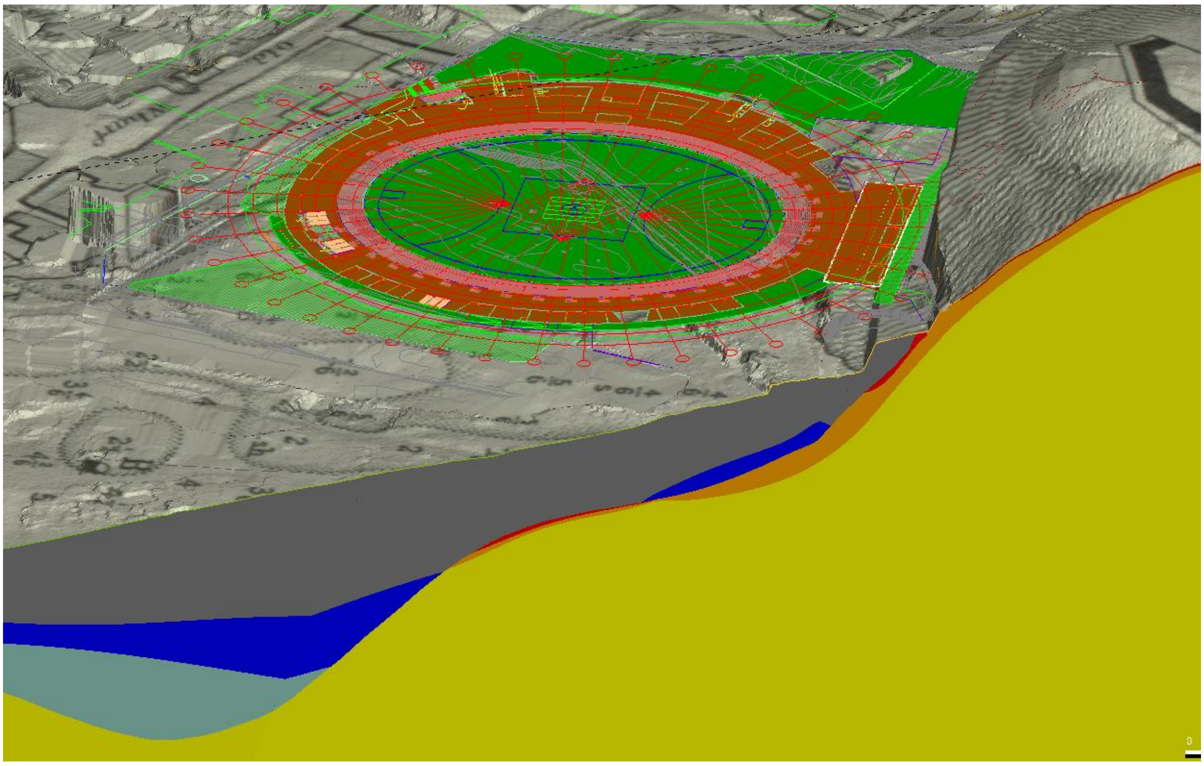
Macquarie Point Development Corporation

Macquarie Point Stadium Development

Geotechnical Interpretive Report

July 2024



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Macquarie Point Stadium Development
Geotechnical Interpretive Report

Macquarie Point Development Corporation

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*John Griffith left WSP between the issue of the draft version and the issue of this Rev0 version.

WSP acknowledges that every project we work on takes place on First Peoples lands.
We recognise Aboriginal and Torres Strait Islander Peoples as the first scientists and engineers and pay our respects to Elders past and present.

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Abbreviations

AEP	Annual Exceedance Probability
ALV	Alluvium
BH	Borehole
CBR	California Bearing Ratio
CPT	Cone Penetration Test
CRR	Cyclic Resistance Ratio
CSR	Cyclic Stress Ratio
DMT	Dilatometer
EST	Estuarine
FR	Fresh
GFR	Geotechnical Factual Report
h	Horizontal
HW	Highly Weathered
m AHD	Metres Australian Height Datum
m bgl	Metres below ground level
Max	Maximum
Min	Minimum
MPa	Megapascals
MPDC	Macquarie Point Development Corporation
MW	Moderately Weathered
PGA	Peak Ground Acceleration
PLT	Point Load Test
PSDs	Particle Size Distribution
q_c	Cone tip resistance
RL	Relative Level
RQD	Rock Quality Designation
RS	Residual Soil
SI	Site Investigation
SLS	Serviceability Limit State

SPT	Standard Penetration Test
SW	Slightly Weathered
UCS	Uniaxial Compressive Strength
ULS	Ultimate limit state
v	Vertical
XW	Extremely Weathered

Nomenclature

Cc	Compression Index
C_h	Coefficient of consolidation (horizontal)
Ch(T=0s)	Site Class Factor
C_r	Recompression Index
C_α	Creep index
C_v	Coefficient of consolidation (vertical)
E’	Drained Youngs Modulus (long term)
E_{ur}	Stiffness of the unload reload modulus, undrained
Φ_g	Geotechnical Strength reduction factor
k_p	Probability Factor
M	Constrained modulus (drained)
N_{kt}	Cone factor
OCR	Overconsolidation Ratio
P_y	Ultimate lateral resistance
q_t	Total cone tip resistance (corrected for porewater pressure effects)
S_u	Undrained shear strength
z	Hazard Factor
σ_v	Vertical effective stress
φ’	Effective Friction Angle

1 Introduction

1.1 The project

The State Government of Tasmania is planning a new multi-purpose stadium to provide a premier sporting, arts, events, and entertainment facility at Macquarie Point, Hobart. To inform the initial feasibility and concept design phases of the project, WSP Australia Pty Ltd (WSP) was commissioned by Macquarie Point Development Corporation (MPDC) to develop a preliminary geotechnical ground model from available subsurface information, and to prepare a data gap analysis to identify where additional information was required to further inform ground conditions. A site investigation was scoped and executed in April and May 2024 and is reported separately in Geotechnical Factual Report reference PS212776-WSP-HOB-GEO-REP-001 (GFR).

This report presents the geotechnical interpretation of the investigations undertaken, incorporating an assessment of recent and historic investigations on the site. The assessment of the presence or absence of contaminated material on site and groundwater level and chemistry is the responsibility of others and is not discussed in this report.

Macquarie Point lies to the north of Sullivans Cove, adjacent to the Hobart City Centre as shown in Figure 1.1. The precinct is approximately 9.3 hectares in size and is bounded by Davey Street to the West, Evans Street to the south, Hunter Street and the ports facility to the east, and the Hobart Cenotaph and Royal Hobart Regatta grounds to the north. The channelised Hobart Rivulet lies to the west and north, crossing the precinct near the northern boundary extent, where it flows into the Derwent River.

Figure 1.1 Project location

1.2 Objective of geotechnical investigations

The objective of the investigation was to provide geotechnical information to inform the foundation design of the Macquarie Point Stadium. The geotechnical investigation was carried out in general accordance with WSP's proposal ref. PS212776 dated 27 March 2024.

At the time of the geotechnical investigations, the design and site layout were conceptual in development. The geotechnical investigation was scoped with the objective of obtaining preliminary information across the site to assist design engineers with the development of the design for the project. A preliminary ground model was developed, and the ground investigation was scoped based on significant areas of unknowns in the ground model. For this reason, locations of investigations were scoped to give an overview of site conditions across the proposed stadium precinct, as such it is likely that further investigation would be required to develop and/or verify the detailed design of the stadium and associated structures.

1.3 Scope of this report

This report provides preliminary recommendations specific to the Macquarie Point Stadium namely:

- description of the site based on available information including its extents, topography and geomorphology, and geology
- summary of and reference to all geotechnical investigation data used in the preparation of the GIR including boreholes, cone penetration testing, in-situ testing, and laboratory testing
- geological and geotechnical interpretation and assessment of the site investigation results
- refinement of the ground model for the site based on the recent site investigation data
- recommendation of geotechnical design parameters for materials on-site
- preliminary foundation design parameters based on the ground model
- summary of seismic conditions at the site with potential impact on the proposed structure
- key risks and limitations that require mitigation or management methods
- advice about additional work that could be beneficial for the detailed design of the stadium.

1.4 Investigation scope

The scope of work for the geotechnical investigation is discussed in the GFR and is summarised below. The fieldwork was undertaken between 14 April 2024 and 21 May 2024. Figure A.1 in Appendix A shows the test locations.

- sixteen (16) cored boreholes spread across the stadium precinct drilled to depths between 10.10 m and 28.82 m
- four (4) acid sulfate boreholes (ABH) drilled to depths between 2.70 m and 9.20 m
- seventeen (17) cone penetration test (CPT) to depths between 1.82 m and 13.59 m
- laboratory testing of soil and rock samples for geotechnical purposes.

Sampling for acid sulfate soil and potential acid sulfate soil testing has been undertaken as part of the investigation scope, but results have been received and reported by others. Groundwater and contamination assessment investigations were not part of this scope of work.

2 Site description

2.1 Topography and terrain

The proposed Macquarie Point stadium development area encompasses an area of approximately 93,000 m^2. The site is relatively flat, predominantly comprising an industrial hard stand area, with an elevation of approximately RL 6 m AHD to RL 3 m AHD, the existing site levels gradually fall to the south-east.

2.2 Current land use description

At the time of the site investigation, there were a number of businesses, operations, and temporary facilities on site that influenced the positioning and timing of the investigation locations. These site operations included the following:

- existing businesses and offices such as Goods Shed, The Hobart Brewing Company, The Loaded Dog Café, and the Spiegeltent at the southern end of the site
- the Taswater construction zone at the northern end of the site
- the active public car park in the south and east area of site
- excavation and remediation of the southwestern corner of the site
- stockpiles of rubble
- shipping containers
- storage areas for temporary fencing and construction equipment.

The site was predominantly covered in gravel or asphalt and all investigation locations could be driven to.

2.2.1 Historical site activity

Historical site activity within Macquarie Point included defence, livestock management, sanitation, waste management, gas works industries, education, transport, port facilities, and freight handling. To accommodate these different land use activities, progressive reclamation of Macquarie Point has been undertaken to increase the usable area of the land, shown in Figure 2.1. Through the late 1800s and into the 1900s, seawall construction and land reclamation was undertaken of both Sullivan's Cove and Macquarie Point to support the various site activities at the time. Ocean Pier built in 1914 at the southern end of Macquarie Point was destroyed by fire in the 1948 and was subsequently removed.

Infrastructure features that have previously existed on site include gasworks infrastructure, underground storage tanks, fuel transfer lines, former building footings, seawalls, revetments, sewer, telecommunication and power cables, and stormwater drains (SEMP, AECOM, 22 Oct 2021).

Contamination remediation activities are understood to have occurred over sections of the site in recent years.

