REPORT

Umina-Ocean Beach Erosion Management Strategy

Terminal Protective Structure Concept Design Report

Client: Central Coast Council

Reference: PA1149-114M&ARP191016

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Table of Contents

Table of Tables

Table of Figures

Appendices

1 Introduction

Central Coast Council are investigating the following proposed works as part of the Umina-Ocean Beach Erosion Management Strategy:

1. Terminal protection of The Esplanade and other landward infrastructure between Kourung Street and Ocean Beach SLSC; and,
2. Beach nourishment.

It is understood that the key project objectives are:

- Protect existing infrastructure from coastal erosion hazards;
- Align the terminal protective structure (TPS) as far landward as possible;
- Minimise disruption to existing dune vegetation and foreshore reserve areas;
- Maintain beach access during phases of erosion;
- Explore opportunities to integrate erosion management with regular maintenance dredging within the entrance channel to Brisbane Water;
- Improve and maintain beach amenity; and,
- Establish a vegetated dune over the TPS.

This report presents an investigation into several TPS concept design options leading to the selection of a preferred TPS proposal. It is noted that the TPS is proposed to be installed in conjunction with beach nourishment works, which are the subject of a separate *Beach Nourishment Strategy* (RHDHV, 2019) report. The investigation herein is set out as follows:

- Basis of Design (**Section 2**);
- Concept Design of TPS Options (**Section 3**);
- Assessment of TPS Options (**Section 4**);
- Preferred TPS Proposal (**Section 5**); and,
- References (**Section 6**)

2 Basis of Design

2.1 Extent of TPS

The length of shoreline under consideration for implementation of TPS comprises the section along Umina-Ocean Beach between Ocean Beach SLSC to the Kourung Street Boat Ramp. This covers a length of approximately 1,300 metres. The beach and foreshore to be protected by the TPS is shown in **Figure 1**.

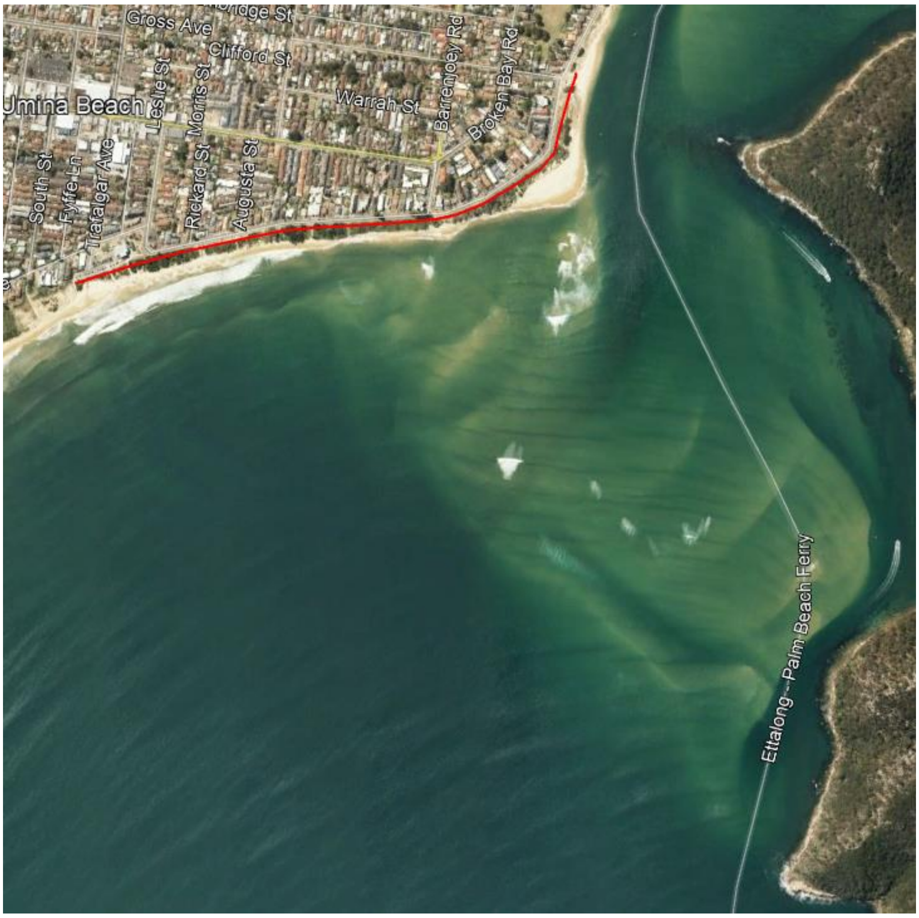

Figure 1: Umina-Ocean Beach showing the extent of terminal protection and beach nourishment from western end of carpark at Ocean Beach SLSC (Trafalgar Ave), to northern end of informal carpark and boat ramp at Kourung Street, a shoreline distance of approximately 1,300m. Entrance shoals and channel to Brisbane Water also shown.

2.2 Design Life

The design life of the TPS is subject to the type of construction materials used (refer **Section 2.11**) and the degree of exposure of the structure during its working life. The amount of exposure that the TPS has to wave action and other environmental loads is related to the frequency and scope of beach nourishment campaigns and the phasing of beach erosion and accretion cycles.

A design working life of 40 years has been nominated for a rock revetment. This is consistent with the design life for armour stone specified within AS 2758.6 Aggregates and Rock for Engineering Purposes and industry practice for rock structures exposed to hydraulic action (e.g. waves, currents, water inundation) in marine environments.

A design working life of 50 years has been nominated for piled or concrete seawall structures. This is consistent with the design working life specified for 'normal maritime structures' within AS4997-2005 Guidelines for design of maritime structures.

The design working life of geotextile container structures is expected to be less than hard engineering structures. The durability of geotextile fabric is affected by several mechanisms including fabric degradation caused by exposure to UV light, abrasion caused by wave action and sand movement, and vandalism such as knife cuts and punctures. Burial of geotextile container with beach nourishment and re-establishment of dune vegetation will increase their design life by limiting exposure to environmental loads. Vandal deterrent geotextile containers are available that are manufactured from a composite geotextile fabric comprising a standard non-woven inner geotextile that is bonded to a coarse fibre geotextile on the outer surface. The coarse outer layer allows sand to be trapped within the geotextile and this trapped sand provides further protection from knife cuts (Hornsey et al, 2011). There are a number of Australian geotextile container installations that have been in place for a significant length of time, including the Maroochydore beach revetment (installed 2000-2001). Maroochydore groynes (installed 2001) and the seawall at Stockton Beach (installed 1996). This suggests that with adequate maintenance as required (e.g. re-stitching, patching or replacement of damaged containers) the design life of geotextile container structures could be up to 15 to 20 years.

2.3 Terrestrial Survey

A detailed terrestrial survey was undertaken by Stephen Thorne & Associates during the week of 4 to 8 June 2018. The survey was completed to MGA horizontal coordinates and AHD vertical datum and captured the following elements:

- ground levels from the seaward side of The Esplanade, over the dune to the low tide mark on the beach;
- crest and toe of erosion scarps on the beach or locations where there is an obvious change in slope;
- kerb/gutter alignment along The Esplanade and within car park areas;
- edge of pavement/footpaths;
- fence lines;
- vegetation lines;
- services (including stormwater drainage pits/outlets, powerlines, power poles, sewer, water, telecommunications);
- existing beach accessways;
- exposed rock structures (e.g. groynes);
- geotextile container wall at Barrenjoey Road;
- building footprint and surrounds of Ocean Beach SLSC;
- park furniture (e.g. picnic tables, bench seats, bollards etc.);
- landscaping features (e.g. retaining walls).

Survey plans are attached within **Appendix A**.

2.4 Services

Services including stormwater drainage, power lines/poles and telecommunications lines were captured in the terrestrial survey where visible. The location of services has also been provided in Council GIS information, which includes stormwater drainage, power lines/pits/poles, water mains, sewer mains and telecommunications. A Dial Before You Dig request has been submitted and any additional services identified will be shown on design drawings.

2.5 Stormwater Drainage

A total of six (6) stormwater outlets currently exist along the beachfront within the study area (refer **Table 1**). These reinforced concrete pipe outlets are typically located within the back beach dune area and are characterised by localised erosion caused by stormwater flows. Where necessary, these outlets would need to be extended through the TPS to maintain stormwater drainage flows.

Table 1: Stormwater outlets within study area

Outlet No.	Diameter (mm)	Invert Level (m AHD)	Location
1	1250	1.98	Opposite Trafalgar Avenue
2	300	5.76	Opposite Rickard Street
3	450	4.56	Opposite 150 The Esplanade
4	375	4.52	Opposite 140 The Esplanade
5	450	4.88	Opposite 134 The Esplanade
6	450	3.73	Opposite Barrenjoey Road

It is noted that a large GPT is currently being installed on the drainage outlet at Trafalgar Avenue (Ocean Beach SLSC carpark).

2.6 Geotechnical Conditions

Geotechnical investigations over the study area were undertaken by JK Geotechnics and are documented within **Appendix B**. 17 cone penetration tests (CPTs) to 15m depth were completed within areas landward of the vegetated dune area and spaced at 65m to 90m along the foreshore. 10 hand-augered boreholes to 1m to 2.6m depth (generally terminated due to borehole collapse) were completed at the toe of the existing back beach dune and spaced at 110m to 150m along the foreshore. The results of the geotechnical investigation indicated that subsurface materials generally comprised orange brown, fine to medium grained or fine to coarse grained, medium dense to very dense sands and silty sands that had increasing density with depth. The presence of hard interfaces (e.g. cemented layers, residual clay or bedrock) that may limit coastal erosion was not detected by the investigations, which in the case of the CPTs extended down to reduced levels of -8.0m AHD to -13.3m AHD.

Soil samples for acid sulfate soils (ASS) testing were collected from 10 points along the beach corresponding to the borehole locations. The testing results indicated that the sandy subsurface material was below the action criteria within the ASS guidelines and that there was considered to be a low potential for ASS or PASS to exist at the site to a depth of 2.1m.

2.7 Design Scour Level

A design scour level of -1m AHD is commonly adopted in NSW as a ‘rule of thumb’ for the design of rigid coastal structures positioned at the back of an active beach area. A deeper scour level of -2m AHD is often adopted for vertical face coastal structures to account for the additional scour associated with increased wave reflections.

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Geotechnical investigations undertaken at the site did not detect any subsurface interfaces that are likely to represent historical scour levels or inerodible materials (e.g. cemented sands, gravel layers, stiff clay, bedrock).

Photogrammetry data indicates that Umina-Ocean Beach has remained relatively stable or slightly accreting over the period of record. It is noted that recent analysis of satellite imagery by WRL (2019) identified a recent recessive trend across the beach between Ocean Beach SLSC and Ettalong Point in response to a shoal blowout event in 2012. The beach recession continued until 2015-2016 and has shown signs of a return to a gradually accreting trend in recent years. Ongoing beach monitoring is required to determine if the beach continues to demonstrate stable or accreting behaviour in the longer term. In addition, it has been reported (WorleyParsons, 2014) that the beach is yet to establish an equilibrium profile to validate long-term shoreline recession under projected sea level rise (i.e. Bruun Rule). As such, given that the TPS is proposed in conjunction with beach nourishment and no significant long-term shoreline recession is forecast, it is considered that the above ‘rule of thumb’ scour levels are appropriate to adopt at this location. These are potentially conservative scour levels if beach nourishment is periodically undertaken to supplement the natural sand buffer for coastal storm erosion and to enhance beach amenity.

2.8 Design Water Level Conditions

Water levels at Umina-Ocean Beach vary primarily in response to astronomical tides, although storm surge (barometric and wind set-up) may also influence water levels from time to time. Other mechanisms and events including coastal trapped waves and tsunamis can also affect water levels on rare occasions. Sea level rise will have a long-term effect on water levels.

2.8.1 Astronomical Tide

Umina-Ocean Beach is subject to semi-diurnal tides (i.e. two high tides and two low tides per day) that propagate through the entrance to Broken Bay and into Brisbane Water. The timing and magnitude of tides is influenced mainly by the gravitational pull of the moon. Within the 28 day lunar cycle alternating Spring and Neap tides occur every 14 days, which correspond to periods of higher and lower tidal range respectively. During the year there are two periods when the earth is closest and furthest from the sun, these are referred to as the summer solstice (late December) and winter solstice (late June) and result in the maximum tides in a year (also known as ‘King tides’). Highest astronomical tides occur every 18.6 years. Astronomical tides can be accurately predicted by analysing water level records and extracting the lunar and tidal constituents over a full 18.6 year lunar cycle.

Manly Hydraulics Laboratory (MHL) operates and maintains a network of tide gauges along the NSW coast and produced a report documenting a harmonic analysis to determine tidal planes at all of its gauges (MHL, 2012). The analysis was based on analysis of data over a 20 year period (where available from 1990 to 2010. Tidal planes derived for several gauges in the vicinity of the study area and for Port Jackson (Sydney Harbour) are summarised in **Table 2**.

Table 2: Tidal planes in vicinity of study area and at Sydney Harbour relative to Australian Height Datum (MHL, 2012)

Tidal Plane	Ettalong	Patonga	Port Jackson
High High Water Solstices Springs (HHWSS)	0.794	1.020	0.995
Mean High Water Springs (MHWS)	0.512	0.683	0.647
Mean High Water (MHW)	0.430	0.558	0.524
Mean High Water Neaps (MHWN)	0.348	0.433	0.401

Mean Sea Level (MSL)	0.066	0.047	0.02
Mean Low Water Neaps (MLWN)	-0.217	-0.339	-0.361
Mean Low Water (MLW)	-0.299	-0.464	-0.484
Mean Low Water Springs (MLWS)	-0.380	-0.589	-0.607
Indian Spring Low Water (ISLW)	-0.581	-0.830	-0.856

Comparison of the tidal planes shown in **Table 2** indicates that the tides at Patonga (located at the mouth of Hawkesbury River and immediately adjacent to Broken Bay) are similar in magnitude to those within Sydney Harbour. The tidal planes at Ettalong are measured within Brisbane Water at a location opposite Kourung Gourung Point and indicate that the tidal range has been reduced due to the constriction to tidal flows provided by the narrow and shallow channel through the extensive entrance shoals.

2.8.2 Storm Surge

The combined effect of barometric pressure setup and wind stress setup is referred to as storm surge.

Barometric pressure setup refers to the increase in mean sea level caused by a drop in atmospheric pressure, such as when a low pressure system is centred over an area. This can be equivalent to a 10mm rise in water level per hectopascal. Wind stress setup is the increase in mean sea level caused by the ‘piling up’ of water on a shoreline by wind action acting on the water surface. In NSW, storm surge can be up to 0.6 m corresponding to barometric setup of up to 0.2m to 0.4m and wind setup of up to 0.1m to 0.2m.

2.8.3 Design Still Water Level

The Fort Denison tide gauge provides one of the longest continuous recordings of ocean water levels in the world, with reliable data considered to have been recorded from 1914. Water levels recorded at the gauge are inclusive of astronomical tide variation as well as non-astronomical sea level forcing factors discussed above. The water level data also inherently includes climate change induced sea level rise over the period of record. The Fort Denison gauge is considered to represent the open coast water level conditions experienced within the study area.

Analysis of the Fort Denison water level record was completed to determine the frequency distribution of measured water levels over the period from 31 May 1914 to 31 December 2017. An extreme value analysis (EVA) was completed on the 102 years of available water level data. Extreme events were extracted from the record based on a threshold value above Highest Astronomical Tide (HAT, 2.1m above ISLW) and a time gap of 7 days between events to ensure independence. This dataset was then used to calculate percentiles between 1 and 100 for the 102 years of data. This information was used to determine water levels for average recurrence intervals (ARI) of up to 100 years, which did not require conventional EVA extrapolation methods[1] due to the extensive length of record. The results of this analysis are summarised in **Table 3** and represent the ‘present day’ design still water levels at Fort Denison.

It is noted that a Weibull distribution was fitted to the data as a cross-check on the analysis. This resulted in similar values for each ARI, however it was observed that the fitted distribution was slightly under-estimating the water level for less frequent events and slightly over-estimating the water level for more frequent events. This was considered to be due to the trend of sea level rise within the data.

[1] That is, fitting an assumed probability distribution function to the data for extrapolation to design values with a recurrence interval significantly longer than that of the data record.

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Table 3: Design still water levels at Fort Denison tide gauge

ARI (years)	m ISLW	m AHD
0.02	1.90	0.975
0.05	1.97	1.045
0.1	2.02	1.095
1	2.16	1.235
2	2.19	1.265
5	2.22	1.295
10	2.26	1.335
20	2.30	1.375
50	2.33	1.405
100	2.37	1.445

2.8.4 Wave Setup

Waves breaking along a shoreline can create a superelevation in coastal water levels known as wave setup. Wave setup at the shoreline is approximately 10-15% of the breaking significant wave height (refer **Section 2.9.3**). It should be noted that wave setup would only apply seaward of the wave break point and contributes to the foreshore still water level (excluding wave runup). Wave setup is inherently included in wave runup estimates.

2.8.5 Sea Level Rise

The latest global mean sea level rise projections are provided in IPCC (2013b) for four (4) representative concentration pathways (RCP) scenarios, namely RCP2.6, RCP4.5, RCP6.0 and RCP8.5. The characteristics of each RCP scenario represent the wide range of greenhouse gas emissions in the literature and are as follows:

- RCP2.6 – stringent mitigation scenario, representative of a scenario that aims to keep global warming likely below 2°C above pre-industrial temperatures;
- RCP4.5 – intermediate scenario with a gradual reduction in emissions;
- RCP6.0 – intermediate scenario without any additional efforts to constrain emissions (‘baseline scenario’); and,
- RCP8.5 – high range scenario without any additional efforts to constrain emissions (‘baseline scenario’) and very high greenhouse gas emissions.

For each scenario a median sea level rise value is provided along with a likely range, corresponding to the 5% (low) to 95% percentile (high) values, for future years up to 2100. Global plots of percentage deviation from the global mean sea level rise are also provided and indicate that the local variation along the east coast of Australia is up to 10% higher than the global mean (refer **Figure 2**).

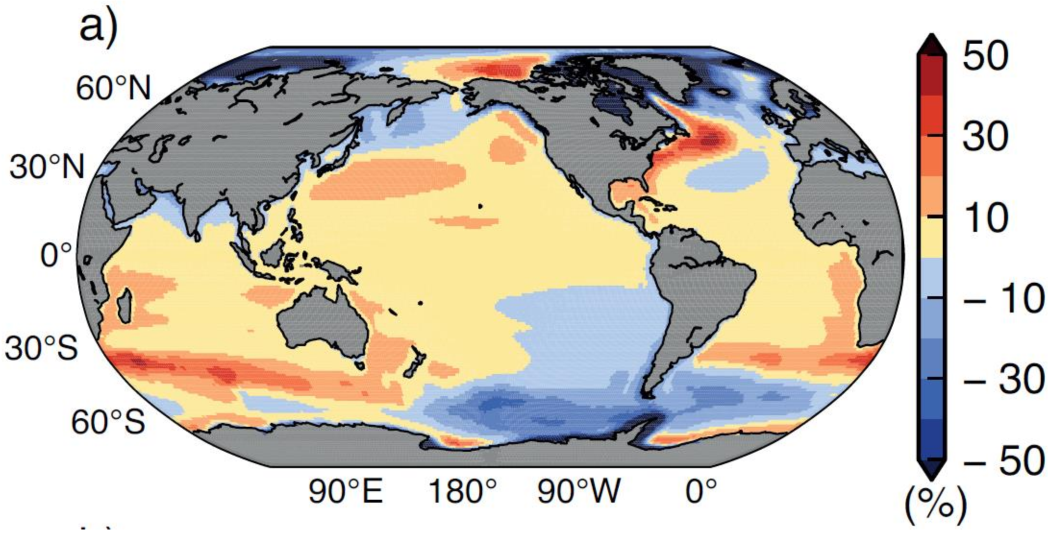

Figure 2: Percentage deviation of regional sea level change from the global mean value (IPCC, 2013b)

At present time, Central Coast Council are yet to adopt a common sea level rise policy for the former Gosford and Wyong local government areas. As such, the sea level rise policy adopted by the former Gosford City Council on 10 March 2015 has been considered to apply to the study area at Umina-Ocean Beach. The future sea level rise projections adopted by this policy are summarised in **Table 4** and are understood to be based on the median values reported for the IPCC RCP8.5 scenario with an additional allowance of +10% for local variation. The values within Council's policy have been adjusted to reflect sea level rise from the present day (2018).

Table 4: Adopted sea level rise projections

Year	2015 SLR Policy (m)	SLR Relative to 2018 (m)
2015	0.00	-
2018	0.014*	0.00
2030	0.07	0.06
2040	0.135*	0.13
2050	0.20	0.19
2070	0.39	0.38
2100	0.74	0.73

* Interpolated value

2.8.6 Design Water Levels

The design 100 year ARI water levels for present (2018), 20 year design life (2040) and 50 year design life (2070) time horizons are summarised in **Table 5**. These have been determined from the summation of the 100 year ARI design still water level (refer **Section 2.8.3**), wave setup estimated as 15% of the significant breaking wave height (refer **Section 2.9.3**) and sea level rise estimates.

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Table 5: 100 year ARI Design Water Levels

Time Horizon	Still Water Level	Wave Setup	SLR	Design Water Level
	(m AHD)	(m)	(m)	(m AHD)
Present (2018)	1.44	0.33	-	1.77
20 year design life (2040)	1.44	0.34	0.13	1.91
50 year design life (2070)	1.44	0.35	0.38	2.17

2.8.7 Coincidence of Extreme Waves and Water Levels

Large coastal storm events on the NSW coast typically result in both large waves and elevated water levels. However, the coincidence (phasing) of worst cases of elevated water levels and wave heights may not occur simultaneously. For example, the 100 year ARI wave height may not always coincide with the 100 year ARI water level. However, there are insufficient studies available that fully consider different phasing (joint probability) of the coincidence of extreme water levels and wave heights for much of the NSW coast (Shand et al (2010)). Based on results by Shand et al (2012) and following the approach of WRL (2015), we conservatively assume that for the ARIs considered, the same ARI be applied to each component.

2.9 Design Wave Conditions

2.9.1 Offshore Wave Climate

The offshore wave climate for the study area is reported within the *Open Coast and Broken Bay Beaches Coastal Processes and Hazard Definition Study* (WorleyParsons, 2014). The extreme value analysis performed on Sydney directional Waverider Buoy data determined that the offshore wave conditions producing the largest inshore wave heights at Umina Beach and Ocean Beach were from the SSE with a 100 year ARI significant wave height (H_s) of 9.3m and a peak energy period (T_p) of 13.3s.

2.9.2 Nearshore Wave Transformation

Wave transformation modelling was undertaken as part of the *Open Coast and Broken Bay Beaches Coastal Processes and Hazard Definition Study* (WorleyParsons, 2014). The results of this SWAN modelling determined that the 100 year ARI nearshore wave conditions at 6.5m water depth for the northern end of Umina Beach and at Ocean Beach had a significant wave height (H_s) of 3.6m.

Additional SWAN modelling was undertaken by RHDHV. A SWAN model was developed for the project site using a 10 m calculation grid for modelling swell waves. This 10 m grid was embedded within a larger 50 m grid for modelling wind-waves. Grid extents are shown in **Figure 3**. The 10 m grid spans 3.8 km by 2.1 km and the 50 m grid spans 15 km by 19 km. The south-eastern boundary of the 10 m grid was set to coincide with output location 1007506 of the OEH NSW Nearshore Wave Hindcast tool, residing at the 10 m depth contour.

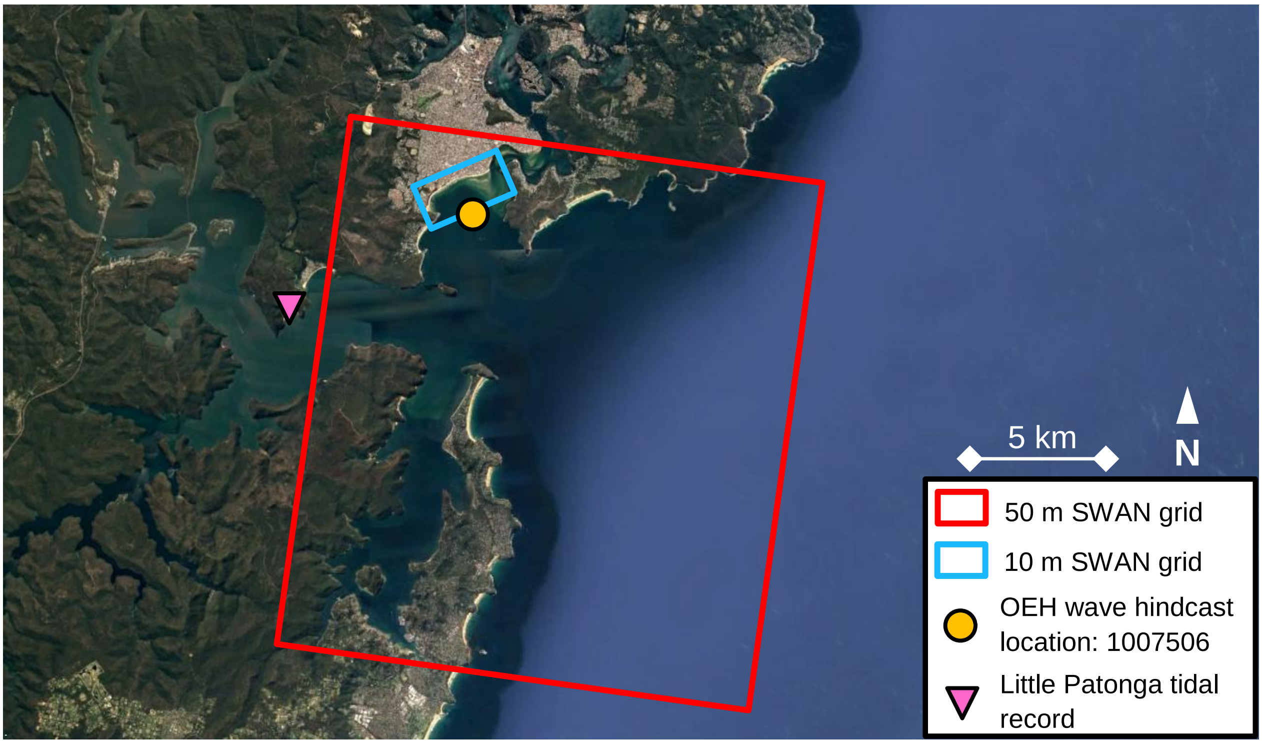

Figure 3: SWAN model grid areas

2.9.2.1 Boundary Conditions

Water levels

Still water levels used to force the SWAN model spanned LAT (-0.88 m AHD) through to HAT (1.2 m AHD) based on the historical Xtide tidal record available at Little Patonga (refer **Figure 3**) located approximately 7 km southwest of the study area.

Swell waves

Wave data for the ~27 year period spanning 1992 – 2018 were extracted from the OEH NSW Nearshore Wave Hindcast tool at location 1007506. This nearshore hindcast includes both locally generated wind waves and swell. The range of data observed in the wave record was used to inform SWAN model runs. A summary of the data is shown in **Table 6**.

Table 6: Summary of wave data used to force SWAN swell runs

	Minimum	Maximum	Mean	Standard Deviation
Significant Wave Height (m)	0	6.2	0.9	0.5
Peak Wave Period (s)	3.5	19.9	9.7	2.4

Wave Direction (degrees N)	84	163	131	13

Wind waves

Wind data for the ~74 year period spanning 1939 – 2012 were available from Sydney Airport and were used to force the SWAN model to calculate wind-waves at the study site. The range of data observed in the wind record is summarised in **Table 7**.

Table 7: Summary of wind data used to force SWAN wind runs

	Minimum	Maximum	Mean	Standard Deviation
Wind Speed (m/s)	0	28.7	4.9	3.4
Wind Direction (degrees N)	0	359	167	116

2.9.2.2 Bathymetry

High resolution nearshore bathymetric data of the study area was available from a combination of sources including a nearshore jet ski survey conducted by WRL, hydrographic survey in Broken Bay completed by Astute Survey, and hydrosurvey within the Brisbane Water entrance obtained from Crown Lands. This survey data extended to approximately the 10 m depth contour. Additional offshore bathymetric data to cover the model grid was sourced from Geoscience Australia. The combined nearshore and offshore bathymetric data was interpolated to the model grid. An example of the interpolated nearshore 10 m grid can be seen in **Figure 4**.

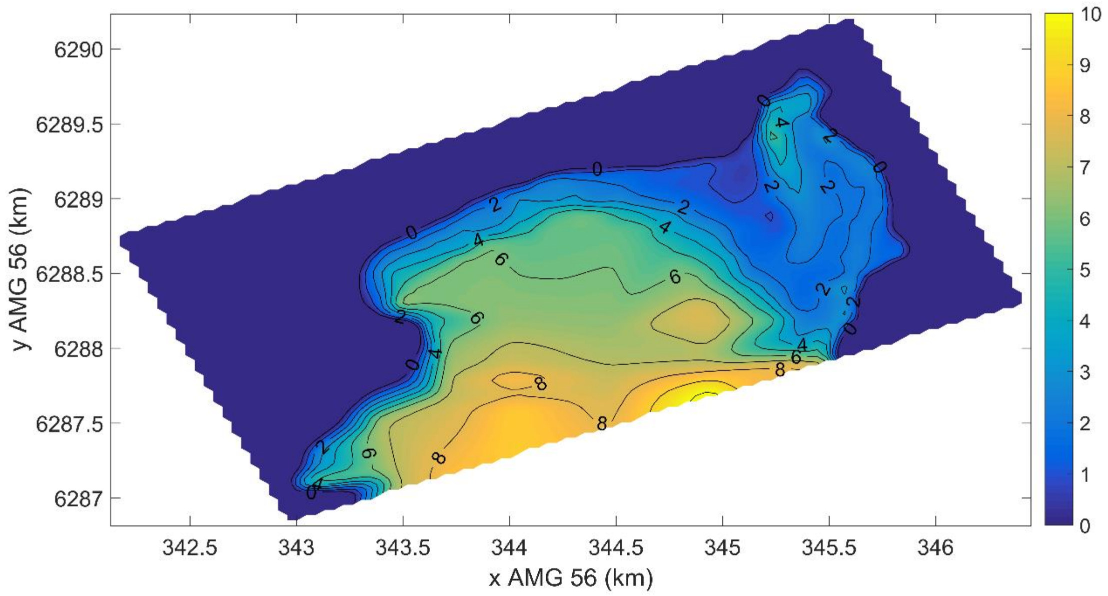

Figure 4: Interpolated bathymetric data to 10 m SWAN grid

2.9.2.3 Model Settings and Input Conditions

Based on the available water level, wind and wave data for the study area, a matrix of input conditions was determined with which to run the SWAN model. The values chosen encompass the full combination of metocean conditions expected within the study area. The range of modelled input conditions for swell modelling is provided in **Table 8** and for wind-wave modelling in **Table 9**. Model settings are provided in **Table 10**.

Table 8: Boundary conditions used to force SWAN for swell wave modelling

Boundary Condition	Modelled Values
Wave Direction (degrees N)	80:10:160*
Hs (m)	0.1, 1, 2, 3, 4, 5, 6.5
Tp (s)	4:4:20*
Water Level (m AHD)	-1.2:0.8:1.2*

* min:increment:max

Table 9: Boundary conditions used to force SWAN for wind wave modelling

Boundary Condition	Modelled Values
Wind Direction (degrees N)	0:11.25:360*
Wind Speed (m/s)	4:5:29*

Water Level (m AHD)	-1.2:0.8:1.2*

* min:increment:max

Table 10: SWAN model settings

Process	SWAN code
Wind growth	GEN3
Wave setup	Activated
Wave breaking	Activated, coef = 0.073
Bottom friction	JONSWAP, coef = 0.067
White capping	Komen
Quadruplets	Activated for wind forcing, deactivated for no wind forcing
Triads	Activated

2.9.2.4 Model Results

SWAN model results show that the 100 year ARI H_s varied between 2.8 – 3.6 m along Umina Beach and Ocean Beach in 6 – 7 m water depth, consistent with modelling results previously obtained by WorleyParsons (2014). It is of particular interest to determine the maximum nearshore wave height that could reach the proposed TPS. Model results were extracted along profile transects spanning from in front of Ocean Beach Surf Life Saving Club eastward to Ettalong Beach Point at 200 m intervals and to approximately 6 m water depth as shown in **Figure 5**. To determine the maximum nearshore wave height that could reach the proposed TPS, a scour level of -1 m AHD at the structure toe was assumed in addition to 100 year ARI water levels for the present-day, 2040 and 2070 as presented in **Table 5**. The water depths corresponding to these conditions were located on each profile transect and the 5-year, 10-year, 20-year, 50-year and 100-year ARI wave height was then extracted.

Figure 5: Location of profile transects used for SWAN model data extraction

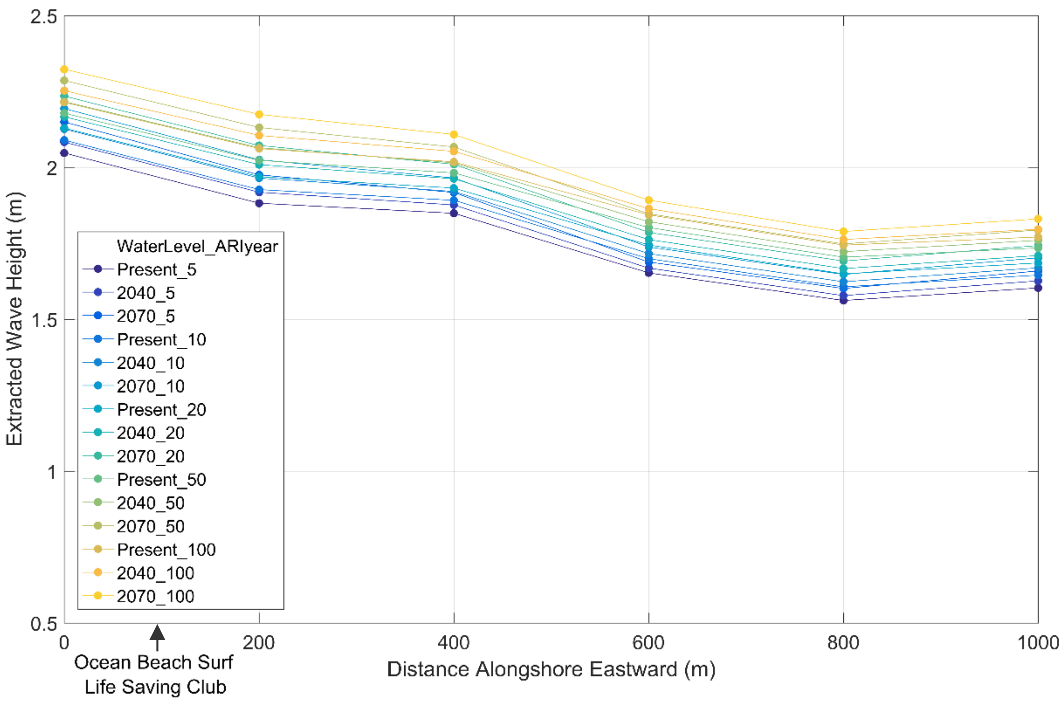

Figure 6: The 10-year, 20-year, 50-year and 100-year ARI swell wave heights extracted from the SWAN model at different alongshore locations and for different water levels

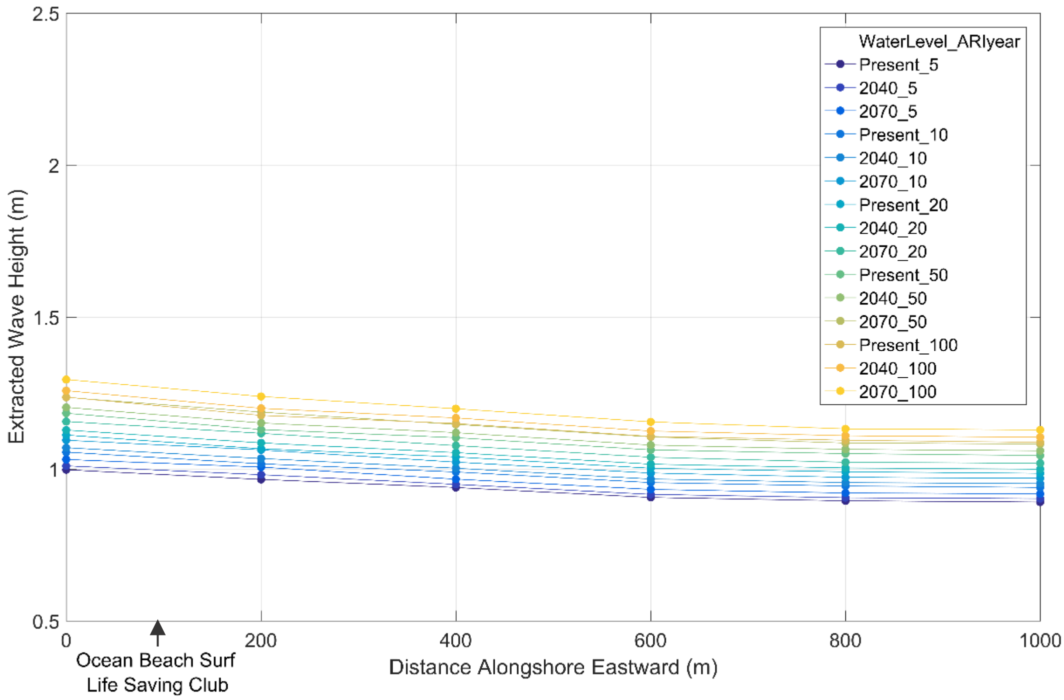

Figure 7: The 10-year, 20-year, 50-year and 100-year ARI wind-wave heights extracted from the SWAN model at different alongshore locations and for different water levels

Figure 6 and **Figure 7** show that modelled wave heights are higher at the western end of Umina-Ocean Beach likely due to greater exposure to offshore conditions. The highest waves occurred directly in front of Ocean Beach Surf Life Saving Club and reduced in height alongshore towards Ettalong Beach. Further, the magnitude of the swell waves extracted from the SWAN model runs are roughly 1 m higher than the modelled wind-wave heights. Wave heights in **Figure 6** and **Figure 7** can be seen to increase with higher water levels and for larger ARI events. However, the difference between different ARI events is marginal, suggesting that waves are depth limited. The overall maximum wave heights that could reach the proposed TPS based on the SWAN modelling occur at the western end of the study area, directly in front of Ocean Beach Surf Life Saving Club, as shown in **Figure 6** and are presented in **Table 11**.

