 100-150 m/day for Tamala Limestone. However, due to the longer duration of the pump out tests and absence of significantly larger impressed heads induced during injection, the pumping test results were considered to be more reliable for determining hydraulic properties.

8.3 *Characteristic Design Parameters*

A summary of the geotechnical and hydrogeological design parameters recommended for tunnel design is presented in Table 2. Design ranges are presented along with the recommended characteristic values. Where geotechnical units comprised both soil and rock materials and could behave as a combined material, unified material parameters (i.e., composite soil and rock parameters) were developed to simplify analysis for design. Unified parameters were developed by assessing an average Geological Strength Index (GSI) for a combined soil and rock within a unit and then the GSI used to assess lower bound rock mass design properties using published correlations. An example of the rock laboratory test results (point load strength index and UCS tests) carried out along the outfall tunnel alignment are shown in Figure 4 and 5 at the end of this section.

Table 2: Characteristic Design Parameters

Unit	Material	UCS	g'	c'	f'	n'	E'	E_m	OCR	K_0	K_h / K_v
-	-	MPa	kN/m^3	kPa	(°)	-	MPa	MPa	-	-	m/day
2	Soil	-	18 (15-19)	0	35 (34-38)	0.3	30 (20-50)	-	1-2	0.52 (0.41-0.62)	-
5a	Rock	4.0 (3.8-5.0)	20	See note 4	See note 4	0.3	-	800 (400-1200)	-	0.80-1.20	
	Soil	-	18 (16-20)	0	36 (34-38)	0.3	60 (30-90)	-	1-3	0.62 (0.41-0.79)	
5b	Rock	2.0 (1.0-3.5)	19	-	-	0.3	-	-	-	0.80–1.20	100-200 / 50-100
	Unified	-	19	See note 4	See note 4	-	-	250 (100-500)	-	0.40-1.20	
5c	Soil	-	19 (18-20)	0	36 (32-38)	0.3	50 (30-90)	-	1-3	0.62 (0.41-0.79)	
	Soil	-	20 (19-21)	0	38 (34-40)	0.3	60 (30-100)	-	1-4	0.68 (0.38-0.90)	
6a	Rock	2.0 (1.0-2.5)	22	-	-	0.3	-	-	-	0.50–2.00	
	Unified	-	21	See note 4	See note 4	-	-	200(100-400)	-	0.40-2.00	30-40 / 3-4
6b	Soil	-	19 (18-20)	0	36 (34-39)	0.3	50 (30-75)	-	1-4	0.71 (0.41-0.93)	
6c	Soil	-	19 (18-20)	0	36 (34-39)	0.3	50 (30-75)	-	1-4	0.71 (0.41-0.93)	
6d	Soil	-	19 (18-21)	0	30 (28-32)	0.3	35 (25-45)	-	1-4	0.79 (0.50-1.00)	
9a	Soil	-	20 (19-21)	0	39 (38-40)	0.3	90 (80-120)	-	1-6	0.82 (0.37-1.15)	10-60 / 1-6

1. γ' = bulk unit weight, c' = effective cohesion, φ' = effective angle of friction, ν' = Poisson's ratio, E' = effective elastic modulus, OCR = overconsolidation ratio, K_0 = at-rest earth pressure coefficient, K_h = horizontal permeability, K_v = vertical permeability.
2. Alluvium can contain interbedded material of variable consistency and relative density. Design parameters are considered representative of soil mass. Location specific parameters were considered in geotechnical design.
3. Range provided and parameter adopted for design selected based on location specific geotechnical investigation data with reference to soil and rock constituents.
4. Mohr-Coulomb parameters assessed utilising RocScience RSData (v1.005) utilising a Generalised Hoek-Brown failure criterion to define c' and φ' for a given depth, relative to the element under assessment

Published correlations used for assessing characteristic parameters appeared to provide parameters that were similar to or of the same order as parameters recommended for local projects which encountered similar ground conditions.

Comparison of the above characteristic parameters for the nearshore works with those of the onshore works indicate the onshore subsurface materials comprise more competent rock, particularly within the Tamala Limestone and Ascot Formation.

8.4 *Karsts*

Tamala Limestone, formed from windblown dunes and cemented through non-marine processes, exhibits karstic characteristics throughout its formation. Observed features include surface pinnacles, subvertical tube structures, minor voids, and pockets of uncemented material within competent layers.

The limestone showed varied leaching states, indicating carbonate depleted and enriched zones. Drilling observations revealed zones of flush loss, possibly due to planar features like relict bedding or historical water tables. Larger uncemented zones (Unit 5c) may have resulted from carbonate depletion or represent relic uncemented areas.

While the risk of large voids or cavities below the water table was considered low based on drilling investigations (drilling fluid loss, small rod drops) and geophysical survey data, their presence could not be completely discounted.

The Ascot Formation presented a low risk of karst development based on the drilling and geophysical results, with the potential for minor cavities. This formation can be characterised as a soil that has undergone authigenic cementation processes, but in most instances, the cementation has not progressed sufficiently to develop a massive rock texture. Any cavities present could possibly be the result of soil material being washed out from within a skeletal cemented matrix, as well as the dissolution of carbonates typically associated with karst formation.