Figure 2.1 Progressive reclamation of Macquarie Point Precinct (Macquarie Point Strategic Framework and Masterplan, 2015)

2.3 Climate and meteorology

2.3.1 Overview

The Hobart area experiences relatively mild temperatures and moderate rainfall, with an annual yearly rainfall of 610 mm from the nearest observation site at Ellerslie Road, Hobart (refer Figure 2.2). Typically, the wettest month (mean rainfall) is October and driest usually February. The annual mean maximum temperature is 17°C and the annual mean minimum temperature is 8°C.

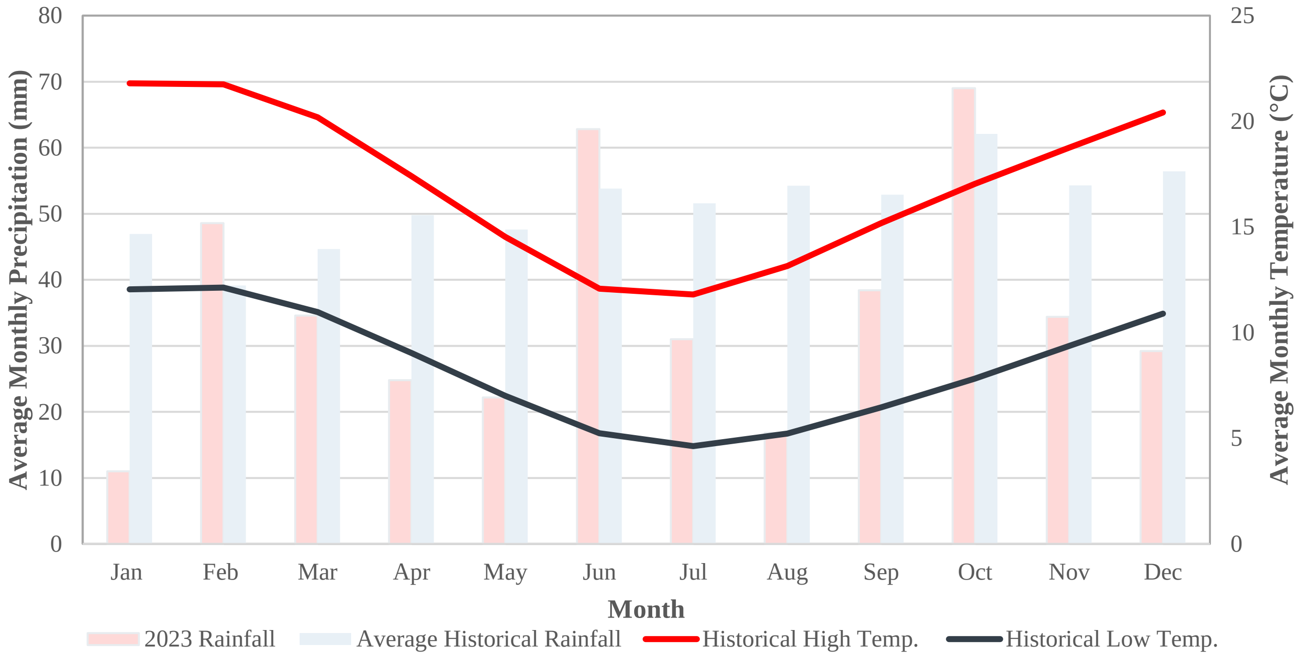


Figure 2.2 Monthly rainfall and temperature records from Ellerslie Road, Hobart (3.1 km from Macquarie Point Stadium)

2.3.2 Rainfall records

Daily rainfall totals during the site investigation are plotted below (Figure 2.3). The rainfall conditions around the time of the site investigation may impact groundwater levels measured during the works.

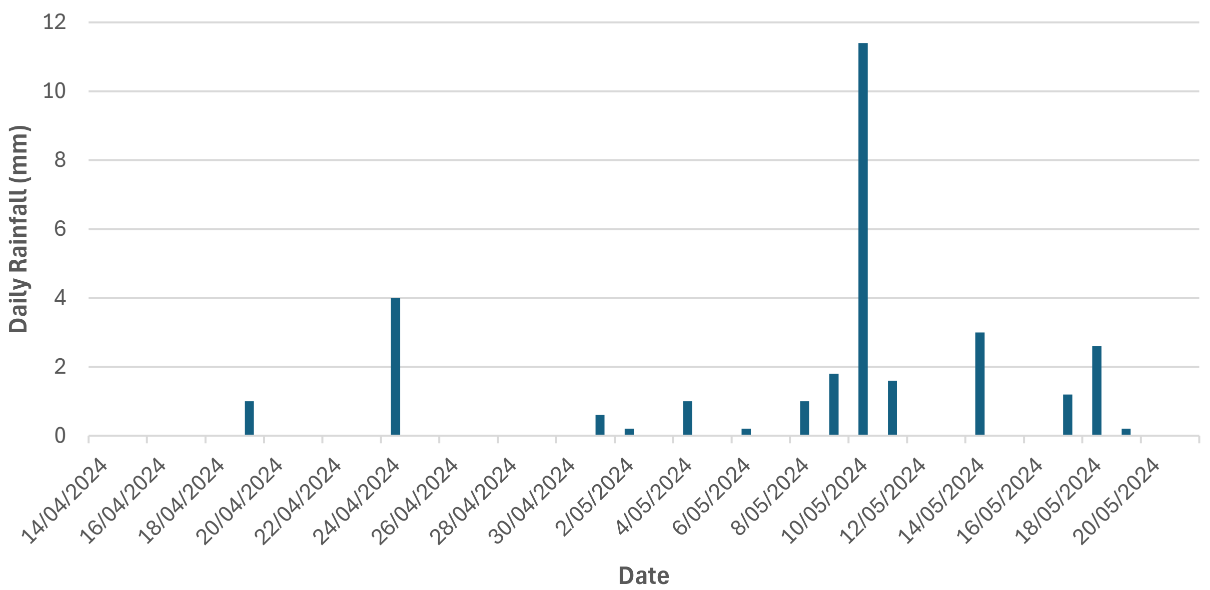


Figure 2.3 Daily rainfall records from Ellerslie Road, Hobart (3.1 km from Macquarie Point Stadium)

2.4 Previous investigations

There have been multiple site investigations carried out around the Macquarie Point area between 1962 and the present. These are summarised below:

- 335 Geo-environmental borehole and test pit investigations undertaken by AECOM (2015, 2017, 2021)
- 103 borehole investigations undertaken by Sinclair Knight Merz (1997)
- 85 borehole investigations undertaken by the Tasmanian Department of Mines (1972, 1984)
- 82 borehole investigations undertaken by GHD (2010, 2014)
- 25 borehole investigations undertaken by Pitt & Sherry (2008)
- 20 borehole investigations undertaken by Roads and Transport division (various state departments) (1993)
- 15 borehole investigations undertaken by public works department (1962, 1976)
- 12 borehole investigations undertaken by Sloane Weldon Engineering (1995)
- 9 borehole investigations undertaken by Douglas Partners, documented in Report on Foundation Constructability, Macquarie Point development (2015)
- 7 test pit investigations undertaken by William Cromer for MPDC (2021)
- 6 borehole investigations undertaken by Department of Growth (2008)
- 1 borehole investigation undertaken by Hydro Tasmania (2000).

A preliminary ground model completed by WSP in March 2024, on behalf of MPDC, summarised the ground investigations above with results documented in P209850-WSP-SYD-GEO-REP-00004. This report provided an overview of the ground conditions based on the investigations above, with ground conditions illustrated on a 3D ground model. Additionally, the report discussed assignment of material characteristics and geotechnical recommendations for the foundations, site preparation, and earthworks. This Geotechnical Interpretive Report supersedes the preliminary ground model report.

3 Geology and soils

3.1 Regional geology

Hobart is situated on the estuary of the River Derwent, a drowned river valley formed at a similar geological time as the extensional forces and crustal thinning that formed the Bass Strait and Gippsland Basins (95–85 million years ago). The Derwent Estuary is located in one of the many extensional fault systems that formed during the separation of Australia from Antarctica (the breakup of the supercontinent of Gondwana).

The Derwent Estuary has formed in the Derwent Graben, where a fault block or series of fault blocks move downwards with respect to the surrounding fault blocks (Horst). This change in elevation created a steep-sided valley where Quaternary glacial meltwater formed channels and pathways that eroded the Parmeener Supergroup and cut down into the Jurassic Dolerite.

The Hobart 1:25,000 geological series map and available literature indicate the ground surface in the Hobart region is predominantly underlain by Triassic Sandstone and Mudstone (Upper Parmeener Supergroup), which has been intruded by Jurassic era dolerite; this intrusion is quite extensive throughout the Hobart region and Tasmania. The Upper Parmeener Supergroup has eroded on the higher elevated horsts, and only dolerite remains on the flanks of the city. The dolerite can be several hundred metres thick, extremely hard and durable when fresh, and caps a number of mountains in the region (for example, Mount Wellington to the west).

At Macquarie Point, the Hobart Rivulet and the Park Street Rivulet have formed and eroded the bedrock on the alignment of faulting in the northwest-to-southeast direction, which generally parallels the southern boundary of the Queens Domain and Brooker Avenue. The water flow over many years has preferentially eroded the more highly weathered rock associated with the fractured zones along the fault alignment. Figure 3.1 provides an extract from the Hobart Geological Series Map, indicating general fault alignment responsible for the geological and geomorphological development of the rock strata underlying the Macquarie Point site.

After the last postglacial marine transgression [20,000–6,000 years ago; Harvey and Caton (2010)], the sea levels rose to fill these rift valleys with more recent Estuarine materials. These Estuarine deposits have formed interbedded sequences with the Alluvium from the smaller tributaries of the Derwent River and overly the bedrock. Fill placement has occurred since European settlement as part of the reclamation of Sullivans Cove, including Macquarie Point, the basins, and wharves on the west side of Sullivan’s Cove and New Wharf (Salamanca) on the south side of Sullivan’s Cove.