Table 11: Maximum wave heights reaching TPS toe extracted from SWAN model

	ARI				
Water Level at Structure Toe	**5**	**10**	**20**	**50**	**100**
Present (1.44 m AHD)	2.05	2.09	2.13	2.17	2.21
2040 (1.57 m AHD)	2.08	2.12	2.17	2.22	2.25
2070 (1.82 m AHD)	2.15	2.19	2.23	2.29	2.32

The large shoal at the entrance to Brisbane Water significantly limits waves penetrating into Brisbane Water and reaching Ettalong Beach. This shoal is highly dynamic and wave heights cannot be confidently extracted for Ettalong Beach from the SWAN model, which is based on an instantaneous bathymetric survey. However, for the results obtained from the SWAN model using this instantaneous bathymetry, it is noted that the height of 10, 20, 50 and 100-year ARI swell and wind waves reaching Ettalong Beach and penetrating into Brisbane Water were in the order of 1 m, with the majority of waves breaking on the shoal.

2.9.3 Design Breaking Wave Height

The design breaking wave height for structures positioned in back-beach areas can be limited by the available depth caused by storm erosion scour in the immediate vicinity of the structure. The available depth is influenced by several factors including the design scour level assumed in front of the structure (refer **Section 2.7**), the nearshore slope in the zone of wave breaking, and the design water level (including allowances for wave setup and sea level rise). Depth-limited significant breaking wave heights have been determined for present day, 2040 (20 year design life) and 2070 (50 year design life) 100 year ARI water level conditions (refer **Section 2.8.6**) using the equations within Goda (2010) and wave plunge distance estimated from Smith and Kraus (1991). A nearshore slope of 1V:12H has been adopted for these calculations based on analysis of average slopes around 0m AHD from available beach profile data (WorleyParsons, 2014). Calculation of the nearshore breaking wave height has also been completed using a nearshore slope of 1V:20H to test the sensitivity to varying seabed bathymetry.

The results of these calculations for H_s are summarised in **Table 12** and demonstrate that the depth-limited breaking wave heights calculated from the 100 year ARI water depths under design scour conditions are generally greater than the 100 year ARI nearshore wave heights determined from wave transformation modelling (refer **Table 11**) and are sensitive to the assumed nearshore bed slope. This indicates that the breaking wave heights in **Table 12** represent wave conditions that are rarer than the maximum 100 year ARI wave heights that are able to reach the TPS. As such, the design wave heights in **Table 11** have been adopted for concept design purposes.

Table 12: Depth-limited breaking wave heights determined from 100 year ARI water level and -1m AHD scour level

Time Horizon	Wave Height (Hs, m)	
	1V:12H nearshore slope	1V:20H nearshore slope
Present (2018)	2.9	2.4
20 year design life (2040)	3.1	2.5
50 year design life (2070)	3.3	2.7

2.10 Wave Runup and Overtopping Performance Criteria

Design wave runup and wave overtopping values will be determined in accordance with the empirical procedures for different types of structures within the EurOtop Manual (EurOtop, 2016). Wave runup heights will be used to define appropriate TPS crest levels in conjunction with other considerations including impacts on beach amenity (e.g. ocean views, blocking of sea breeze and beach access) and the existing crest level of dune system.

There are several guideline publications (including different versions of the EurOtop Manual and USACE publications) that nominate tolerable average overtopping discharges for different types of activity (e.g. vehicles and pedestrian access) and for limiting structural damage. These limits all vary as they are based on subjective interpretation of safety aspects in particular. The guideline values published within the

Coastal Engineering Manual (USACE, 2006) and the EurOTop Manual (2016) are reproduced below in **Figure 8**, **Figure 9**, **Figure 10**, and **Figure 11**. These guideline values cover limits for structural damage to the seawall and landward areas (e.g. grassed or paved areas), damage to buildings or equipment, and safety of pedestrian and vehicle traffic, and will be considered in the assessment of different TPS design options. It is noted that while the Coastal Engineering Manual limits the overtopping criteria to a mean overtopping discharge, EurOtop includes a maximum overtopping volume as well, taken to represent the maximum overtopping volume for an individual wave that is predicted over the duration of a storm event.

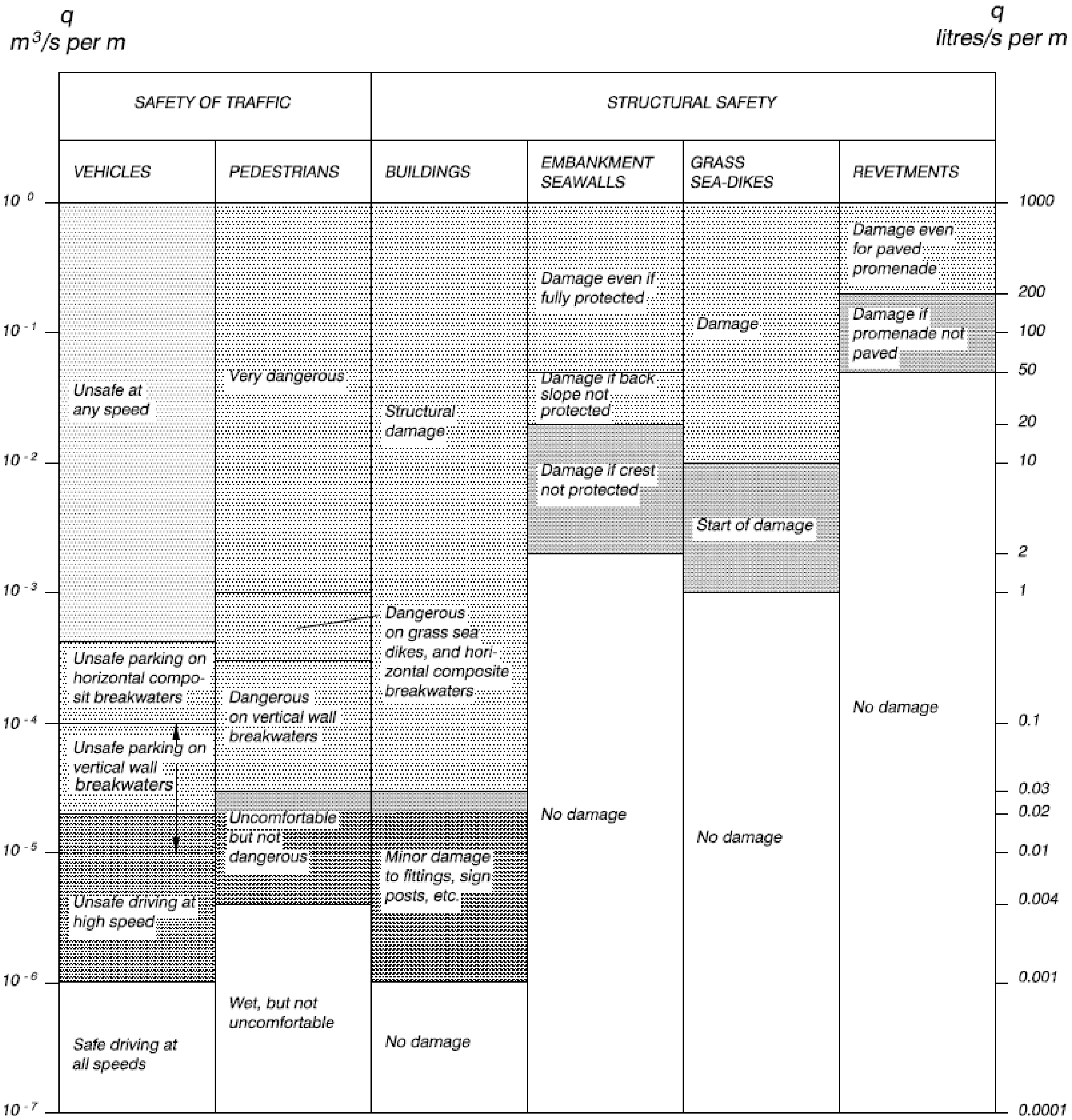

Figure 8: Critical Values of Average Overtopping Discharges (USACE, 2006)

Hazard type and reason	Mean discharge q (l/s per m)	Max volume V_{max} (l per m)
Rubble mound breakwaters; H_{m0} > 5 m; no damage	1	2,000-3,000
Rubble mound breakwaters; H_{m0} > 5 m; rear side designed for wave overtopping	5-10	10,000-20,000
Grass covered crest and landward slope; maintained and closed grass cover; H_{m0} = 1 – 3 m	5	2,000-3,000
Grass covered crest and landward slope; not maintained grass cover, open spots, moss, bare patches; H_{m0} = 0.5 – 3 m	0.1	500
Grass covered crest and landward slope; H_{m0} < 1 m	5-10	500
Grass covered crest and landward slope; H_{m0} < 0.3 m	No limit	No limit

Figure 9: Limits for wave overtopping for structural design of breakwaters, seawalls, dykes and dams (EurOtop, 2016)

Hazard type and reason	Mean discharge q (l/s per m)	Max volume V_{max} (l per m)
Significant damage or sinking of larger yachts; H_{m0} > 5 m	>10	>5,000 – 30,000
Significant damage or sinking of larger yachts; H_{m0} = 3-5 m	>20	>5,000 – 30,000
Sinking small boats set 5-10 m from wall; H_{m0} = 3-5 m Damage to larger yachts	>5	>3,000-5,000
Safe for larger yachts; H_{m0} > 5 m	<5	<5,000
Safe for smaller boats set 5-10 m from wall; H_{m0} = 3-5 m	<1	<2,000
Building structure elements; H_{m0} = 1-3 m	≤1	<1,000
Damage to equipment set back 5-10m	≤1	<1,000

Figure 10: General limits for overtopping for property behind the defence (EurOtop, 2016)

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Hazard type and reason	Mean discharge q (l/s per m)	Max volume V_{max} (l per m)
People at structures with possible violent overtopping, mostly vertical structures	No access for any predicted overtopping	No access for any predicted overtopping
People at seawall / dike crest. Clear view of the sea. H_{m0} = 3 m H_{m0} = 2 m H_{m0} = 1 m H_{m0} < 0.5 m	0.3 1 10-20 No limit	600 600 600 No limit
Cars on seawall / dike crest, or railway close behind crest H_{m0} = 3 m H_{m0} = 2 m H_{m0} = 1 m	<5 10-20 <75	2000 2000 2000
Highways and roads, fast traffic	Close before debris in spray becomes dangerous	Close before debris in spray becomes dangerous

Figure 11: Limits for overtopping for people and vehicles (EurOtop, 2016)

2.11 TPS Design Options

The TPS design options under consideration for application at Ocean-Umina Beach comprise:

- rock revetment;
- piled seawall;
- Stepped concrete seawall; and,
- Geotextile container wall.

2.12 Storm Damage and Maintenance

The rock, piled and concrete TPS structures would be designed to withstand a 100 year ARI storm event. The tolerable level of damage to rock structures in a 100 year ARI storm event would be 5%.

It is assumed that the nominated design life for structures (refer **Section 2.2**) would be achieved with periodic inspection and maintenance of any identified defects. Maintenance of minor damage that may occur during large storms is important, particularly for flexible structures such as rock revetments. If minor damage (e.g. movement of armour rocks) is not repaired post-storm this leaves a section of the revetment more vulnerable to more significant damage in a future storm. Exposure of the underlayer to wave action due to loss of armour rock protection can lead to eventual failure of the structure and damage to landward assets. As such, it is important that the implementation of a TPS is accompanied with an appropriate program and allocated budget for ongoing inspection and maintenance over its design life.

2.13 Foreshore and Beach Access

A key objective of the project is to maintain beach access during phases of erosion. All existing beach accessways are to be reinstated following installation of the TPS. Accessways are to be designed in a manner that accommodates storm erosion demand and periodic beach nourishment campaigns. The TPS is to be located as far landward as possible so as to minimise encroachment on the beach area and to facilitate burial of the TPS in the back-beach area and establishment of a vegetated dune.

2.14 Beach Nourishment

It is assumed that the TPS would be installed in conjunction with periodic beach nourishment campaigns that would be undertaken on an opportunistic basis to supplement the natural sand buffer for coastal storm erosion and to enhance beach amenity.

3 Concept Design of TPS Options

3.1 Rock Revetment

This option comprises a conventional rock revetment, which would be constructed from three(3) layers of rock armour underlain by a heavy grade geotextile fabric (refer **Appendix C** for typical section and example photo in **Figure 12**). The rock revetment structure would be positioned as far landward as is practicable to minimise encroachment on the available beach width. The rock revetment crest would be set at 5.5m AHD so that it is generally lower than the crest of the existing natural dune profile, which varies along the beach from around 7.5m AHD immediately east of Ocean Beach SLSC and progressively lowers to around 5-5.5m AHD at Ettalong Point. This should allow for subsequent dune re-building and revegetation so that the rock revetment is a buried structure that is only uncovered during major coastal storms that cause depleted beach conditions and erosion of the dune. The toe of the rock revetment would extend down to the design scour level of -1m AHD. The width of the rock revetment from the crest (seaward face) to the toe would be approximately 13.5m.

Figure 12: Seawall constructed from igneous rock at Stockton Beach, NSW

The latest guidance on the hydraulic stability of rock structures is contained in The Rock Manual (CIRIA, 2012). For design of revetments in shallow water conditions the Van Gent (2004) method for armour design in shallow water is considered to be the most applicable. This method was applied with the following input parameters:

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- H_s = 2.3m and $H_{2\%}$ = 3.2m[2]
- T_p = 13.2s, with $T_{m-1,0}$ determined to be 10.7s from wave modelling
- 2.5 hour storm duration (wave depth limited and can only occur around high tide) giving number of waves (N) during the storm of 840 for 10.7s period waves
- structure slope of 1V:1.5H
- notional permeability of structure (P) of 0.4
- coefficient values of c_{pl} = 7.25 and c_s = 1.05 for assessment of the 5% confidence limit
- S_d = 2 for 0-5% damage on a double layer armour structure
- sea water density = 1,025 kg/m^3
- igneous rock density = 2,600 kg/m^3

The median primary armour mass (M50) was determined to be 4.6 tonnes. The resultant rock size grading for the primary armour layer, first underlayer and second underlayer is summarised in **Table 13**. The nominal diameter (D_n) of each rock mass is provided in brackets and is based on a rock density of 2,600 kg/m^3.

Table 13: Rock Size Grading

Rock Layer	Minimum Rock Mass	Median Rock Mass	Maximum Rock Mass
Primary Armour Layer	3,450 kg (1.10 m)	4,600 kg (1.21 m)	5,750 kg (1.30 m)
First Underlayer	320 kg (0.50 m)	460 kg (0.56 m)	600 kg (0.61 m)
Second Underlayer	12 kg (0.16 m)	23 kg (0.21 m)	35 kg (0.24 m)

Each rock layer is required to be a minimum of two rocks thick. In accordance with The Rock Manual (CIRIA, 2012), this thickness is determined based on the D_{n50} (median nominal diameter) multiplied by the number of rocks in the layer and a layer thickness coefficient (k_t). A k_t value of 0.92 has been adopted to correspond with double standard placement of irregular rock using the highest point survey method. The resultant layer thicknesses (two rocks thick) are summarised in **Table 14** and have been used on the Drawings. The crest width would be a two rocks wide, which corresponds to a width of 2.4m.

Table 14: Rock Layer Thicknesses

Rock Layer	D_{n50}	Layer Thickness (two rocks thick)
Primary Armour Layer	1.21 m	2.2 m
First Underlayer	0.56 m	1.0 m
Second Underlayer	0.21 m	0.4 m

3.2 Piled Seawall

This option comprises a vertical seawall installed at the rear of the dune and adjacent to the existing footpath and carparking areas. The seawall would comprise a secant pile wall formed by overlapping reinforced (hard piles) and unreinforced (soft piles) concrete piles installed to form a continuous vertical wall along the shoreline (refer **Appendix C** for typical section). The crest height of the seawall would vary to match the retained ground levels behind the existing dune, which vary from around 7.5m AHD at the western end of the project area to around 5m AHD at the eastern end near Ettalong Point. A summary of

[2] *Design wave height obtained from nearshore wave transformation was modified to reflect wave height statistics associated with waves in the surf zone using the method of Battjes and Groenendijk (2000).*

alternative piled wall configurations for different crest heights is provided in **Table 15**, and the worst case for a crest height of 7.5m AHD is shown on the typical section (refer **Appendix C**).

Table 15: Piled Seawall Concept Configurations

Crest RL (AHD)	Pile Diameter (mm) x Length (m)	Hard Pile Centres (mm)	ULS Anchor Load (tonnes)	Anchor Spacing (mm)
7.5	900 x 19	1500	36	1500
6.0	900 x 16 750 x 16	1500 1200	28 44	1500 2400
5.0	600 x 13	900	25	1800

A reinforced concrete capping beam would be cast in situ along the top of the piles and ground anchors could be installed at intervals along the capping beam to minimise the depth of piling required to retain the significant height of soil exposed during a design scour event. The anchors would be installed at mid-depth within the capping beam at an angle of 30 degrees beneath the adjacent footpath and road pavement. The required anchor lengths are estimated to be 16-22m. An alternative would be to install tie rods at intervals along the wall back to deadman anchors, however this would require demolition of the existing road pavement.

Architectural finishes could be applied to the top of the capping beam such as low-level block walls and handrails/balustrades. The concrete capping beam could also be buried beneath natural ground level to improve visual amenity. It is anticipated that the existing bitumen footpath would be demolished as part of the works and reinstated with a wider shared path to Council standards.

An example of a secant pile seawall with a concrete capping beam at Cudgen Surf Club, Kingscliff Beach is shown in **Figure 13** and installation of secant pile walls at Kingscliff Beach and South Curl Beach are shown in **Figure 14** and **Figure 15**.

Figure 13: Secant pile wall with concrete capping beam installed in front of Cudgen Surf Club, Kingscliff (Source: ASP)

Figure 14: Installation of secant pile wall at Kingscliff Beach (Source: SEE Civil)

Figure 15: Construction photos during installation of an anchored secant pile wall and capping beam installed at South Curl Curl Beach

3.3 Stepped Concrete Seawall

This option comprises a stepped concrete seawall in the form of bleacher-style steps (refer **Appendix C** for typical section) to provide both shoreline protection from coastal erosion and the amenity of a formalised seating area facing the ocean. The stepped seawall structure would be constructed from marine grade reinforced concrete (precast or in situ concrete) and would be supported by reinforced concrete bored piles that are integrated into the base of the stepped seawall with reinforced concrete pile caps/beams. The piles would comprise three (3) rows of 600mm diameter continuous flight auger (CFA) piles at 2.4m centres. The rear, central and front piles would require approximate toe levels at -5.5m AHD, -9.5m AHD and -10.0m AHD respectively in the medium dense sand expected to exist at the founding levels.

This type of structure is relatively high cost, and therefore best suited to existing public domain areas that are high use, popular with the local community, and are supported with other amenities and facilities (e.g. toilets, playgrounds, cafes etc.). As such, the frontage of Ocean Beach SLSC does present an opportunity to consider a stepped concrete seawall. At this location the seawall could be designed to tie into the existing concrete footpath and pavement area around the SLSC building, which are at ground levels of around 5.5-6m AHD. The conceptual geometry of the seawall shown on the typical section (refer **Appendix C**) comprises a crest level at 5.5m AHD and 900x500mm bleachers stepping down to a base bleacher nominal level of 0m AHD. The bleacher step profile would accommodate two steps inside each bleacher to facilitate stepped beach access in nominated locations along with stainless steel handrails (refer **Figure 16** and **Figure 18**).

While potentially adding significantly to cost, the structure could incorporate several features to reduce a monolithic appearance and maximise amenity including:

- bleachers for seating;
- steps and handrails to facilitate access the beach;
- ramps to provide all-ability access; and,
- showers.

Architectural design input is recommended to integrate the structure into the local setting and to incorporate urban and landscape design futures. For example, the stepped concrete seawall recently constructed at Kingscliff Beach in Tweed Shire (refer **Figure 18**) included a varied plan alignment for sections of bleachers within the upper structure, separated by footpaths and planters. Here public artwork is included in the form of stencilled phrases imprinted along selected bleacher risers, conveying a cultural connection to the local area and landscape. Colouring can also be introduced into the concrete to match the native sand as has been very successfully achieved within the stepped seawall at Dee Why Beach (refer **Figure 17**).

Figure 16: Stepped seawall at Sandy Bay, Hobart incorporating beach access steps

Figure 17: Stepped seawall at Dee Why Beach constructed in 1999, coloured to match natural colour of beach sand

Figure 18: Recent stepped seawall installation at Kingscliff Beach (2018)

3.4 Geotextile Container Wall

This option comprises a wall constructed from stacked sand-filled geotextile containers (refer **Appendix C** for typical section). This wall would be similar to the existing geotextile container wall at Barrenjoey Road (refer **Figure 19**), which has been in place for around 3 years now. The proposed wall would be constructed from 2.5m^3 geocontainers (ELCOROCK manufactured by Geofabrics Australasia or approved equivalent) stacked in two layers at a 1V:1.5H slope from a maximum crest level of 7.5m AHD down to a toe level of -1m AHD. To accommodate toe scour along the wall an additional 2.5m^3 geotextile 'scour flap' container would be provided along the base of the wall to act as a 'falling toe' in event of localised scour. This may allow for minor lifting of the toe level as part of the detailed design. The scour flap containers would be wrapped in geotextile fabric to integrate them with the wall structure. The outer layer of geocontainers would be manufactured from vandal-deterrent geotextile fabric to minimise damage over their design life.

Figure 19: Geotextile container wall installed at Barrenjoey Road

To minimise impacts on beach amenity, the alignment of the wall would be positioned as far landward as possible and buried beneath subsequent dune restoration and suitable revegetation works.

Based on field measurements, the 2.5m^3 geotextile containers would have typical dimensions of 2.6m length, 1.9m width and 0.6m height (Blacka, Carley et al (2007)). Geofabrics Australasia ELCOROCK Installation Guidelines (ref M155-06/11) state the following typical filled dimensions for these containers:

- Length = 2.4 to 2.6 m
- Width = 1.6 to 1.9 m
- Height = 0.6 to 0.7 m

A filled geotextile container height of 0.65m, width of 1.8m and length of 2.4m has been adopted for design purposes.

The design wave climate (refer **Section 2.9**) was applied to the design plot for stability of 2.5m^3 geocontainers (sand filled geotextile containers) under wave action within Coghlan et al (2009) (refer **Figure 20**). This determined that 2.5m^3 geotextile containers would not be stable in the most exposed area at the western end of the site (Ocean Beach SLSC) and would likely incur significant damage under the 100 year ARI design conditions (Hs=2.2-2.3m, Tp=13-13.2s), even if a reduced design life of 20 years (2040 water level) is considered. The 100 year ARI design wave conditions reduce moving eastward along the beach to around Hs=1.7-1.8m and Tp=12s, which would result in marginal stability of 2.5m^3 geotextile containers based on **Figure 20**. However, these reduced alongshore wave heights are dependent on the configuration of the offshore shoals at the entrance to Brisbane Water, which are dynamic and cannot be relied upon to provide wave attenuation to eastern areas of the beach at all times.

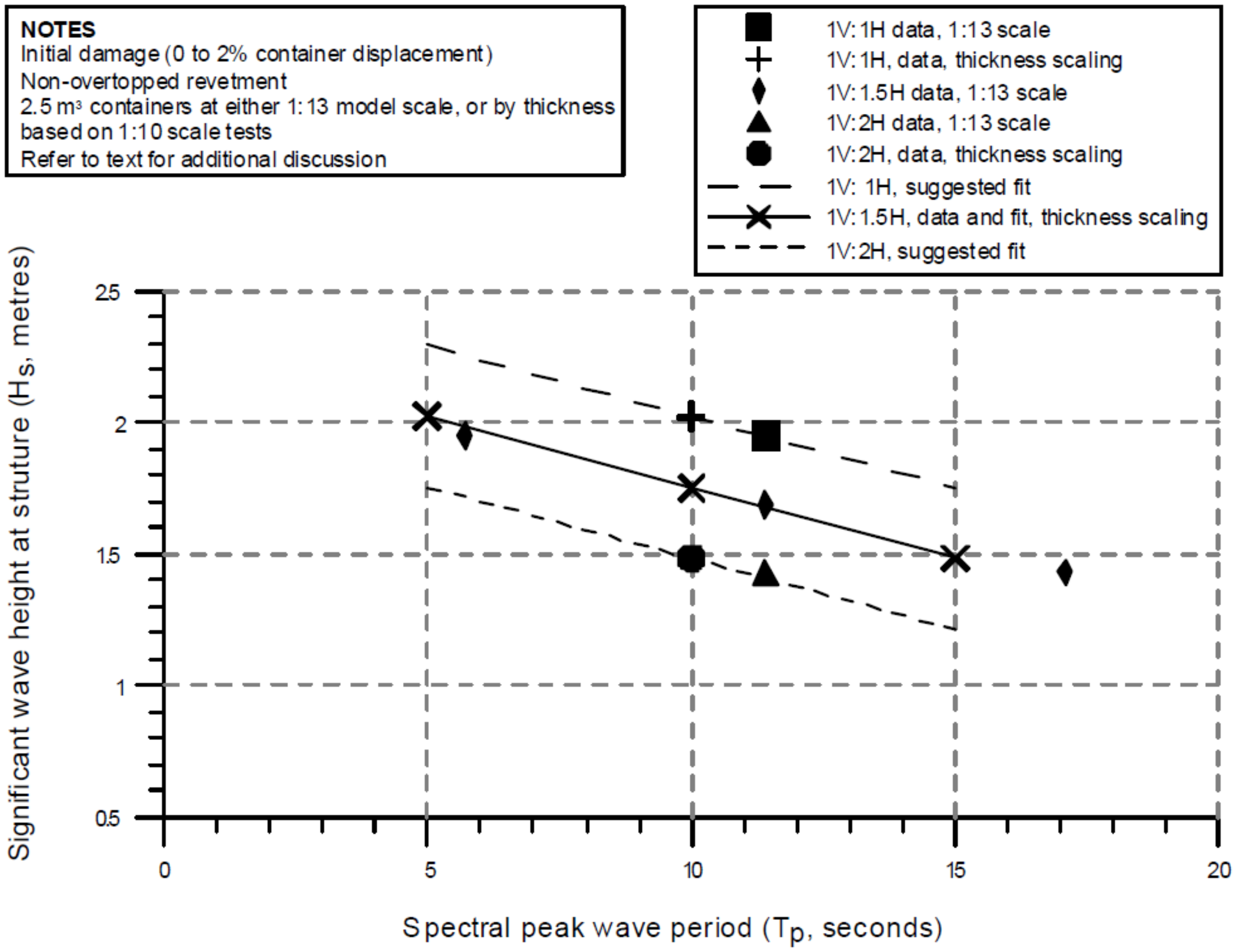

Figure 20: Design wave height for "initial damage" criterion for 2.5m³ geocontainers (Coghlan et al, 2009)

Due to the logistics of sand transport and the risk of damage to filled geotextile containers during transport to site, it is considered that filling the geotextile containers on site with local beach sand in the vicinity of the seawall location is a more practical solution. It is anticipated that this volume would be sourced from the excavation of dune and beach sand required to prepare the founding level (-1m AHD) of the wall. Sourcing sand from the local beach for a terminal geotextile container structure is a reasonable proposition and a common approach. The dune volume occupied by the TPS is essentially outside of the beach 'swept prism'. The structure results in no net loss of beach volume in the long term since sand from the containers would be returned to the system at the end of the seawall design life.

4 Assessment of TPS Options

4.1 Rock Revetment

To minimise encroachment on the beach, a rock revetment would need to be positioned as far landward as possible. Due to the bulk and slope of the rock structure, this would require significant excavation of the existing dune. This would result in loss of existing dune vegetation which in some areas comprises mature banksia trees and other native species, particularly immediately east of Ocean Beach SLSC and Augusta Street. Reinstatement of the dune over the top of the rock revetment would require the investment of time and effort, with the dune vegetation likely to take some time to be restored to its current level of maturity and coverage.

Beach amenity would be impacted by the presence of the rock structure, particularly during coastal storms when the face of the seawall may become exposed. Installation of beach access stairs supported on piles would be required to maintain beach access during these times. To reduce the bulk of the rock revetment, denser igneous rock is proposed which is more visually intrusive (i.e. in comparison to sandstone) due to its grey/black colouration.

A rock revetment would be a durable structure that would be resilient under wave action during coastal storms. Any damage caused by storms can be accommodated without catastrophic failure by the flexible nature of the structure. Maintenance requirements for the structure would be relatively low, comprising re-grading and/or top-up with additional armour rock following major coastal storms.

Being a porous and permeable structure with roughness over its sloped seaward face, a rock revetment with a crest height at 5.5m AHD is not expected to be overtopped with estimated wave runup levels at around 5.3m AHD when subjected to a 100 year ARI wave condition and 2070 extreme water level (including SLR).

The estimated comparative per metre costs (excluding fixed costs) of a rock revetment are approximately $15,000 per metre length, excluding any costs of subsequent dune restoration works.

The advantages and disadvantages of this option are summarised below.

<u>Advantages</u>

- good absorption of wave energy limiting wave reflection and overtopping run-up
- low maintenance and high durability
- flexible structure which can accommodate foundation movements
- wider range of potential contractors than for a concrete seawall
- relatively short construction period less susceptible to stoppages due to weather
- cheaper option subject to availability of rock
- flexible design, lower construction tolerances
- adaptive to sea level rise
- relatively simple repairs if damaged in storms

<u>Disadvantages</u>

- construction requires significant excavation of the existing dune and subsequent restoration works
- impacts beach access and requires installation of beach access stairs
- can provide habitat for undesired wildlife (rodents, snakes) where exposed
- local quarries may not supply suitable rock (size, durability etc) which may then need to be hauled in from a distance away from the site

- may reduce beach width subject to location of crest of structure
- toe and seaward face of sloping structure may be exposed more regularly, reducing beach amenity

4.2 Piled Seawall

A significant benefit of secant piled seawall is that its installation requires significantly less excavation of the existing dune and removal of vegetation in comparison to other options requiring the preparation of the TPS toe foundation. The secant piles could be installed from the roadside with drilling rigs and the in situ concreting of the capping beam would be constructed with formwork and delivery of concrete from the road. It is anticipated that a corridor of ground clearing and levelling would be required at the back of the dune to allow drilling equipment access to the seaward side of the capping beam for installation of ground anchors.

A secant piled seawall would be a substantial structure (e.g. 900mm pile diameter) and would require installation of high capacity ground anchors at relatively close spacings and with anchor lengths of 16-22m. Preliminary discussions with civil contractors that are familiar with ground anchor installation have indicated that a 30 tonnes ULS capacity in medium dense sand is comfortable and anchor lengths of up to 20m are manageable. It was advised that ULS capacity of greater than 30 tonnes is possible but experience has found that some higher capacity anchors can fail load tests in medium dense sand. As such, the proposed design (particularly for the highest wall height at 7.5m AHD) is considered to be at the limit of anchor constructability. There is a risk that the design may be found to be unfeasible with further detailed design or that the high capacity ground anchors required may fail load tests in the field if there is insufficient strength in the soil. An alternative would be to install tie rods at intervals along the wall back to deadman anchors, however this would require demolition of the existing road pavement.

Due to its position at the rear of the dune and its vertical face, a secant piled seawall would have minimal impact on beach amenity under typical conditions. The seawall would act as an effective ‘safety net’ to protect the road and other assets in the event that coastal storms erode away the entire dune. The vertical face of the seawall would be uncovered only in exceptional circumstances when critical assets are threatened by major coastal storms (refer **Figure 13**).

A secant piled seawall could be designed with appropriate concrete specifications to provide durability over the design life with minimal maintenance. It would provide a solid wall of defence against wave action and would not be expected to need any repairs following coastal storms.

Under design scour conditions (-1m AHD scour level), a secant piled seawall would present as a vertical wall subject to wave action. For a 100 year ARI wave condition and 2070 extreme water level (including SLR), a vertical piled seawall would have a wave runup height of around 5m AHD. For a piled seawall with a crest at 6-7.5m AHD (shallow water Hmo=1.6m), this would result in a mean overtopping discharge of 2-6 l/s/m, which should be acceptable for people and cars in accordance with EurOtop criteria (refer **Figure 11**). It should also be noted that a piled seawall structure would be readily adaptable under future sea level rise, as the height of the wall could be increased at any time by building low-level walls on top of the existing crest.

The estimated comparative per metre costs (excluding fixed costs) of a secant piled seawall are approximately $26,000 per metre length for a wall with crest at 7.5m AHD, excluding any costs of subsequent dune restoration works. Walls with lower crest levels of 6m AHD and 5m AHD are estimated to cost approximately $23,000/metre and $19,000/metre respectively.

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The advantages and disadvantages of this option are summarised below.

Advantages

- installation does not require significant excavation of existing dune
- occupies substantially less space than sloped protective structures
- does not encroach on beach and would only be exposed following major erosion event
- existing beach access provisions can be maintained
- low visual impact
- can incorporate low-level retaining wall and balustrade/handrails
- readily adaptable under future sea level rise
- acceptable durability and low maintenance requirements

Disadvantages

- high degree of construction expertise required (piling and ground anchors)
- high capacity anchors are required, which are at the limit of constructability
- tie rods back to deadman anchors may be necessary, which would require demolition of the road pavement
- more onerous construction tolerances required
- corridor of ground clearing and levelling would be required at the back of the dune to install ground anchors
- rigid design is more susceptible to catastrophic failure
- no absorption of wave energy with wave reflection against vertical wall and overtopping during severe storms once beach and dune are fully eroded
- relatively high cost

4.3 Stepped Concrete Seawall

A stepped concrete seawall would provide both protection from coastal processes and a formal seating area to enhance the recreational amenity of beachfront areas. It is envisaged that this type of protective structure would be appropriate in an area that already provides open access to the beach and is supported with other infrastructure such as cafes or other facilities that encourage congregation of the local community. As such, the frontage of Ocean Beach SLSC would be well-suited for this type of structure.

Construction of a stepped concrete seawall would require significant excavation and clearing of existing dunes in front of Ocean Beach SLSC. If the seawall was constructed to tie-in to the seaward edge of the existing footpath and paved area, its position would be recessed into the existing dune face. This may create problems with sand accumulation on the steps if vegetation and/or sediment fencing is not maintained on the adjacent dunes.

A structure such as this would enhance amenity from a beach access and recreational amenity perspective. However, visual amenity would be an important aspect of the design with high quality landscape architecture input recommended.

A stepped concrete seawall could be designed with appropriate concrete specifications to provide durability over the design life with minimal maintenance. It would provide a solid wall of defence against wave action and would not be expected to need any repairs following coastal storms.

The rate of overtopping on a stepped concrete seawall has been found to be relatively high in other studies involving physical modelling of design conditions (e.g. 100 year ARI event), such as that completed for the Kingscliff Beach seawall. However, if overtopping is found to be problematic over time due to sea level rise and other factors this could be managed by incorporation of a wave deflector at the crest of the structure as an adaptation measure.

The estimated comparative per metre costs (excluding fixed costs) of a stepped concrete seawall are approximately $40,000 per metre length.

The advantages and disadvantages of this option are summarised below.

Advantages

- occupies less space than a rock revetment
- provides high quality seating at the back of the beach
- more accommodating to incorporation of beach amenity such as access stairs, showers, ramps etc.
- can be used to enhance the amenity of existing public domain areas
- acceptable durability and low maintenance requirements

Disadvantages

- installation requires significant modification of existing dunes and change to visual character
- stepped beach access reduces accessibility in comparison to sand ramps
- high degree of construction expertise required (piling and marine concrete construction)
- more onerous construction tolerances required
- rigid design is more susceptible to catastrophic failure
- less adaptive to sea-level rise
- moderate absorption of wave energy but still prone to wave reflection and overtopping (however, can incorporate a wave deflection barrier)
- not constructed from natural material and requires architectural input to reduce visual impact and enhance amenity
- likely to be the highest cost option

4.4 Geotextile Container Wall

Similar to a rock revetment, a geotextile container wall would need to be positioned at the back of the beach to minimise encroachment on the beach width and associated amenity impacts. Excavation of the toe of the wall would require removal and subsequent reconstruction and revegetation of the existing dune. The construction methodology for the wall is relatively simple and would utilise the local beach sand to fill the containers. Geotextile manufacturers and suppliers such as Geofabrics Australasia offer proprietary filling frames and container handling attachments for excavators along with detailed installation instructions to guide contractors and can offer onsite support during the works. As such, the construction process is well defined as can be completed with conventional earthmoving equipment with a high level of support from suppliers.

Beach amenity would be impacted by the presence of the geotextile container structure, particularly during coastal storms when the face of the wall may be exposed. Piled beach access stairs could be installed over the top of the wall to maintain access during eroded beach conditions. However, due to the soft nature of the filled geotextile containers able-bodied people could still access the beach by stepping down the face of the wall. The visual impact of the structure would be relatively low with the colour of the geotextile material being similar to that of beach sand and likely integration of dune sand and vegetation within the wall (refer **Figure 19**).

The long-term durability of the geotextile containers is unlikely to extend over the desired 50 year design life and could be up to 15 to 20 years. The durability of geotextile fabric is affected by several mechanisms including fabric degradation caused by exposure to UV light, abrasion caused by wave action and sand movement, and vandalism such as knife cuts and punctures. Damage to the wall is likely to occur during major coastal storms due to instability of the geotechnical containers under wave action. Periodic repairs are likely to be required to the wall following coastal storms to re-pack the containers in failed sections and replace or repair damaged containers.