8.5 *Mixed Face Tunnelling Conditions*

The Tamala Limestone and Ascot Formation will likely present challenging tunnelling conditions due to their highly variable rock strength and structure. Mixed face conditions, including uncemented sand bodies and interbedded layers of soft clay and loose sand, are likely to be encountered based on the EGM developed for the tunnels. These formations can range from very low to medium strength, with occasional high-strength intervals, and often consist of fractured rock masses with high-strength fragments in a sandy or clayey matrix. These mixed face conditions, which may also occur at the interfaces between the Tamala Limestone, Ascot Formation and TQ-Sand, could potentially impact TBM operations.

The varying permeability and strength of materials could lead to preferential excavation difficulties in maintaining face pressures. Careful design and control of TBM operations will be crucial to mitigate these challenges and prevent adverse impacts on performance, productivity, and ground movements. Ground movements are of risk for the onshore portion of the tunnels which will be constructed below existing onshore structures and therefore instrumentation and monitoring will be installed to monitor ground movements. Environmental issues with regards to the use of bentonite slurry in the TBM operations in permeable ground is considered to be a very low risk based on the pumping tests conducted onshore within the Tamala Limestone and Ascot Formation.

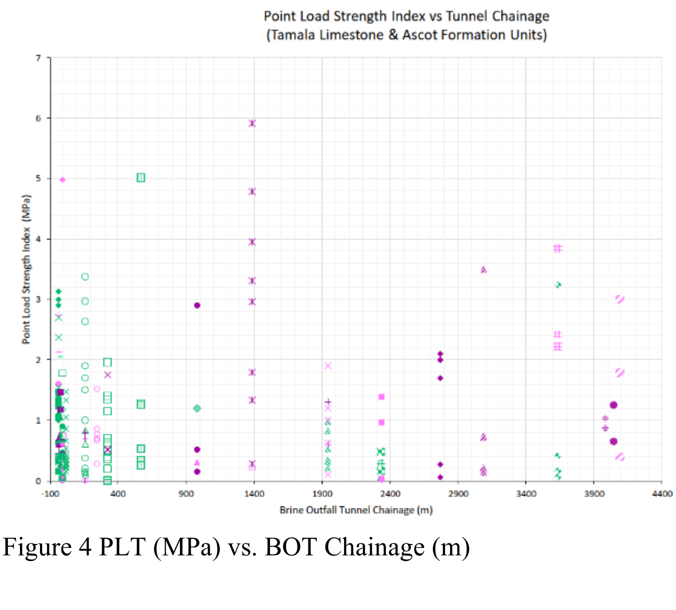

Figure 4 PLT (MPa) vs. BOT Chainage (m)

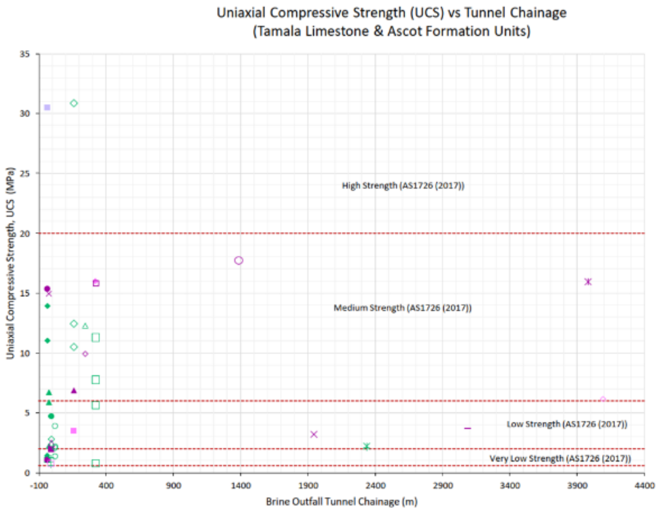

Figure 5 UCS (MPa) vs. BOT Chainage (m)

9 CONCLUSIONS

The project site presents complex geotechnical and hydrogeological conditions, primarily due to the variable properties of the Tamala Limestone and Ascot Formation. While the risk of large voids in the Tamala Limestone is considered low, smaller karstic features are likely present, which could potentially impact tunneling operations.

Pumping and injection well tests below the water table yielded permeability results consistent with published data and local projects, indicating the potential presence of small karstic voids. The Ascot Formation poses challenges in accurate characterisation due to its variable nature and poor sample recovery during drilling investigations. An EGM has been developed to convey the area's complex geological and engineering characteristics. Nearshore geotechnical and hydrogeological design parameters have been recommended for each unit, considering material property variability and uncertainty. These parameters are in good agreement with the onshore parameters and compare well with published data and parameters adopted on other local projects with similar ground and groundwater conditions.

The project will require careful planning, continuous monitoring, and adaptable design approaches to address the challenges posed by these complex geological and hydrogeological conditions.

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