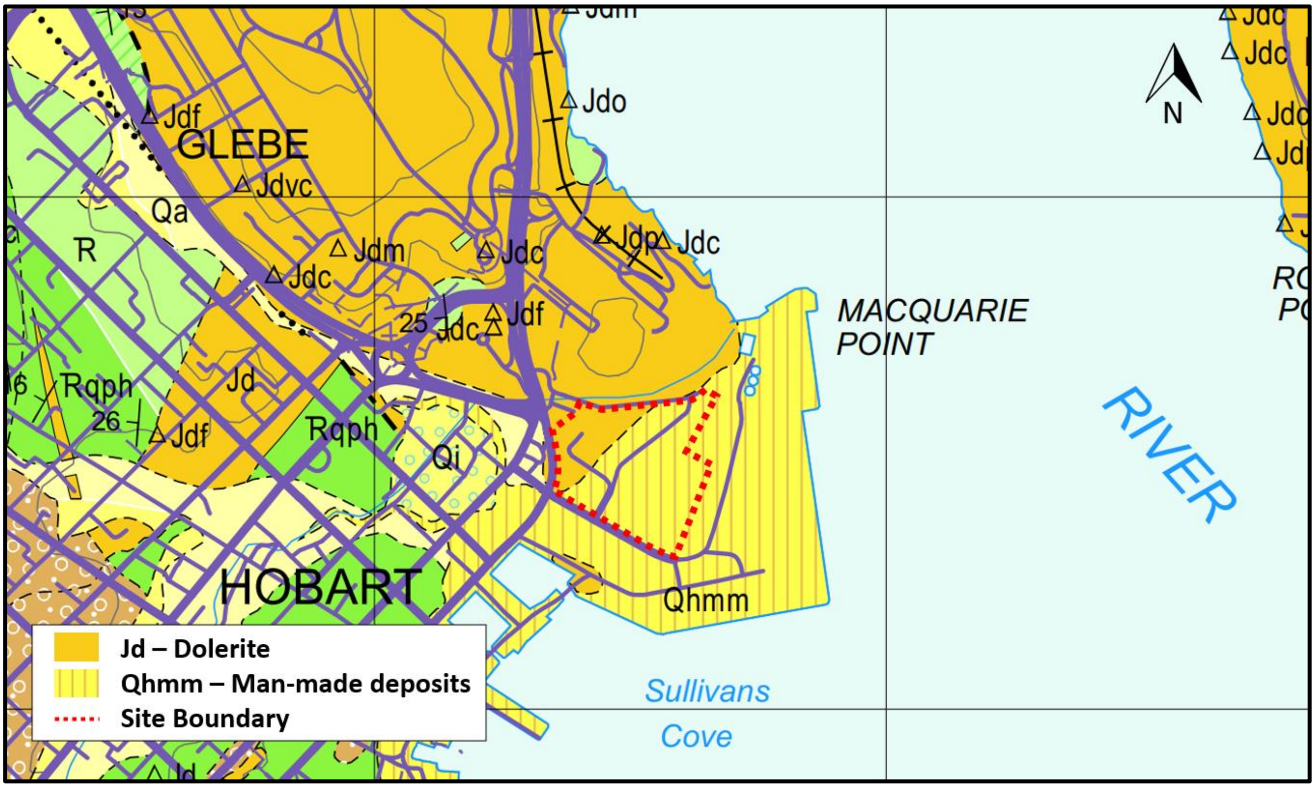


Figure 3.1 Macquarie Point Site Geological Map 1:25000 (Mineral Resources Tasmania)

3.2 Quaternary geology

3.2.1 Fill

This unit includes engineered fill (concrete, asphalt, bitumen and road base materials) and uncontrolled fill (waste building materials, excavated materials and anthropogenetic waste) and has been deposited over a 200-year time frame. The uncontrolled fill varies in material types and is highly variable in density and stiffness. The long history of the development of the site and the lag time between fill deposits has allowed the Derwent River to deposit estuarine soils between fill layers (predominantly Clays). The quality of engineered fill varies from ballast used in constructing piers from the mid to late 1800s (typically poorly graded, well-rounded material) to well-graded, locally sourced materials used in building roads.

3.2.2 Estuarine

The Estuarine deposits consist mainly of sand; a typical intersection consists of silty Sand with trace shells; some cohesive materials have also been identified within this unit. This deposit is higher in the stratigraphic column than the Alluvial material. The source of the Estuarine material is the Derwent River. This material is confined to the centre and to the south of the site. The central area thickness is highly variable and laterally irregular. The irregular distribution of the estuarine in the central area is inferred to have been caused by the excavation for the development and settling of the fill material.

3.2.3 Alluvial

The Alluvium logged encountered in the most recent investigation consists of granular soil (76%) and cohesive soil (24%). The Alluvial cohesive soils are confined to the main channel of the rivulet west of the mouth of the creek system. The material transported by the Hobart Rivulet into the paleo-channel is likely the further transported tertiary aged poorly-sorted large boulder to gravel grade deposit (noted as Tcbd on the 1:25k geological map of Hobart). The coarser grain cobble to boulder size materials are found along the deepest parts of the paleo-channel (thalweg), that approximately follows Evans Street in the vicinity of the southern site boundary. The cohesive materials were likely deposited during low-flow periods of the Park Steet and Hobart Rivulets. Future investigations may seek to refine the Alluvium further to separate the cohesive and granular soils, as 24% of the cohesive soils appear to make up a single large bed of clay.

3.3 Jurassic geology

3.3.1 Dolerite

The dolerite is a Jurassic-aged intrusive volcanic rock within the older Triassic Parmeener Supergroup. This intrusive rock is laterally extensive and represents a large area of Gondwana. Post Breakup these areas include Hobart, Antarctica, and South America. Due to uplift and extension, the Triassic sandstone and mudstone have been weathered away, exposing the dolerite on the surface. Typically the dolerite is in good condition, with high strength, it is slightly to moderately weathered, with confined zones of low Rock Quality Designation (RQD). This overall positive general assessment should accurately describe the dolerite across much of the site. Please refer to the following descriptions of lower rock quality features, and for their locations, please refer to sections and the 3D model.

The typical defects found within the dolerite are orthogonal jointing, vertical veins, and some vertical breccia pipes. Three known causes for these features are cooling of the Dolerite intrusion (Cooling joints or columnar jointing), hydrothermal fluids (younger fluid intrusion, possibly syntectonic) and regional normal faulting.

The observed hydrothermal breccia is found in BH-006 15.3 to 17.8 m, 19.0 to 21.3 m, and 22.8 to 24.5 m. A more ambiguous texture that could be hydrothermal breccia or a younger intrusion that incorporated and metamorphosed wall rock (either Dolerite or Parmeener Sandstone/Mudstone) can be found in BH-008 15.6–16.56 m. These textures indicate multiple pulses of intrusive volcanic and hydrothermal fluids.

The local tectonic processes have increased the pathways for meteoric water to dissolve and weaken calcite veining (examples of this can be found in BH001 5.0 to 6.3 m, BH-002 7.2 to 8.0 m, and BH-007 down to 12.8 m) and cause an increased depth of weathering along fracturing just below the top of the Dolerite contact (modelled as Extremely weathered). A deep, extremely weathered zone has been observed in BH-011, 6.0 m to 12.9 m, with multiple clay seams below this zone. This weathered zone contains slightly to moderately weathered core stones (Cobble to boulder-sized) supported in a clay matrix. More information is needed to determine whether this deep weathering is a localized feature controlled by a combination of veining, brecciation, and cooling joints or whether this deep weathering is connected to a more regional structure. We have modelled this as a localised feature based on the drilling and site observations. However, we strongly recommend that future investigations attempt to define or restrict this feature as it poses a potential hazard as a false top of rock due to the possible large (cobble-boulder) core stones. A crushed zone was observed in BH-013 that could be related to this feature and a more regional structure, which could make deep weathering more extensive on this site. More data is needed to confirm the connection between the two features. We have not modelled this way because the zone of deep weathering in BH-011 does not display offset, while the crushed zone does.

Our assessment of the dolerite reveals the presents of detrimental defects and weathering depths; however, they are currently confined to small areas of the model within the dolerite.

3.4 Structural geology

The Derwent Graben formed during the breakup of Gondwana (95–85 million years ago) and is one of several horsts and Graben structural settings in Tasmania (the Tamar Graben in northern Tasmania and Macquarie Graben on the west coast are others). All have structural trends in the general north-northwest direction. Similar structural settings in Victoria with the same age range are found within the Bass Strait and off the Gippsland and Otway coasts.

Central Hobart and some of its suburbs (including Macquarie Point) lie within the Derwent Graben, where the River Derwent has flowed for tens of millions of years. The horsts bordering the graben are higher ground to the West of Hobart, including Kunanyi/Mt Wellington, and lesser hills east of the river. Kunanyi/Mt Wellington and the Queens Domain Hill define the smaller horst and graben within the system in which Macquarie Point resides.

3.4.1 Faults

There is only one fault interpreted to be on site, a normal fault striking along the thalweg of the paleo channel under the Park Street rivulet. This structure is a splay (branch) of a north-west trending structure found on the Hobart 1:25,000 geological series map. This structure does not appear to influence ground mass quality on site (RQD results generally show an improvement in RQD score with increased depth). Some historical holes further to the West along the strike of the structure do show lower RQD values with depth.

The steep incline in the northwestern part of the site up to the Queens Domain represents the historical excavation of the local Dolerite and does not represent a geologically caused offset. The original site topography and morphology represent a gently sloping hill, which is consistent with the weathering down of the northeastern Horst (elevated section of the structural system).

3.4.2 Rock defects

Generally the rock defects encountered during the investigation were jointing of the dolerite. The dips of the joints are variable, ranging from 0° to 90° (horizontal to vertical). The majority of joints dip <10°, as illustrated below.

Zones of extremely weathered material were encountered in BH-001, BH-002, BH-006, BH-007, and BH-009. The extremely weathered zones had thicknesses of between 2 and 100 mm, with a clay infill. Pocket penetrometer results recorded 110 to 150 kPa. Vertical drilling preferentially selects horizontal jointing. The site has a high interaction with vertical structures when compared to other geological settings and sites.

3.5 Morphological features

3.5.1 Palaeochannel

During the late Tertiary and Quaternary periods, the Hobart Rivulet and its tributary Park Street Rivulet eroded their valleys through the Permian and Triassic rocks and Jurassic dolerite. This erosion process involved the transportation of mud, silt, sand and gravel into the graben. Many of these drainage lines are preferentially aligned along faults, a geological feature that results in the rocks being more weathered and erodible. The palaeochannel in the south of the project site aligned parallel to Evans Street is a prime example of such a channel.

The lower half of the deposits in this palaeochannel has been modelled as Alluvium (ALV). The Alluvium comprises cobble to boulder sized materials; these are located along the base of the channel, hard to very stiff clays (SPT N=R) at the centre of the unit and dense to very dense sands at the top (CPT did not penetrate the upper Alluvium and the mean SPT was N=40). The remaining natural soil modelled in the palaeochannel is Estuarine which consists of sands with minor silts and clays and variable CPT and SPT results (see Section 5 for more information).

3.5.2 Historic quarrying

The West side of the Queen's Domain Hill was a quarry for engineered fill and stone (the North-West area of the Macquarie Point site). Originally, the steep ascent from the site to the war memorial did not exist; it was a shallow graded slope. The quarrying of the hill has left the top of rock in this area to be irregular and less reliable than areas with natural contact. The low reliability of the top of rock in these areas is due to the lack of historical survey or design excavation levels – this is typical of early colonisation sites and a common problem with historically developed sites. More modern excavations without designs or surveys also reduce the confidence of the assessment of the top of rock surface and are in the north and central areas of the model where we have fill contacting the top of rock.