Once the geotextile containers have settled into place and the sand within the containers is wet, the wall would act as an impermeable stepped structure with a relatively low roughness. As a result, measured wave runup over the geotextile containers structures is high and based on testing by Coghlan et al (2009) would be in the order of 2.7 to 3.4 times the significant wave height (above the still water level) for typical storm wave periods of between 10 and 14 seconds (refer **Figure 21**). For the 100 year ARI design event at the present time (Hs = 2.2m, Tp=13s) this would result in a runup height of 7m above the 100 year ARI still water level (1.44m AHD) and significant overtopping of the structure. This would worsen over time due to sea level rise. The geotextile container wall would need to extend up to the full height of the dune in order to completely protect landward areas from wave action, although wave overtopping would still create a public safety hazard and cause potential damage to property.

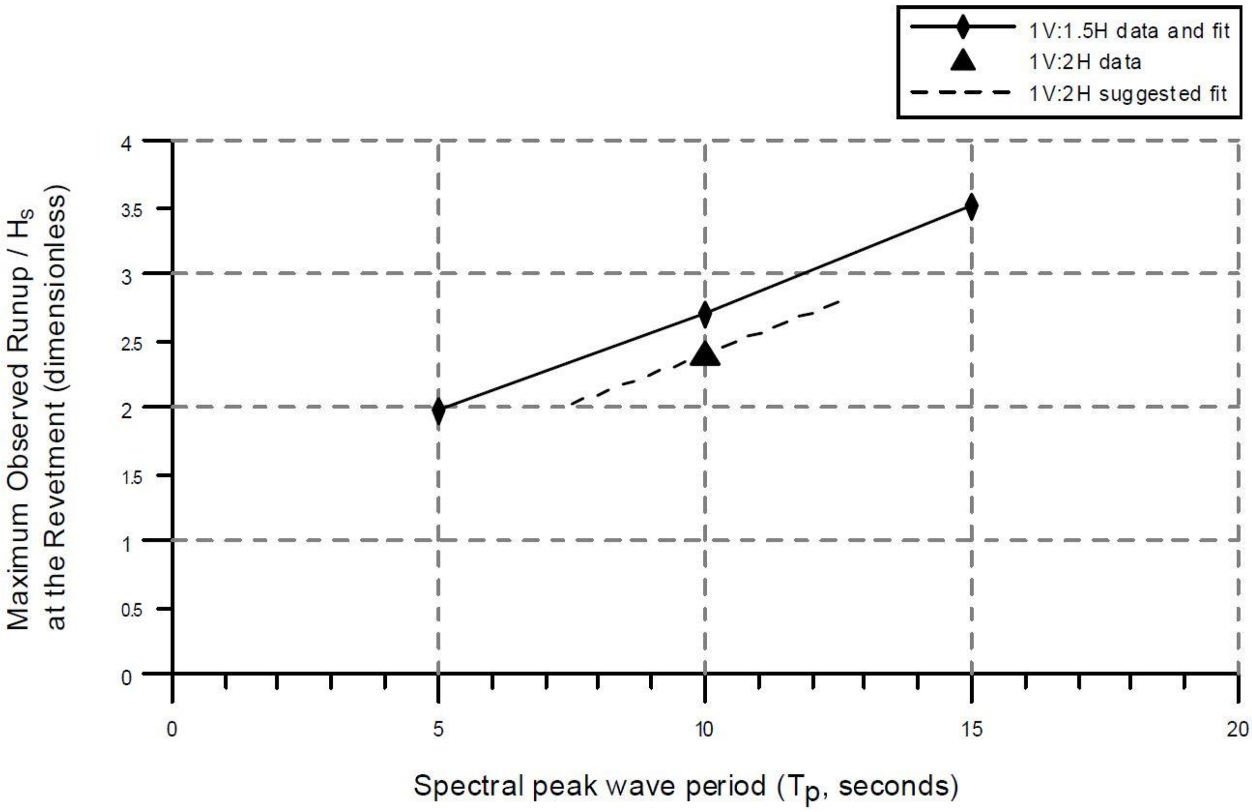

Figure 21: Maximum observed wave runup for 0.75m³ geocontainers (Coghlan et al, 2009)

The estimated comparative per metre costs (excluding fixed costs) of a geotextile container wall are approximately $7,000 per metre length, excluding any costs of subsequent dune restoration works.

The advantages and disadvantages of this option are summarised below.

Advantages

- can utilise local beach sand from toe excavation as a resource to fill geotextile containers
- flexible structure which can accommodate foundation movements
- wider range of potential contractors than for a concrete seawall
- relatively short construction period less susceptible to stoppages due to weather
- likely to be the lowest cost option
- flexible design, lower construction tolerances
- adaptive to sea level rise
- low visual impact in comparison to other options

Disadvantages

- construction requires significant excavation of the existing dune and subsequent restoration works
- impacts beach access and requires installation of beach access stairs, although able-bodied people could still climb down the face of the wall
- may reduce beach width subject to location of crest of structure
- toe and seaward face of sloping structure may be exposed more regularly, reducing beach amenity
- low absorption of wave energy resulting in high wave runup levels and expected overtopping during major coastal storms
- periodic maintenance expected due to instability of geotextile containers when subject to major coastal storms, possible vandalism and their deterioration over time
- durability of geotextile containers would not extend over the desired 50 year design life

5 Preferred TPS Proposal

5.1 General Scheme

Based on the above assessment and discussions with Council, the preferred TPS proposal comprises:

- future installation of a rock revetment along a section(s) of the beach, if triggered by unacceptable risk to coastal infrastructure; and,
- future installation of a stepped concrete seawall in front of Ocean Beach SLSC, if triggered by failure to maintain a natural dune profile at this location.

5.2 Rock Revetment

The coastal erosion hazard experienced at Umina-Ocean Beach is typically episodic and associated with short-term erosion resulting from coastal storms and the cyclical process of erosion and subsequent beach recovery. However, longer term progressive beach erosion has also occurred in recent times as a result of the disruption to onshore sediment supply caused by a shoal blowout event in 2012. As such, it is considered that triggers for TPS implementation should be linked to the risk to coastal infrastructure and the persistence of the risk to take into account potential beach recovery under the action of natural processes, which would also be assisted by beach scraping and opportunistic beach nourishment campaigns.

The trigger for Council to call tenders for the construction of a rock revetment would be unsustainable dune maintenance and management (e.g. following one calendar year of little progress) in attempts to replenish and stabilise the dune following a severe storm or closely connected series of storms, leading to high and unacceptable risk to coastal infrastructure (e.g. stormwater outlets, footpaths, carparking areas, road pavements).

The trigger for Council to commence construction of a rock revetment would be when the existing footpath is threatened by erosion (i.e. no dune buffer) for a continuous period of greater than 3 months. Threats to stormwater outlets should be manageable locally without the need to commit to a rock revetment.

It should be noted that sectional installation of a rock revetment along vulnerable areas of the beach would need to consider potential ‘end effects’ at the extents of the structure. This could be mitigated by tying the structure into hard points (e.g. existing structures or natural features), positioning the end points away from at risk infrastructure or designing landward returns into areas of the natural dune system with a good sand buffer volume.

5.3 Ocean Beach SLSC

It is Council’s strong preference to maintain a natural dune profile in front of Ocean Beach SLSC for as long as this remains feasible. Council would endeavour to maintain and enhance the existing dune buffer by a combination of vegetation planting and maintenance, beach scraping to enhance beach recovery following storm erosion, and opportunistic beach nourishment campaigns using sand sourced from entrance maintenance dredging. As such, the installation of a TPS in the form of a stepped concrete seawall would only be considered as a last resort option if progressive erosion results in the complete loss of the foredune. Maintenance of a healthy vegetated dune system has a number of benefits including management of wind blown sand, high ecological value, ease of beach access and maintaining the existing natural aesthetics.

The trigger for implementation of a stepped concrete seawall would be unsustainable dune maintenance and management, indicated by one calendar year of little progress in attempts to replenish and stabilise the dune following a severe storm or closely connected series of storms resulting in significant or complete loss of the foredune in front of the existing concrete footpath and paved area.

It is noted that the shoreline length from the western end of the carpark to the eastern end of the paved area around the Ocean Beach SLSC building is approximately 150m. As such, application of a stepped concrete seawall over this entire length would involve a significant capital cost. Consideration should be given to reducing the length of the concrete seawall to a more cost-effective extent. This could comprise the paved area in front of Ocean Beach SLSC, which covers a shoreline length of approximately 40m. Depending on the severity and extent of erosion in the area, the implementation of a stepped concrete seawall could be supplemented with a rock revetment in adjacent shoreline areas (e.g. stepped seawall and adjacent rock revetments at South Avoca Beach SLSC).

5.4 Monitoring

To ensure the timely identification of the above triggers and implementation of the TPS, ongoing monitoring of the beach condition is required. This would comprise:

- Periodic visual, photographic and structural inspections of beach condition and coastal infrastructure that may be at risk from coastal erosion. The suggested frequency for general inspections would be quarterly with additional targeted inspections to be completed following any major coastal storms or reports of significant threats to stormwater outlets or the existing footpath.

- Periodic beach surveys and analysis to monitor the sand buffer volume, width of dune and position of erosion scarps. Beach surveys could be undertaken by UAV photogrammetry techniques supplemented with topographic survey as required to determine accurate ground levels in heavily vegetated areas (e.g. dune vegetation). The suggested frequency for general surveys would be biannually with additional targeted surveys to be completed following any major coastal storms or following significant beach scraping and/or beach nourishment campaigns.

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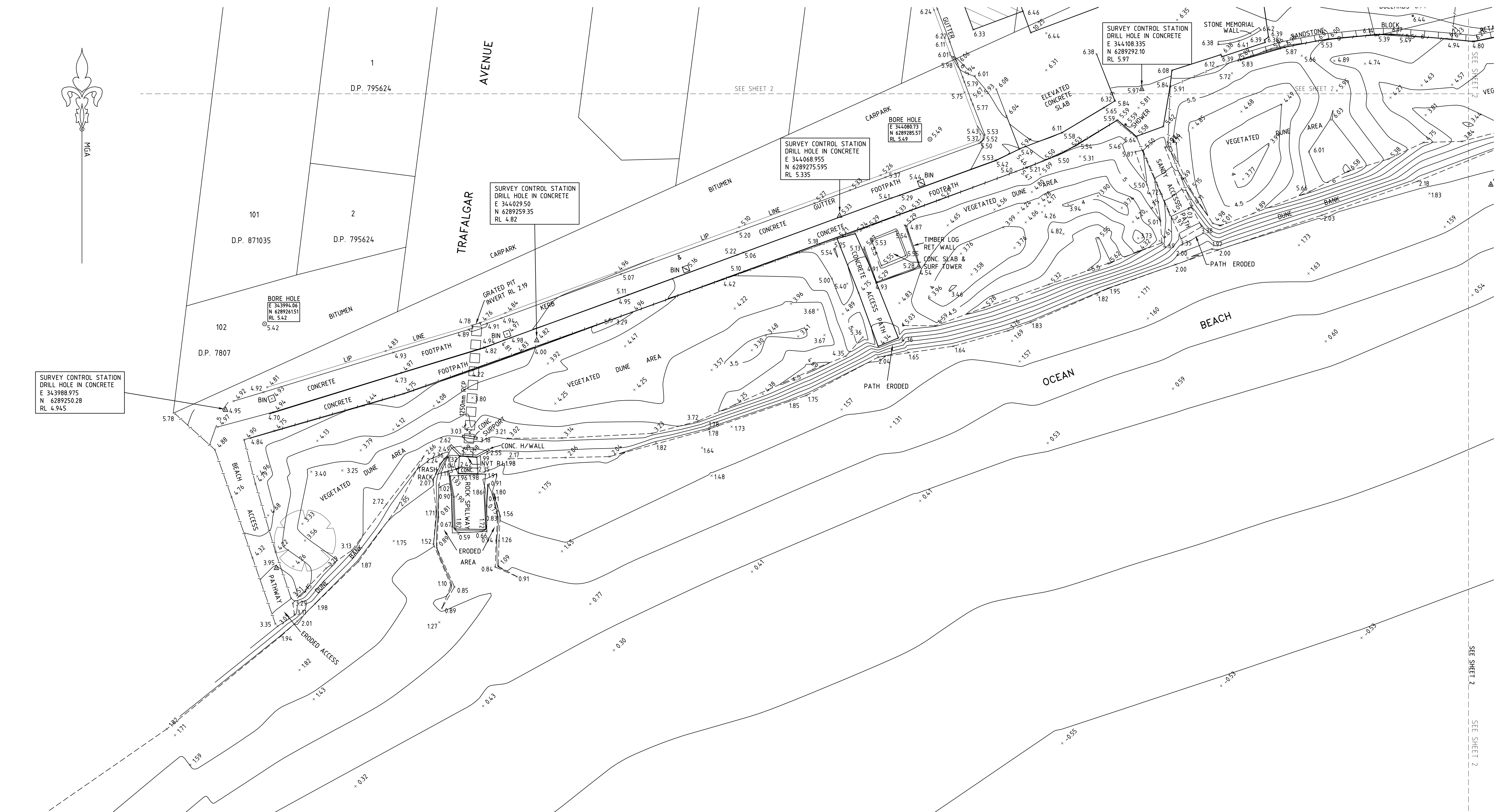
Appendix A: Terrestrial Survey



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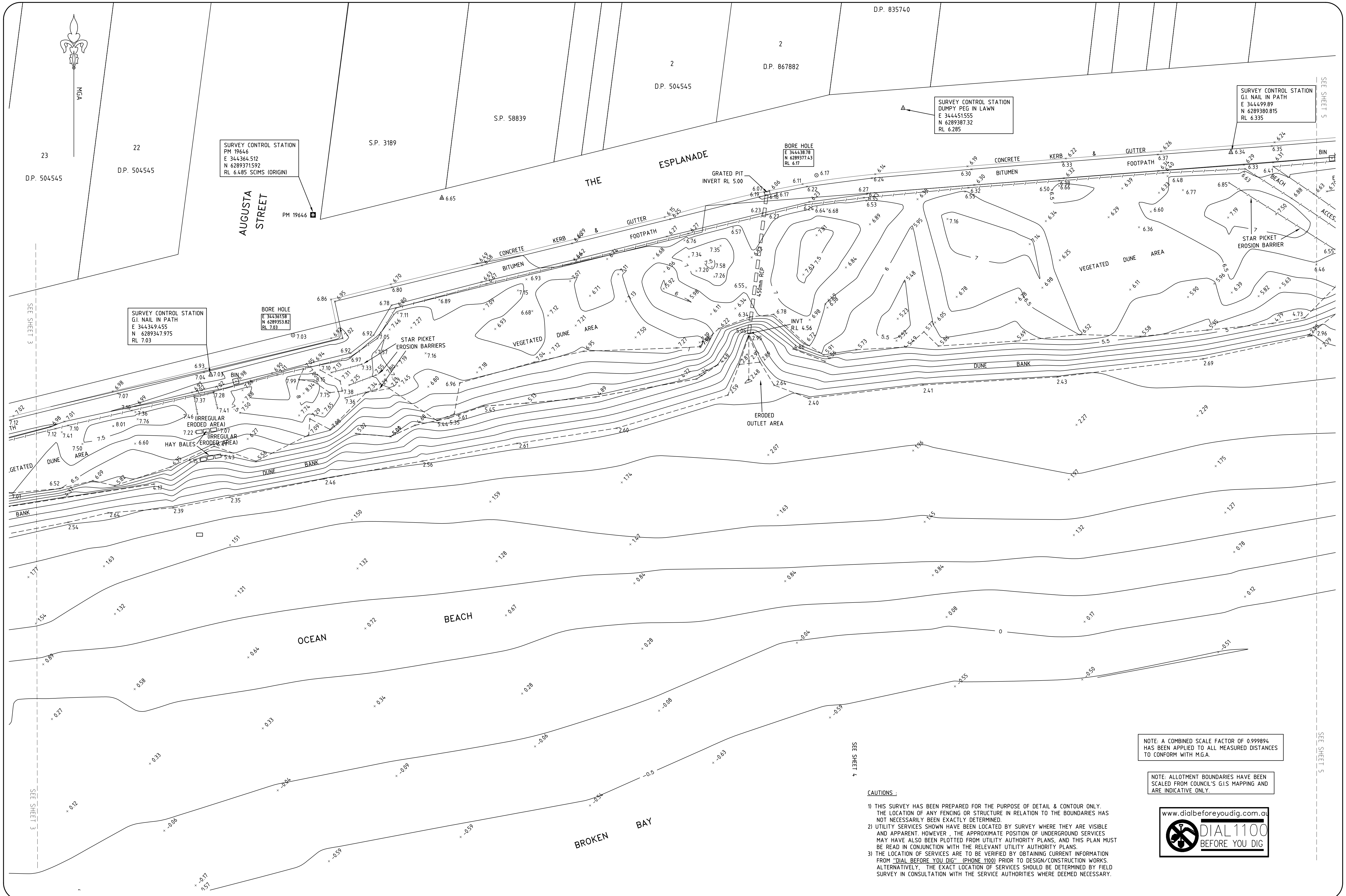

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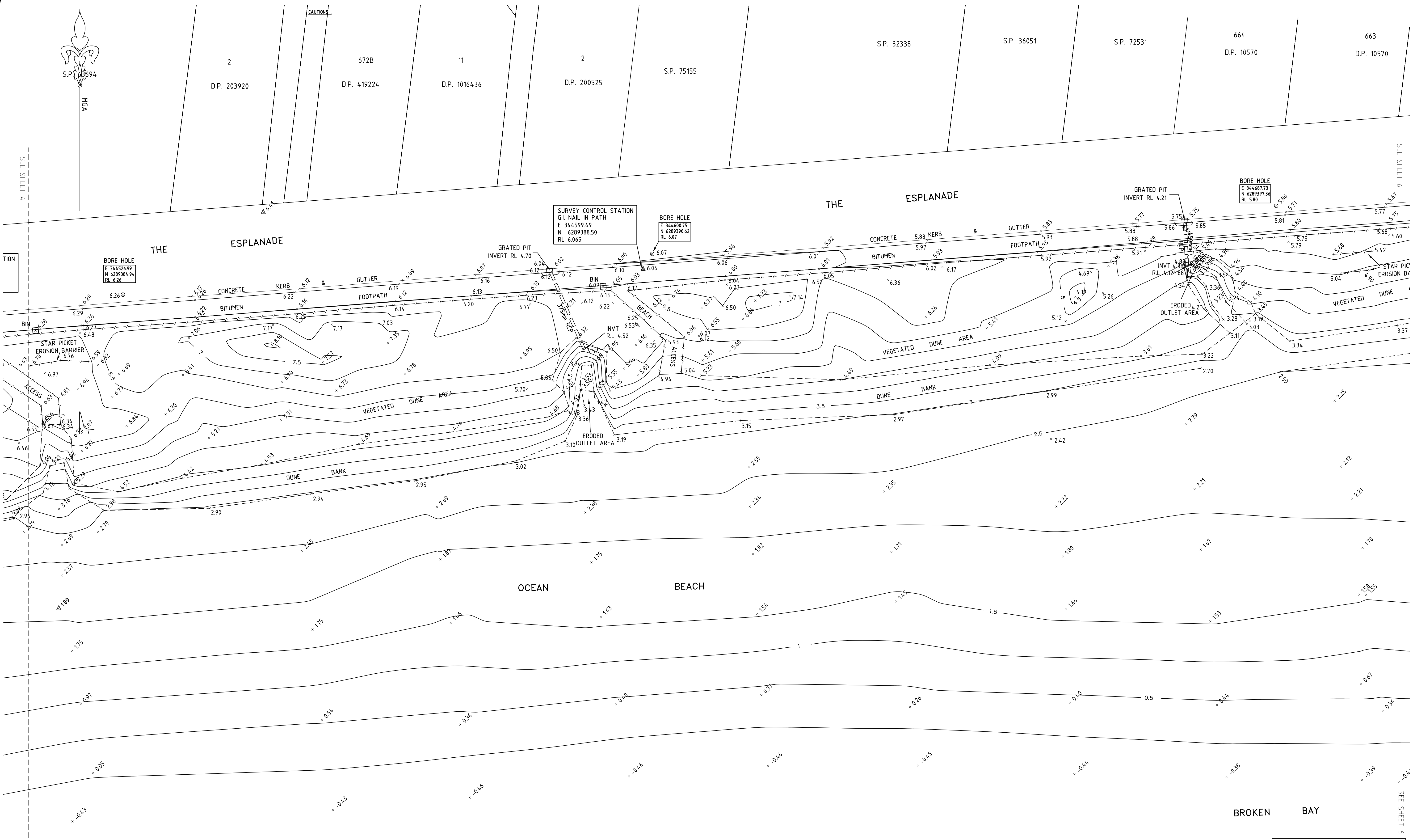

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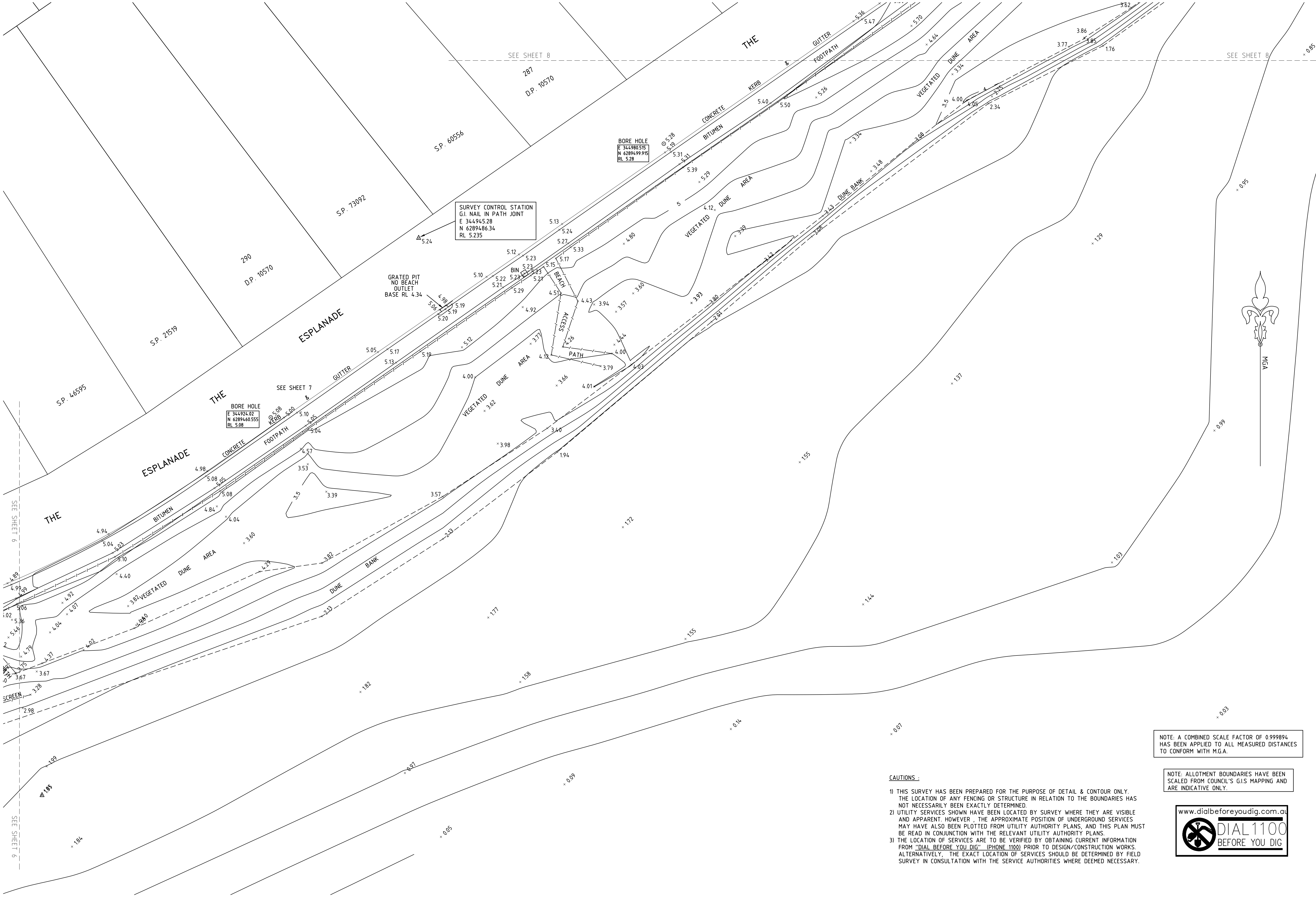

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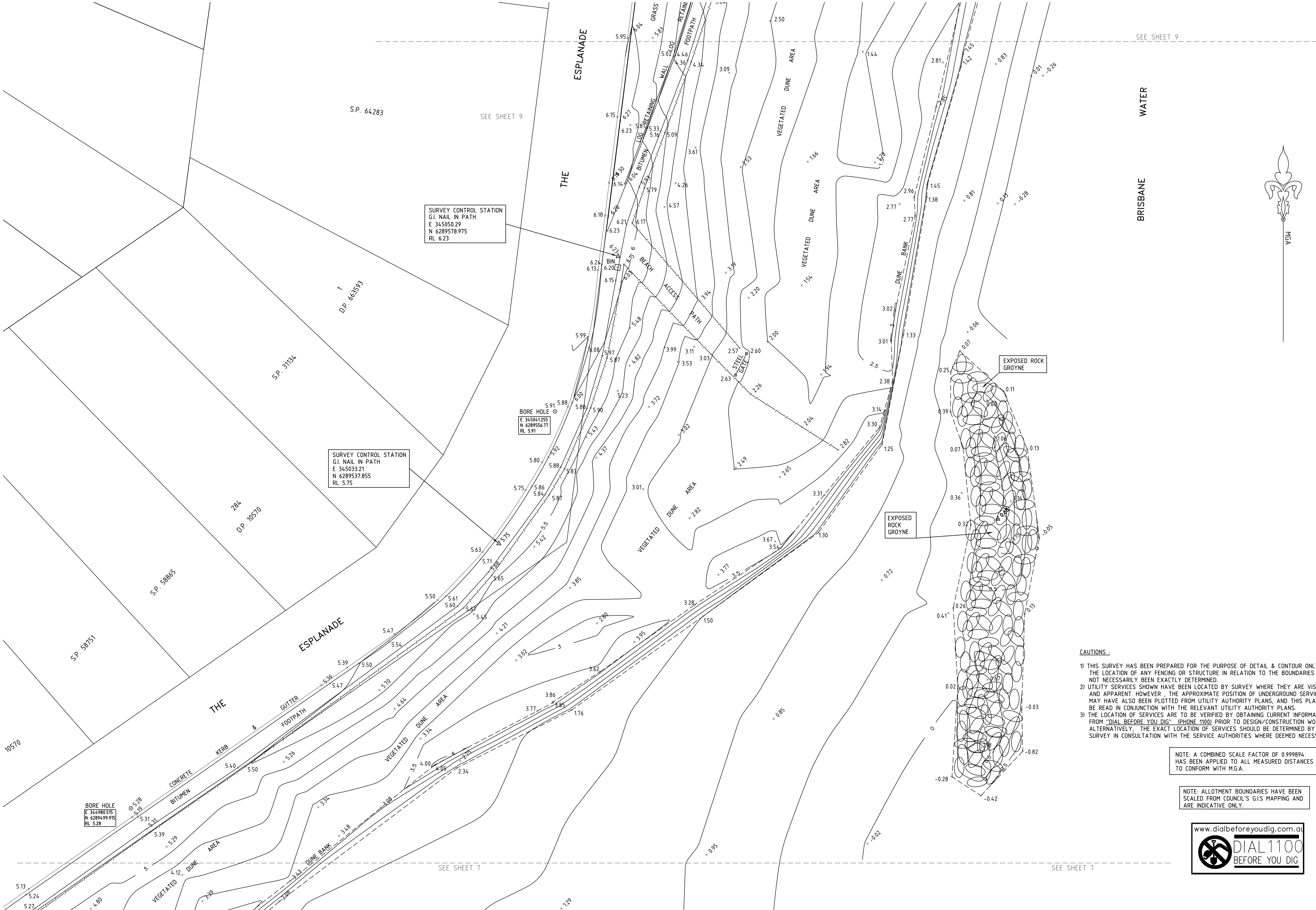

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SCALE: 1:250 @ A1 SHEET 1:500 @ A3 SHEET	STEPHEN THORNE AND ASSOCIATES TSS Total Surveying Solutions Pty Ltd 3/127 ERINA STREET, GOSFORD. 2250 Phone (02)4323-1255 Email stsurvey@bigpond.net.au P.O. BOX 1315 GOSFORD NSW 2250 A.C.N. 603 458 546 A.B.N. 53 603 458 546	DETAIL AND CONTOUR SURVEY FOR OCEAN BEACH FORESHORE UPGRADE AT THE ESPLANADE OCEAN BEACH, LGA CENTRAL COAST	5 0 5 10 15 20 25 SCALE OF METRES 1:250	amendments CAD FILE: 7825_DET.dwg THIS PLOT: 21-06-2018 14:39 REFERENCE: 7825	PLAN NUMBER 7825	ISSUE A
DATUM: A.H.D.	DATE 21/06/2018				SHEET 8 OF 9 SHEETS	

NOTE: A COMBINED SCALE FACTOR OF 0.999894 HAS BEEN APPLIED TO ALL MEASURED DISTANCES TO CONFORM WITH M.G.A.

NOTE: ALLOTMENT BOUNDARIES HAVE BEEN SCALED FROM COUNCIL'S G.I.S MAPPING AND ARE INDICATIVE ONLY.

CAUTIONS :

1) THIS SURVEY HAS BEEN PREPARED FOR THE PURPOSE OF DETAIL & CONTOUR ONLY. THE LOCATION OF ANY FENCING OR STRUCTURE IN RELATION TO THE BOUNDARIES HAS NOT NECESSARILY BEEN EXACTLY DETERMINED.
2) UTILITY SERVICES SHOWN HAVE BEEN LOCATED BY SURVEY WHERE THEY ARE VISIBLE AND APPARENT. HOWEVER , THE APPROXIMATE POSITION OF UNDERGROUND SERVICES MAY HAVE ALSO BEEN PLOTTED FROM UTILITY AUTHORITY PLANS, AND THIS PLAN MUST BE READ IN CONJUNCTION WITH THE RELEVANT UTILITY AUTHORITY PLANS.
3) THE LOCATION OF SERVICES ARE TO BE VERIFIED BY OBTAINING CURRENT INFORMATION FROM "DIAL BEFORE YOU DIG" (PHONE 1100) PRIOR TO DESIGN/CONSTRUCTION WORKS. ALTERNATIVELY, THE EXACT LOCATION OF SERVICES SHOULD BE DETERMINED BY FIELD SURVEY IN CONSULTATION WITH THE SERVICE AUTHORITIES WHERE DEEMED NECESSARY.

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DIAL 1100 BEFORE YOU DIG

SCALE: 1:250 @ A1 SHEET 1:500 @ A3 SHEET	STEPHEN THORNE AND ASSOCIATES TSS Total Surveying Solutions Pty Ltd 3/127 ERINA STREET, GOSFORD. 2250 Phone (02)4323-1255 Email stsurvey@bigpond.net.au P.O. BOX 1315 GOSFORD NSW 2250 A.C.N. 603 458 546 A.B.N. 53 603 458 546	DETAIL AND CONTOUR SURVEY FOR OCEAN BEACH FORESHORE UPGRADE AT THE ESPLANADE OCEAN BEACH, LGA CENTRAL COAST	5 0 5 10 15 20 25 SCALE OF METRES 1:250	amendments CAD FILE: 7825_DET.dwg THIS PLOT: 21-06-2018 14:42 REFERENCE: 7825	PLAN NUMBER 7825	ISSUE A
DATUM: A.H.D. DATE 21/06/2018					SHEET 9 OF 9 SHEETS	

Appendix B: Geotechnical Investigations

AERIAL IMAGE SOURCE: GOOGLE EARTH PRO 7.1.5.1557
AERIAL IMAGE ©: 2015 GOOGLE INC.

This plan should be read in conjunction with the JK Geotechnics report.

Title: **SITE LOCATION PLAN**

Location: OCEAN BEACH SURF LIFE SAVING CLUB TO KUORONG STREET BOAT RAMP, UMINA, NSW

Report No: 31518ZR

Figure No: 1

JK **Geotechnics**

PLOT DATE: 28/06/2018 3:15:47 PM DWG FILE: S:\6 GEOTECHNICAL\6F GEOTECHNICAL JOBS\31000'S\31518ZR UMINA\CAD\31518ZR.DWG

NOTE:
AERIAL IMAGE SOURCE: GOOGLE EARTH PRO, 7.1.5.1557, 2015.
(NOT GEO-REFERENCE TO SURVEY DRAWING)

LEGEND

- BOREHOLE
- CONE PENETRATION TEST
- APPROXIMATE CHAINAGE

SCALE 0 5 10 15 20 25 METRES 1:500 @A3

This plan should be read in conjunction with the JK Geotechnics report.

Title:	GEOTECHNICAL SITE PLAN (CH0m - CH200m)
Location:	OCEAN BEACH SURF LIFE SAVING CLUB TO KUORONG STREET BOAT RAMP, UMINA, NSW
Report No:	31518ZR
Figure No:	2

JK Geotechnics

PLOT DATE: 28/06/2018 4:54:27 PM DWG FILE: S:\6 GEOTECHNICAL\6F GEOTECHNICAL JOBS\31000's\31518ZR UMINA\CAD\31518ZR.DWG

LEGEND

- ● BOREHOLE
- ▼ CONE PENETRATION TEST
- APPROXIMATE CHAINAGE

SCALE 0 5 10 15 20 25 METRES 1:500 @A3

NOTE:
AERIAL IMAGE SOURCE: GOOGLE EARTH PRO, 7.1.5.1557, 2015.
(NOT GEO-REFERENCE TO SURVEY DRAWING)

This plan should be read in conjunction with the JK Geotechnics report.

Title:	GEOTECHNICAL SITE PLAN (CH200m - CH400m)
Location:	OCEAN BEACH SURF LIFE SAVING CLUB TO KUORONG STREET BOAT RAMP, UMINA, NSW
Report No:	31518ZR
Figure No:	3

JK Geotechnics

PLOT DATE: 28/06/2018 4:33:55 PM DWG FILE: S:\16 GEOTECHNICAL\6F GEOTECHNICAL JOBS\31000'S\31518ZR UMINA\CAD\31518ZR.DWG

LEGEND

- BOREHOLE
- CONE PENETRATION TEST
- APPROXIMATE CHAINAGE

NOTE:
AERIAL IMAGE SOURCE: GOOGLE EARTH PRO, 7.1.5.1557, 2015.
(NOT GEO-REFERENCE TO SURVEY DRAWING)

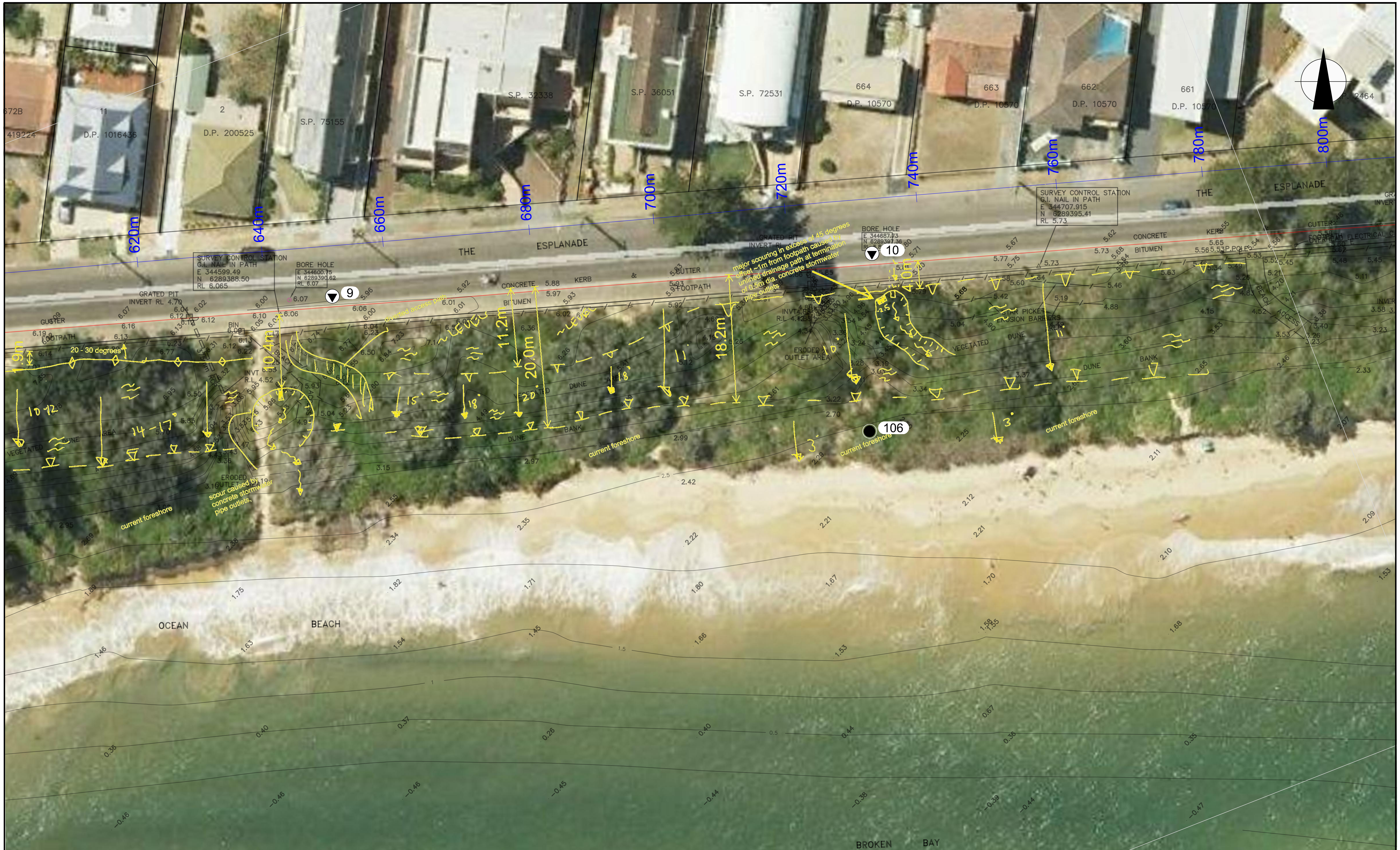
SCALE 0 5 10 15 20 25 METRES 1:500 @A3

This plan should be read in conjunction with the JK Geotechnics report.