3.6 Hydrogeology

Groundwater level and chemistry assessment is the scope of others, however the groundwater level measurements were recorded, where encountered, during the borehole investigations. Typically groundwater was encountered in the central to eastern portion of the site between RL1.3 m AHD and RL 1.6 m AHD reducing to 0 m AHD at the south-eastern corner of the site.

Groundwater levels will depend on sea levels and fluctuations due to precipitation. Site levels should take into account design sea levels considering tidal and storm surge levels, as well as design event rainfall. Consideration on the effects of climate change on design levels is likely to be required.

4 Classification of soil and rock units

4.1 Geotechnical model

Based on the site investigation information obtained, the following geotechnical units have been assigned to the subsurface materials at Macquarie Point.

Table 4.1 Geotechnical model units

Unit	Unit reference	Material type	Stiffness / density / inferred strength
Fill Granular (Uncontrolled)	1A	Varaible in nature and composition, generally described as gravelly sand, clayey gravelly sand to silty sandy gravels. Overlies the entire site, varaible thickness across the site.	Variable
Fill Cohesive (Uncontrolled)	1B	Variable in nature and composition, generally described as silty clay, silty sandy clay or gravelly clay. Overlies the entire site, varaible thickness across the site.	Variable
Estuarine	2	Predominantly granular in nature but some cohesive material present. Generally described as a sand with variable secondary constituents of clay, silt and gravels. Confined to the southern and the central areas of the site.	Very Loose to Loose
Alluvium Granular	3A	Alluvium encountered was generally granular in nature, comprising predominantly sand and gravels with variable secondary constituents of clay, silts. Confined to the southern part of the site, trends in a SE-NW direction.	Medium Dense to Dense
Alluvium Cohesive	3B	The Alluvium is primarily granular in nature, but there are areas where cohesive material described as a sandy clay is interbedded with the granular deposits (Sheet 3 in Appendix B with cohesive material). Confined to the south-eastern area of the site.	Stiff to Hard
Extremely weathered Dolerite and Residual Soil	4A	Localised occurrences of extremely weathered dolerite are present on the site. Described as a gravelly clay or clayey gravel, fine to coarse grained, brown or grey-green, extremely weathered, very low to low strength rock. Generally found underlying the Fill, Estuarine or Alluvium.	Hard / Dense / Very Low Strength
Highly to moderately weathered Dolerite	4B	Described as a fine to coarse grained, dark grey-blue, crystalline, moderately to highly weathered. Generally overlies the fresh dolerite within the northern part of the site.	High and Very High Strength
Slightly weathered to Fresh dolerite	4C	Underlies the entire site described as a fine to coarse grained, dark grey-blue crystalline, fresh rock. Anticipated top of rock across the site varies from 8 m AHD at the north extent to -20 m AHD to the south-east of the site.	Very High Strength

5 Project ground conditions

5.1 Information

The interpreted geotechnical model as outlined in this section is based on the recent site investigation data comprising 20 boreholes and 17 CPTs, and historical site investigation data. Refer to Section 4 for the soil and rock units adopted for the project. The locations of intrusive investigations are provided in Appendix A. Reference should be made to the geotechnical long sections and cross sections provided in Appendix B, that have been cut through the 3-Dimensional geological model.

5.2 Subsurface conditions

The site is characterised by 3 zones – the Northern, Central, and Southern – as shown in Figure 5.1.

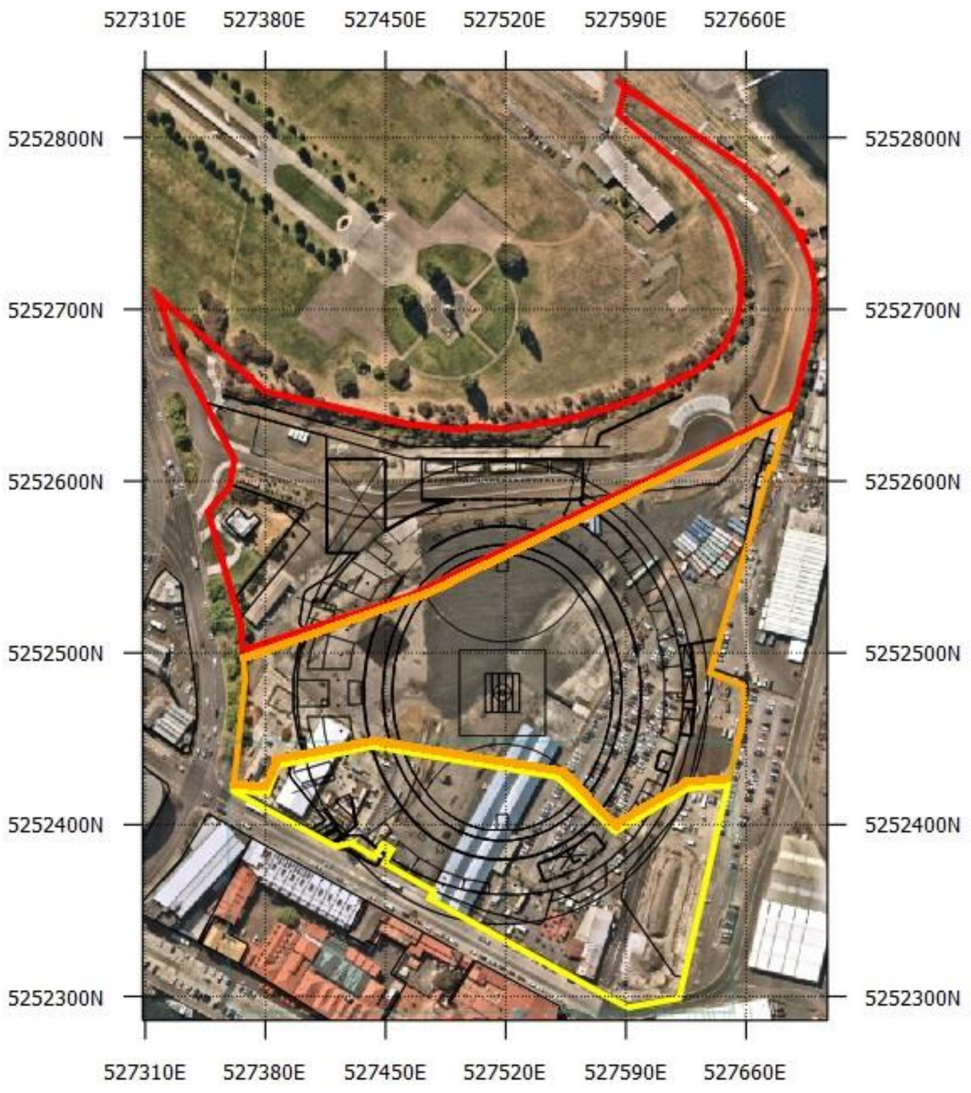


Figure 5.1 Site zones northern (red) – central (orange) – southern (yellow)

The Northern Zone has highly modified terrain from the original natural surface due to historic quarrying activity. The historic excavations removed the southern slope of the hill to the north of the site on which the Cenotaph and Regatta grounds are situated. The ground profile within this zone predominantly comprises uncontrolled Fill overlying Dolerite. The dolerite stratum was penetrated to a maximum depth of approximately 10 m during the investigation. The depth to competent dolerite (moderately weathered to Fresh) is typically shallow (about 3 m to 5 m). There is some increased uncertainty when assessing the Fill to Dolerite contact as the interface is manmade, and there is a lack of historical survey or design information indicating excavation extents. It is unlikely that the base of excavation is below high tide levels.

The Central Zone was likely a tidal wash zone and includes Fill overlying Dolerite, Fill overlying Estuarine deposits and Estuarine deposits overlying Dolerite. The Dolerite is Fresh to extremely weathered. The Dolerite contact in this area has more natural surfaces and is relatively flat (approx. 0.5 to 3°). The western area of this zone also includes a modified top of rock surface from excavation activities.

The Southern Zone comprises the area that is dominated by the underlying palaeochannel. This zone has a small area of uncontrolled Fill to Dolerite contact in the north-west. Fill in this zone predominantly overlies the Estuarine unit. The Estuarine unit overlies the Alluvium and Dolerite. The Alluvium overlies the Dolerite within the palaeochannel. In this zone, the Dolerite surface that defines the palaeochannel can be moderately steep; inferred to be about 25 degrees at the steepest part of the channel.

5.3 Geological model reliability

WSP has prepared a 3-dimensional geological model to inform the design of the Stadium and associated structures. Stratum boundary elevations and stratum thicknesses (isopachytes) and geological cross-sections, extracted from this model are presented in Appendix A and Appendix B.

It is important to understand factors which affect the reliability of the geological model. These factors include the quantity of site investigations available which affect the spacing between investigation points, the depths of the investigations which affect the available information on the various stratum boundaries and the purpose for which the various investigations were carried out which affects the quality of the logging (from a geotechnical perspective) and the types of in situ and laboratory investigations carried out. A further critical input to the geological modelling is the engineering geological understanding of the processes which have been involved in the formation of the existing strata and the constraints that these processes impose on the strata boundaries and engineering properties.

Prior to the current geotechnical investigation, a large number of previous investigations had been undertaken on the site. These investigations were mostly shallow and predominantly carried out for environmental purposes. Consequently, the pre-existing information on the highly variable fill material was extensive, providing information on its depth from some of the investigation points but only providing limited information on the geotechnical properties of the fill. Information on the strata underlying the fill was patchier although the general sequence of strata and the main site stratigraphic features of the site could be inferred, including the presence of the infilled palaeochannel extending across the southern zone of the site. Geotechnical understanding of the properties of the estuarine and alluvial soils and the properties of the dolerite rock and the variation of these engineering properties were poorly established prior to the current investigation.

The current investigation was planned following a detailed evaluation of the pre-existing site investigation information and taking into account preliminary information on the location of the planned stadium facility. The current investigation has greatly improved the understanding of the stratigraphy and key engineering properties of the site. Nevertheless, it is important to understand that the current investigation is preliminary. The updated geological model is considered to be suitable and of sufficient reliability for preliminary design of the facilities. Further targeted investigations are likely to be required for detailed design.

5.4 Site investigation data

To inform the material classification and unit characterisation, visual and tactile descriptions of core samples have been supplemented with in-situ testing during the site investigation and laboratory testing after the investigation. The in-situ and laboratory testing that has been used to characterise the subsurface material includes the following:

- Borehole in-situ testing comprising SPT testing. Refer to Appendix C for the figures providing the SPT N value for the different material units.
- CPT testing and interpretation outputs for individual locations is provided in Appendix D. For each CPT undertaken, the figures provide the measured cone resistance, measured sleeve friction and pore pressure traces, as well as the interpreted Soil behaviour type, friction angle, undrained shear strength, Young's modulus, relative density, and SPT N60 correlation.
- Laboratory testing results of bulk samples, bag samples, U63 samples and rock specimens are provided in Appendix E. The figures provide results of the following tests:
  - Atterberg limits
  - Moisture content
  - Particle size distribution (PSD) curves
  - 1-D consolidation test (Oedometer)
  - Unconfined Compressive Strength (UCS) and Point Load Test (PLT) results
- SPT N values and CPT Tip resistance q_c can be used in correlations to assess material properties such as the friction angle, material density, and design parameters such as pile shaft adhesion, end bearing and Youngs Modulus. The preliminary design values and assessed correlations are provided in Appendix F.

Detailed material descriptions are provided below.

5.5 Unit 1 – Fill

Fill overlies the entire site, with thicknesses ranging from 0.5 m to 13 m, but more typically between 5 m and 7 m. The fill in the northern zone of the site is under 2 m in thickness, increasing towards the southeast. The inferred base elevation ranges from 6 m AHD in the northwest to -6 m AHD in the southeast with the majority of the stadium footprint having a base of Fill elevation between RL -2 m AHD and RL 0 m AHD. Due to the site's history and the unknown origin and placement of the fill, it is considered "uncontrolled". Uncontrolled fill is fill which has not been selected, placed and compacted to a known and appropriate engineering standard under engineering supervision. Figures A4 and A5 (Appendix A) illustrate the inferred base and thickness of the fill across the site.

Table 5.1 presents a summary of the Atterberg limits and PSDs for the Fill unit. The fill is subdivided into cohesive and granular material, with mean values and ranges provided for the laboratory results. The laboratory results obtained for the grading and Atterberg limits indicate the variable nature of the fill unit. Four (4) Atterberg limit tests highlight the variability in the Fill behaviour, indicating generally high plasticity soil. One (1) result was non-plastic. Six (6) PSDs have been performed in the Fill at depths between RL 1.2 m BGL and RL 6.9 m BGL. The PSDs included of four (4) hydrometer tests – the results are illustrated on Figure E4 (Appendix E). Generally no clear trend is evident in the particle size distribution envelope highlighting the variability in the composition of the Fill.

Table 5.1 Summary of fill laboratory test data for the project area

Unit	Grading			Atterberg			
	% Gravel	% Sand	% Fines	Liquid limit	Plastic limit	Plasticity index	Linear shrinkage
1A (Granular)	35 (0–72)	45 (15–85)	20 (13–33)	49	27	22	5.5
1B (Cohesive)	13 (0–25)	41 (39–42)	47 (36–58)	77 (57–97)	24 (21–26)	54 (31–76)	11 (8–15)

Notes:

(1) The range of upper/lower bound values are provided with the mean shown in parentheses.

(2) Unit 1A granular material samples are mostly non-plastic.

Table 5.2 summarises the SPT N values obtained for the fill from historical and recent investigations.

Table 5.2 Summary of SPT N values for the Fill

Item	SPT (N)	
	Full data set	Refusal values removed
Data Points	52	47
Mean N Value	14	9
Median N Value	7	7
Interquartile Range (range between 25th and 75th percentile of the values)	5 to 13	5 to 11
Median q_c value (MPa)	1.9	
Mean q_c value (MPa)	2.0	
q_c range (MPa)	1.0 to 6.0	
Density	Predominantly Loose	

SPT values generally range from N=1 to 60, with a median value of 7. The interquartile range is N = 5 to 11, omitting refusal values. Due to the fill containing brick fragments/boulders of dolerite, which lead to SPT N refusals, the density has been interpreted based on both the full data set and refusal values, as outlined in Table 5.2. Based on the interquartile SPT N values, the relative density is interpreted as loose. Figure C2 (Appendix C) shows the SPT N values against depth for Fill, indicating that the majority of results are less than N = 10.

The CPT data revealed significant variation in cone tip resistance, indicating layered fill with a friction ratio generally varying between 1% and 10%, suggesting a mix of cohesive and granular material. The q_c values generally ranged from 1 MPa to 6 MPa (Figure F1 and Figure F2, Appendix F). Figures F1 and F2 show the same test data with Figure F2 plotted at an expanded qc scale to provide greater resolution at low resistance values). The CPT results indicate that the Fill consists of a mixture of cohesive, granular, and interbedded materials across the site with frequent large inclusions (e.g., cobble sized or larger man-made or rock materials).

CBR testing on 8 samples showed values ranging from 2% to 13%, with no recorded swell. Due to this variability and lack of swell values, a conservative design value of 2% is proposed, but we note that this is not consistent with the predominantly granular nature of the fill. Further testing is recommended following the establishment of pavement subgrade locations to refine the assessment of CBR for design purposes and to identify any foundation treatment or replacement requirements.

Eight (8) moisture content tests were scheduled at depths between 0.1 m BGL and 6.8 m BGL. The moisture content values range from 12.5% and 56.8% with a median of 32.2%. Figure E1 (Appendix E) provides the Atterberg limits and Moisture Content vs depth.

5.6 Unit 2 – Estuarine

The Estuarine deposits overly Alluvium at the southern end of the site and overlie Dolerite to the northeast of the site. The thickness ranges from 0.1 m to 11.0 m, is 1 m to 4 m in the central and north-eastern regions where present and is typically 6 m to 9 m in thickness in the southern zone as shown in Figure A7 (Appendix A). To the north thickness is between 1 m and 4 m thick. Generally, the inferred base ranges from RL 0.0 to -10.0 m AHD, Figure A6 (Appendix A).

The composition of the Estuarine deposits typically includes sand with some gravel and clay. According to the available PSD curves, approximately 70% of the sand falls within the fine sand range, as illustrated in Figure E5 (Appendix E). The deposit generally falls above the A-line in the low and high plasticity domains, as illustrated on Figure E2 (Appendix E). Review of the CPT Soil Behaviour Type assessment provided in Appendix D indicates the deposits can be interbedded, which will affect the behaviour of the Unit and how structures founded on the Unit perform. Consolidation, Strength, Stiffness and bearing capacity will all be influenced by the interbedded nature of the Unit.

Table 5.3 presents a summary of available geotechnical laboratory data for the Estuarine soil. Both mean and range of values are recorded.

Table 5.3 Summary of estuarine laboratory test data for the project area

Unit	Grading			Atterberg			
	% Gravel	% Sand	% Fines	Liquid limit	Plastic limit	Plasticity index	Linear shrinkage
2	3 (1–5)	80 (70–94)	17 (5–26)	56 (30–76)	21 (13–30)	35 (17–46)	11 (4–16)

Notes:

(1) Atterberg limits values are of limited use in the predominantly granular Estuarine unit but they indicate some material sampled that has been classified as Estuarine is fine grained and plastic in nature.

Table 5.4 summarises the SPT N values obtained for the Estuarine unit from historical and recent investigations.

Table 5.4 Summary of Estuarine SPT values

Item	SPT (N) Full Data Set	SPT (N) Without N=60
Data Points	79	71
Mean N Value	12 (medium dense)	7 (loose)
Median N Value	4	4
Interquartile Range	3 to 11	3 to 9
Mean qc value (MPa)	2.2	
Median qc value (MPa)	1.8	

Item	SPT (N) Full Data Set	SPT (N) Without N=60
Qc range (MPa)	1.0 to 4.0	
Density	Very loose to loose	

SPT values generally range from N=0 to 60, with a median value of 4. The interquartile range is N =3 to 9 when omitting refusal values. Table 5.4 summarises the SPT N values obtained for the Estuarine deposit from historical and recent investigations. Refusal values were excluded due to potential cobble or boulder sized obstructions, which may not be representative of the material in general. Based on the median SPT N values, the relative density can be interpreted as very loose to loose. The SPT plots are presented in Figure C3 (Appendix C).

Generally, the tip resistance for the CPT probe was below 4 MPa with a friction ratio of generally <2%, indicating the Estuarine deposits comprise loose and very loose sand. Figure F1 and Figure F2 (Appendix F) provide an overlay of the cone tip resistance traces and an inferred preliminary design value for general behaviour assessment. Location-specific data should always be used when designing specific structures. Generally, q_c values exceeding 3.0 MPa were encountered at the boundary between the Estuarine and Alluvial deposits.

5.7 Unit 3 – Alluvial

The Alluvial unit is confined to the southern zone of the site and lies within the general east-west trending main palaeochannel. The thickness of alluvium along the centre of the palaeochannel varies between 6 m and 10 m, with maximum thickness being to the south-east of the site. Generally, the base of the Alluvium is between RL -14 and -20 m AHD, with the upper surface of alluvium typically between RL -6 m AHD and RL -10 m AHD. Figures A8 and A9 (Appendix A) present the thickness and inferred elevation base of the Alluvial deposits.

The composition of the Alluvial deposit is predominantly sand and gravels with approximately 5% fines. Two PSDs completed in the Alluvial deposits indicate that the soil is either a medium-coarse grained sand or sandy gravel. The PSD plots are presented in Figure E5 (Appendix E). No other testing has been completed in the Alluvial deposit due to the limited sample recovery achieved in this material.

Table 5.5 presents a summary of available geotechnical laboratory data for the Alluvial soil. Only the range of values are recorded as the mean values would not represent the typical distribution.

Table 5.5 Summary of alluvial laboratory test data for the project area

Unit	Grading		
	% Gravel	% Sand	% Fines
3	(2–64)	(32–93)	(4–5)

Table 5.6 summarises the SPT N values obtained for the Alluvium from historical and recent investigations.

Table 5.6 Summary of Alluvial SPT values

Item	SPT (N) All	SPT (N) Without N=60
Data Points	21	10
mean N Value	40	19
Median N Value	60	16
Interquartile Range	19-60	9-24

SPT values generally range from N= 6 to 60, with a median value of 60, when omitting refusal values the median value is 16 and the interquartile range is 9–24. A plot of the historical and recent SPT results are outlined in Figure C4 (Appendix C). Table 5.6 provides a summary of the SPT N values obtained for the Alluvium from historic and recent investigations. Generally, SPTs refused in the Alluvium indicating either: 1) material too coarse for the SPT sampler to penetrate (cobble sized or coarser; 2) a very dense for granular material (sand or sandy gravel); or 3) a hard cohesive material. If the evaluation is reassessed, omitting the values of N=60, for the remaining SPT values are indicative of medium-dense granular deposits and/or very stiff cohesive material. Additionally, there are some isolated pockets of lower SPT values (<10) within the Alluvium, spatially the lower SPT values occur in deeper (larger thicknesses) of Alluvium. Our judgement, based on the available data, is that the Alluvium is most likely to be a medium dense granular material with a grading that varies vertically and laterally from sand and gravel through to cobbles and boulders, probably within a sand or gravel matrix.