Title:	GEOTECHNICAL SITE PLAN (CH420m - CH600m)
Location:	OCEAN BEACH SURF LIFE SAVING CLUB TO KUORONG STREET BOAT RAMP, UMINA, NSW
Report No: 31518ZR	Figure No: 4

JK Geotechnics

PLOT DATE: 28/06/2018 4:34:36 PM DWG FILE: S:\6 GEOTECHNICAL\6F GEOTECHNICAL JOBS\31000's\31518ZR UMINA\CAD\31518ZR.DWG



NOTE:
AERIAL IMAGE SOURCE: GOOGLE EARTH PRO, 7.1.5.1557, 2015.
(NOT GEO-REFERENCE TO SURVEY DRAWING)

LEGEND

- BOREHOLE
- CONE PENETRATION TEST
- APPROXIMATE CHAINAGE

0 5 10 15 20 25
SCALE 1:500 @A3 METRES

This plan should be read in conjunction with the JK Geotechnics report.

Title: **GEOTECHNICAL SITE PLAN (CH620m - CH800m)**

Location: OCEAN BEACH SURF LIFE SAVING CLUB TO KUORONG STREET BOAT RAMP, UMINA, NSW

Report No: 31518ZR

Figure No: 5

JK **Geotechnics**

PLOT DATE: 28/06/2018 4:34:57 PM DWG FILE: S:\16 GEOTECHNICAL\6F GEOTECHNICAL JOBS\31000's\31518ZR UMINA\CAD\31518ZR.DWG

PLOT DATE: 28/06/2018 4:35:15 PM DWG FILE: S:\6 GEOTECHNICAL\6F GEOTECHNICAL JOBS\31000's\31518ZR UMINA\CAD\31518ZR.DWG

LEGEND

- ● BOREHOLE
- ▼ CONE PENETRATION TEST
- APPROXIMATE CHAINAGE

SCALE 0 5 10 15 20 25 METRES 1:500 @A3

This plan should be read in conjunction with the JK Geotechnics report.

Title:	GEOTECHNICAL SITE PLAN (CH820m - CH980m)
Location:	OCEAN BEACH SURF LIFE SAVING CLUB TO KUORONG STREET BOAT RAMP, UMINA, NSW
Report No:	31518ZR
Figure No:	6

JK Geotechnics

NOTE:
AERIAL IMAGE SOURCE: GOOGLE EARTH PRO, 7.1.5.1557, 2015.
(NOT GEO-REFERENCE TO SURVEY DRAWING)

LEGEND

BOREHOLE

CONE PENETRATION TEST

APPROXIMATE CHAINAGE

0 5 10 15 20 25
SCALE 1:500 @A3 METRES

This plan should be read in conjunction with the JK Geotechnics report.

Title: **GEOTECHNICAL SITE PLAN (CH980m - CH1160m)**

Location: OCEAN BEACH SURF LIFE SAVING CLUB TO KUORONG STREET BOAT RAMP, UMINA, NSW

Report No: 31518ZR

Figure No: 7

JK **Geotechnics**

PLOT DATE: 28/06/2018 4:35:36 PM DWG FILE: S:\6 GEOTECHNICAL\6F GEOTECHNICAL JOBS\31000'S\31518ZR UMINA\CAD\31518ZR.DWG

LEGEND

- ● BOREHOLE
- ▼ CONE PENETRATION TEST
- APPROXIMATE CHAINAGE

NOTE:
AERIAL IMAGE SOURCE: GOOGLE EARTH PRO, 7.1.5.1557, 2015.
(NOT GEO-REFERENCE TO SURVEY DRAWING)

SCALE 0 5 10 15 20 25 METRES 1:500 @A3

This plan should be read in conjunction with the JK Geotechnics report.

Title:	GEOTECHNICAL SITE PLAN (CH1120m - CH1292m)
Location:	OCEAN BEACH SURF LIFE SAVING CLUB TO KUORONG STREET BOAT RAMP, UMINA, NSW
Report No: 31518ZR	Figure No: 8

JK Geotechnics

PLOT DATE: 28/06/2018 4:36:08 PM DWG FILE: S:\6 GEOTECHNICAL\6F GEOTECHNICAL JOBS\31000'S\31518ZR UMINA\CAD\31518ZR.DWG

JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

Borehole No.
101
1 / 1

BOREHOLE LOG

Client: ROYAL HASKONINGDHV AUSTRALIA PTY LTD
Project: PROPOSED BEACH EROSION MANAGEMENT STRATEGY
Location: OCEAN BEACH SLSC TO KOORUNG STREET BOAT RAMP, THE ESPLANADE, UMINA, NSW

Job No.: 31518ZR **Method:** HAND AUGER **R.L. Surface:** ~1.2 m
Date: 4/6/18 **Datum:** AHD
Plant Type: **Logged/Checked By:** A.F./P.R.

Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
									SP	SAND: orange brown, fine to medium grained.	M	(VL)		MARINE
						1.0						(L)		
	ES						0.5							
groundwater level						0.5					W	(MD)		
	ES											(VD)		
							1.0			END OF BOREHOLE AT 1.00 m				HOLE TERMINATED DUE TO BOREHOLE COLLAPSE
						0.0								
							1.5							
						-0.5								
							2.0							
						-1.0								
							2.5							
						-1.5								
							3.0							
						-2.0								

JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

Borehole No.
102
1 / 1

BOREHOLE LOG

Client: ROYAL HASKONINGDHV AUSTRALIA PTY LTD
Project: PROPOSED BEACH EROSION MANAGEMENT STRATEGY
Location: OCEAN BEACH SLSC TO KOORUNG STREET BOAT RAMP, THE ESPLANADE, UMINA, NSW

Job No.: 31518ZR **Method:** HAND AUGER **R.L. Surface:** ~2 m
Date: 4/6/18 **Datum:** AHD
Plant Type: **Logged/Checked By:** A.F./P.R.

Groundwater Record	SAMPLES ES U50 DB DS	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
	ES					SP	SAND: orange brown, fine to medium grained.	M	(VL)		MARINE
			1.5	0.5					(L)		
			1.0	1.0					(MD)		
	ES					SW	SAND: orange brown, fine to coarse grained, trace of fine shell fragments.				
			0.5	1.5							
Groundwater level	ES							W	(D - VD)		
			0.0	2.0			END OF BOREHOLE AT 1.90 m				HOLE TERMINATED DUE TO BOREHOLE COLLAPSE
			-0.5	2.5							
			-1.0	3.0							

BOREHOLE LOG

Borehole No. 103

Client: ROYAL HASKONINGDHV AUSTRALIA PTY LTD
Project: PROPOSED BEACH EROSION MANAGEMENT STRATEGY
Location: OCEAN BEACH SLSC TO KOORUNG STREET BOAT RAMP, THE ESPLANADE, UMINA, NSW

Job No.: 31518ZR **Method:** HAND AUGER **R.L. Surface:** ~1.9 m
Date: 4/6/18 **Datum:** AHD
Plant Type: **Logged/Checked By:** A.F./P.R.

Groundwater Record	SAMPLES ES U50 DB DS	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
				0.0		SP	SAND: orange brown, fine to medium grained.	M	(VL)		MARINE
									(MD)		
				1.3		SW	SAND: orange brown, fine to coarse grained, trace of fine shell fragments.				
								W			
									(D - VD)		
				2.0			END OF BOREHOLE AT 2.00 m				HOLE TERMINATED DUE TO BOREHOLE COLLAPSE

JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

Borehole No.
104
1 / 1

BOREHOLE LOG

Client: ROYAL HASKONINGDHV AUSTRALIA PTY LTD
Project: PROPOSED BEACH EROSION MANAGEMENT STRATEGY
Location: OCEAN BEACH SLSC TO KOORUNG STREET BOAT RAMP, THE ESPLANADE, UMINA, NSW

Job No.: 31518ZR **Method:** HAND AUGER **R.L. Surface:** ~1.9 m
Date: 4/6/18 **Datum:** AHD
Plant Type: **Logged/Checked By:** A.F./P.R.

RL (m AHD)	Depth (m)	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
1.9 to 0.4	0.0 to 1.5	SP	SAND: orange brown, fine to medium grained.	M	(VL)		MARINE
					(L)		
					(MD)		
0.4 to -0.2	1.5 to 2.1	SW	SAND: orange brown, fine to coarse grained, trace of fine shell fragments.	W	(D - VD)		
	2.10		END OF BOREHOLE AT 2.10 m				HOLE TERMINATED DUE TO BOREHOLE COLLAPSE

Column headings: Groundwater Record; SAMPLES (ES, U50, DB, DS); Field Tests; RL (m AHD); Depth (m); Graphic Log; Unified Classification; DESCRIPTION; Moisture Condition/ Weathering; Strength/ Rel Density; Hand Penetrometer Readings (kPa); Remarks

RL scale: 1.5, 1.0, 0.5, 0.0, -0.5, -1.0, -1.5
Depth scale: 0.5, 1.0, 1.5, 2.0, 2.5, 3.0

JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No.
105
1 / 1

Client: ROYAL HASKONINGDHV AUSTRALIA PTY LTD
Project: PROPOSED BEACH EROSION MANAGEMENT STRATEGY
Location: OCEAN BEACH SLSC TO KOORUNG STREET BOAT RAMP, THE ESPLANADE, UMINA, NSW

Job No.: 31518ZR **Method:** HAND AUGER **R.L. Surface:** ~2.5 m
Date: 4/6/18 **Datum:** AHD
Plant Type: **Logged/Checked By:** A.F./P.R.

Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
									SP	SAND: orange brown, fine to medium grained.	M	(VL)		MARINE
						2.0	0.5							
									SW	SAND: orange brown, fine to coarse grained, trace of fine shell fragments.				
						1.5	1.0							
												(L)		
						1.0	1.5							
											W			
						0.5	2.0					(MD)		
						0.0	2.5					(D - VD)		
										END OF BOREHOLE AT 2.60 m				HOLE TERMINATED DUE TO BOREHOLE COLLAPSE
						-0.5	3.0							

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GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

Borehole No.
106
1 / 1

BOREHOLE LOG

Client: ROYAL HASKONINGDHV AUSTRALIA PTY LTD
Project: PROPOSED BEACH EROSION MANAGEMENT STRATEGY
Location: OCEAN BEACH SLSC TO KOORUNG STREET BOAT RAMP, THE ESPLANADE, UMINA, NSW

Job No.: 31518ZR **Method:** HAND AUGER **R.L. Surface:** ~2.1 m
Date: 4/6/18 **Datum:** AHD
Plant Type: **Logged/Checked By:** A.F./P.R.

Groundwater Record	SAMPLES (ES, U50, DB, DS)	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
			2.0	0		SP	SAND: orange brown, fine to medium grained.	M	(VL)		MARINE
	ES		1.5	0.5					(L)		
			1.0	1.0					(MD)		
▶	ES		0.5	1.5		SW	SAND: orange brown, fine to coarse grained, trace of fine shell fragments.	W			
	ES			2.0					(D - VD)		
			0.0	2.1			END OF BOREHOLE AT 2.10 m				HOLE TERMINATED DUE TO BOREHOLE COLLAPSE
			-0.5	2.5							
			-1.0	3.0							

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GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

Borehole No.
107
1 / 1

BOREHOLE LOG

Client: ROYAL HASKONINGDHV AUSTRALIA PTY LTD
Project: PROPOSED BEACH EROSION MANAGEMENT STRATEGY
Location: OCEAN BEACH SLSC TO KOORUNG STREET BOAT RAMP, THE ESPLANADE, UMINA, NSW

Job No.: 31518ZR | **Method:** HAND AUGER | **R.L. Surface:** ~2 m
Date: 4/6/18 | **Datum:** AHD
Plant Type: | **Logged/Checked By:** A.F./P.R.

Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
							0.0		SP	SAND: orange brown, fine to medium grained.	M	(VL)		MARINE
												(L)		
						1.5	0.5							
												(MD)		
						1.0	1.0							
									SW	SAND: orange brown, fine to coarse grained, trace of fine shell fragments.				
						0.5	1.5							
											W			
												(D - VD)		
						0.0	2.0							
										END OF BOREHOLE AT 2.10 m				HOLE TERMINATED DUE TO BOREHOLE COLLAPSE
						-0.5	2.5							
						-1.0	3.0							

JK 9.01.2 LIB.GLB Log JK AUGERHOLE - MASTER 31518ZR UMINA.GPJ <<DrawingFile>> 20/06/2018 12:28 10.0.000 Datgel Lab and In Situ Tool - DGD | Lib: JK 9.01.2 2018-04-02 Prj: JK 9.01.0 2018-03-20

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GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No.
108
1 / 1

Client: ROYAL HASKONINGDHV AUSTRALIA PTY LTD
Project: PROPOSED BEACH EROSION MANAGEMENT STRATEGY
Location: OCEAN BEACH SLSC TO KOORUNG STREET BOAT RAMP, THE ESPLANADE, UMINA, NSW

Job No.: 31518ZR **Method:** HAND AUGER **R.L. Surface:** ~1.3 m
Date: 4/6/18 **Datum:** AHD
Plant Type: **Logged/Checked By:** A.F./P.R.

Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
							0.0		SP	SAND: orange brown, fine to medium grained.	M	(VL)		MARINE
	ES					1.0								
							0.5					(MD)		
						0.5								
							1.0							
Water level							1.2		SW	SAND: orange brown, fine to coarse grained, trace of fine shell fragments.	W			
	ES					0.0								
							1.5							
							1.6			END OF BOREHOLE AT 1.60 m				HOLE TERMINATED DUE TO BOREHOLE COLLAPSE
						-0.5	2.0							
						-1.0	2.5							
						-1.5	3.0							
						-2.0								

JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No.
109
1 / 1

Client: ROYAL HASKONINGDHV AUSTRALIA PTY LTD
Project: PROPOSED BEACH EROSION MANAGEMENT STRATEGY
Location: OCEAN BEACH SLSC TO KOORUNG STREET BOAT RAMP, THE ESPLANADE, UMINA, NSW

Job No.: 31518ZR **Method:** HAND AUGER **R.L. Surface:** ~0.7 m
Date: 4/6/18 **Datum:** AHD
Plant Type: **Logged/Checked By:** A.F./P.R.

Groundwater Record	SAMPLES (ES, U50, DB, DS)	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
						SP	SAND: orange brown, fine to medium grained.	M	(VL)		MARINE
			0.5								
	ES			0.5					(L)		
			0.0						(MD)		
▼								W			
	ES			1.0					(D)		
			-0.5				END OF BOREHOLE AT 1.10 m				HOLE TERMINATED DUE TO BOREHOLE COLLAPSE
				1.5							
			-1.0								
				2.0							
			-1.5								
				2.5							
			-2.0								
				3.0							
			-2.5								

JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

Borehole No.
110
1 / 1

BOREHOLE LOG

Client: ROYAL HASKONINGDHV AUSTRALIA PTY LTD
Project: PROPOSED BEACH EROSION MANAGEMENT STRATEGY
Location: OCEAN BEACH SLSC TO KOORUNG STREET BOAT RAMP, THE ESPLANADE, UMINA, NSW

Job No.: 31518ZR **Method:** HAND AUGER **R.L. Surface:** ~0.8 m
Date: 4/6/18 **Datum:** AHD
Plant Type: **Logged/Checked By:** A.F./P.R.

Groundwater Record	SAMPLES (ES, U50, DB, DS)	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
				0		SP	SAND: orange brown, fine to medium grained.	M	(VL)		MARINE
	ES		0.5								
				0.5					(L)		
			0.0								
				1.0					(MD)		
									(VD)		
Groundwater level						SW	SAND: orange brown, fine to coarse grained, trace of fine shell fragments.	W			
	ES		-0.5								
				1.5			END OF BOREHOLE AT 1.50 m				HOLE TERMINATED DUE TO BOREHOLE COLLAPSE
			-1.0								
				2.0							
			-1.5								
				2.5							
			-2.0								
				3.0							
			-2.5								

JK 9.01.2 LIB.GLB Log JK AUGERHOLE - MASTER 31518ZR UMINA.GPJ <<DrawingFile>> 20/06/2018 12:28 10.0.000 Datgel Lab and In Situ Tool - DGD | Lib: JK 9.01.2 2018-04-02 Prj: JK 9.01.0 2018-03-20

JK Geotechnics

GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

DYNAMIC CONE PENETRATION TEST RESULTS

Client:	ROYAL HASKONINGDHV AUSTRALIA PTY LTD		
Project:	PROPOSED BEACH EROSION MANAGEMENT STRATEGY		
Location:	OCEAN BEACH SLSC TO KOORUNG ST BOAT RAMP, THE ESPLANADE, UMINA		
Job No.	31518ZR	Hammer Weight & Drop: 9kg/510mm	
Date:	4-6-18	Rod Diameter: 16mm	
Tested By:	A.F.	Point Diameter: 20mm	

	Number of Blows per 100mm Penetration						
Test Location	RL≈1.2m	RL≈2.0m	RL≈1.9m	RL≈1.9m	RL≈2.5m	RL≈2.1m	RL≈2.0m
Depth (mm)	**101**	**102**	**103**	**104**	**105**	**106**	**107**
0 - 100	1	1	1	1	1	1	1
100 - 200	2	1	1	1	1	2	1
200 - 300	3	1	1	2	2	2	2
300 - 400	4	1	1	1	1	3	2
400 - 500	3	1	1	2	2	4	2
500 - 600	4	2	1	2	1	4	3
600 - 700	5	3	1	3	1	4	5
700 - 800	8	4	1	3		3	5
800 - 900	13	5	1	4		3	7
900 - 1000	17	5	2	5	1	5	8
1000 - 1100	25	4	2	7	1	5	10
1100 - 1200	28	5	3	5	2	6	8
1200 - 1300	REFUSAL	4	4	6	3	7	8
1300 - 1400		7	5	8	3	7	7
1400 - 1500		6	6	9	2	8	7
1500 - 1600		6	4	10	3	9	8
1600 - 1700		7	5	15	3	9	6
1700 - 1800		10	7	21	3	8	6
1800 - 1900		16	14	28	2	9	13
1900 - 2000		23	19	REFUSAL	4	15	20
2000 - 2100		30	27		4	21	27
2100 - 2200		REFUSAL	REFUSAL		4	21	REFUSAL
2200 - 2300					4	25	
2300 - 2400					8	REFUSAL	
2400 - 2500					19		
2500 - 2600					27		
2600 - 2700					28		
2700 - 2800					REFUSAL		
2800 - 2900							
2900 - 3000							

Remarks:
1. The procedure used for this test is described in AS1289.6.3.2-1997 (R2013)
2. Usually 8 blows per 20mm is taken as refusal
3. Datum of levels is AHD

JK Geotechnics

GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

DYNAMIC CONE PENETRATION TEST RESULTS

Client:	ROYAL HASKONINGDHV AUSTRALIA PTY LTD		
Project:	PROPOSED BEACH EROSION MANAGEMENT STRATEGY		
Location:	OCEAN BEACH SLSC TO KOORUNG ST BOAT RAMP, THE ESPLANADE, UMINA		
Job No.	31518ZR	Hammer Weight & Drop: 9kg/510mm	
Date:	4-6-18	Rod Diameter: 16mm	
Tested By:	A.F.	Point Diameter: 20mm	

Number of Blows per 100mm Penetration							
Test Location	RL≈1.3m	RL≈0.7m	RL≈0.8m				
Depth (mm)	**108**	**109**	**110**				
0 - 100	1	1	1				
100 - 200	1	2	1				
200 - 300	1	2	2				
300 - 400	2	2	2				
400 - 500	2	2	3				
500 - 600	5	3	2				
600 - 700	6	5	2				
700 - 800	7	6	3				
800 - 900	7	8	3				
900 - 1000	6	10	10				
1000 - 1100	6	14	14				
1100 - 1200	4	16	20				
1200 - 1300	4	14	20				
1300 - 1400	6	8	24				
1400 - 1500	6	10	REFUSAL				
1500 - 1600	10	13					
1600 - 1700	5/20mm	20					
1700 - 1800	REFUSAL	30					
1800 - 1900		32					
1900 - 2000		REFUSAL					
2000 - 2100							
2100 - 2200							
2200 - 2300							
2300 - 2400							
2400 - 2500							
2500 - 2600							
2600 - 2700							
2700 - 2800							
2800 - 2900							
2900 - 3000							

Remarks:
1. The procedure used for this test is described in AS1289.6.3.2-1997 (R2013)
2. Usually 8 blows per 20mm is taken as refusal
3. Datum of levels is AHD

JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

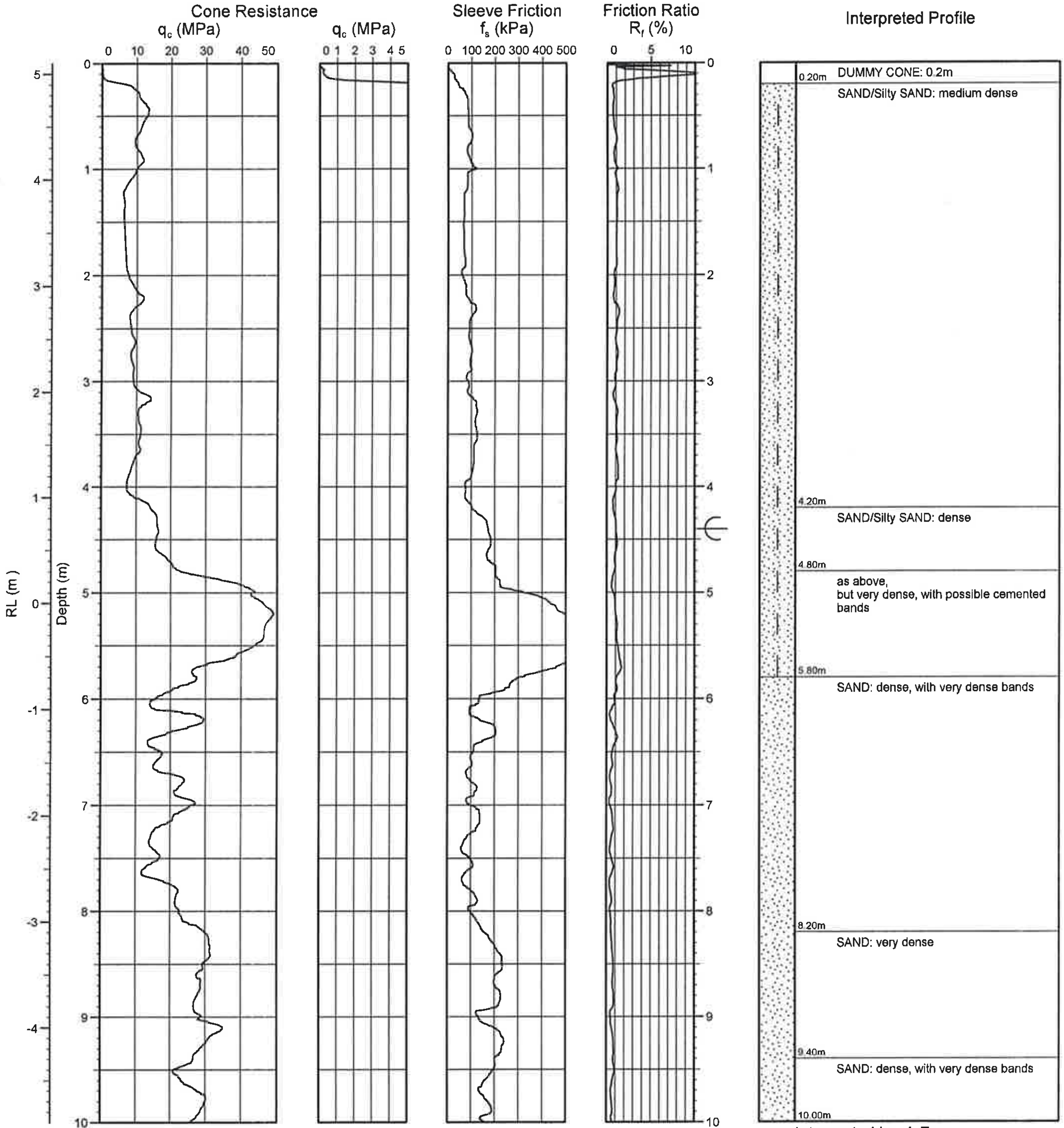
CPT No.
CPT1
1 / 2

CONE PENETROMETER TEST RESULTS

KOURUNG

Client:	ROYAL HASKONINGDHV AUSTRALIA PTY LTD
Project:	PROPOSED BEACH EROSION MANAGEMENT STRATEGY
Location:	OCEAN BEACH SLSC TO KOORUNG STREET BOAT RAMP, THE ESPLANADE, UMINA, NSW

Job No.: 31518ZR	**R.L. Surface:** ~5.1 m	**Data File:** 31518ZR
Date: 5/6/18	**Datum:**	**Operator:** J.B.



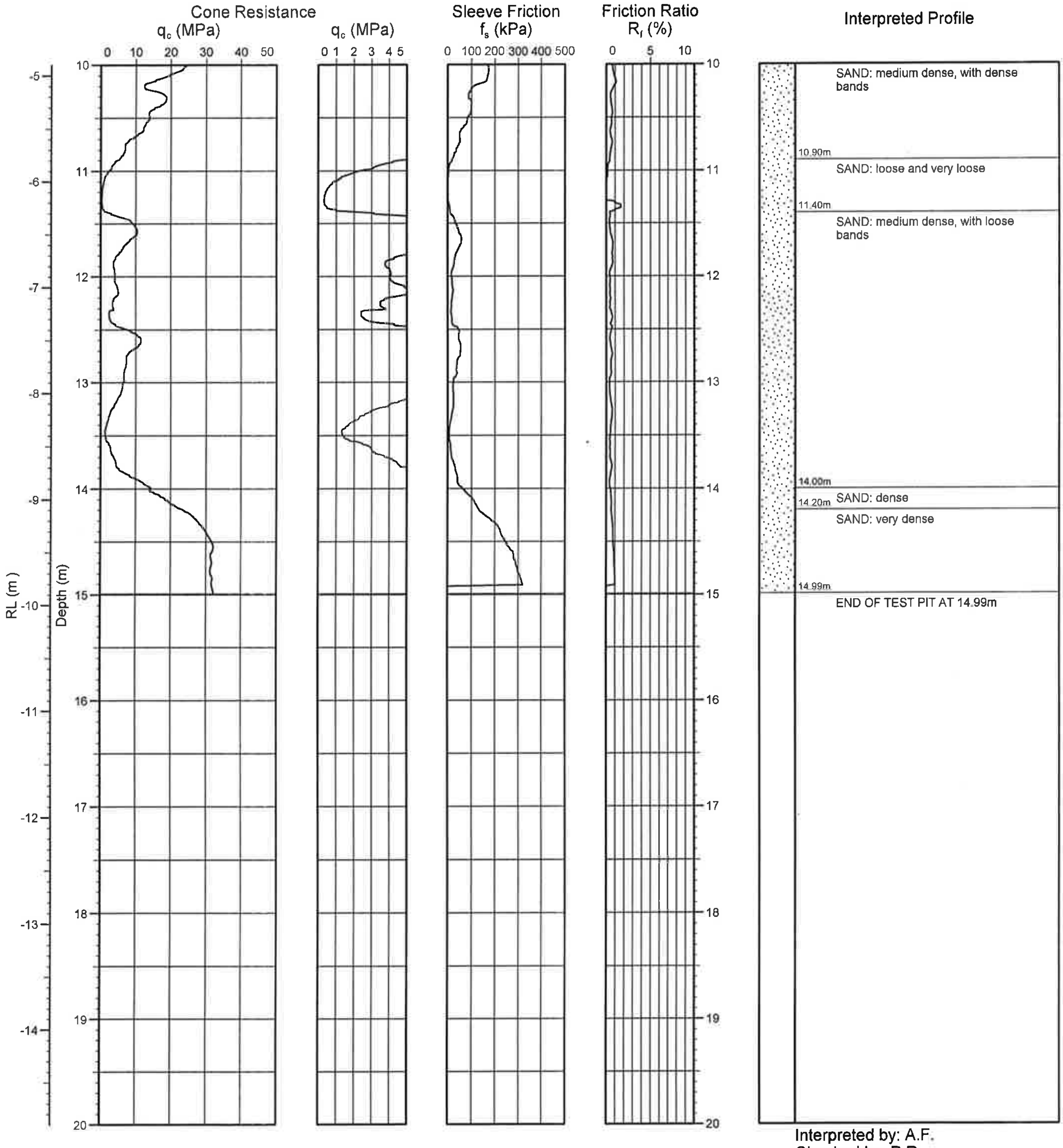
Interpreted by: A.F.
Checked by: P.R.

JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

CPT No.
CPT1
2 / 2

CONE PENETROMETER TEST RESULTS

Client:	ROYAL HASKONINGDHV AUSTRALIA PTY LTD				
Project:	PROPOSED BEACH EROSION MANAGEMENT STRATEGY				
Location:	OCEAN BEACH SLSC TO KOORUNG STREET BOAT RAMP, THE ESPLANADE, UMINA, NSW				
Job No.:	31518ZR	**R.L. Surface:**	~5.1 m	**Data File:**	31518ZR
Date:	5/6/18	**Datum:**		**Operator:**	J.B.



Interpreted by: A.F.
Checked by: P.R.

JK 9.01.2 LIB.GLB Log JK CPT MATERIAL - MASTER 31518ZR UMINA.GPJ <<DrawingFile>> 20/06/2018 16:44 10.0.000 Datgel Lab and In Situ Tool - DGD | Lib: JK 9.01.2 2018-04-02 Prj: JK 9.01.0 2018-03-20

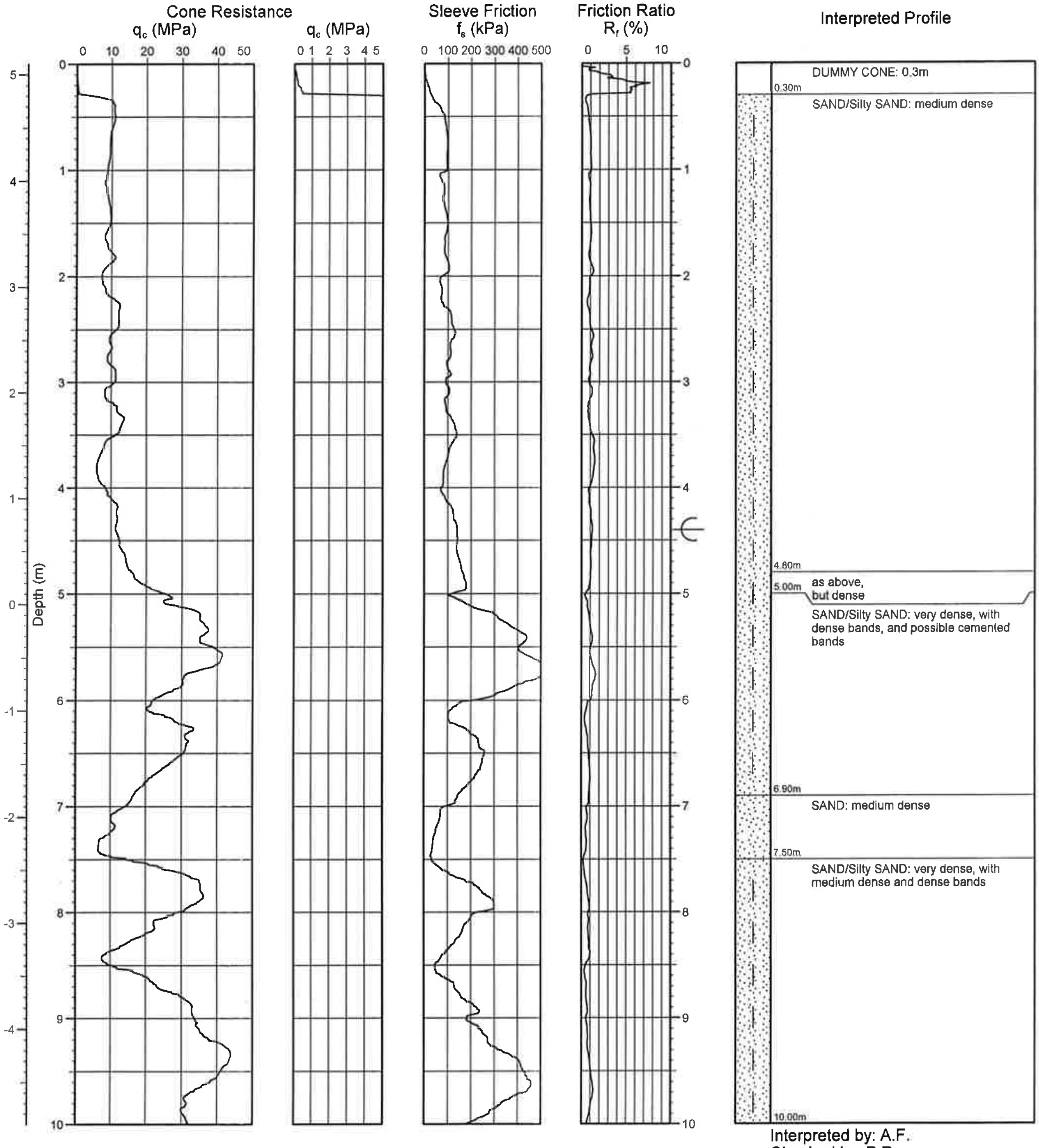
JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

CPT No.
CPT2
1 / 2

CONE PENETROMETER TEST RESULTS

Client: ROYAL HASKONINGDHV AUSTRALIA PTY LTD
Project: PROPOSED BEACH EROSION MANAGEMENT STRATEGY
Location: OCEAN BEACH SLSC TO KOORUNG STREET BOAT RAMP, THE ESPLANADE, UMINA, NSW

Job No.: 31518ZR	**R.L. Surface:** ~5.1 m	**Data File:** 31518ZR Umina
Date: 5/6/18	**Datum:**	**Operator:** J.B.



Interpreted by: A.F.
Checked by: P.R.

JK 9.01.2 LIB.GLB Log JK CPT MATERIAL - MASTER 31518ZR UMINA.GPJ <<DrawingFile>> 20/06/2018 16:44 10.0.000 Datgel Lab and In Situ Tool - DGD | Lib: JK 9.01.2 2018-04-02 Prj: JK 9.01.0 2018-03-20

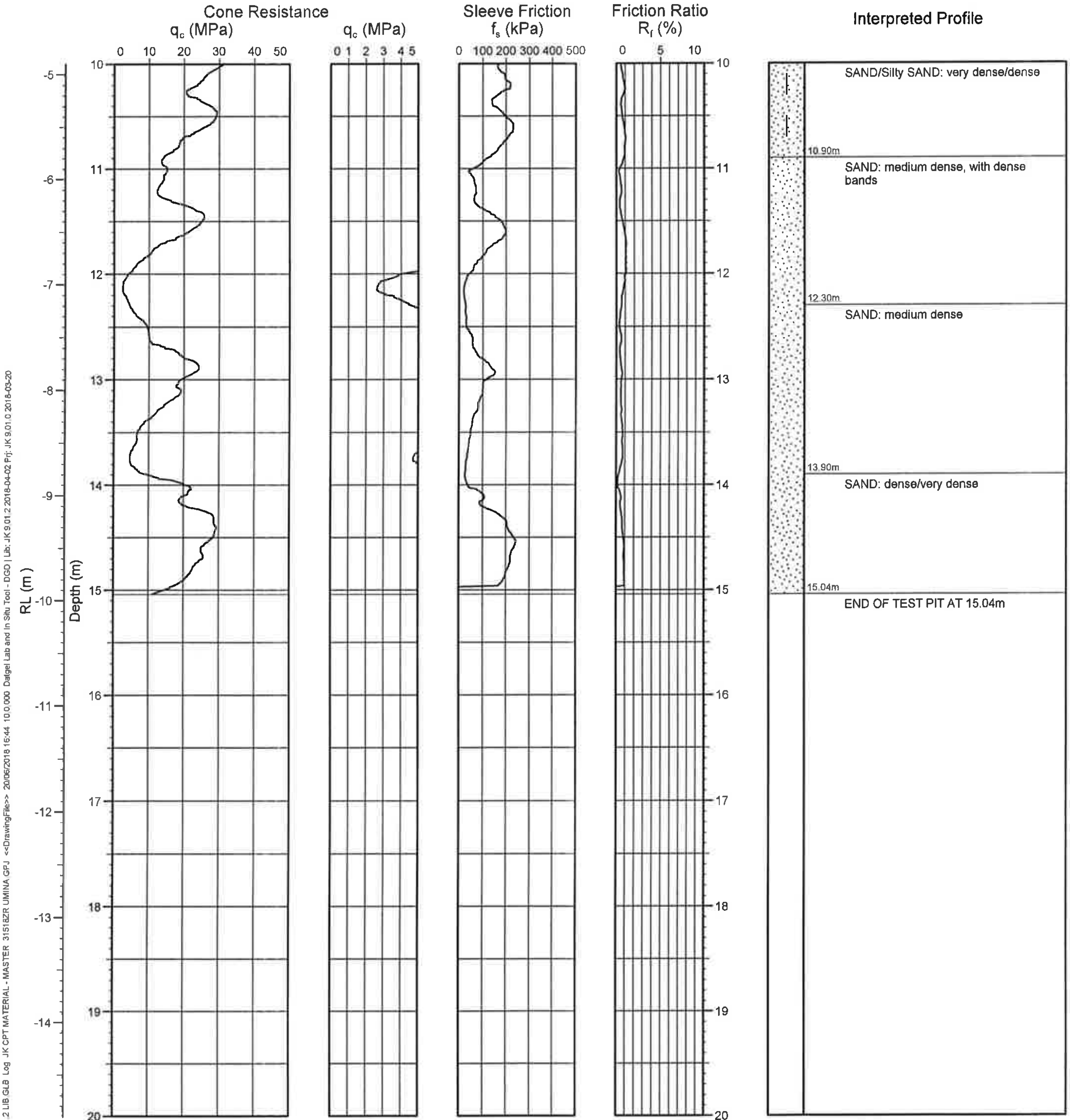
JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

CPT No.
CPT2
2 / 2

CONE PENETROMETER TEST RESULTS

Client: ROYAL HASKONINGDHV AUSTRALIA PTY LTD
Project: PROPOSED BEACH EROSION MANAGEMENT STRATEGY
Location: OCEAN BEACH SLSC TO KOORUNG STREET BOAT RAMP, THE ESPLANADE, UMINA, NSW

Job No.: 31518ZR	**R.L. Surface:** ~5.1 m	**Data File:** 31518ZR Umina
Date: 5/6/18	**Datum:**	**Operator:** J.B.



Interpreted by: A.F.
Checked by: P.R.

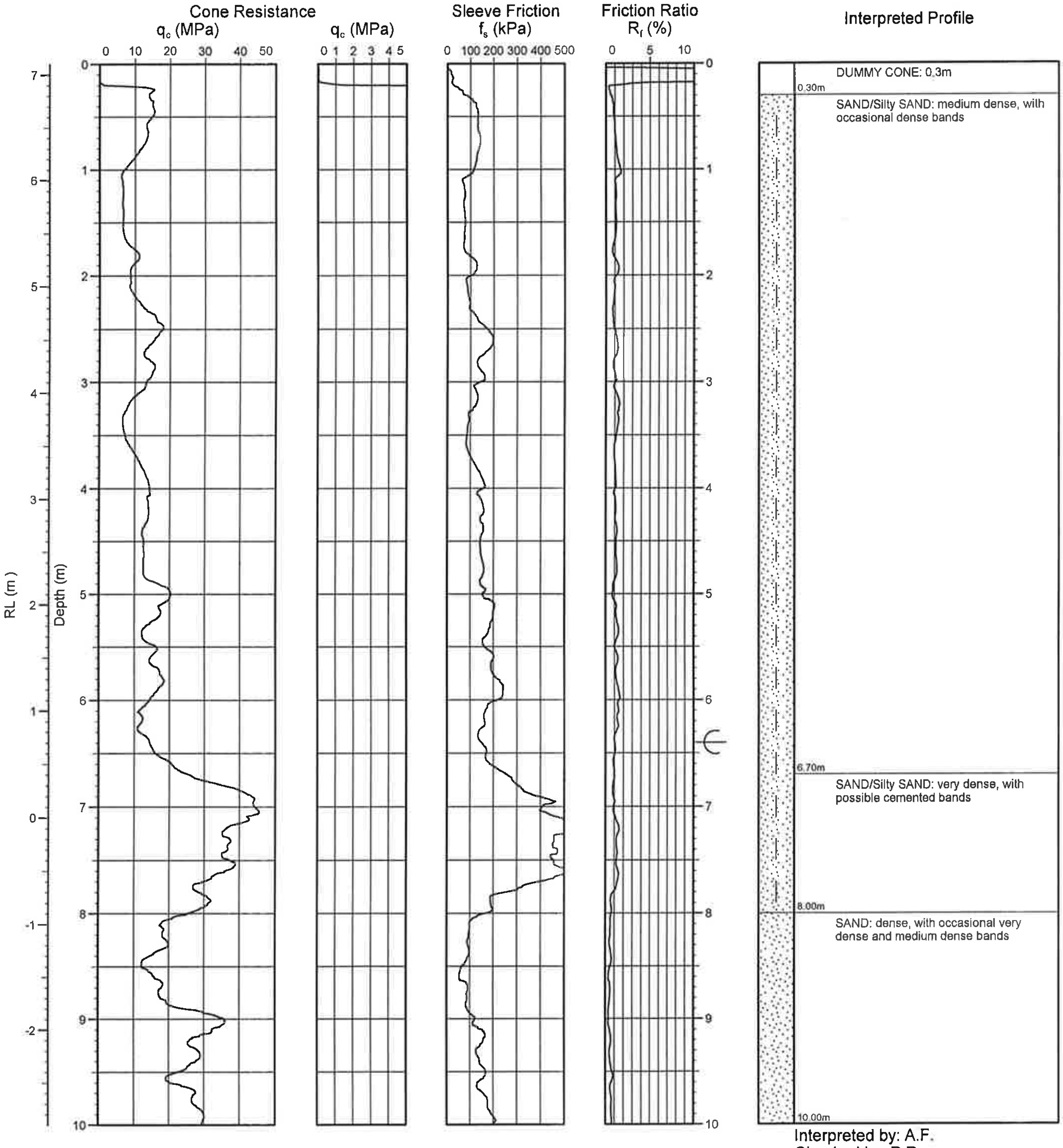
JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

CPT No.
CPT3
1 / 2

CONE PENETROMETER TEST RESULTS

Client:	ROYAL HASKONINGDHV AUSTRALIA PTY LTD
Project:	PROPOSED BEACH EROSION MANAGEMENT STRATEGY
Location:	OCEAN BEACH SLSC TO KOORUNG STREET BOAT RAMP, THE ESPLANADE, UMINA, NSW

Job No.: 31518ZR	**R.L. Surface:** ~7.1 m	**Data File:** 31518ZR Umina
Date: 6/6/18	**Datum:**	**Operator:** J.B.



Interpreted by: A.F.
Checked by: P.R.

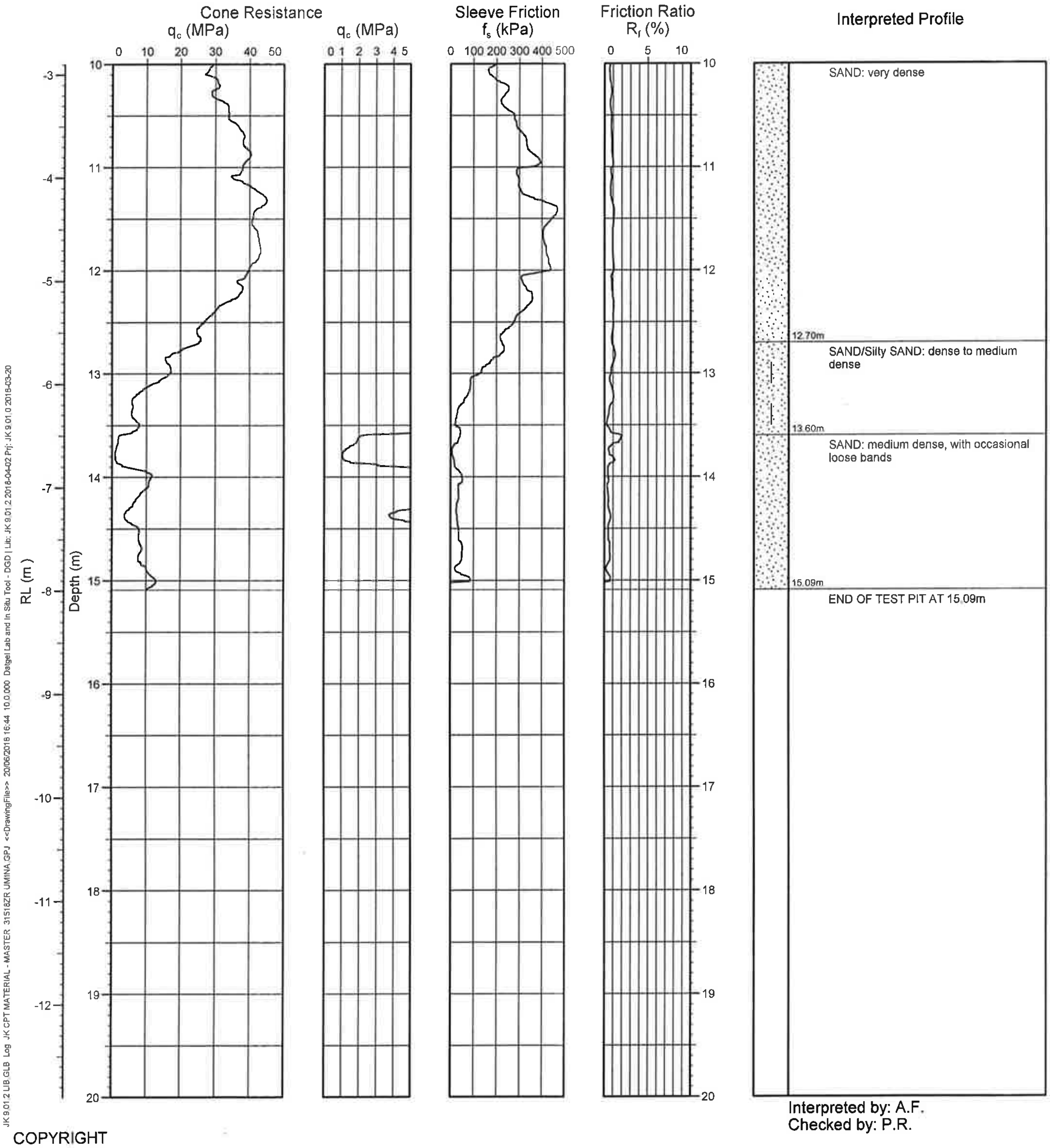
JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

CPT No.
CPT3
2 / 2

CONE PENETROMETER TEST RESULTS

Client:	ROYAL HASKONINGDHV AUSTRALIA PTY LTD
Project:	PROPOSED BEACH EROSION MANAGEMENT STRATEGY
Location:	OCEAN BEACH SLSC TO KOORUNG STREET BOAT RAMP, THE ESPLANADE, UMINA, NSW

Job No.: 31518ZR	**R.L. Surface:** ~7.1 m	**Data File:** 31518ZR Umina
Date: 6/6/18	**Datum:**	**Operator:** J.B.



Interpreted by: A.F.
Checked by: P.R.

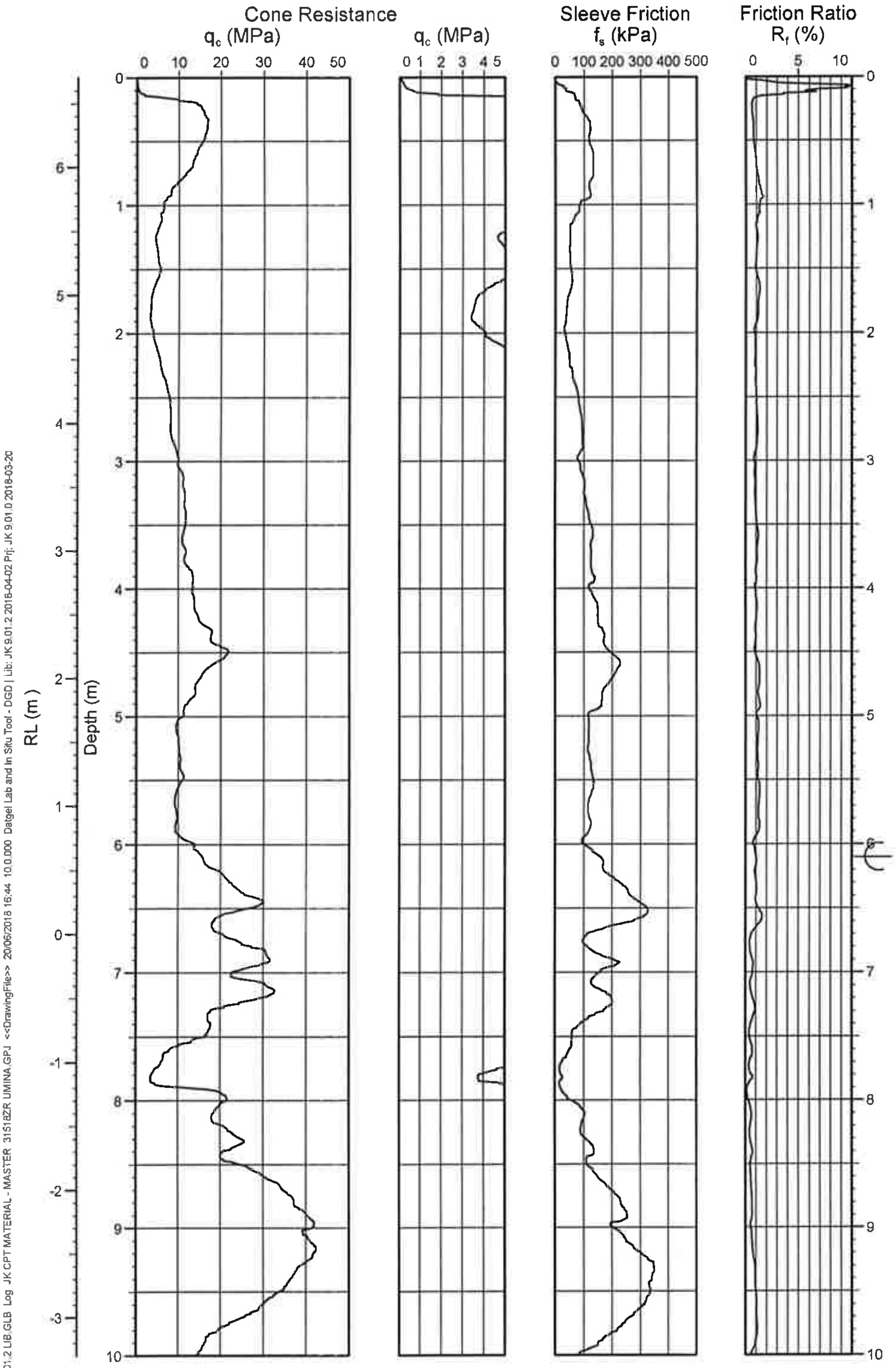
JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

CPT No.
CPT4

CONE PENETROMETER TEST RESULTS

Client: ROYAL HASKONINGDHV AUSTRALIA PTY LTD
Project: PROPOSED BEACH EROSION MANAGEMENT STRATEGY
Location: OCEAN BEACH SLSC TO KOORUNG STREET BOAT RAMP, THE ESPLANADE, UMINA, NSW

Job No.: 31518ZR	**R.L. Surface:** ~6.7 m	**Data File:** 31518ZR Umina
Date: 6/6/18	**Datum:**	**Operator:** J.B.



Interpreted Profile

Depth	Description
0.30m	DUMMY CONE: 0.3m
6.00m	SAND/Silty SAND: medium dense, with occasional loose and dense bands
7.50m	SAND: dense, with very dense bands
7.90m	as above, but loose and medium dense
8.50m	SAND: dense
9.70m	SAND: very dense
	SAND: medium dense and dense

Interpreted by: A.F.
Checked by: P.R.

JK 9.01.2 LIB.GLB Log JK CPT MATERIAL - MASTER 31518ZR UMINA.GPJ <<DrawingFile>> 20/06/2018 16:44 10.0.000 Datgel Lab and In Situ Tool - DGD | Lib: JK 9.01.2 2018-04-02 Prj: JK 9.01.0 2018-03-20

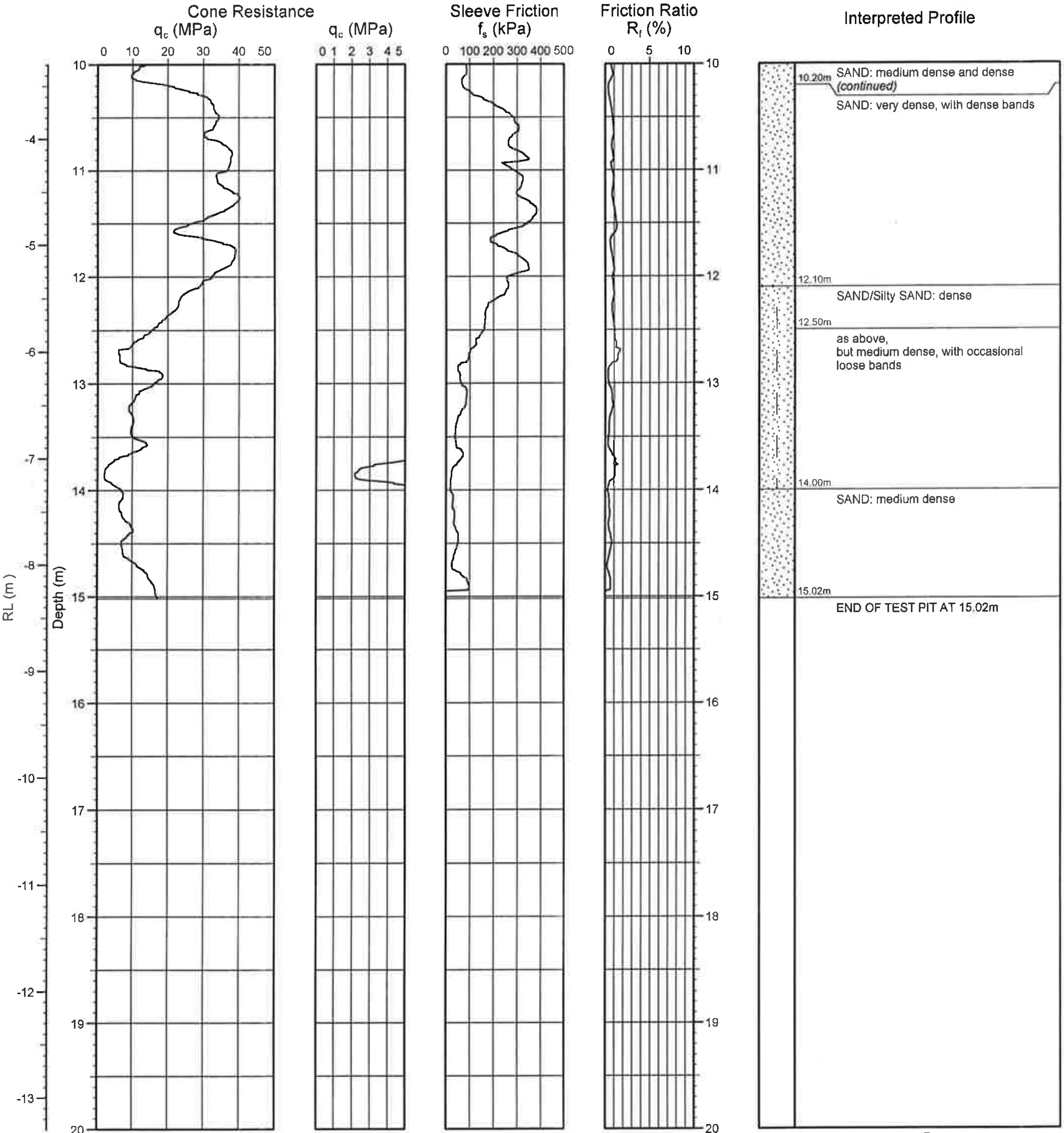
JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

CPT No.
CPT4
2 / 2

CONE PENETROMETER TEST RESULTS

Client:	ROYAL HASKONINGDHV AUSTRALIA PTY LTD
Project:	PROPOSED BEACH EROSION MANAGEMENT STRATEGY
Location:	OCEAN BEACH SLSC TO KOORUNG STREET BOAT RAMP, THE ESPLANADE, UMINA, NSW

Job No.: 31518ZR	**R.L. Surface:** ~6.7 m	**Data File:** 31518ZR Umina
Date: 6/6/18	**Datum:**	**Operator:** J.B.



Interpreted by: A.F.
Checked by: P.R.

JK 9.01.2 LIB.GLB Log JK CPT MATERIAL - MASTER 31518ZR UMINA.GPJ <<DrawingFile>> 20/06/2018 16:44 10.0.000 Datgel Lab and In Situ Tool - DGD | Lib: JK 9.01.2 2018-04-02 Prj: JK 9.01.0 2018-03-20

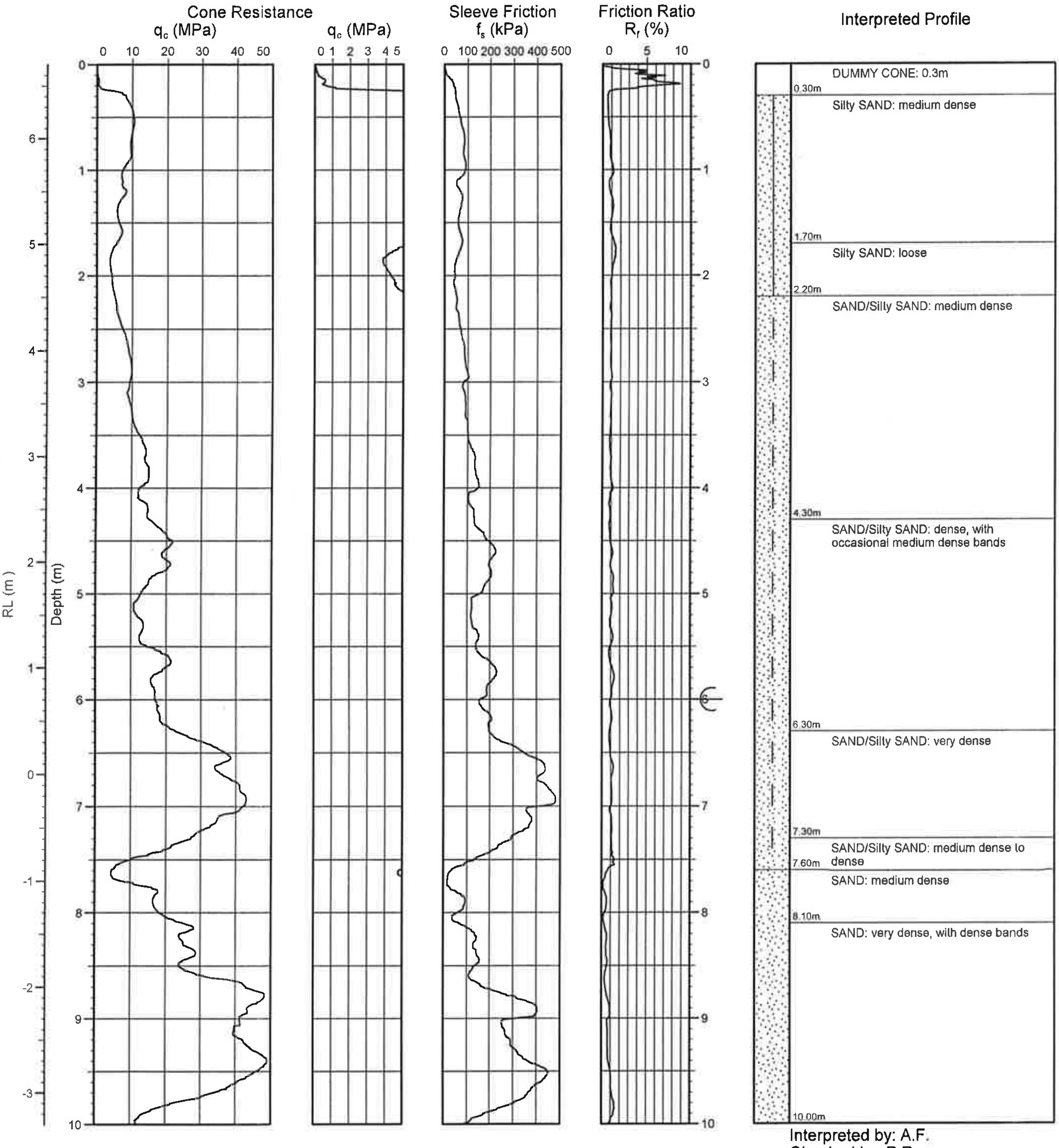
JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

CPT No.
CPT5
1 / 2

CONE PENETROMETER TEST RESULTS

Client:	ROYAL HASKONINGDHV AUSTRALIA PTY LTD
Project:	PROPOSED BEACH EROSION MANAGEMENT STRATEGY
Location:	OCEAN BEACH SLSC TO KOORUNG STREET BOAT RAMP, THE ESPLANADE, UMINA, NSW

Job No.: 31518ZR	**R.L. Surface:** ~6.7 m	**Data File:** 31518ZR Umina
Date: 5/6/18	**Datum:**	**Operator:** J.B.



Interpreted by: A.F.
Checked by: P.R.

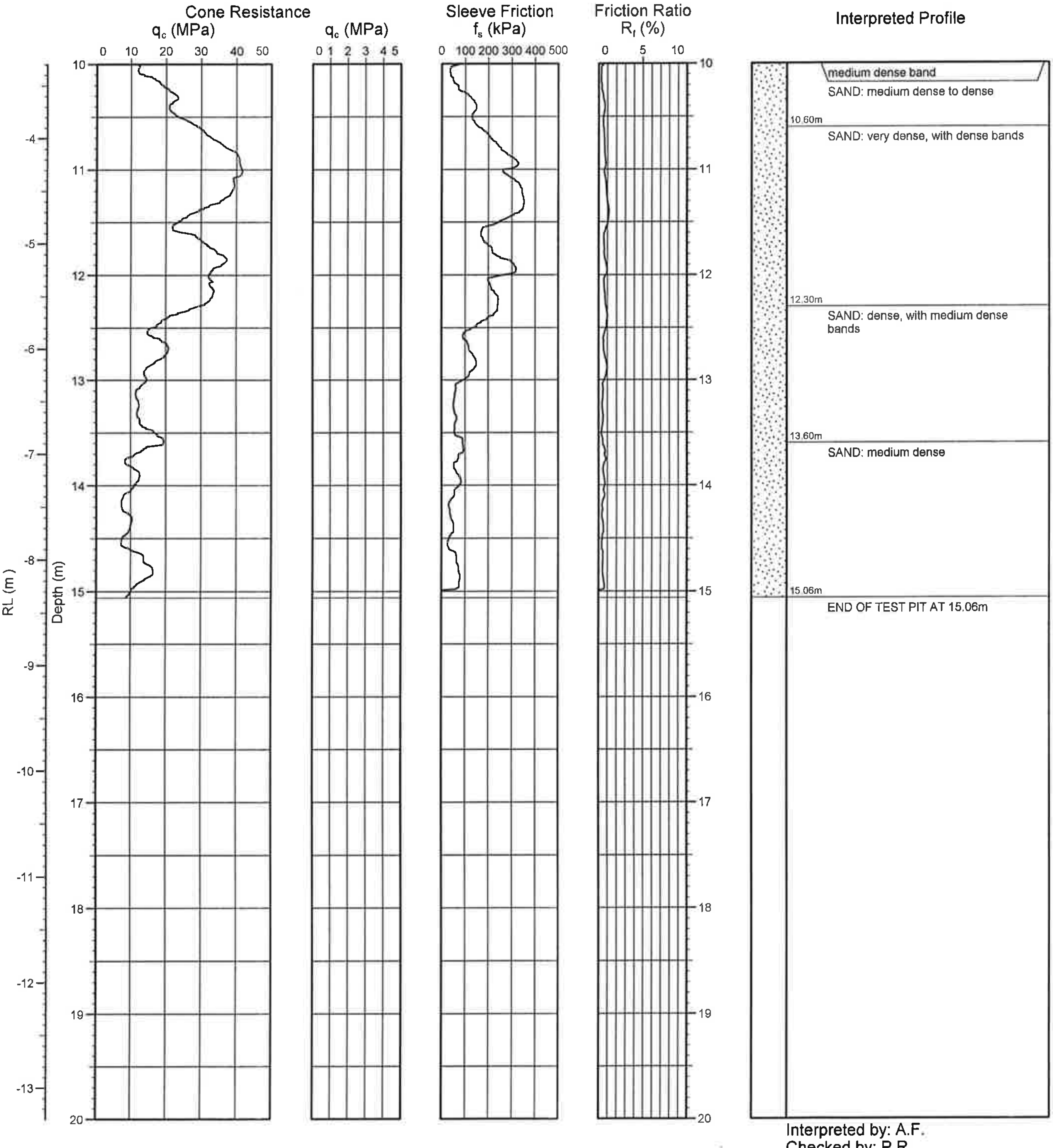
JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

CPT No.
CPT5

CONE PENETROMETER TEST RESULTS

Client: ROYAL HASKONINGDHV AUSTRALIA PTY LTD
Project: PROPOSED BEACH EROSION MANAGEMENT STRATEGY
Location: OCEAN BEACH SLSC TO KOORUNG STREET BOAT RAMP, THE ESPLANADE, UMINA, NSW

Job No.: 31518ZR	**R.L. Surface:** ~6.7 m	**Data File:** 31518ZR Umina
Date: 5/6/18	**Datum:**	**Operator:** J.B.



Interpreted by: A.F.
Checked by: P.R.

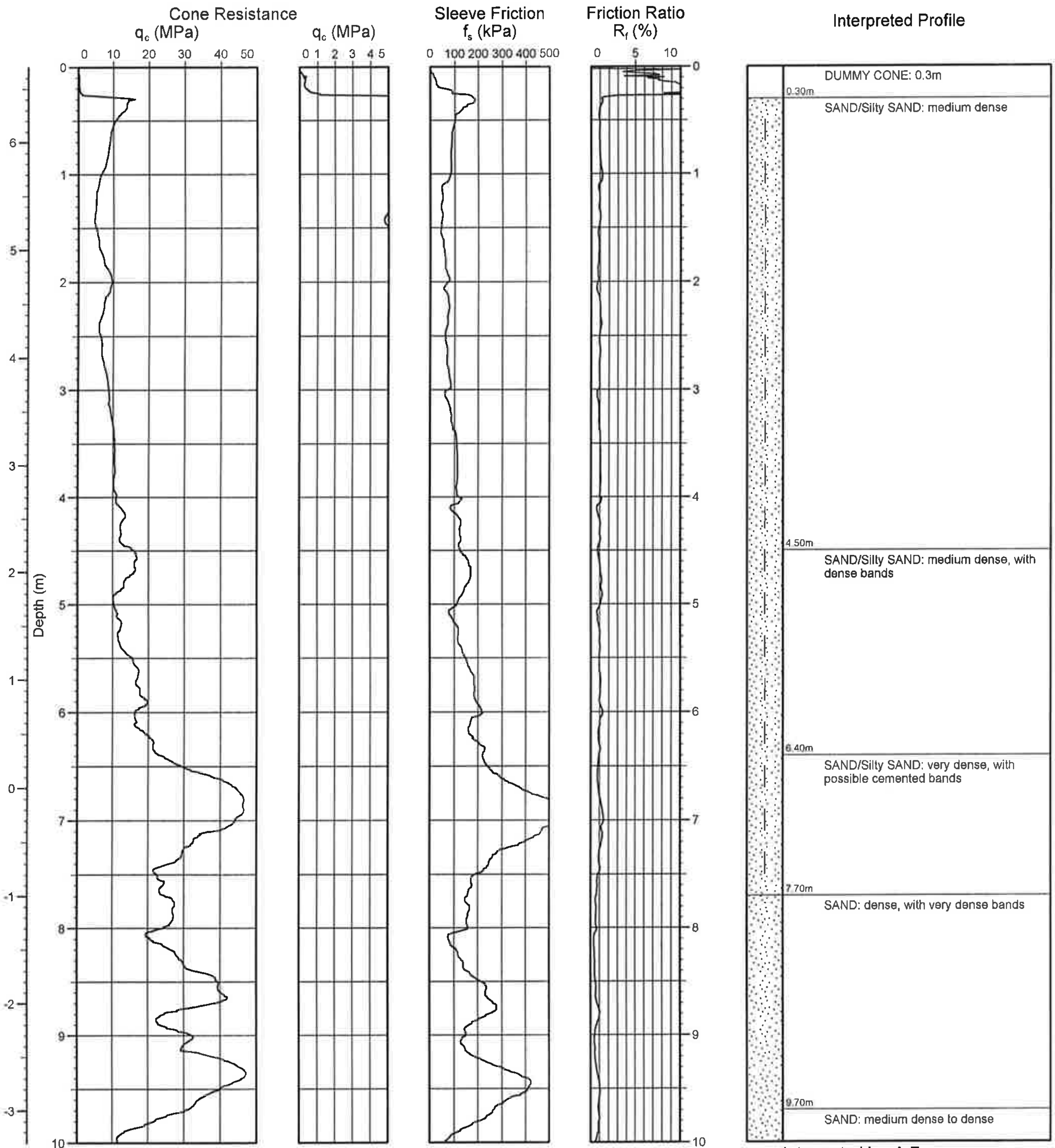
JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

CPT No.
CPT6
1 / 2

CONE PENETROMETER TEST RESULTS

Client:	ROYAL HASKONINGDHV AUSTRALIA PTY LTD
Project:	PROPOSED BEACH EROSION MANAGEMENT STRATEGY
Location:	OCEAN BEACH SLSC TO KOORUNG STREET BOAT RAMP, THE ESPLANADE, UMINA, NSW

Job No.: 31518ZR	**R.L. Surface:** ~6.7 m	**Data File:** 31518ZR Umina
Date: 5/6/18	**Datum:**	**Operator:** J.B.



Interpreted by: A.F.
Checked by: P.R.

JK Geotechnics

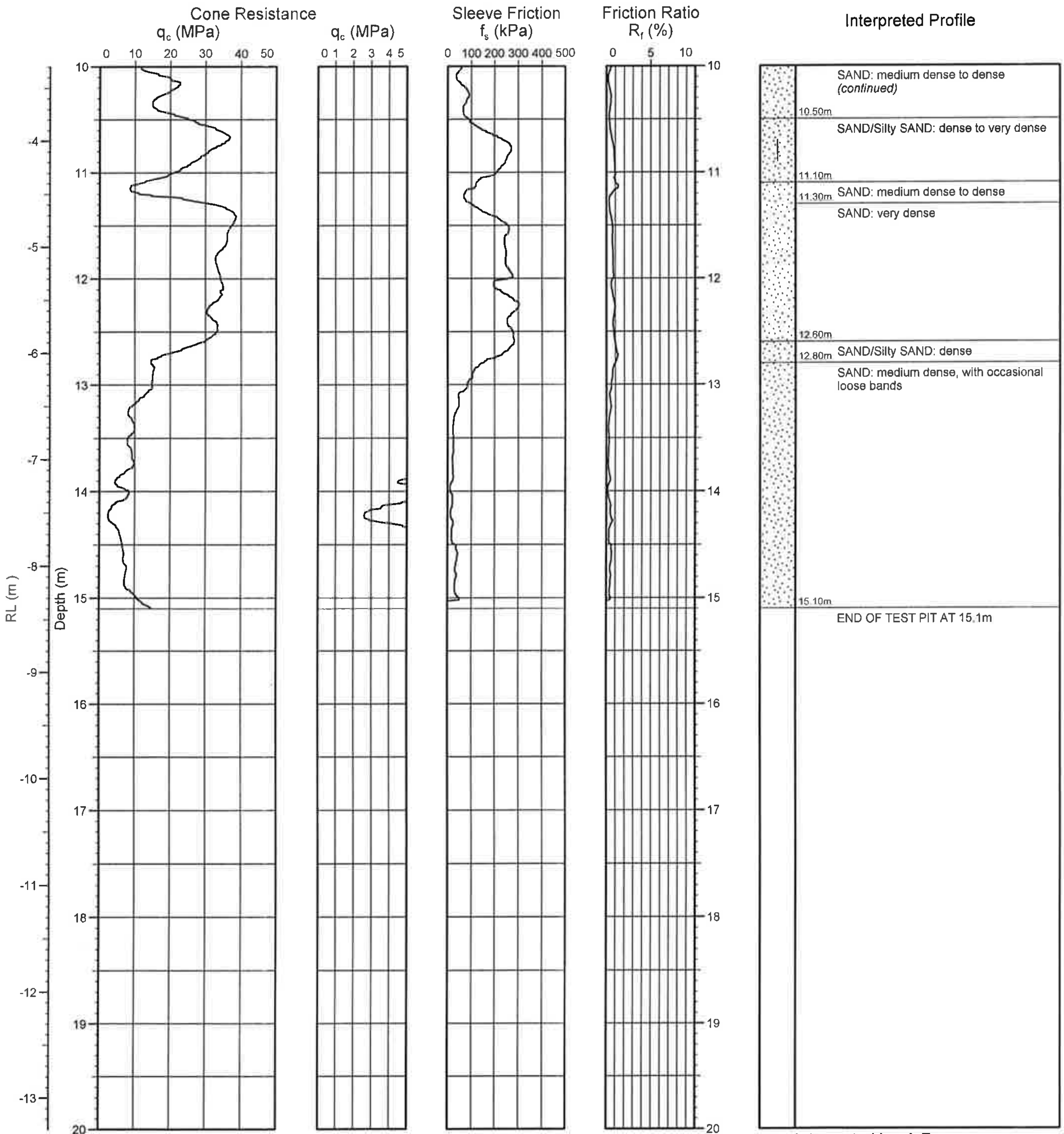
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

CPT No.
CPT6

CONE PENETROMETER TEST RESULTS

Client: ROYAL HASKONINGDHV AUSTRALIA PTY LTD
Project: PROPOSED BEACH EROSION MANAGEMENT STRATEGY
Location: OCEAN BEACH SLSC TO KOORUNG STREET BOAT RAMP, THE ESPLANADE, UMINA, NSW

Job No.: 31518ZR	**R.L. Surface:** ~6.7 m	**Data File:** 31518ZR Umina
Date: 5/6/18	**Datum:**	**Operator:** J.B.



Interpreted by: A.F.
Checked by: P.R.