All CPT data refused on the Alluvial deposit, evident by a marked increase in tip resistance for the CPT probe, followed by the CPT being terminated due to the risk of damage to the CPT tip. It is inferred that practical refusal on gravel or cobble material occurred.

5.8 Unit 4 – Rock (dolerite)

Unit 4 is divided into sub-units according to weathering as follows:

- 4A (residual soil and extremely weathered dolerite)
- 4B (moderately to highly weathered dolerite)
- 4C (fresh to slightly weathered dolerite).

Dolerite underlies the entire site with depth of top of rock varies across the scheme from RL 4 m AHD in the northern zone of the site to RL -16 m AHD to the south-east of the site, as depicted on Figure A12 (in Appendix A). This figure provides the inferred top of rock surface (i.e., top of highly weathered or better dolerite).

Unit 4A comprises extremely weathered dolerite and residual soil. The extremely weathered dolerite occurs in isolated pockets across the site generally of thickness ranges between 1 m and 5 m (refer Figure A11 in Appendix A). The base of the extremely weathered dolerite varies between RL -6 m AHD in the middle of the site and RL 10 m AHD to the north of the site (refer Figure A10 in Appendix A).

Unit 4B comprises moderately and highly weathered dolerite, of thickness ranging between 1 m and 6 m, with a typical thickness between 3 m and 4 m, as illustrated on Figure A13 in Appendix A).

Point load testing (PLT) was carried out on Units 4B and 4C at approximate 1 m intervals and UCS strength tests were undertaken on selected rock samples. Results from the recent investigation are outlined below in Table 5.7. PLT tests were undertaken on irregular lumps of dolerite obtained during sonic coring. PLT tests were undertaken both axially and in a diametral manner on cored dolerite. It should be noted that many of the PLT test values for Unit 4C were limited by the capacity of the PLT machine rather than by rock breakage. Hence many of the recorded PLT values are “greater than” readings.

The results indicate the dolerite encountered is typically very high strength rock. To assess UCS strength based on the PLT tests undertaken, a comparison of axial PLT results with nearby UCS values was undertaken (refer Figure E7 in Appendix E). Additionally, plots of UCS, PLT axial, and diametral values are provided in Figure E8 (Appendix E).

Table 5.7 Summary of geotechnical laboratory rock test data

Item		Unit 4B	Unit 4C
PLT (Axial)	Data Points	9	87
	Mean $I_{s(50)}$ Value	4.4	5.7
	Median $I_{s(50)}$ Value	5.3	5.9
	Range	0.6–7.16	0.14–9.31

Item		Unit 4B	Unit 4C
PLT (Diametral)	Data Points	10	88
	Mean $I_{s(50)}$ Value	5.6	6.2
	Median $I_{s(50)}$ Value	6.0	6.2
	Range	3.79–6.94	1.04–10.17
PLT (Irregular)	Data Points	5	2
	Mean $I_{s(50)}$ Value	1.3	2.7
	Median $I_{s(50)}$ Value	1.0	2.7
	Range	0.45–2.74	1.14–4.23
UCS (MPa)	Data Points	0	15
	Mean Value	N/a	120.6
	Median Value	N/a	114.0
	Range	N/a	23–229

The assessment of Rock Quality Designation (RQD) with respect to weathering for the units is based on historical and current investigations. RQD is expressed as a percentage of intact rock core pieces greater than 100mm length, with respect to the total length considered (usually the core run). Table 5.8 provides the RQD values separated into material unit, to assess the amount of fracturing that is typical for each material unit. As an example, 10.4 m of core was classified as being within the Unit 4A unit. 7.1 m of the 10.4 m had a RQD between 0% and 10%. So 68% of the core that had RQD data that was assessed to fall within the Unit 4A unit has an RQD of 0% to 10%. This data can be used to infer what might be the typical RQD range for each material unit.

A histogram of weathering and RQD is presented in Figure E9 (Appendix E).

Table 5.8 Summary of RQD against weathering class

Unit*		4A		4B		4C	
Weathering		RS-XW		MW-HW		FR-SW	
Aggregate length of cores (m)		10.4		53.7		159.0	
		Length (m)	%	Length (m)	%	Length (m)	%
RQD Band (%)	RQD = 0 to 10	7.1	68%	20.5	38%	12.3	8%
	RQD = 11 to 20	0.0	0%	1.2	2%	3.9	2%
	RQD = 21 to 30	0.0	0%	0.3	1%	6.5	4%
	RQD = 31 to 40	0.0	0%	3.8	7%	6.7	4%
	RQD = 41 to 50	1.9	18%	4.4	8%	8.2	5%
	RQD = 51 to 60	0.5	5%	1.8	3%	4.5	3%
	RQD = 61 to 70	0.9	9%	8.9	16%	6.8	4%
	RQD = 71 to 80	0.0	0%	4.7	9%	10.0	6%
	RQD = 81 to 90	0.0	0%	4.0	7%	18.5	12%
	RQD = 91 to 100	0.0	0%	4.2	8%	81.6	51%

*Data has been obtained from predominantly the current investigation though also supplemented with historical investigation data where applicable.

Unit 4A generally exhibits RQD values within the 0 to 10% band, indicating its highly fractured and lower strength nature. Some values are recorded between 41% and 70%, suggesting the presence of more competent dolerite or potential inaccuracies in historical data classification.

Unit 4B shows a significant amount of highly fractured material, with 38% of RQD values falling within the 0 to 10% range. However, values are relatively evenly spread across all other bands.

Unit 4C predominantly displays RQD values in the 70+ band, indicating competent material. However, lower RQD values are observed occasionally, primarily associated with crush seams and fault zones.

The overall trend is that with decreasing weathering the RQD increases, i.e., reduced number of rock fractures.

5.9 Ground-related risk

A summary of geotechnical risks and potential mitigation measures is provided in Table 5.9. The key geotechnical risks have been developed from the issues raised in the sections above. This section does not encompass all geotechnical risks, but aims to highlight significant items. The risks vary in severity due to the changes in ground conditions across the site. Project risk related to site contamination is not part of this scope and is reported elsewhere.

Table 5.9 Summary of key risk items and mitigation measures

No.	Issue	Risk item	Discussion	Potential mitigation measure
1	Total and differential ground settlements of soils	— High lateral variability within fill (buried foundations – loose/soft clayey fill) — Collapse settlement of fill — Consolidation of cohesive soils — Creep settlements of granular soils	— Excessive total settlement can adversely affect structures and services. Particularly when uncontrolled fill introduces a high degree of uncertainty. The presence of degradable materials in fill (e.g., timber, rubber, other putrescible items) can result in localised settlement. — Lateral variability of thickness and density of natural deposits can result in differential settlement — Collapse settlement of the fill may occur where increased loading causes and groundwater level rise causes compression of fill through realignment of the granular particle matrix. — The ongoing creep of the foundation materials should be considered and the potential effects on the structures. — Ground movements after pile installation can induce unexpected loading.	— Uncontrolled fill may need to be treated to achieve performance requirements, through excavation and replacement, foundation improvement (e.g., high energy impact compaction, rapid impact compaction, dynamic compaction) or engineered inclusions (e.g., stone columns, vibro-compaction) — Bridging layers or load transfer platforms can be an option to mitigate risk of variable foundation performance over short distances. — Founding structures though the compressible stratum onto bedrock — Design structures that can tolerate the predicted settlement, incorporating sensitivity assessments for local variability. — Undertake earthworks prior to installation of piles so that earthworks induced ground movements do not adversely affect constructed piles.
2	Seismic effects on granular soils – liquefaction and lateral spreading	— Liquefaction is likely to lead to settlements of granular strata — Liquefaction may cause lateral spreading	— Ground movements induced by earthquakes could trigger liquefaction in loose granular layers below the water table. Settlement and loss of strength could adversely affect structures and civil works. — Due to the proximity to the Derwent River, there is potential for lateral spreading of land towards the River in seismic events.	— Footings should be designed to consider the effects of liquefaction on the structures. — A lateral spreading assessment can be undertaken to assess if it is a credible design scenario for the site.

No.	Issue	Risk item	Discussion	Potential mitigation measure
3	Effects of high groundwater levels / flooding	— Potential to initiate collapse settlement of fill — Flotation of buried structures — Disruption of drainage provisions	— The potential increase in groundwater level can exert an uplift force on the structures, causing flotation issues of tanked structures. — The reduced effective strength and loss of soil suction associated with an increase in groundwater level could result in particle realignment (collapse settlement).	— Design of footings should consider the pressures from hydrostatic uplift and either be sufficiently heavy to counteract the uplift forces or have retention systems in place (anchors, piles) to resist the uplift forces. — Drainage measures can be put in place to reduce the buoyancy effect (i.e., through the use of drained basement design) — Drainage systems should consider potential levels of groundwater and water levels at outlet points to prevent system backup.
4	Buried obstacles to piling	— Various types of large obstructions in fill — Boulders in palaeochannel	— The presence of boulders and cobbles in the palaeochannel could adversely affect the installation of piles, preventing them from reaching target depth and achieving design loading.	— Pile construction methods should consider the potential for refusal above target levels. Mitigation measures may include the use of appropriately sized piling rigs to penetrate dense materials, the use of boring and temporary casing to retain collapsible materials, and the use of driving shoes for driven piles. — Bored piles are assessed to have a lower risk of founding on a boulder or cobble layer given the ability to excavate through such layers and achieve the design socket length. — For driven piles, testing should be undertaken to verify load capacity. — Where there is significant variability of driven pile refusal from the design level, consideration should be given to installation of additional piles due to the uncertainty associated with verification of founding stratum. — End bearing assessments for piles should consider the potential for adverse founding conditions.

No.	Issue	Risk item	Discussion	Potential mitigation measure
5	Variable Rock Strength for foundations	— Variable rock socket capacities — Variable capacities of spread foundations on rock	— The potential variable weathering profile of the founding stratum could result in variable performance for adjacent piles and adverse performance. — The potential variable weathering across a spread footing could result in adverse performance.	— Sufficient socket length into founding stratum (min. 2 pile diameters) should be adopted to develop design capacity values. — Additional site investigation may be required at critical locations where boreholes have not been undertaken to verify ground conditions. — Sensitivity analyses should be undertaken to consider potential variability in weathering and strength. — Sufficient embedment into founding stratum for spread footings should be undertaken to verify consistent conditions across the footing. Coring of rock beneath the footing can be undertaken to assess the presence of weathered seams and fractured zones.
6	Rock slope instability	— Potential for rockslides of various dimensions to occur	— Where rock cuts are present on the site, there is the potential for wedge instability where rock fragments can slide or topple out of the face and impact the area at the toe of the slope.	— The face of all cuts should be mapped by an engineering geologist or geotechnical engineer to assess risk of rock wedge failure. — Where there is a risk of adverse rock wedges affecting structures or the public, measures should be implemented to reduce the risk such as flatter batters, permanent rock support and/or catch fences.