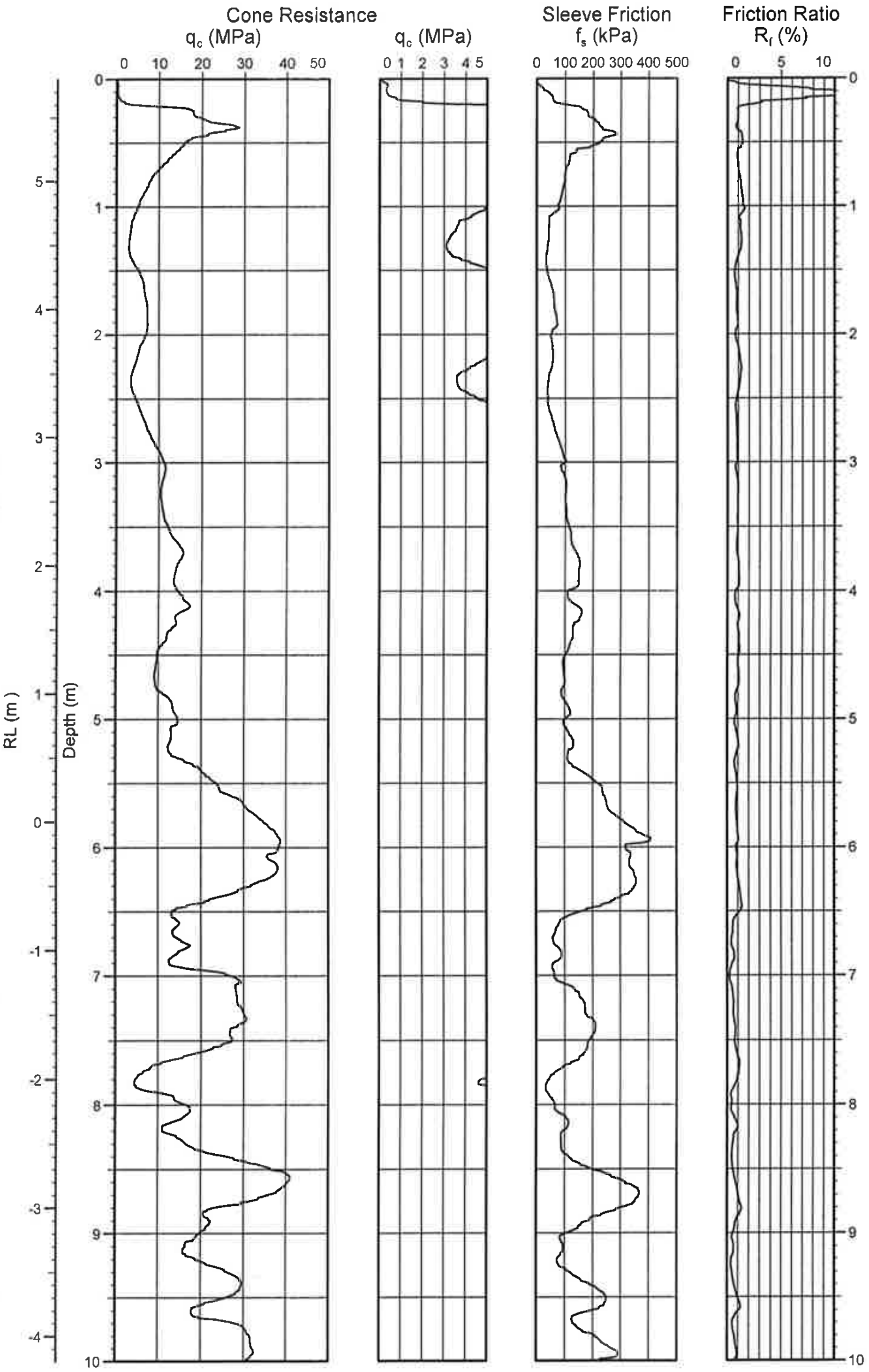
JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

CPT No.
CPT7
1 / 2

CONE PENETROMETER TEST RESULTS

Client: ROYAL HASKONINGDHV AUSTRALIA PTY LTD
Project: PROPOSED BEACH EROSION MANAGEMENT STRATEGY
Location: OCEAN BEACH SLSC TO KOORUNG STREET BOAT RAMP, THE ESPLANADE, UMINA, NSW

Job No.: 31518ZR | **R.L. Surface:** ~5.8 m | **Data File:** 31518ZR Umina
Date: 6/6/18 | **Datum:** | **Operator:** J.B.



Interpreted Profile

Depth	Description
0.30m	DUMMY CONE: 0.3m
5.30m	SAND/Silty SAND: medium dense, with loose bands
6.50m	SAND/Silty SAND: dense to very dense
6.90m	SAND: medium dense to very dense
7.70m	SAND/Silty SAND: dense to very dense
8.40m	SAND/Silty SAND: medium dense to dense
10.00m	SAND/Silty SAND: very dense, with dense bands

Interpreted by: A.F.
Checked by: P.R.

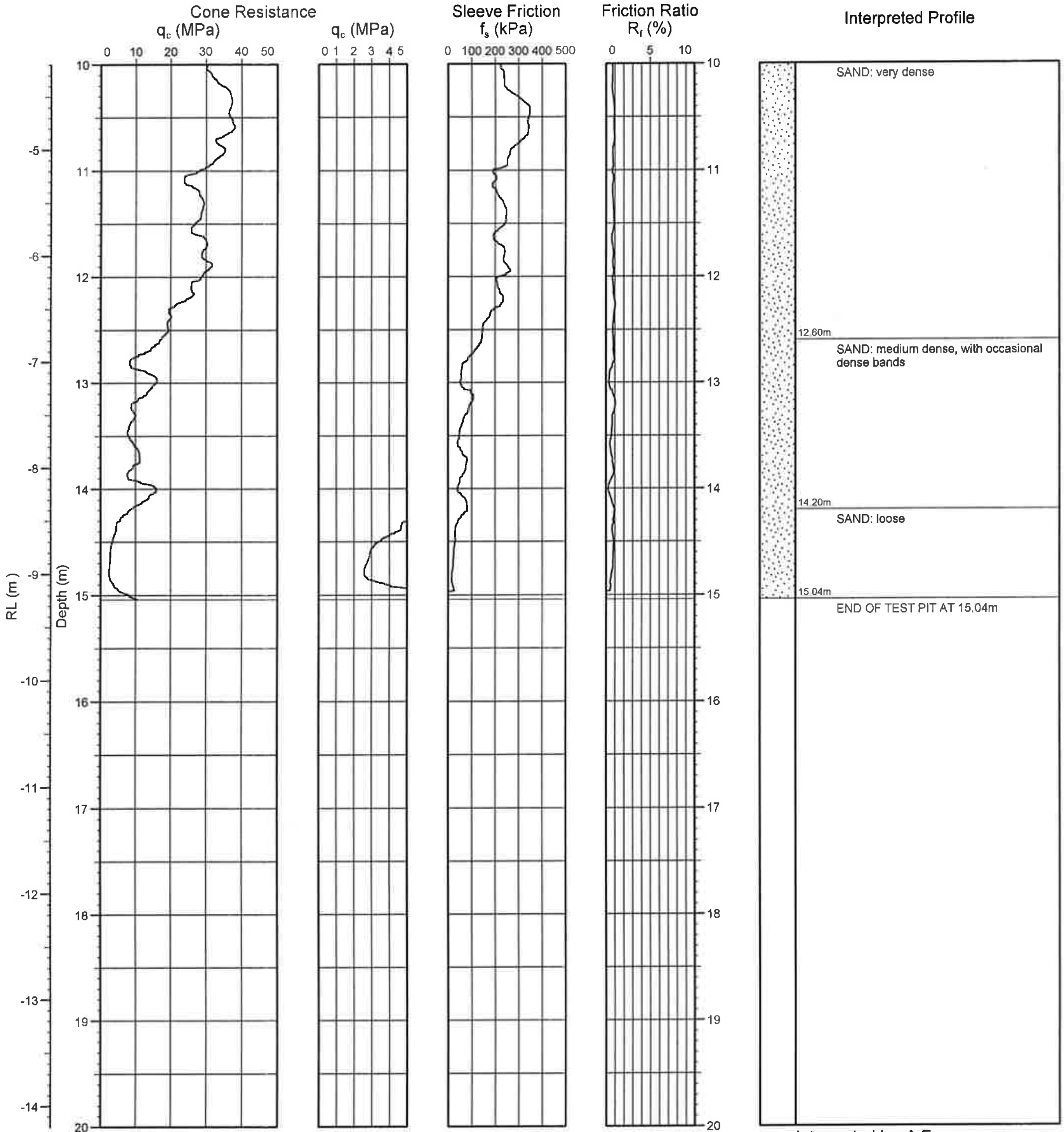
JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

CPT No.
CPT7
2 / 2

CONE PENETROMETER TEST RESULTS

Client:	ROYAL HASKONINGDHV AUSTRALIA PTY LTD
Project:	PROPOSED BEACH EROSION MANAGEMENT STRATEGY
Location:	OCEAN BEACH SLSC TO KOORUNG STREET BOAT RAMP, THE ESPLANADE, UMINA, NSW

Job No.: 31518ZR	**R.L. Surface:** ~5.8 m	**Data File:** 31518ZR Umina
Date: 6/6/18	**Datum:**	**Operator:** J.B.



Interpreted by: A.F.
Checked by: P.R.

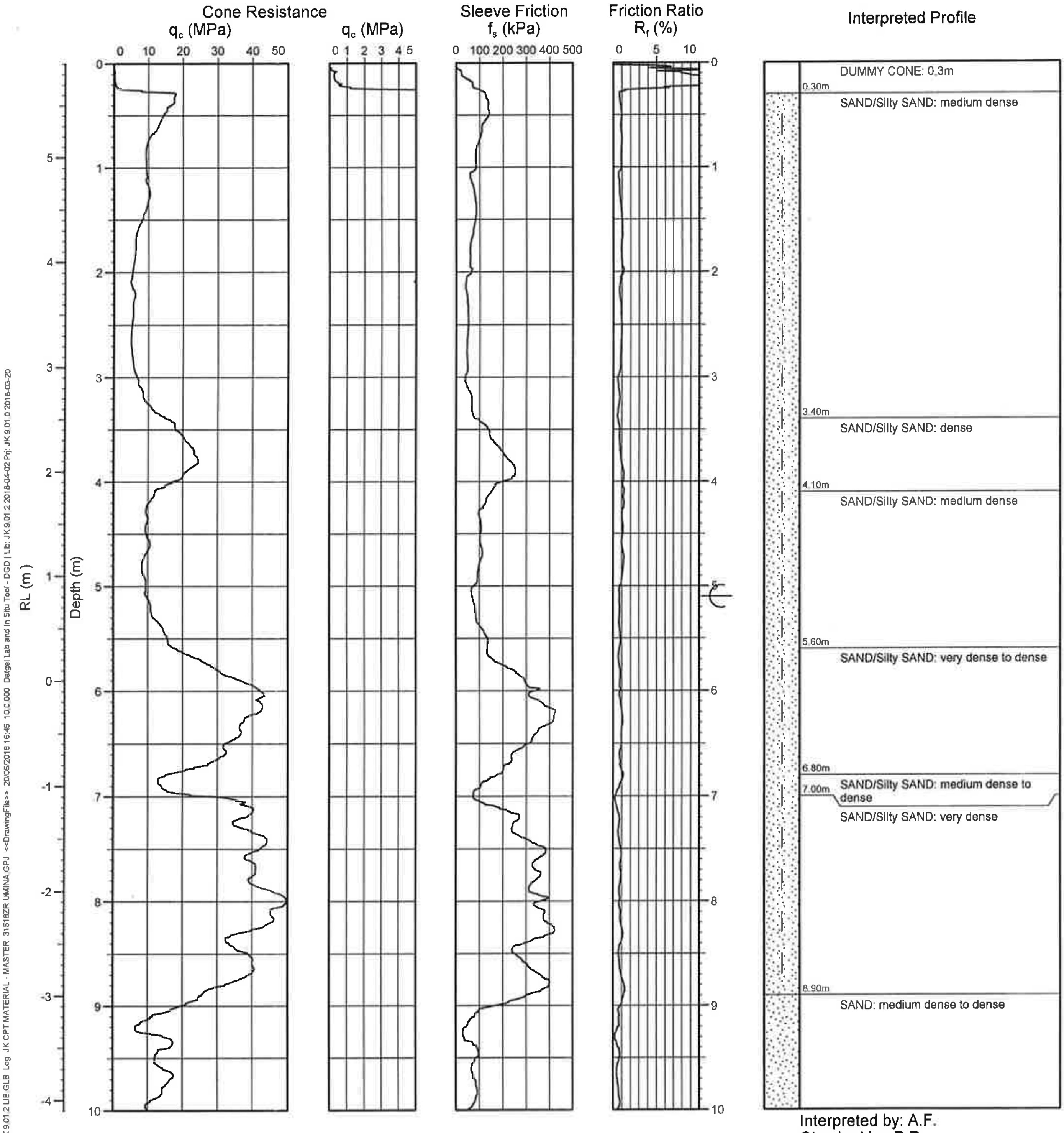
JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

CPT No.
CPT8
1 / 2

CONE PENETROMETER TEST RESULTS

Client:	ROYAL HASKONINGDHV AUSTRALIA PTY LTD
Project:	PROPOSED BEACH EROSION MANAGEMENT STRATEGY
Location:	OCEAN BEACH SLSC TO KOORUNG STREET BOAT RAMP, THE ESPLANADE, UMINA, NSW

Job No.: 31518ZR	**R.L. Surface:** ~5.9 m	**Data File:** 31518ZR Umina
Date: 6/6/18	**Datum:**	**Operator:** J.B.



Interpreted by: A.F.
Checked by: P.R.

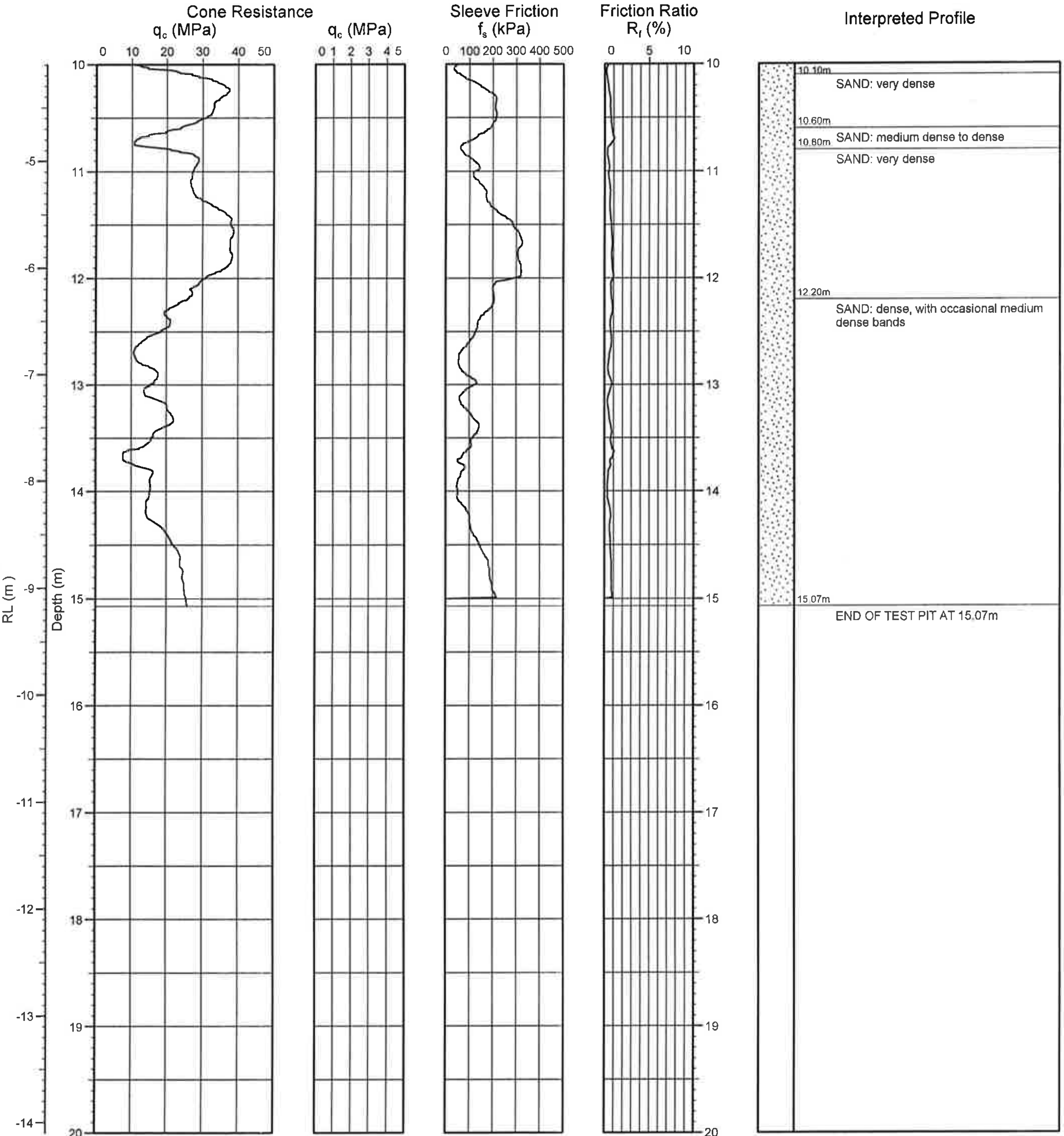
JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

CPT No.
CPT8
2 / 2

CONE PENETROMETER TEST RESULTS

Client: ROYAL HASKONINGDHV AUSTRALIA PTY LTD
Project: PROPOSED BEACH EROSION MANAGEMENT STRATEGY
Location: OCEAN BEACH SLSC TO KOORUNG STREET BOAT RAMP, THE ESPLANADE, UMINA, NSW

Job No.: 31518ZR	**R.L. Surface:** ~5.9 m	**Data File:** 31518ZR Umina
Date: 6/6/18	**Datum:**	**Operator:** J.B.



Interpreted by: A.F.
Checked by: P.R.

JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

CPT No.
CPT9
1 / 2

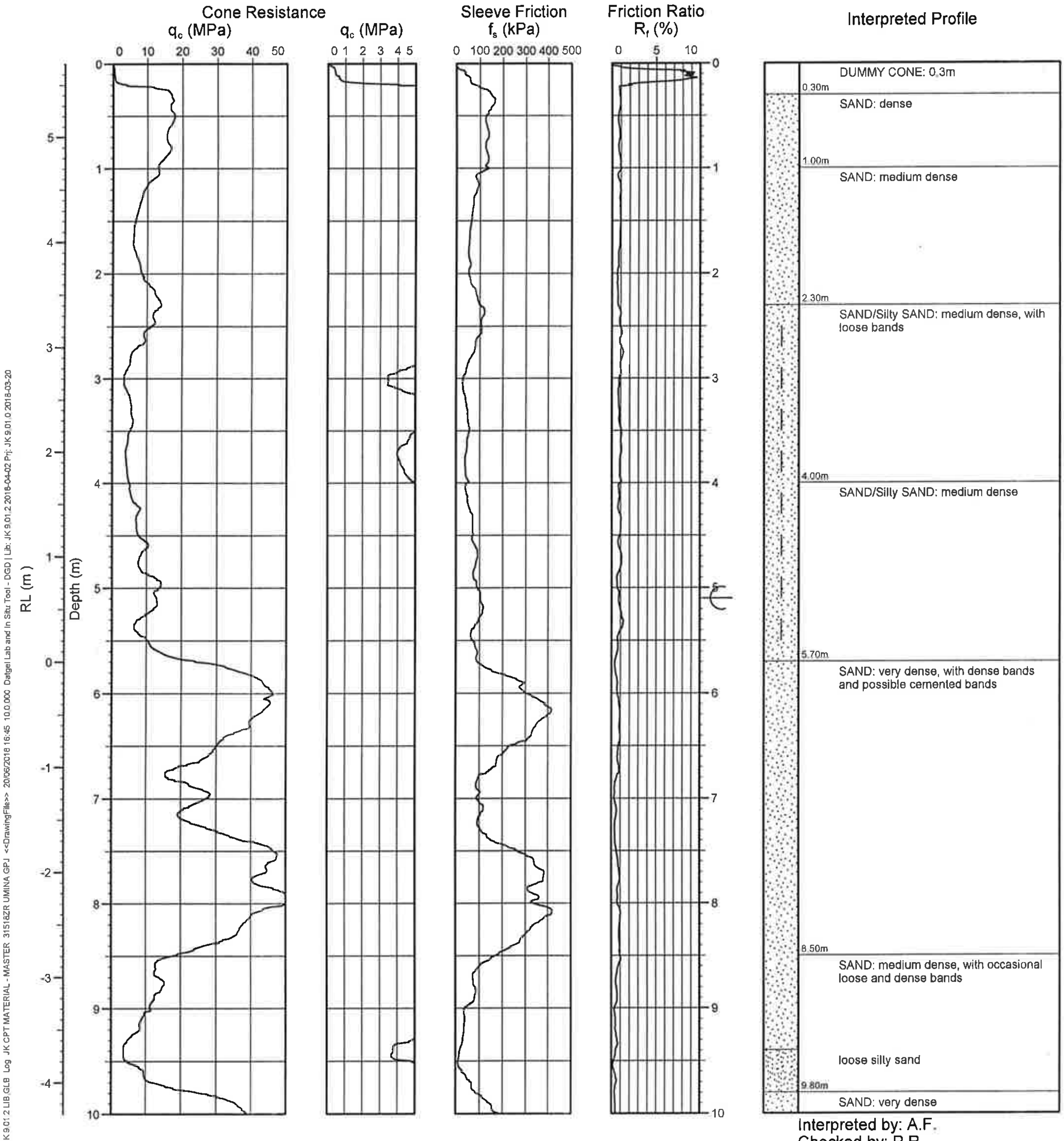
CONE PENETROMETER TEST RESULTS

Client: ROYAL HASKONINGDHV AUSTRALIA PTY LTD

Project: PROPOSED BEACH EROSION MANAGEMENT STRATEGY

Location: OCEAN BEACH SLSC TO KOORUNG STREET BOAT RAMP, THE ESPLANADE, UMINA, NSW

Job No.: 31518ZR	**R.L. Surface:** ~5.7 m	**Data File:** 31518ZR Umina
Date: 6/6/18	**Datum:**	**Operator:** J.B.



Interpreted by: A.F.
Checked by: P.R.

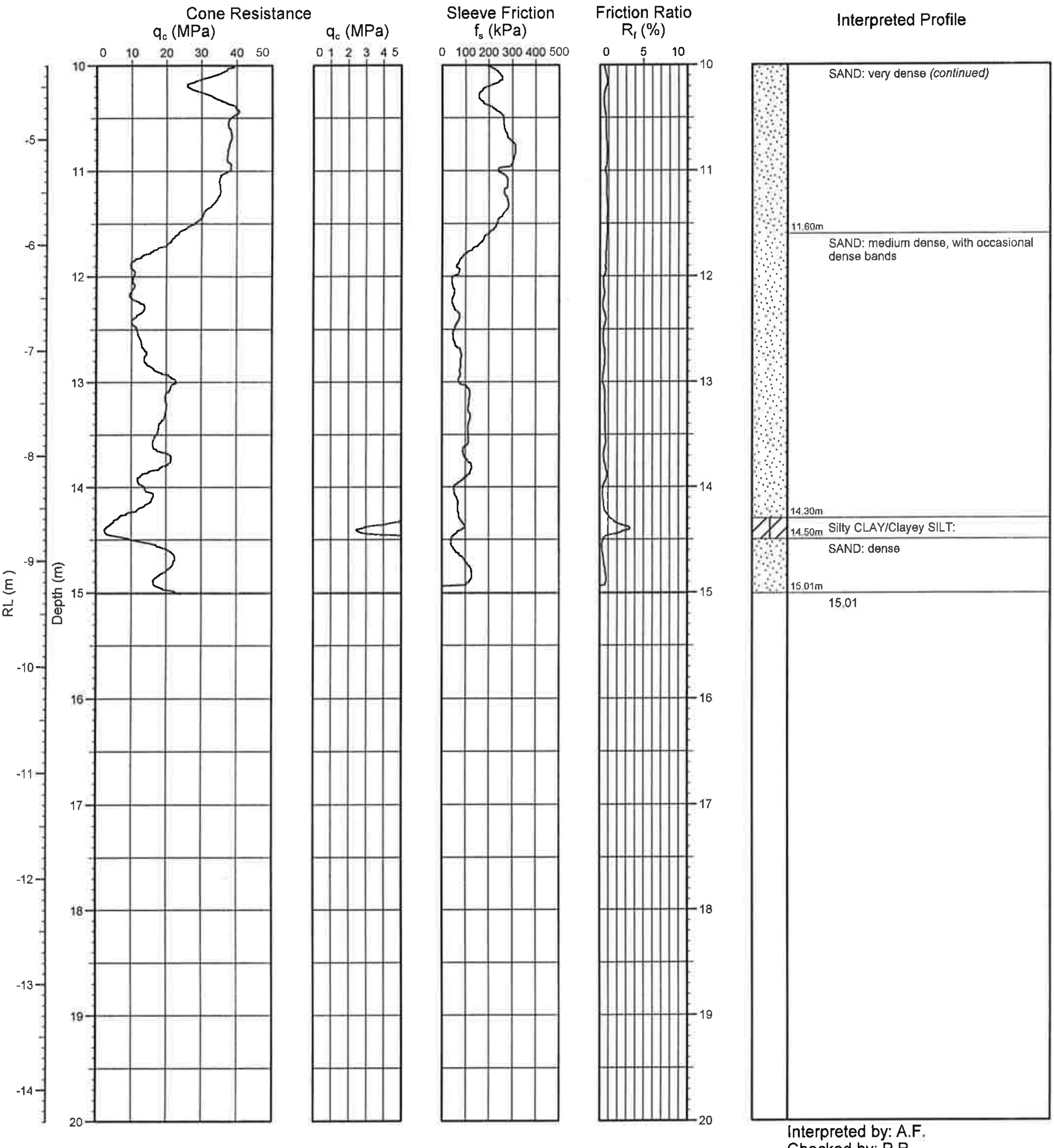
JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

CPT No.
CPT9
2 / 2

CONE PENETROMETER TEST RESULTS

Client:	ROYAL HASKONINGDHV AUSTRALIA PTY LTD
Project:	PROPOSED BEACH EROSION MANAGEMENT STRATEGY
Location:	OCEAN BEACH SLSC TO KOORUNG STREET BOAT RAMP, THE ESPLANADE, UMINA, NSW

Job No.: 31518ZR	**R.L. Surface:** ~5.7 m	**Data File:** 31518ZR Umina
Date: 6/6/18	**Datum:**	**Operator:** J.B.



JK 9.01.2 LIB.GLB Log JK CPT MATERIAL - MASTER 31518ZR UMINA.GPJ <<DrawingFile>> 20/06/2018 16:45 10.0.000 Datgel Lab and In Situ Tool - DGD | Lib: JK 9.01.2 2018-04-02 Prj: JK 9.01.0 2018-03-20

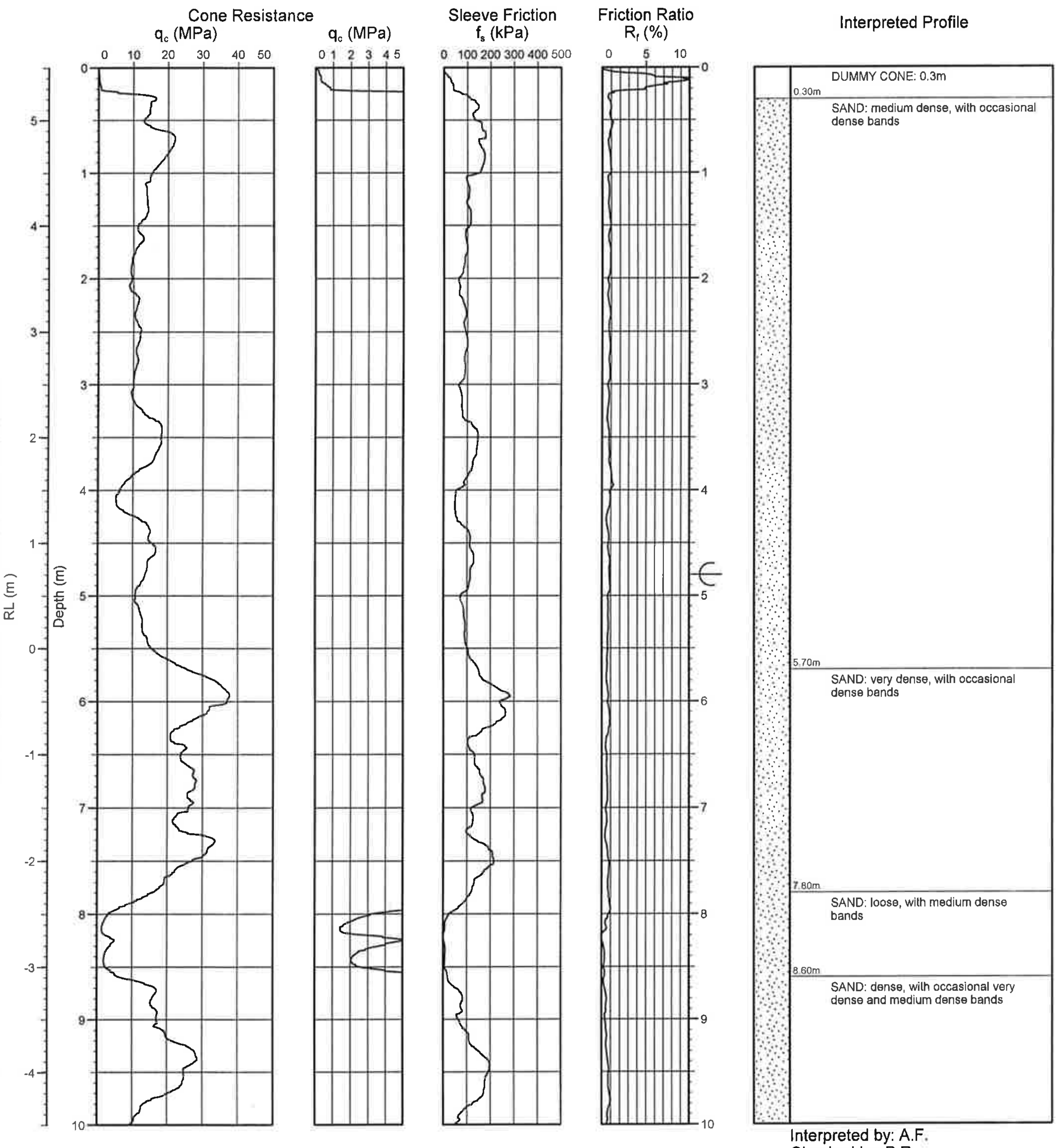
Interpreted by: A.F.
Checked by: P.R.

JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

CPT No.
CPT10
1 / 2

CONE PENETROMETER TEST RESULTS

Client:	ROYAL HASKONINGDHV AUSTRALIA PTY LTD				
Project:	PROPOSED BEACH EROSION MANAGEMENT STRATEGY				
Location:	OCEAN BEACH SLSC TO KOORUNG STREET BOAT RAMP, THE ESPLANADE, UMINA, NSW				
Job No.:	31518ZR	**R.L. Surface:**	~5.5 m	**Data File:**	31518ZR
Date:	6/6/18	**Datum:**		**Operator:**	J.B.



Interpreted by: A.F.
Checked by: P.R.

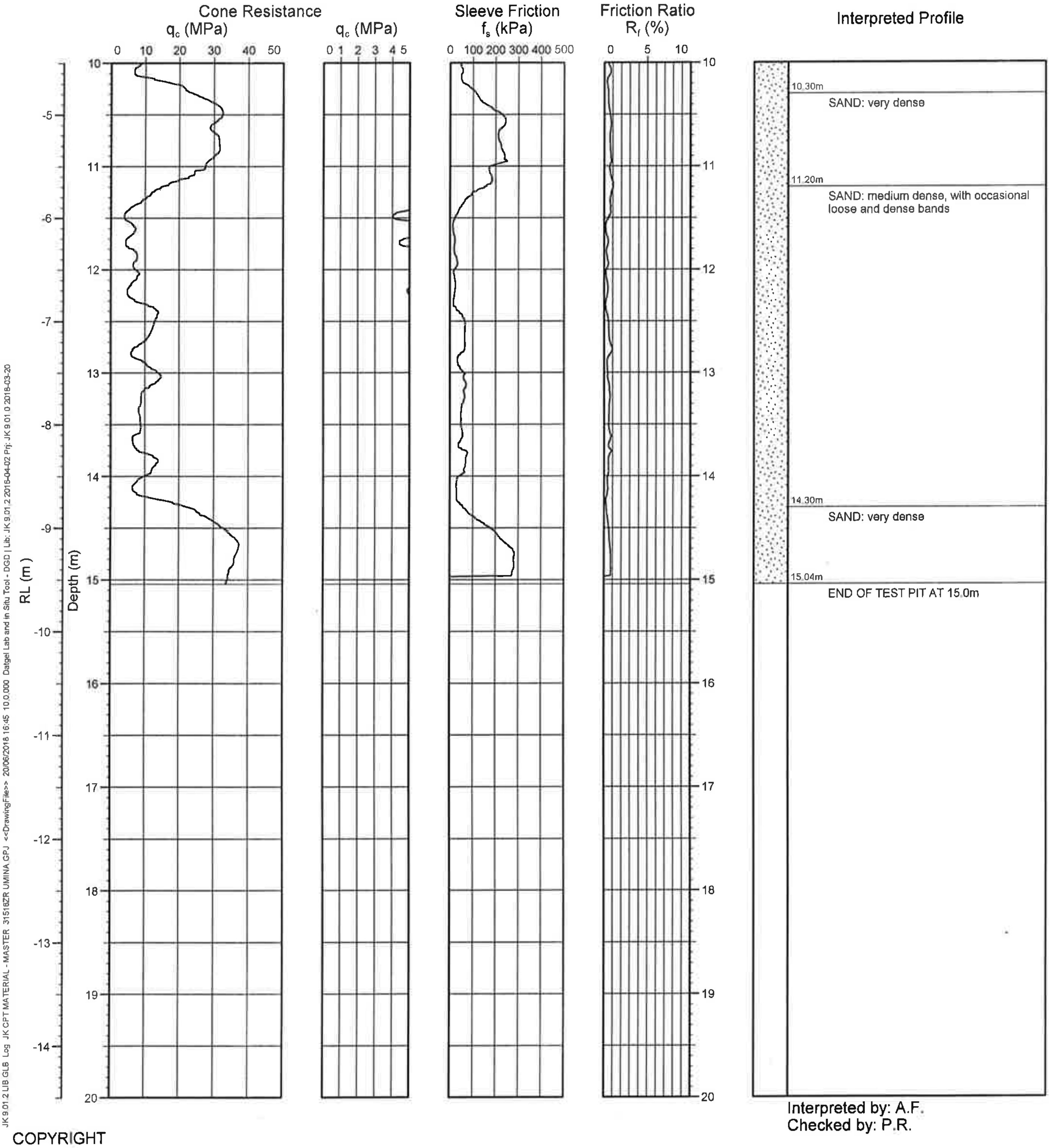
JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

CPT No.
CPT10
2 / 2

CONE PENETROMETER TEST RESULTS

Client:	ROYAL HASKONINGDHV AUSTRALIA PTY LTD
Project:	PROPOSED BEACH EROSION MANAGEMENT STRATEGY
Location:	OCEAN BEACH SLSC TO KOORUNG STREET BOAT RAMP, THE ESPLANADE, UMINA, NSW

Job No.: 31518ZR	**R.L. Surface:** ~5.5 m	**Data File:** 31518ZR
Date: 6/6/18	**Datum:**	**Operator:** J.B.



Interpreted by: A.F.
Checked by: P.R.

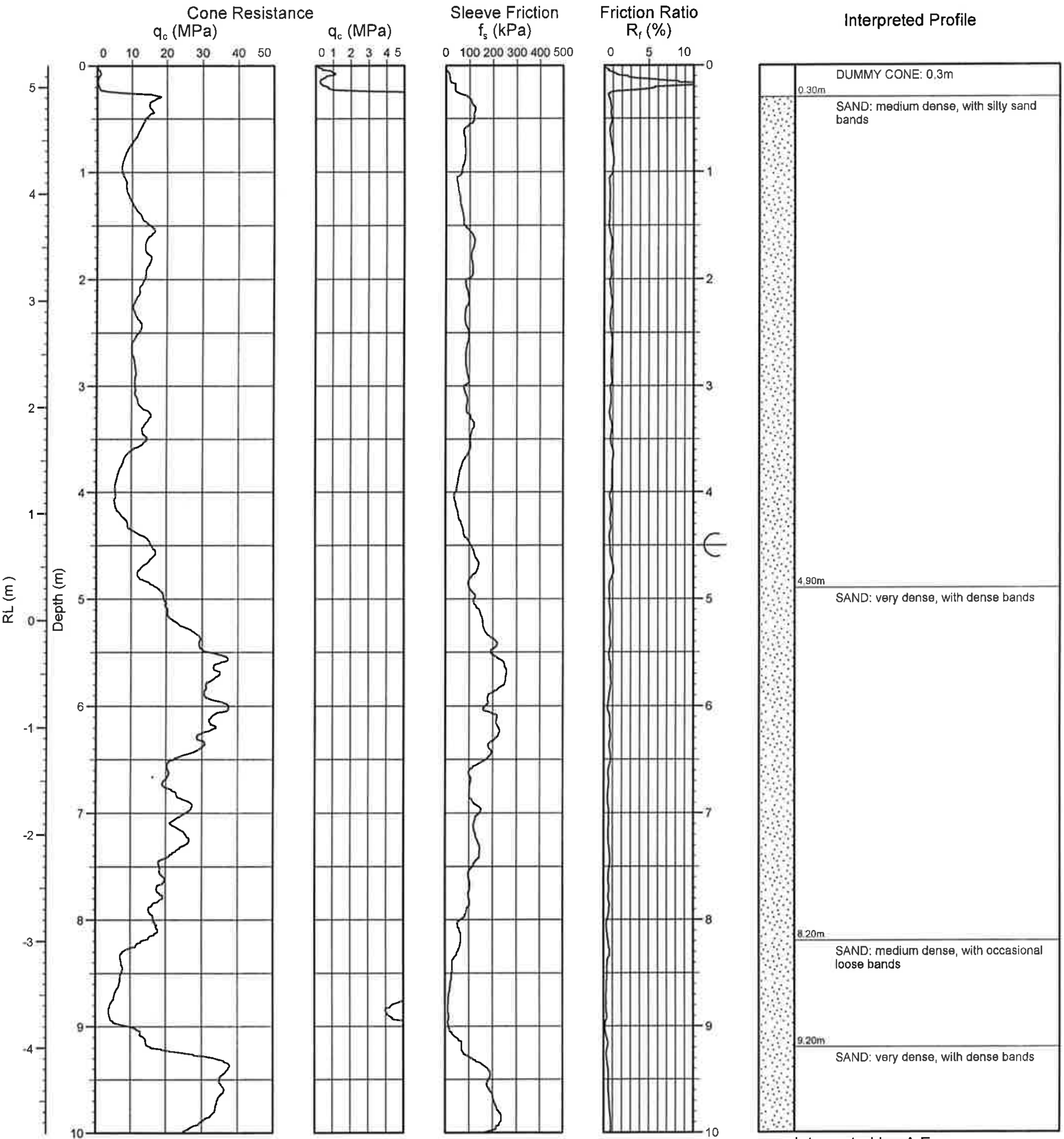
JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

CPT No.
CPT11
1 / 2

CONE PENETROMETER TEST RESULTS

Client:	ROYAL HASKONINGDHV AUSTRALIA PTY LTD
Project:	PROPOSED BEACH EROSION MANAGEMENT STRATEGY
Location:	OCEAN BEACH SLSC TO KOORUNG STREET BOAT RAMP, THE ESPLANADE, UMINA, NSW

Job No.: 31518ZR	**R.L. Surface:** ~5.2 m	**Data File:** 31518ZR Umina
Date: 7/6/18	**Datum:**	**Operator:** J.B.