6 Design approach and method

The following section discusses the approach, method and assumptions adopted in deriving geotechnical design parameters and developing geotechnical input for the design of structures addressed in this report.

6.1 Derivation of geotechnical parameters

The geotechnical parameters were derived based on the interpretation and correlations with available borehole, SPT, CPT, and laboratory testing data. CPeT-IT (version 3) is a commercially available software package which was used to interpret and infer various soil properties based on available CPT data. The interpreted CPT plots are provided in Appendix D and other plots derived from the interpretation of other borehole and laboratory test information are presented in Appendix C, Appendix E, and Appendix F. The sections below provide additional commentary around the derivation of key geotechnical parameters.

6.1.1 Effective friction angle

The effective friction angle (φ') can be obtained from the CPeT-IT software package based on correlations to CPT data as presented in Robertson and Cabal (2002). However, in our experience, we note that these can occasionally over-estimate the peak friction angle in some instances. Therefore, the design friction angle was reviewed with consideration of the following additional correlations listed below:

- empirical correlations with SPT-N values as presented by Kulhawy and Mayne (1990); and
- AS4678-2002 Table D2 which takes into account angularity, grading, and SPT-N values.

6.1.2 Undrained shear strength

The undrained shear strength (S_u) for cohesive units was primarily derived based on CPT cone tip resistance based on the following correlation:

$$S_u = \frac{q_t - \sigma_v}{N_{kt}}$$

Where:

- q_t is the corrected cone tip resistance;
- σ_v is the vertical effective stress;
- N_{kt} is the cone factor, which may range between 10 and 20 depending on the soil type encountered.
  - N_{kt} =14 was adopted and considered appropriate for the type of soils encountered.

Correlations with SPT-N values based on published approximate variation in consistency and undrained shear strength for clays as presented by Terzaghi and Peck (1967)), and other commonly adopted empirical correlations (e.g., $S_u/N = 5$) were also used and considered in developing the design undrained shear strength parameters.

6.1.3 Stiffness

The elastic ground stiffness parameters were assessed using correlations between the constrained modulus (M) and CPT cone tip measurements. A corresponding Young's Modulus (E') profile was also obtained by considering the corresponding Poisson's Ratio (refer to Section 6.2 for more details).

Additionally, E' values were also compared against multiple in situ data sources, which include:

- estimation using SPT-N values based on E'/N = α, where α is between 1.6 and 2.3
- estimation based on cone tip resistance based on $E' = \beta q_c$, where β is between 4 and 7
- correlations based on dilatometer (DMT) testing
- directly measured during the UCS test (rock cores only).

These elastic stiffness parameters (i.e., E' and M') may be used to assess elastic settlement of granular materials, stiff/over-consolidated clays and rock (i.e., Units 1, 3, and 4) where the settlement within these layers is expected to occur relatively quickly after loading. The stiffness of the unload/reload modulus (E_{ur}) is generally stiffer and is suggested to be taken as 5 times the loading elastic modulus (E') for sands and 3 times the virgin loading elastic modulus (E') for clays. For Unit 2 soils, however, use of a simple elastic model is less appropriate and deformation due to consolidation should be considered. Further details are discussed in Section 6.1.4.2 below.

It is important to note that ground stiffness is a soil property which is strain dependent. Where strains are small, the soil stiffness tends to be high and conversely, where strains are large, the soil stiffness reduces. It is important that the recommended stiffness values are reviewed during detailed design to ensure they are applicable for the level of strain anticipated for the structure.

6.1.4 Consolidation parameters

Consolidation is the process where water is expelled from the soil matrix which results in deformation (settlement) over time. Depending on the permeability and thickness of the soil, consolidation can occur relatively quickly (e.g., in sands, gravels and other granular soils) or can take place over an extended period of time (e.g., for soft clays or other cohesive/fine grained soils).

As previously mentioned in Section 6.1.3 above, consolidation is likely to occur relatively quickly and therefore can be modelled using elastic theory for Units 1, 3, and 4. Although the Unit 2 estuarine deposits mostly comprise granular soils, it was also observed to contain fine cohesive soils in variable proportions and at various depths. These cohesive sub-units are likely to affect the permeability and compression behaviour of the Estuarine Unit. This has been considered in the derivation of relevant Unit 2 consolidation parameters and further details are discussed below.

6.1.4.1 Primary consolidation settlement

Consolidation settlements may be assessed using the approach originally developed by Terzaghi (1927) where the Compression Index (C_c) and Recompression Index (C_r) are applicable where soil loads are above or below the in-situ pre-consolidation pressure, respectively (i.e., where OCR is equal to, or greater than 1).

Laboratory oedometer testing was undertaken on Unit 2 Estuarine soil samples and the results were used to assess the C_c value for various vertical effective stress ranges. Other correlations with moisture content, plasticity index and liquid limit (e.g., by Wroth and Wood (1978), Koppula (1981) and Wilkes (1974)) were also adopted based on available laboratory test results to refine the potential range of C_c values. USACE (1990) defines typical compression indices for various soil types, including uniform loose and dense sands as well as silts, among others, which was also used as a check against the recommended parameters.

The C_r values were obtained by adopting a C_c/C_r ratio of 5, which is considered appropriate for Unit 2 Estuarine soils and consistent with results of the oedometer testing and typical values presented in the literature.

6.1.4.2 Creep settlement

Creep or secondary settlement may be modelled for Unit 2 Estuarine soils to represent the ongoing, time-dependent settlement which occurs towards the end of Primary Consolidation (e.g., once 90% of Primary Consolidation has completed). Creep is typically modelled as a function of log time and the Creep Index (C_α) was determined by considering the correlation relationship to the Compression Index, C_c, as developed by Mesri and Godlewski (1977), where:

$$\frac{C_{\alpha NC}}{C_c} = 0.02 \pm 0.01$$

It is noted that this approach is typically applied to cohesive soils and the Estuarine soils are predominantly granular in nature. Burland and Coatsworth (1987) provide an alternative procedure that may be considered for assessing settlement of foundations in sands and gravels.

Additionally, long term creep may occur within uncontrolled or poorly compacted existing or new fill and is estimated to be in the order of 0.2%H per log time cycle, where H is the thickness of the fill.

6.1.5 Coefficient of consolidation

The coefficients of consolidation (C_v and C_h) were developed through reviewing the results of laboratory oedometer testing and in-situ dissipation test data completed at selected CPT locations. A C_h/C_v ratio of 2 was adopted to recognise the anisotropic nature of most natural soil deposits. These tests predominantly targeted the Unit 2 Estuarine soils and resulted in relatively large range of inferred C_v and C_h values, which reflects the highly variable nature of the granular/fines mixture observed within the estuarine deposits.

6.1.6 Rock strength

Unconfined Compressive Strength (UCS) testing was undertaken on fifteen dolerite rock core samples which resulted in UCS values of between 23 MPa to 229 MPa. A median UCS value of 114 MPa was obtained and it was noted that the lower values were recorded due to failures influenced by defects and may not be reflective of the overall rock mass strength or quality.

Point Load Testing (PLT) was also undertaken at 1 m depth intervals of recovered rock core to obtain an Is_{50} point load strength index. The Is_{50} values can be used to estimate the Unconfined Compressive Strength (UCS) of the rock by applying a UCS:Is_{50} ratio. Based on a review of available UCS and Is_{50} test results, as shown in Figure E7 (Appendix E), a ratio of 19:1 was adopted which correlates to UCS values of 2.7 MPa to 380 MPa for the dolerite rock. This is considered appropriate for the purpose of foundation design for the proposed development.

6.1.7 Rock parameters

For the assessment of preliminary rock strength parameters, Hoek Brown parameters have been assessed by considering the assigned UCS value of the rock unit and assessing the mass properties through the use of ROCLAB. The inferred Geological Strength Index (GSI) for each rock unit has been used in combination with the Intact Rock Modulus and a figure by Hoek and Diederichs to assess Rock Mass Modulus, utilising a disturbance value of 0.7.

6.2 Geotechnical design parameters

The suggested geotechnical design parameters for soils are summarised in Table 6.1. The consolidation parameters for Unit 2 Estuarine soils are presented in Table 6.2.

Table 6.1 Geotechnical design parameters for soils

Unit	Unit name	Bulk unit weight, γ (kN/m^3)	Undrained shear strength, S_u (kPa)	Cohesion, c' (kPa)	Friction angle, Φ' (deg)	Drained elastic modulus (E') (MPa)
1A	Uncontrolled Fill (Granular)	17 to 19 (18)	-	0	30 to 35 (32)	5 to 20 (10)
1B	Uncontrolled Fill (Cohesive)	17 to 19 (18)	20 to 120 (40)	2 to 7 (3)	25 to 30 (28)	5 to 20 (8)
2	Estuarine (Granular)	16 to 18 (17)	-	0	28 to 34 (30)	3 to 10 (6)
3A	Alluvium (Granular)	18 to 20 (19)	-	0	32 to 36 (34)	20 to 60 (40)

Unit	Unit name	Bulk unit weight, γ (kN/m^3)	Undrained shear strength, S_u (kPa)	Cohesion, c' (kPa)	Friction angle, Φ' (deg)	Drained elastic modulus (E') (MPa)
3B	Alluvium (Cohesive)	18 to 20 (19)	50 to 200 (75)	2 to 7 (5)	25 to 30 (29)	10 to 50 (20)

Notes:

(1) Parameters reported as a range and recommended design value, i.e., Min to Max (Selected)

(2) If required, unload reload elastic modulus (E_{ur}) is suggested to be taken as 5 times the loading modulus (E') for granular materials and 3 times the virgin loading modulus (E') for cohesive materials.

(3) Relationship between E_u and E' can be taken as $E_u = 3E'/2(1+\upsilon)$

(4) Sensitivity of 50% and 200% on provided elastic modulus values should be considered in design to assess structure sensitivity to variation in material stiffness.

(5) Poisson's ratio (υ) of 0.3 is considered appropriate for drained materials. Suggest υ_u ~ 0.5 for the undrained condition in cohesive materials.

Table 6.2 Consolidation parameters for Unit 2 Estuarine soils

Unit	Unit name	Initial void ratio (e_0)	Coefficient of consolidation, C_v (m^2/yr)	Compression index, C_c	Recompression index, C_r	Creep index, C_α
2	Estuarine	0.7 to 1.0 (1.0)	0.5 to 80 (5)	0.1 to 0.3 (0.2)	0.001 to 0.042 (0.04)	0.0001 to 0.003 (0.005)

Notes:

(1) Parameters reported as a range and recommended design value, i.e., Min to Max (Selected)

Indicative intact and rock mass properties for weathered dolerite units are presented in Table 6.3. Site specific and structure specific conditions should be checked to ensure that the preliminary values provided are suitable for the assessment undertaken.