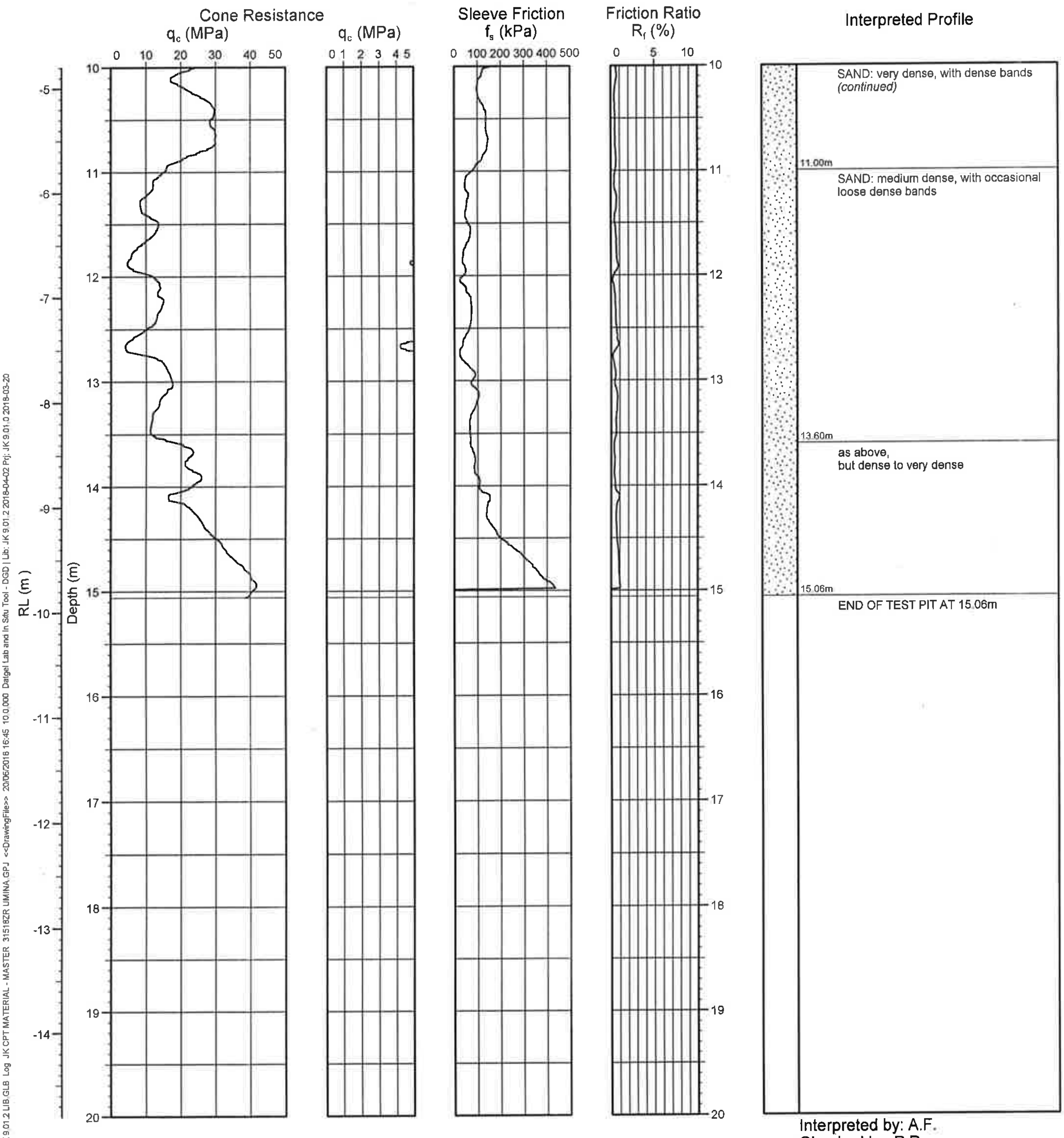
Interpreted by: A.F.
Checked by: P.R.

JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

CPT No.
CPT11
2 / 2

CONE PENETROMETER TEST RESULTS

Client:	ROYAL HASKONINGDHV AUSTRALIA PTY LTD		
Project:	PROPOSED BEACH EROSION MANAGEMENT STRATEGY		
Location:	OCEAN BEACH SLSC TO KOORUNG STREET BOAT RAMP, THE ESPLANADE, UMINA, NSW		
Job No.: 31518ZR	**R.L. Surface:** ~5.2 m	**Data File:** 31518ZR Umina	
Date: 7/6/18	**Datum:**	**Operator:** J.B.	



Interpreted by: A.F.
Checked by: P.R.

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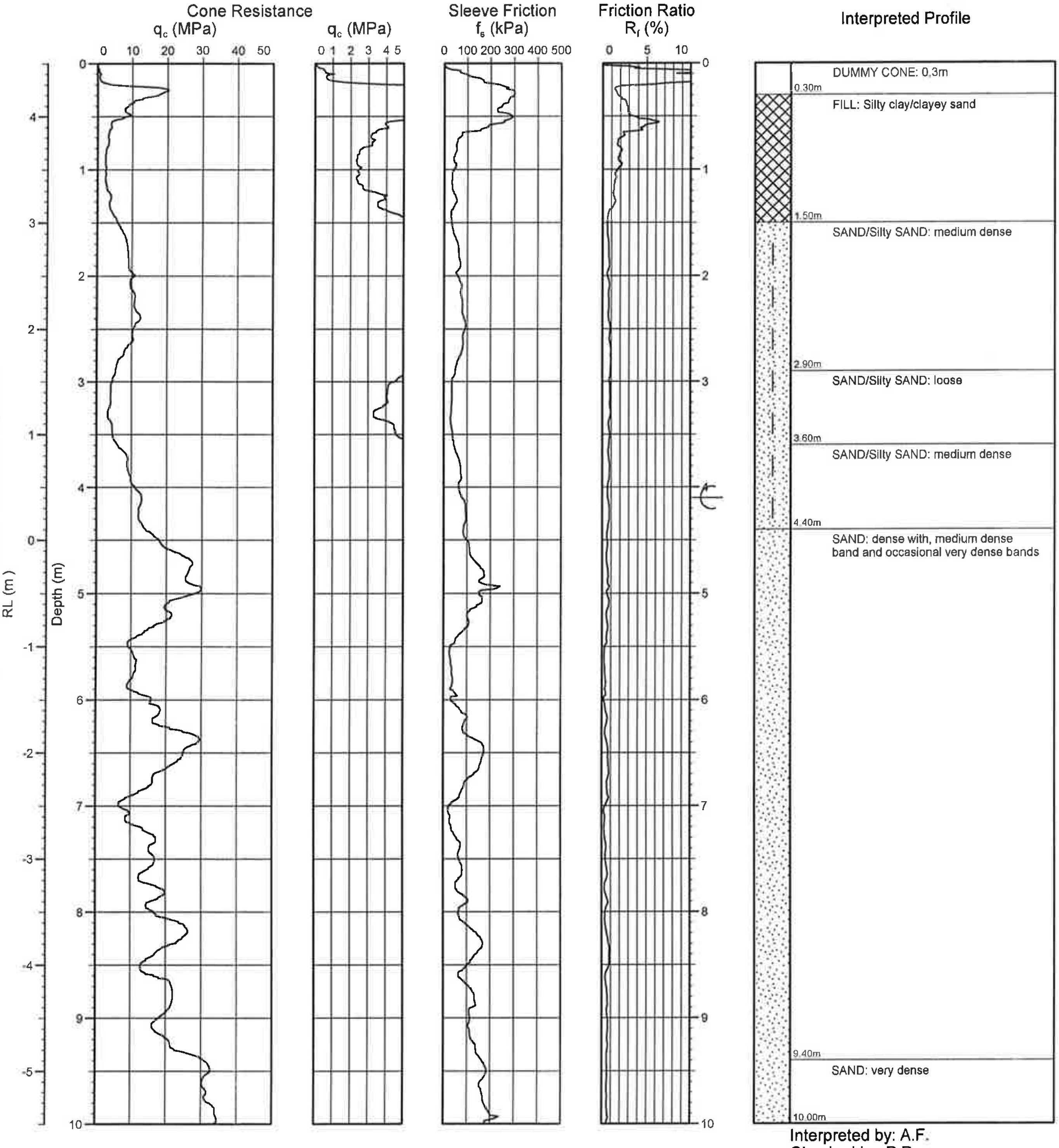
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

CPT No.
CPT12
1 / 2

CONE PENETROMETER TEST RESULTS

Client: ROYAL HASKONINGDHV AUSTRALIA PTY LTD
Project: PROPOSED BEACH EROSION MANAGEMENT STRATEGY
Location: OCEAN BEACH SLSC TO KOORUNG STREET BOAT RAMP, THE ESPLANADE, UMINA, NSW

Job No.: 31518ZR	**R.L. Surface:** ~4.5 m	**Data File:** 31518ZR Umina
Date: 7/6/18	**Datum:**	**Operator:** J.B.



Interpreted by: A.F.
Checked by: P.R.

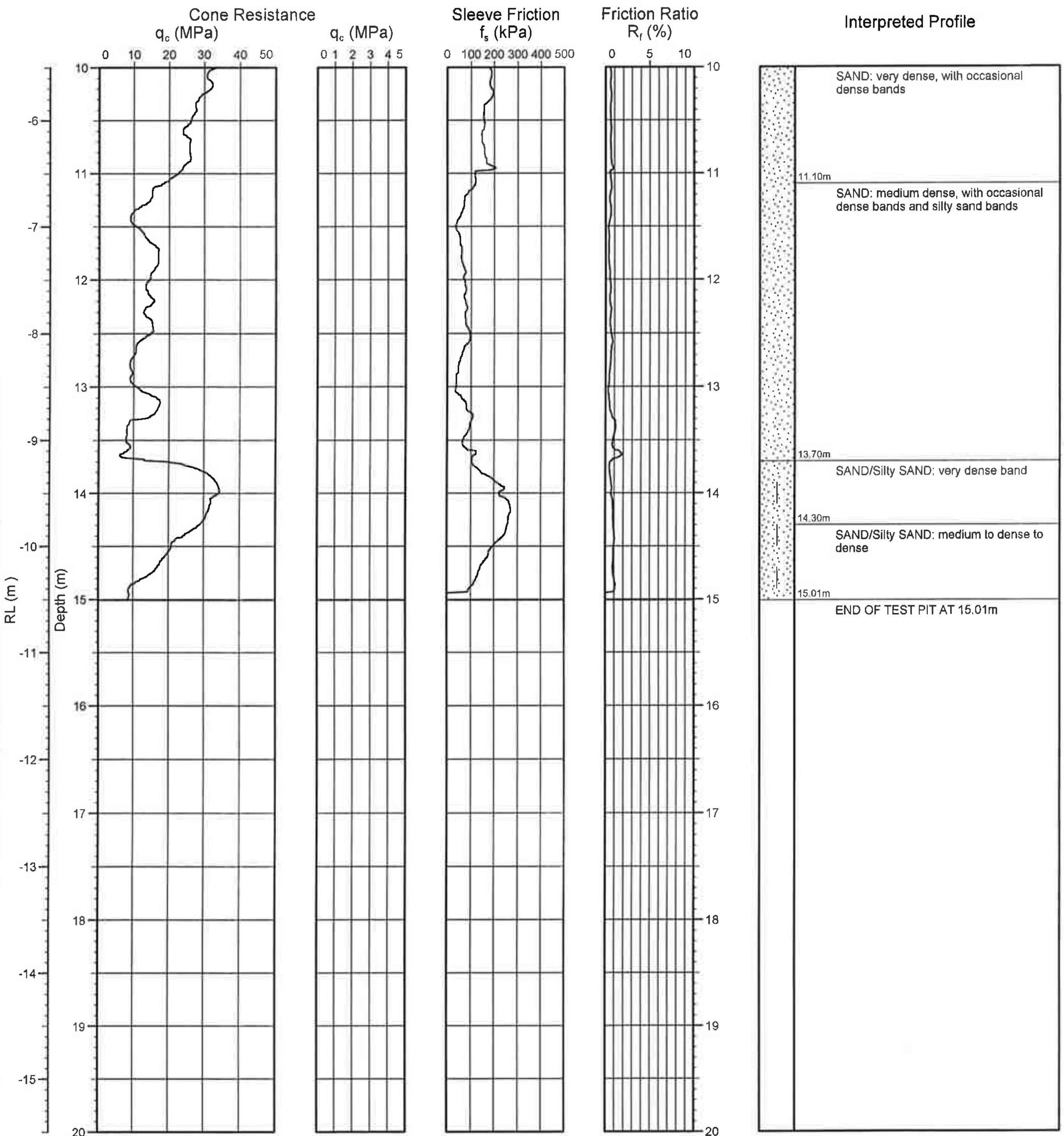
JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

CPT No.
CPT12
2 / 2

CONE PENETROMETER TEST RESULTS

Client:	ROYAL HASKONINGDHV AUSTRALIA PTY LTD
Project:	PROPOSED BEACH EROSION MANAGEMENT STRATEGY
Location:	OCEAN BEACH SLSC TO KOORUNG STREET BOAT RAMP, THE ESPLANADE, UMINA, NSW

Job No.: 31518ZR	**R.L. Surface:** ~4.5 m	**Data File:** 31518ZR Umina
Date: 7/6/18	**Datum:**	**Operator:** J.B.



JK 9.01.2 LIB.GLB Log JK CPT MATERIAL - MASTER 31518ZR UMINA.GPJ <<DrawingFile>> 20/06/2018 16:45 10.0.000 Datgel Lab and In Situ Tool - DGD | Lib: JK 9.01.2 2018-04-02 Prj: JK 9.01.0 2018-03-20

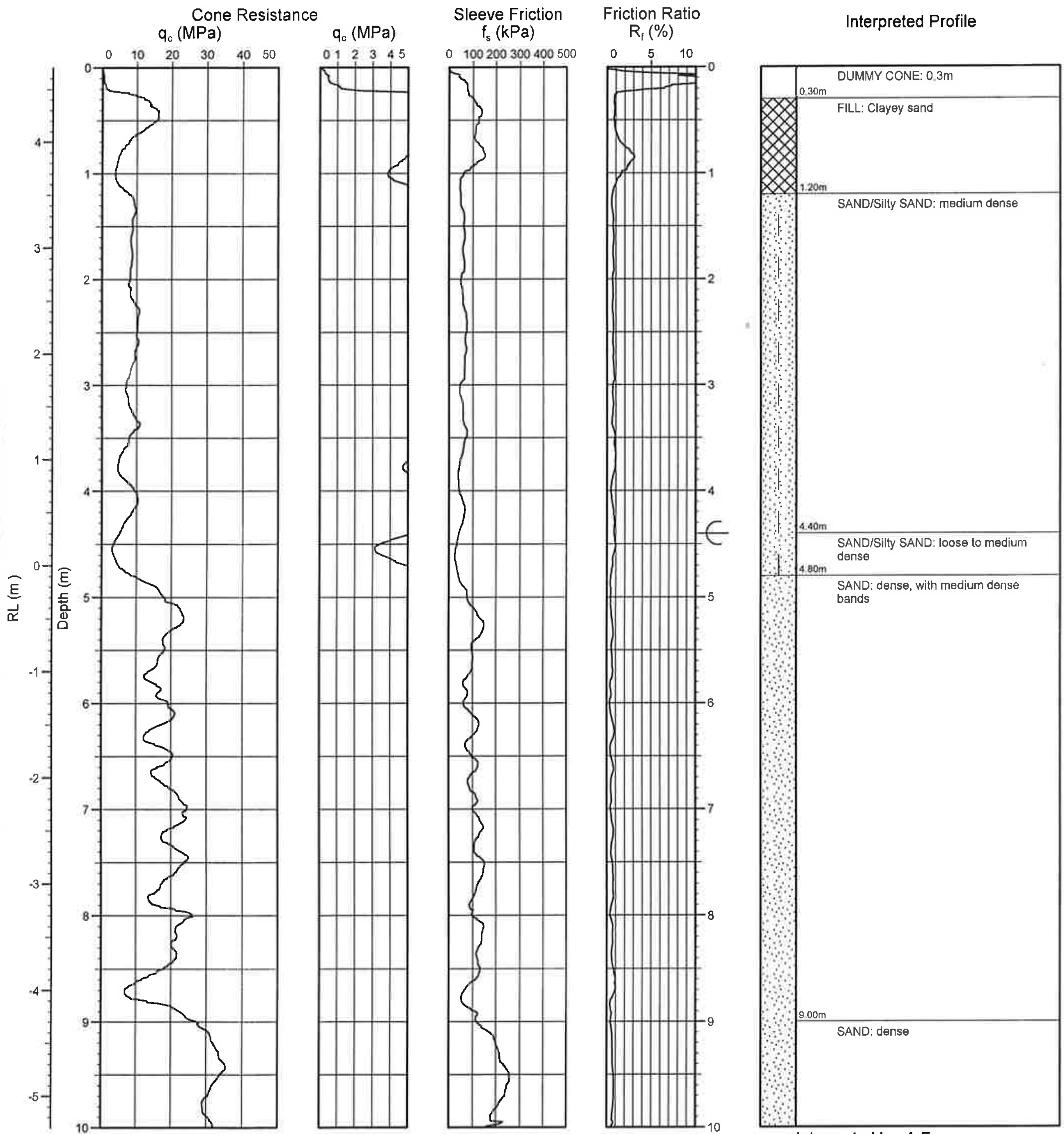
Interpreted by: A.F.
Checked by: P.R.

JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

CPT No.
CPT13
1 / 2

CONE PENETROMETER TEST RESULTS

Client:	ROYAL HASKONINGDHV AUSTRALIA PTY LTD				
Project:	PROPOSED BEACH EROSION MANAGEMENT STRATEGY				
Location:	OCEAN BEACH SLSC TO KOORUNG STREET BOAT RAMP, THE ESPLANADE, UMINA, NSW				
Job No.:	31518ZR	**R.L. Surface:**	~4.7 m	**Data File:**	31518ZR Umina
Date:	7/6/18	**Datum:**		**Operator:**	J.B.



Interpreted by: A.F.
Checked by: P.R.

JK 9.01.2 LIB.GLB Log JK CPT MATERIAL - MASTER 31518ZR UMINA.GPJ <<DrawingFile>> 20/06/2018 16:45 10.0.000 Datgel Lab and In Situ Tool - DGD | Lib: JK 9.01.2 2018-04-02 Prj: JK 9.01.0 2018-03-20

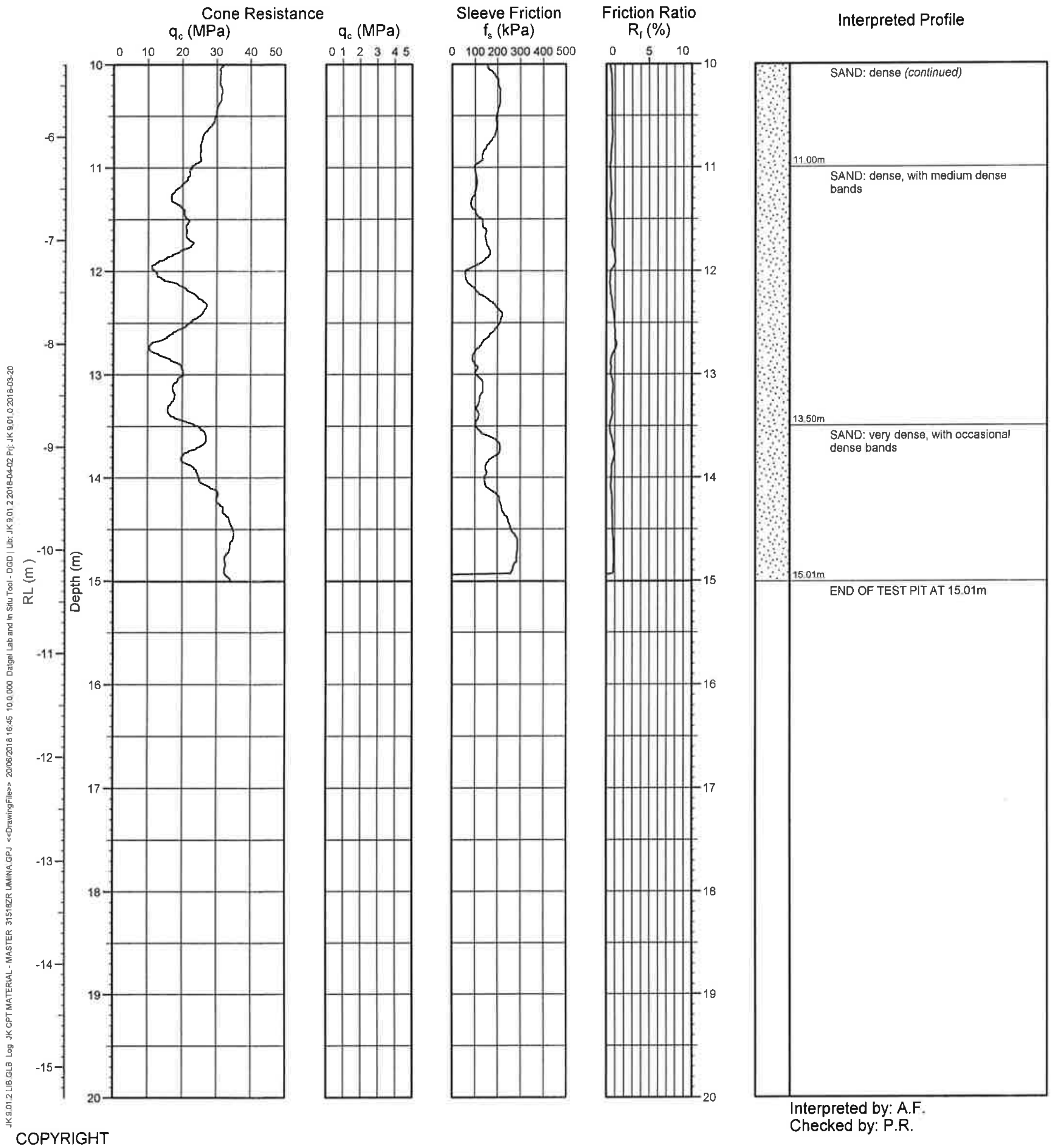
JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

CPT No.
CPT13
2 / 2

CONE PENETROMETER TEST RESULTS

Client:	ROYAL HASKONINGDHV AUSTRALIA PTY LTD
Project:	PROPOSED BEACH EROSION MANAGEMENT STRATEGY
Location:	OCEAN BEACH SLSC TO KOORUNG STREET BOAT RAMP, THE ESPLANADE, UMINA, NSW

Job No.: 31518ZR	**R.L. Surface:** ~4.7 m	**Data File:** 31518ZR Umina
Date: 7/6/18	**Datum:**	**Operator:** J.B.



Interpreted by: A.F.
Checked by: P.R.

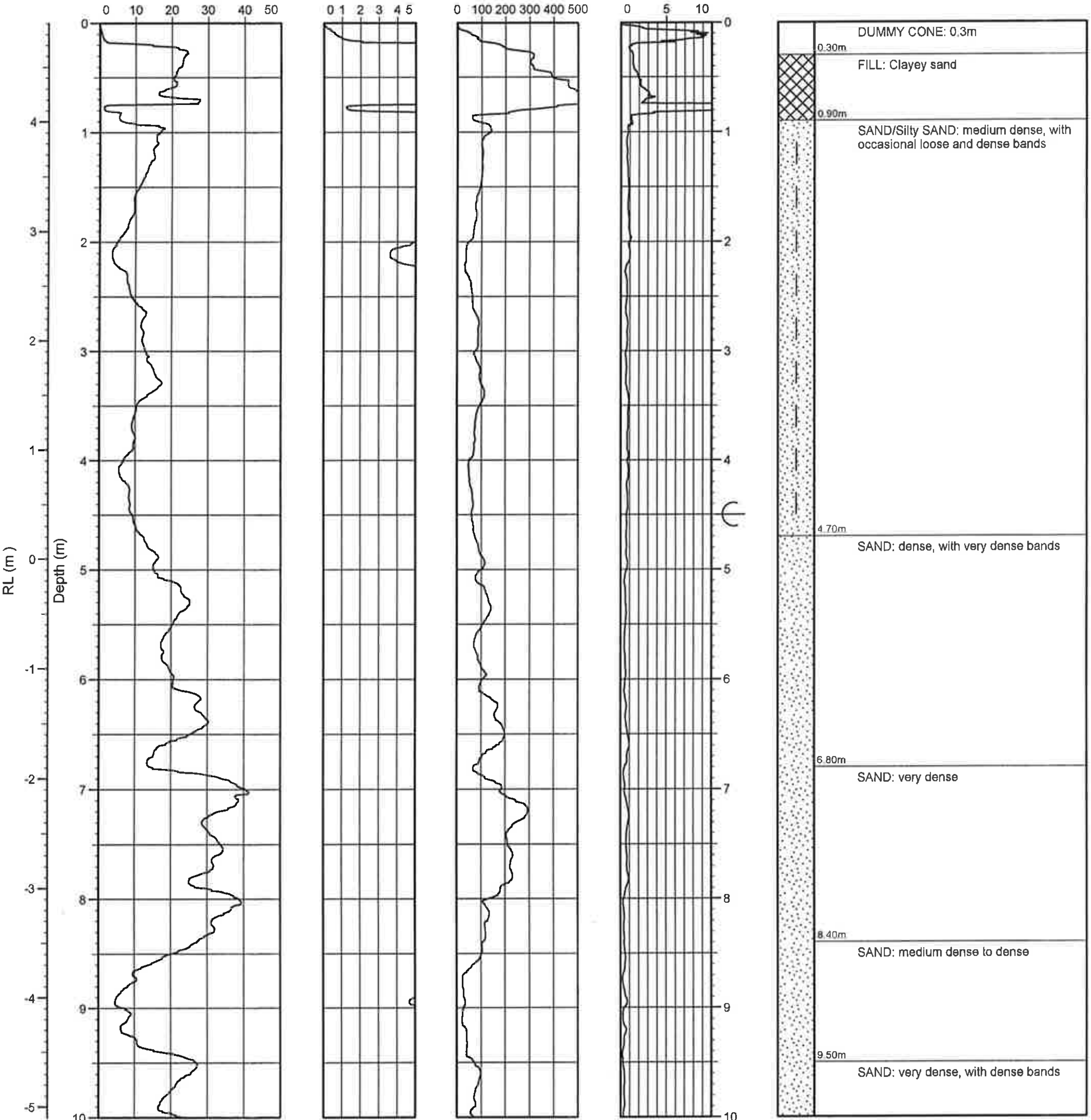
JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

CPT No.
CPT14
1 / 2

CONE PENETROMETER TEST RESULTS

Client: ROYAL HASKONINGDHV AUSTRALIA PTY LTD
Project: PROPOSED BEACH EROSION MANAGEMENT STRATEGY
Location: OCEAN BEACH SLSC TO KOORUNG STREET BOAT RAMP, THE ESPLANADE, UMINA, NSW

Job No.: 31518ZR	**R.L. Surface:** ~4.9 m	**Data File:** 31518ZR Umina
Date: 7/6/18	**Datum:**	**Operator:** J.B.



Interpreted by: A.F.
Checked by: P.R.

JK 9.01.2 LIB.GLB Log JK CPT MATERIAL - MASTER 31518ZR UMINA.GPJ <<DrawingFile>> 20/06/2018 16:45 10.0.000 Datgel Lab and In Situ Tool - DGD | Lib: JK 9.01.2 2018-04-02 Prj: JK 9.01.0 2018-03-20

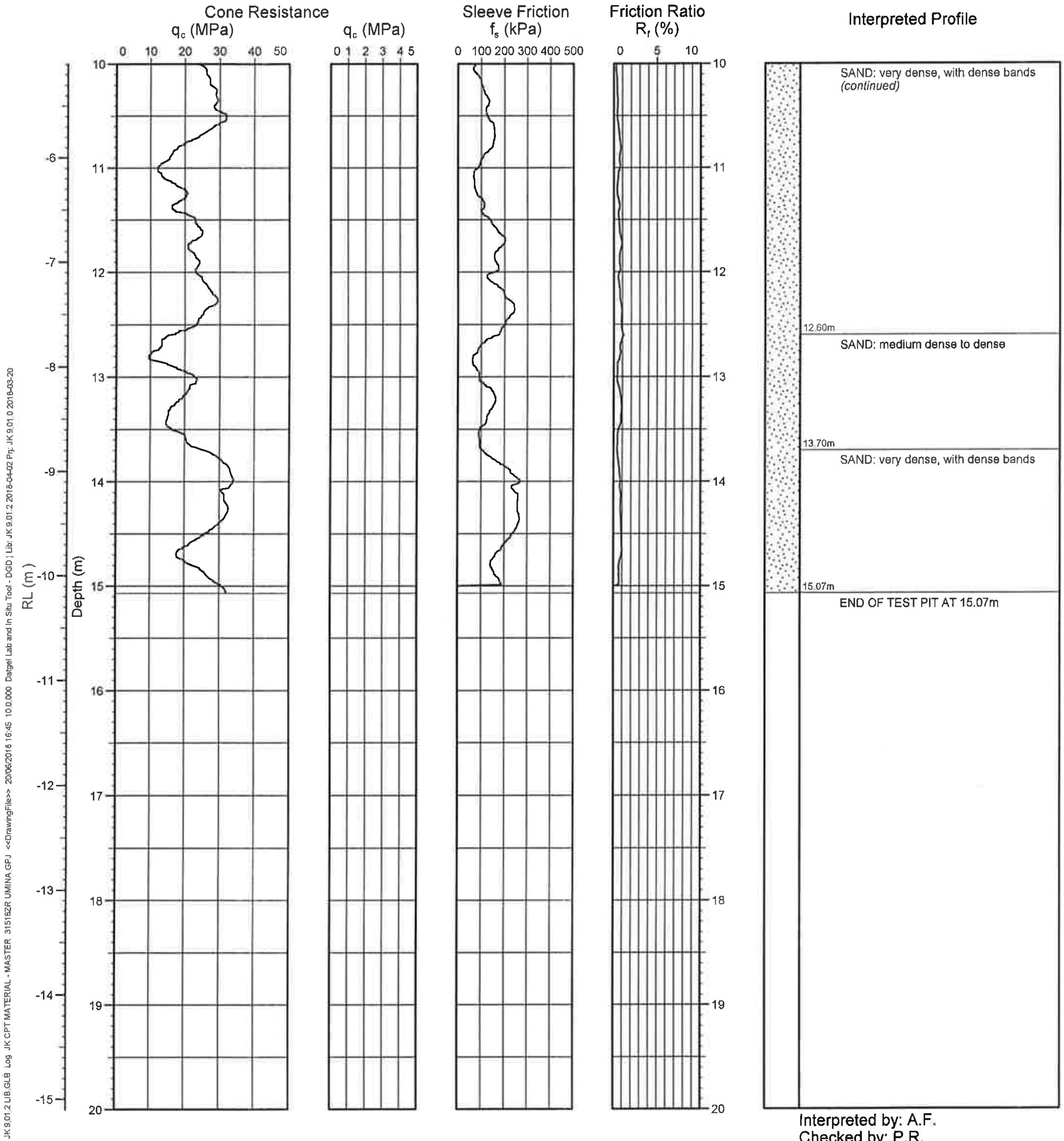
JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

CPT No.
CPT14

CONE PENETROMETER TEST RESULTS

Client:	ROYAL HASKONINGDHV AUSTRALIA PTY LTD
Project:	PROPOSED BEACH EROSION MANAGEMENT STRATEGY
Location:	OCEAN BEACH SLSC TO KOORUNG STREET BOAT RAMP, THE ESPLANADE, UMINA, NSW

Job No.: 31518ZR	**R.L. Surface:** ~4.9 m	**Data File:** 31518ZR Umina
Date: 7/6/18	**Datum:**	**Operator:** J.B.



Interpreted by: A.F.
Checked by: P.R.

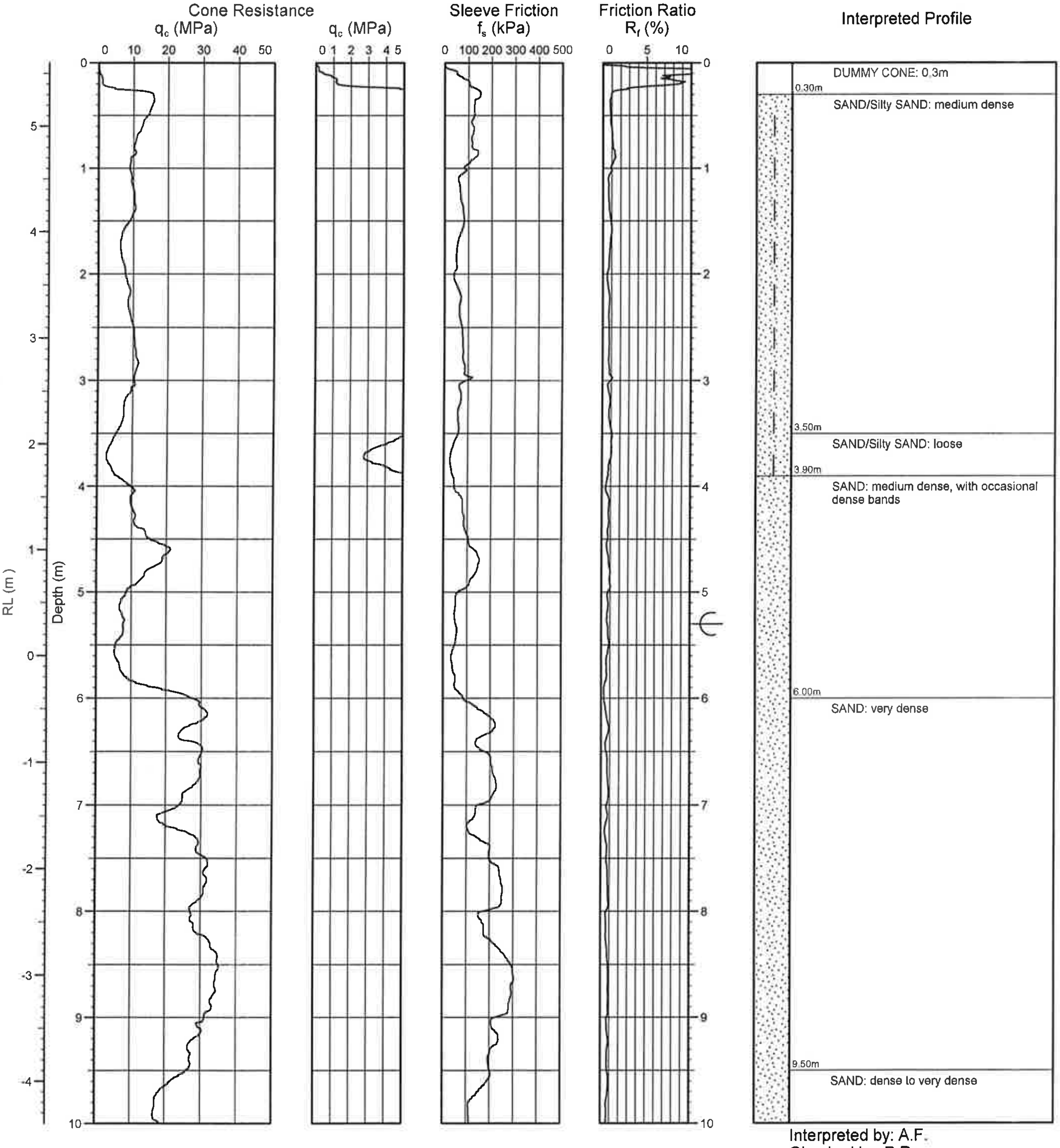
JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

CPT No.
CPT15
1 / 2

CONE PENETROMETER TEST RESULTS

Client: ROYAL HASKONINGDHV AUSTRALIA PTY LTD
Project: PROPOSED BEACH EROSION MANAGEMENT STRATEGY
Location: OCEAN BEACH SLSC TO KOORUNG STREET BOAT RAMP, THE ESPLANADE, UMINA, NSW

Job No.: 31518ZR	**R.L. Surface:** ~5.6 m	**Data File:** 31518ZR Umina
Date: 7/6/18	**Datum:**	**Operator:** J.B.



Interpreted by: A.F.
Checked by: P.R.

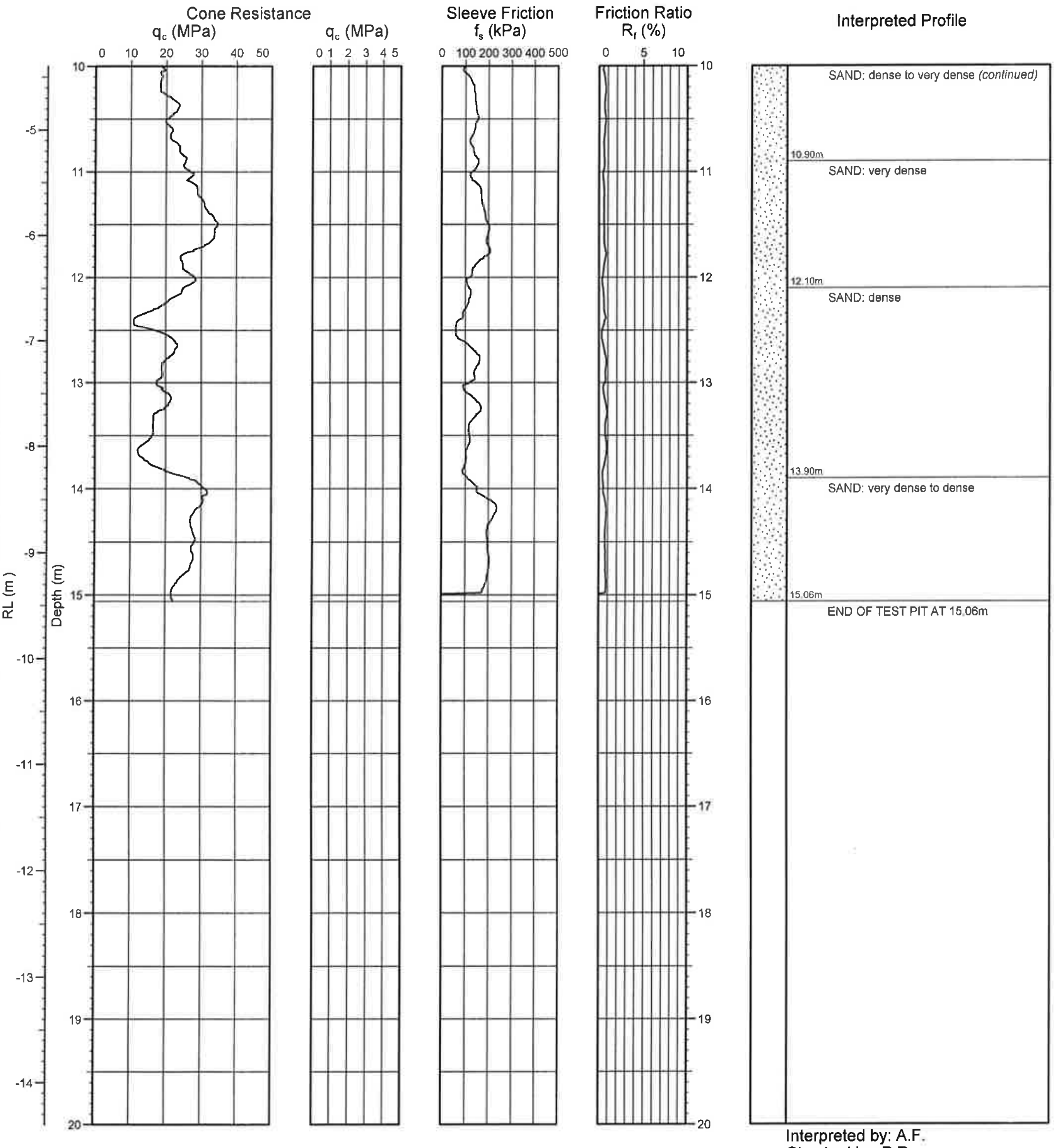
JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

CPT No.
CPT15
2 / 2

CONE PENETROMETER TEST RESULTS

Client:	ROYAL HASKONINGDHV AUSTRALIA PTY LTD
Project:	PROPOSED BEACH EROSION MANAGEMENT STRATEGY
Location:	OCEAN BEACH SLSC TO KOORUNG STREET BOAT RAMP, THE ESPLANADE, UMINA, NSW

Job No.: 31518ZR	**R.L. Surface:** ~5.6 m	**Data File:** 31518ZR Umina
Date: 7/6/18	**Datum:**	**Operator:** J.B.



JK 9.01.2 LIB.GLB Log JK CPT MATERIAL - MASTER 31518ZR UMINA.GPJ <<DrawingFile>> 20/06/2018 16:45 10.0.000 Datgel Lab and In Situ Tool - DGD | Lib: JK 9.01.2 2018-04-02 Prj: JK 9.01.0 2018-03-20

Interpreted by: A.F.
Checked by: P.R.

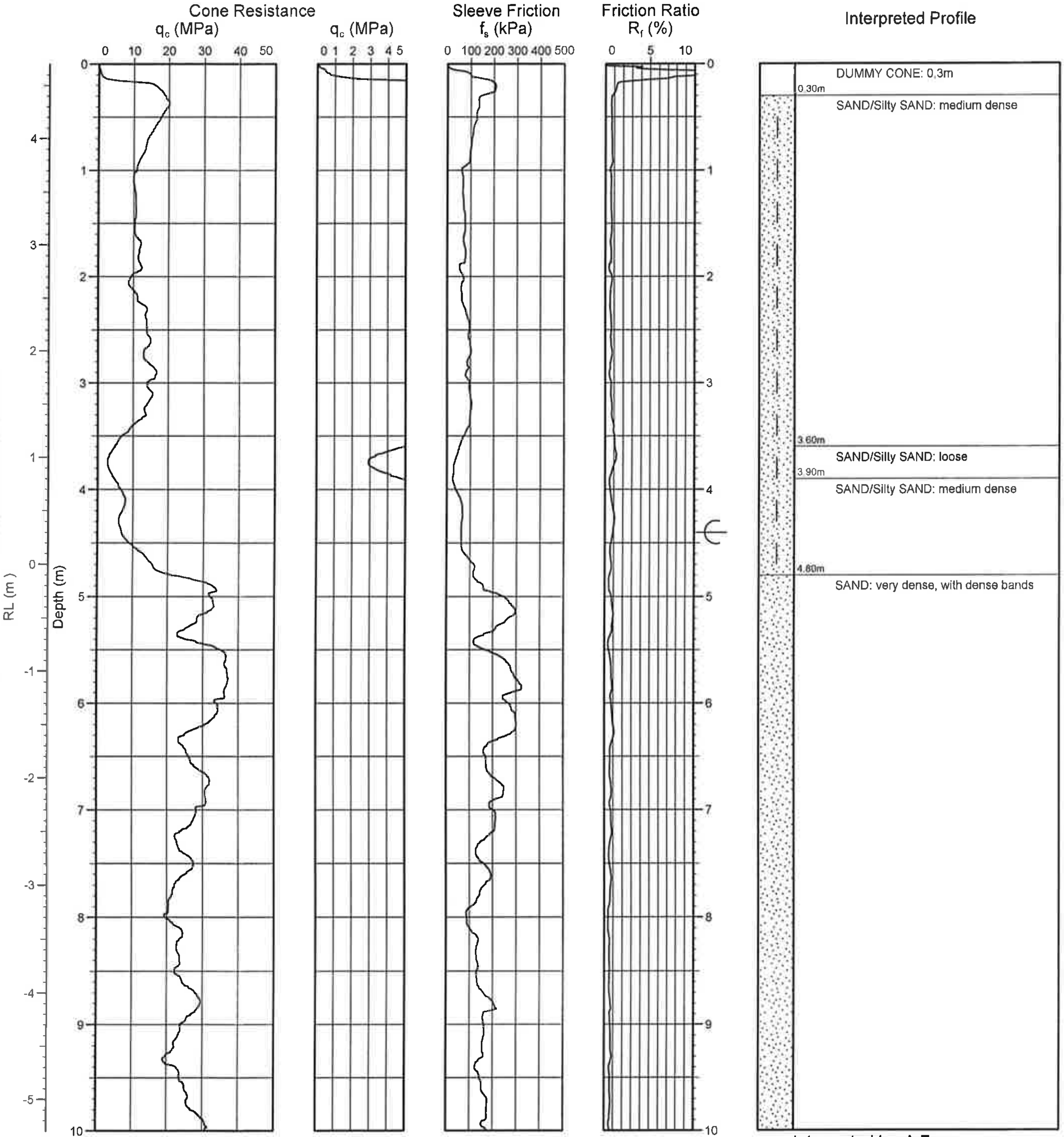
JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

CPT No.
CPT16
1 / 2

CONE PENETROMETER TEST RESULTS

Client:	ROYAL HASKONINGDHV AUSTRALIA PTY LTD
Project:	PROPOSED BEACH EROSION MANAGEMENT STRATEGY
Location:	OCEAN BEACH SLSC TO KOORUNG STREET BOAT RAMP, THE ESPLANADE, UMINA, NSW

Job No.: 31518ZR	**R.L. Surface:** ~4.7 m	**Data File:** 31518ZR
Date: 7/6/18	**Datum:**	**Operator:** J.B.



Interpreted by: A.F.
Checked by: P.R.

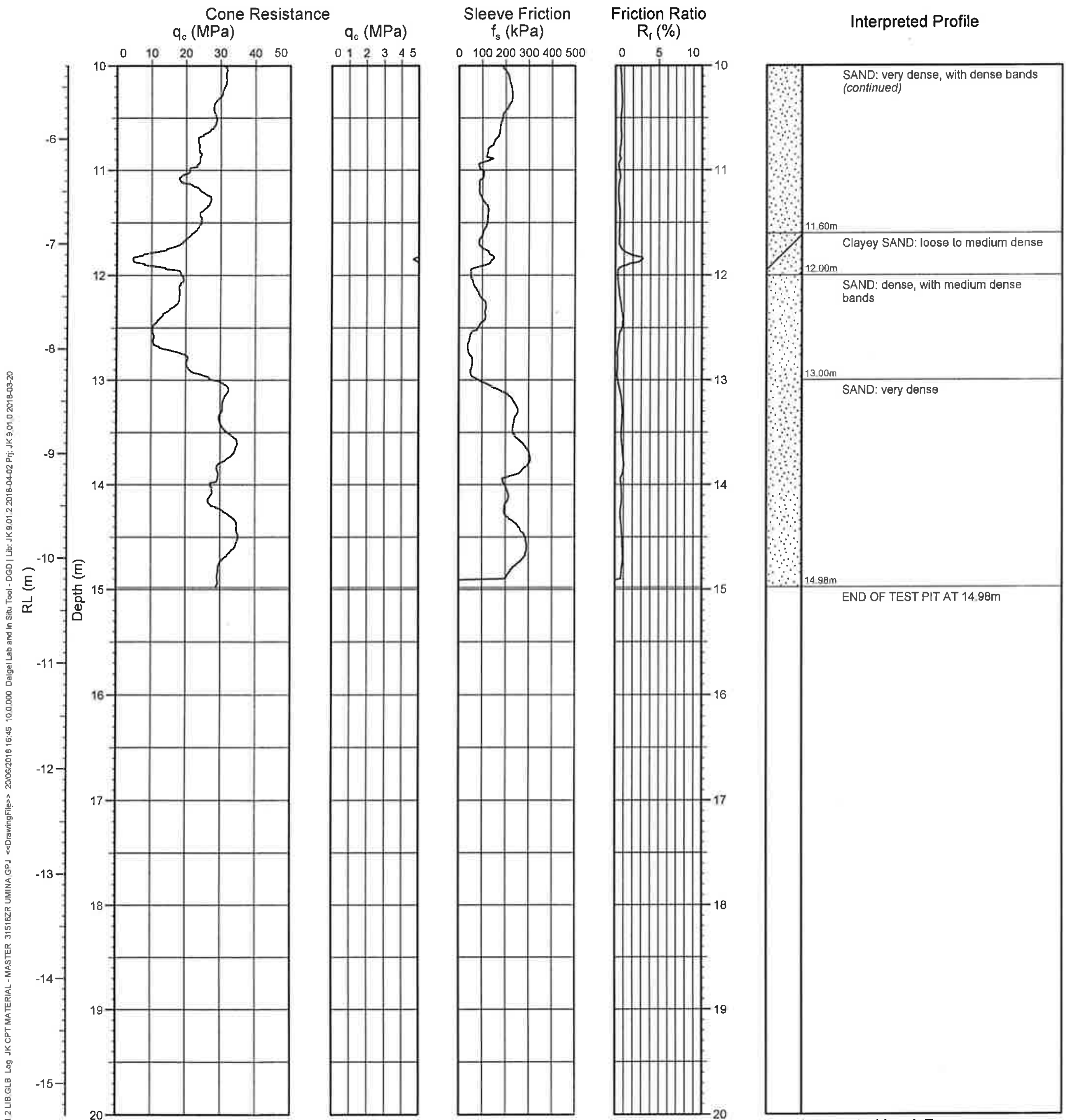
JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

CPT No.
CPT16
2 / 2

CONE PENETROMETER TEST RESULTS

Client:	ROYAL HASKONINGDHV AUSTRALIA PTY LTD
Project:	PROPOSED BEACH EROSION MANAGEMENT STRATEGY
Location:	OCEAN BEACH SLSC TO KOORUNG STREET BOAT RAMP, THE ESPLANADE, UMINA, NSW

Job No.: 31518ZR	**R.L. Surface:** ~4.7 m	**Data File:** 31518ZR
Date: 7/6/18	**Datum:**	**Operator:** J.B.



Interpreted by: A.F.
Checked by: P.R.

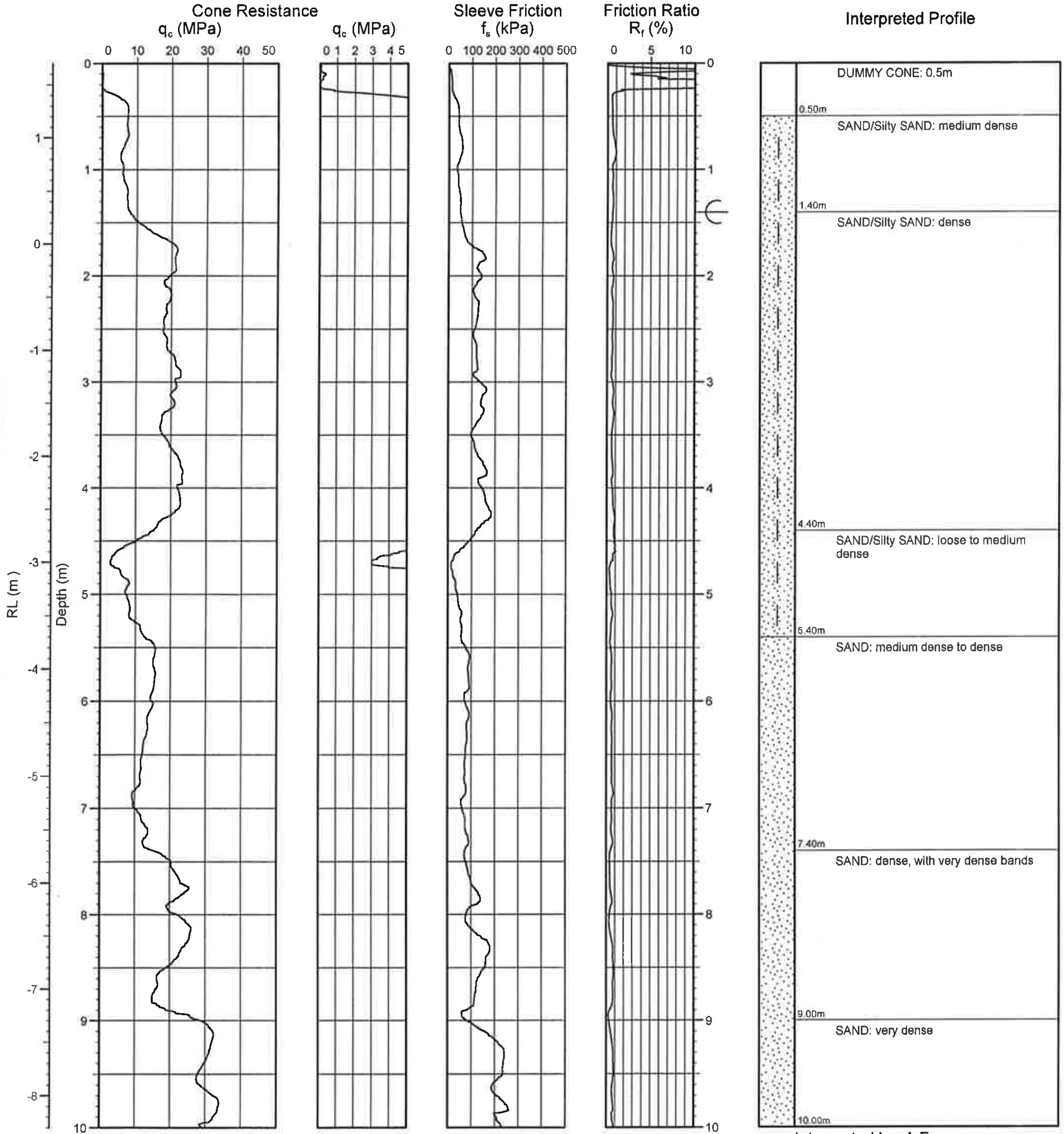
JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

CPT No.
CPT17
1 / 2

CONE PENETROMETER TEST RESULTS

Client: ROYAL HASKONINGDHV AUSTRALIA PTY LTD
Project: PROPOSED BEACH EROSION MANAGEMENT STRATEGY
Location: OCEAN BEACH SLSC TO KOORUNG STREET BOAT RAMP, THE ESPLANADE, UMINA, NSW

Job No.: 31518ZR	**R.L. Surface:** ~1.7 m	**Data File:** 31518ZR
Date: 7/6/18	**Datum:**	**Operator:** J.B.



Interpreted by: A.F.
Checked by: P.R.

JK Geotechnics

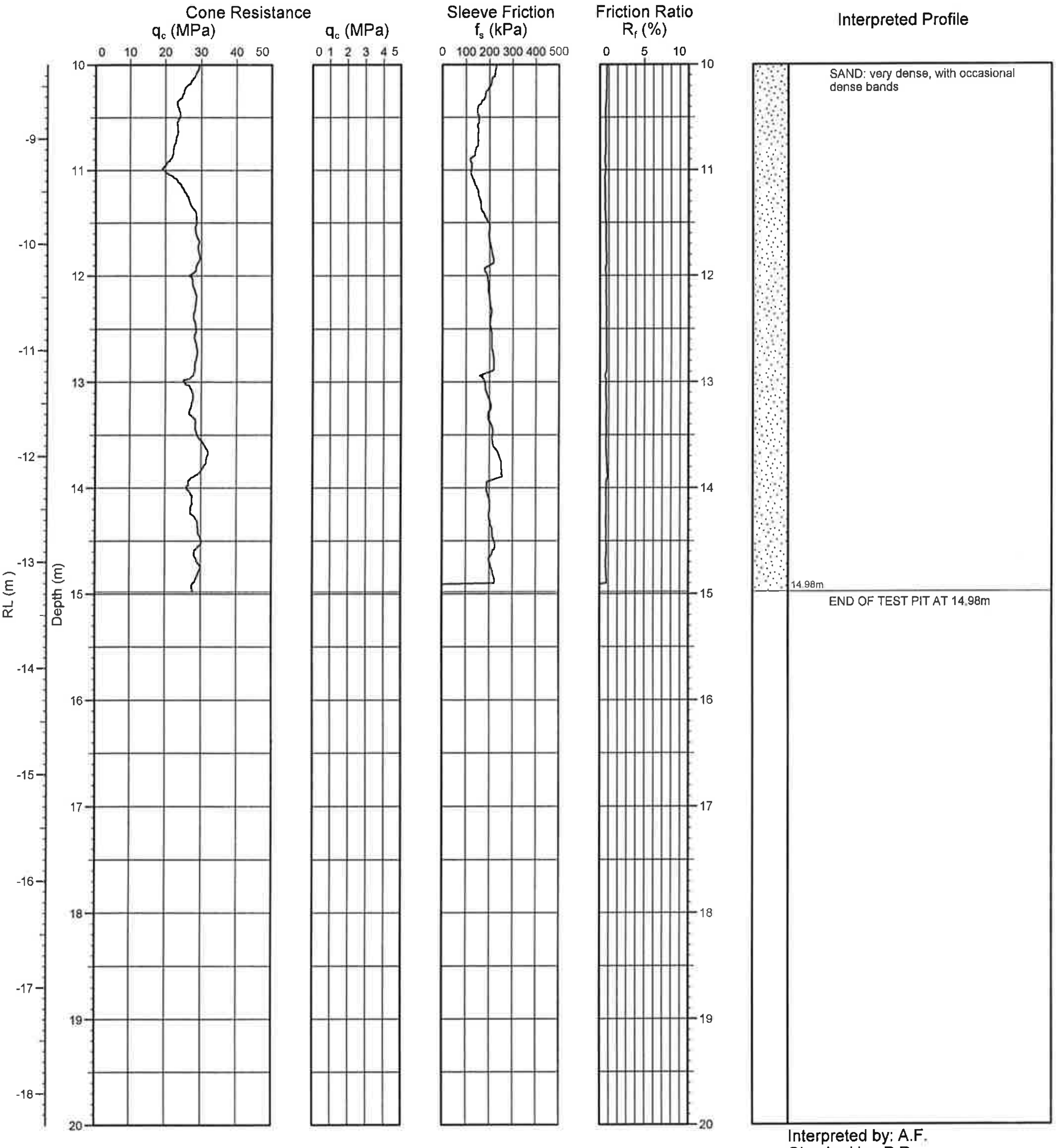
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

CPT No.
CPT17
2 / 2

CONE PENETROMETER TEST RESULTS

Client:	ROYAL HASKONINGDHV AUSTRALIA PTY LTD
Project:	PROPOSED BEACH EROSION MANAGEMENT STRATEGY
Location:	OCEAN BEACH SLSC TO KOORUNG STREET BOAT RAMP, THE ESPLANADE, UMINA, NSW

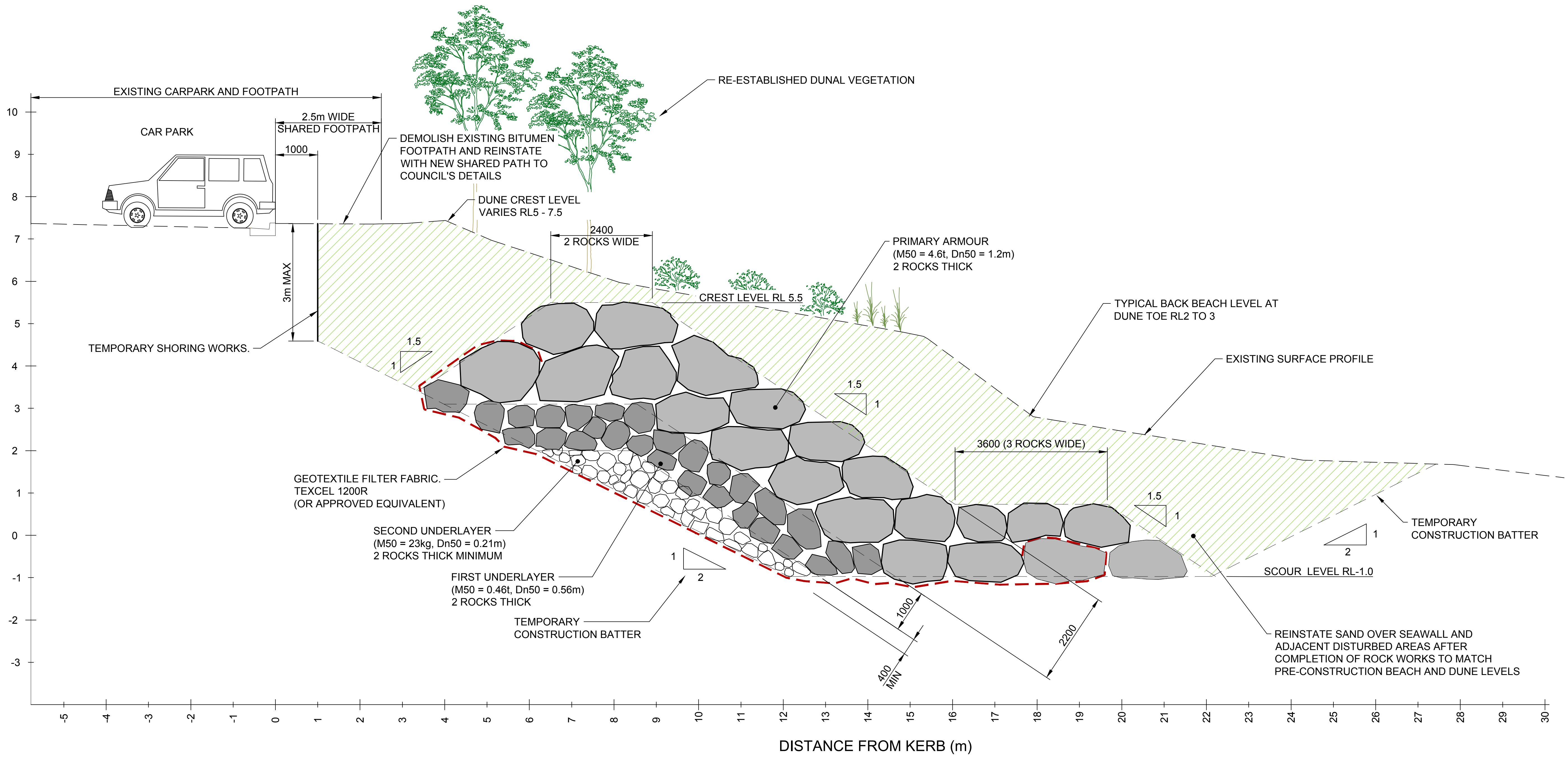
Job No.: 31518ZR	**R.L. Surface:** ~1.7 m	**Data File:** 31518ZR
Date: 7/6/18	**Datum:**	**Operator:** J.B.



Interpreted by: A.F.
Checked by: P.R.

Appendix C: Typical Sections

SKETCH C1
ROCK REVETMENT



AUSTRALIAN HEIGHT DATUM

NOT FOR CONSTRUCTION

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1:50 (A1) 1:100 (A3)

SAVED: 26-Apr-19

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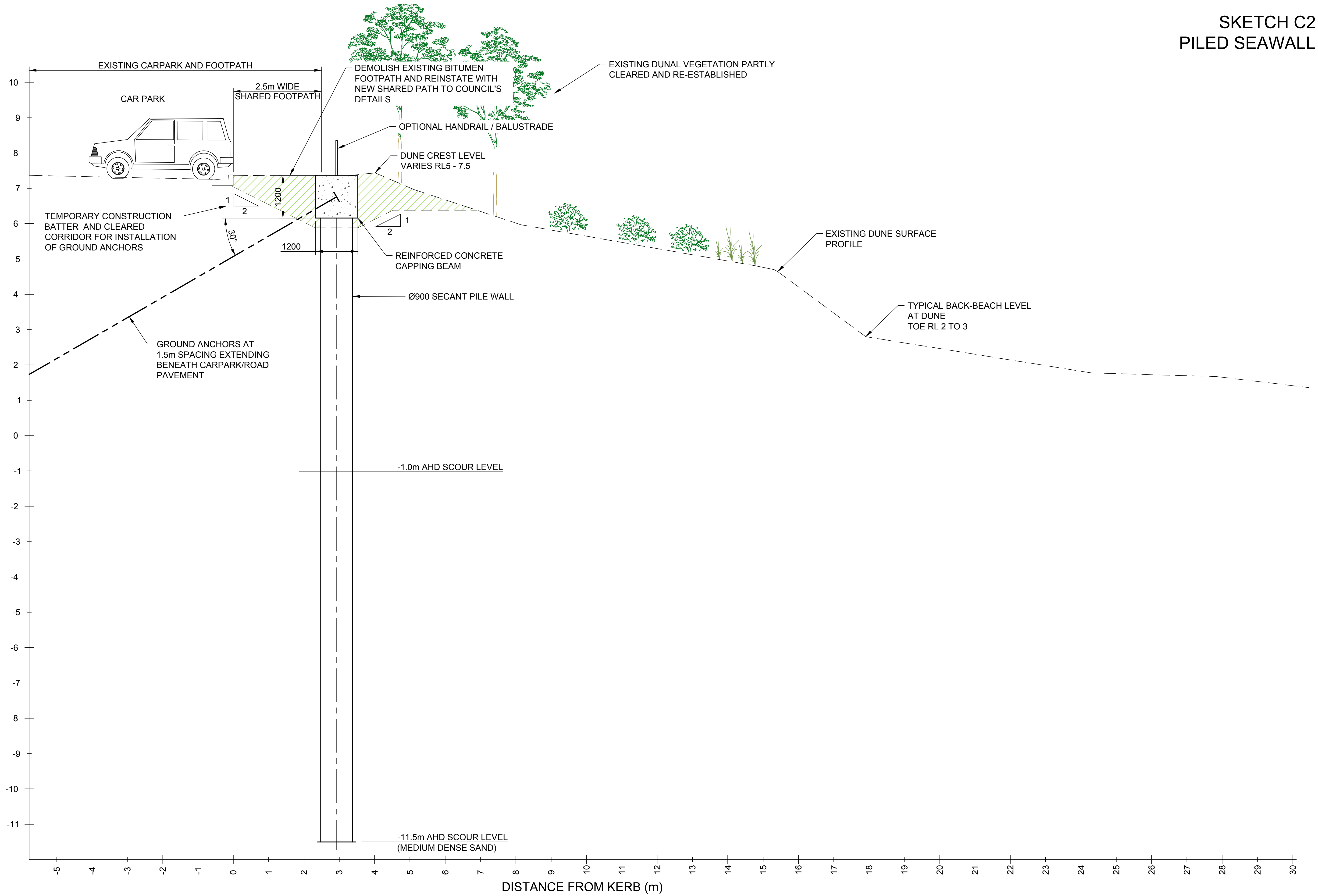
Royal HaskoningDHV
Enhancing Society Together

SKETCH C2
PILED SEAWALL



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AUSTRALIAN HEIGHT DATUM

NOT FOR CONSTRUCTION

SAVED: 26-Apr-19

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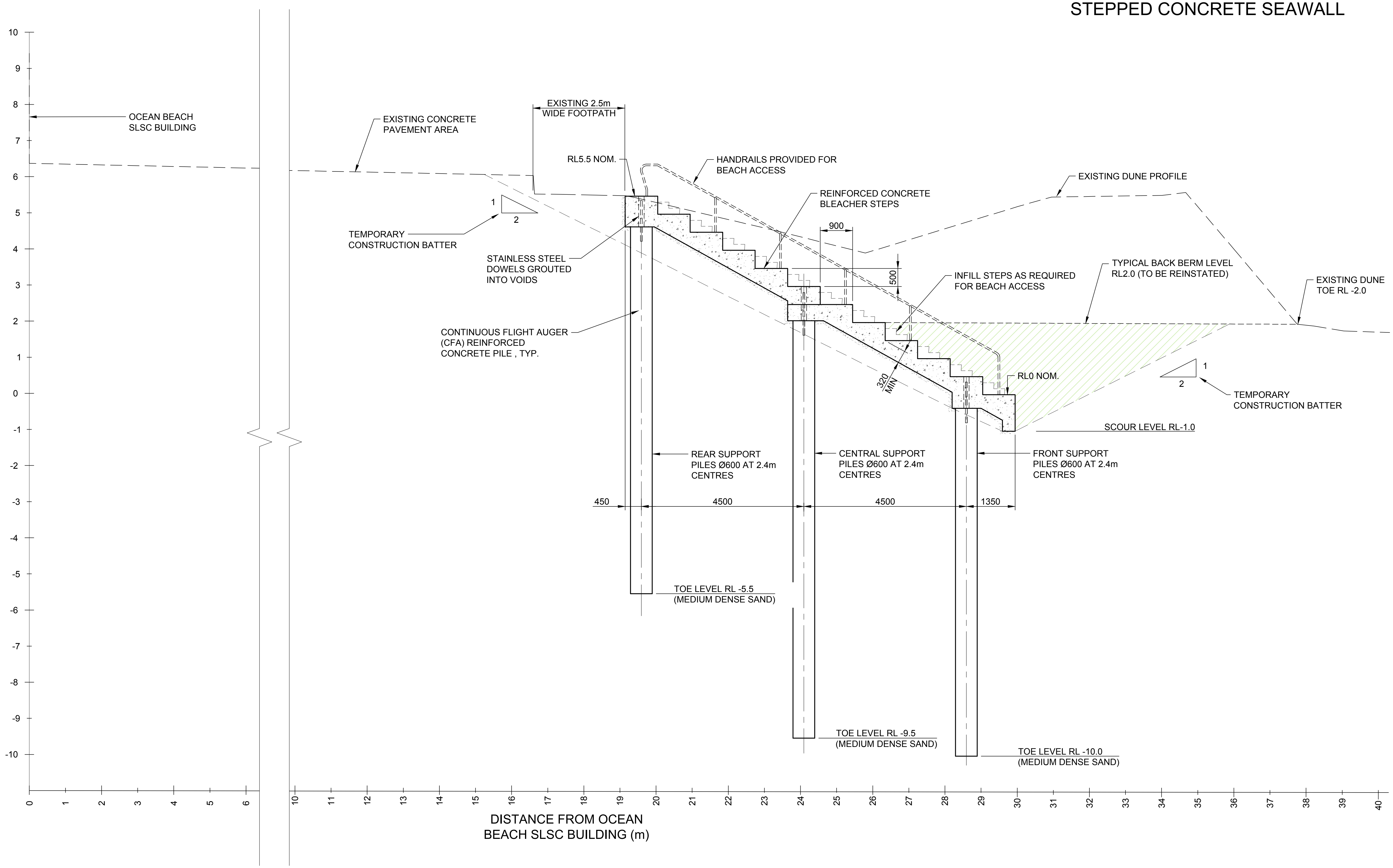
SKETCH C3
STEPPED CONCRETE SEAWALL



AUSTRALIAN HEIGHT DATUM

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SAVED: 26-Apr-19

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Working Drawings\PA1149-Sketches.dwg

Royal HaskoningDHV
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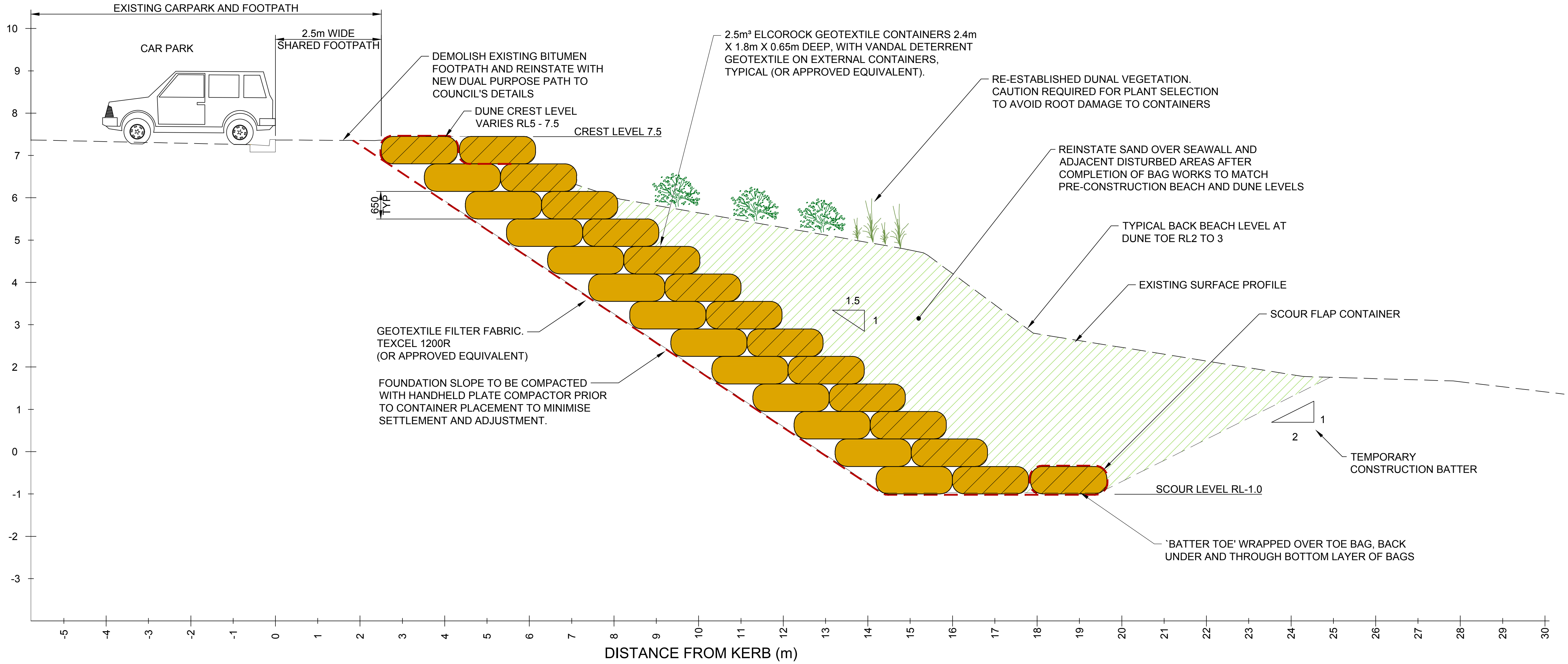
SKETCH C4
GEOTEXTILE CONTAINER WALL



AUSTRALIAN HEIGHT DATUM

NOT FOR CONSTRUCTION

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SAVED: 26-Apr-19

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