Table 6.3 Preliminary rock strength design parameters

Parameters		Unit 4B	Unit 4C
Intact Rock Parameters	UCS (MPa)	40	100
	Poisson's Ratio, υ	0.2	0.15
	Unit Weight (kN/m^3)	25	26
	mi	16	16
Block Scale (e.g., 1m^3)	GSI	35 to 45	75 to 85
	Rock Intact Modulus (GPa)	60	85
	Rock Mass Modulus (GPa)	6	60
Hoek Brown Parameters	Mb	0.592	5.332
	s	0.0002	0.0551
	a	0.511	0.501

6.3 Soil aggressivity to concrete and steel

The laboratory tests were compared with the guidelines for durability presented in Table 6.4.2 and 6.5.2 of AS2159-2009 *Piling – Design and Installation.* A summary of the aggressivity exposure classification for the soil is presented in Table 6.4 below. The groundwater table has been assumed to be 3 m BGL, if the groundwater table changes revision of soil conditions may be appropriate.

Table 6.4 Summary of exposure classifications for steel and concrete piles

Sample ID	Depth from (m)	Depth to (m)	Geology	Material description	Soil condition type	Exposure classification	
						Steel	Concrete
BH-001	0.3	0.4	FILL	Gravelly SAND (Fill)	B	Non-Aggressive	Non-Aggressive
BH-001	1.1	1.2	FILL	Clayey Gravelly SAND (Fill)	B	Non-Aggressive	Non-Aggressive
BH-002	2.7	2.8	FILL	Sandy Gravelly CLAY (Fill)	B	Non-Aggressive	Non-Aggressive
BH-003	1.8	1.9	FILL	Sandy GRAVEL (Fill)	B	Non-Aggressive	Non-Aggressive
BH-004	12.5	12.6	ALV	Sandy Gravelly CLAY	B	Mild	Non-Aggressive
BH-004	15.6	15.7	ALV	Sandy CLAY	B	Moderate	Non-Aggressive
BH-005	2.2	2.3	FILL	Silty Clayey SAND (Fill)	A	Mild	Mild
BH-006	4.2	4.3	FILL	Sandy CLAY (Fill)	B	Mild	Non-Aggressive
BH-007	5.2	5.3	FILL	SAND (Fill)	A	Severe	Mild
BH-008	5.9	6	DOL	SAND with gravel (XW rock)	A	Mild	Mild
BH-009	4	4.1	FILL	Clayey SAND (Fill)	A	Mild	Mild
BH-010	4.4	4.5	EST	Silty CLAY	B	Non-Aggressive	Non-Aggressive
BH-011	6	6.1	DOL	SAND with gravel (XW rock)	A	Severe	Mild
BH-012	3.5	3.6	FILL	Silty Sandy CLAY / Clayey Silty SAND (Fill)	B	Non-Aggressive	Non-Aggressive

Sample ID	Depth from (m)	Depth to (m)	Geology	Material description	Soil condition type	Exposure classification	
						Steel	Concrete
BH-013	0.5	0.6	FILL	Sandy Silty Gravel (Fill)	B	Non-Aggressive	Non-Aggressive
BH-014	4.7	4.8	EST	Clayey SAND	A	Mild	Mild
BH-015	3.7	3.8	FILL	SAND (Fill)	A	Non-Aggressive	Mild
BH-016	3.7	3.8	FILL	Gravelly CLAY	B	Moderate	Non-Aggressive

Notes:

(1) For the assessment of Soil Condition A or B, a groundwater level of RL 1.6 m AHD has been assumed across the site. The designers should review the Soil Condition on a location-by-location basis.

6.4 Seismic liquefaction assessment

In accordance with AS1170.4-2007, a pseudo-static analysis for an annual exceedance (AEP) of 1/500 and 1/2500 has been completed using C-liq software. The adopted characteristics, in accordance with AS1170.4-2007 are outlined below in Table 6.5.

Table 6.5 Summary of earthquake characteristics for an AEP of 1/500 and 1/2500

Characteristics	Value	Justification (if applicable)
Hazard factor (z)	0.08	
Site sub-soil classification	Ce	Applicable for 3 m depth or more of weathered rock or soil
Site class factor (Ch(T=0s))	1.3	Table 6.4 from AS1170.4-2007
Probability factor (k_p)	1.0	1 in 500 AEP
	1.8	1 in 2500 AEP
Importance Level	3	AS1170.0
Earthquake Magnitude	5.5	1 in 500 AEP; assessed from historical earthquake data within the Hobart area from 1958 to current day.
	6.5	1 in 2500 AEP; assessed from historical earthquake data within the Hobart area from 1958 to current day.

Based on the parameters above a design horizontal Peak Ground Acceleration (PGA) for a 1 in 500 year AEP and 1 in 2500 year AEP are outlined below based on $PGA = Z * (\mathrm{Ch}(\mathrm{T} = 0\mathrm{s}) * k_p$:

A cyclic liquefaction using C-liq software was performed to understand the cyclic liquefaction potential of the soils within the CPT tests. The procedure adopted is outlined below:

- "Simplified procedure – Seed & Idriss"
- obtain seismic demand, cyclic stress ratio (CSR) and cyclic resistance of sand (CRR)
- compare resistance with demand.

Seismic liquefaction analysis has been performed under a 1 in 500 yr Annual Exceedance Probability (AEP) and 1 in 2500 yr AEP. No liquefaction is assessed to occur in the 1 in 500 yr but there was the risk of liquefaction in the 1 in 2500 yr AEP as outlined in Table 6.6. Refer to Appendix G for outputs of the liquefaction assessment.

Table 6.6 Summary of C-liq results

AEP	PGA	Risk of liquefaction
1 in 500 year	0.104g	No
1 in 2500 year	0.187g	Yes CPT-04a*, CPT-05, CPT-07*, CPT-08*, CPT-13, CPT-13a, CPT-14, CPT-16, CPT-17

*Less than 0.2 m

Under the 1 in 2500 case, lenses of sand of thickness between 0.01 and 2.63 m liquefy, the adopted factor of safety was 1.1. Beds less than 0.20m has been considered as a negligible risk. Table 6.7 outlines the CPT locations at risk of potential liquefaction, the depth and thickness at which liquefaction could occur.

Table 6.7 Depth and thickness of potential liquefaction

CPT	Depth of liquefaction (m bgl)	Thickness (m)
CPT-05	5.81–6.02	0.21
	6.19–6.39	0.20
	6.98–9.33	2.35
	9.39–11.14	1.75
CPT-13	6.05–6.35	0.30
	6.51–7.25	0.74
	7.44–7.90	0.46
CPT-13a	8.80–9.60	0.80
	10.02–10.76	0.74
	11.22–11.55	0.33
	12.38–12.84	0.46
CPT-14	6.70–8.84	2.14
CPT-16	4.02–4.25	0.23
	4.60–4.89	0.29
	5.08–7.71	2.63
	8.62–9.72	1.10
	10.49–12.18	1.69
CPT-17	5.79–6.13	0.34
	8.49–9.37	0.88
	9.72–10.67	0.95
	11.10–11.37	0.27

Thin and impersistent granular layers are less likely to undergo liquefaction than thicker, potentially continuous, layers. Hence thin potentially liquefiable layers identified by CLiq in the Fill material are less likely to liquefy in practice. Most of the thicker layers indicated as potentially liable to liquefy occur in the Estuarine stratum. Potentially liquefiable layers with thicknesses greater than about 0.8 m in the Estuarine soils merit further design consideration. Liquefaction of these thicker layers in the Estuarine sands would lead to post-liquefaction settlement and may also lead to lateral spreading. If lateral spreading were to occur it is most likely to produce displacements in an eastwards or south-eastwards direction. Such displacements would be largest at the waterfront structures (i.e., wharfs) and would reduce in magnitude with increasing distance behind the wharf. Lateral spreading on gently sloping land surfaces has been observed to affect ground to a distance of 200 m back from the waterfront. Hence lateral spreading resulting from liquefaction within the Estuarine sands has some potential to produce small lateral ground displacements within the eastern and south-eastern site boundary of the MPDC.

7 Design recommendations

7.1 Geotechnical design parameters – piles

Recommended geotechnical end bearing and shaft resistance for design of bored piled foundations are provided in Table 7.1 below. Piles should penetrate at least two pile diameters into the respective founding soil or rock units used for design.

These values are considered suitable for design of bored piles between about 450 mm and 1500 mm diameter in accordance with requirements presented in Australian Standard AS 2159 – 2009, 'Piling – Design and Installation'. This assumes good construction practices such as roughening of sockets, cleaning of pile base, and appropriate concreting practices (tremie pouring underwater, adherence to maximum drop heights, etc.).

Table 7.1 Geotechnical design parameters for bored piles

Unit ID	Unit name	Ultimate unit shaft resistance (kPa)	Ultimate unit end bearing resistance (MPa)	Design Young's Modulus E' MPa	Ultimate lateral resistance P_y (kPa)
1A	Fill (Uncontrolled) Granular	10–25 (15)	–	5–20 (10)	100–350 (250)
1B	Fill (Uncontrolled) Cohesive	20–50 (25)	–	5–20 (10)	100–450
2	Estuarine	5–20 (10)	0.2–1.5 (0.5)	5–15 (6)	25–250 (75)
3A	Alluvium (Granular)	30–100 (40)	(2.5)	20–60 (40)	500–1250 (1000)
3B	Alluvium (Cohesive)	(30)	(0.9)	10–50 (25)	(900)
4A	Dolerite (EW)	(100)	2–5 (3)	50–200 (100)	(1250)
4B	Dolerite (HW to MW)	500–1000 (600)	10–40 (15)	1,000–10,000 (2,000)	5,000–12,000 (8,000)
4C	Dolerite (SW to FR)	1400–2800 (2000)	35–100 (50)	40,000–70,000 (50,000)	18,000–35,000 (25,000)

Notes:

(1) The indicative range of upper/lower bound design values are provided with the selected value shown in parentheses. Unit shaft resistance for rock sockets will decrease with increasing pile diameter and increasing socket length.

(2) In the uppermost 1 m of each layer, the design ultimate lateral resistance values should be multiplied by 0.5 to account for the increased possibility of failure along fracture planes in rock. Where rock is encountered at the existing ground surface level, the reduced lateral resistance value should be increased to 1.5 m, due to the increased likelihood of failure along fracture planes at low overburden pressures.

(3) Ultimate lateral resistance increases from 2 x S_u at the surface to 9 x S_u at a depth of 4.5 x pile diameter.

(4) Adopt sensitivity of 50% and 200% to stiffness values for structural analysis. If structure is unable to withstand such variation in stiffness, more detailed stress/strain dependent geotechnical design analysis should be undertaken.

(5) The designer should consider the potential effect of deep weathered zones along fractured and sheared zones (where present) when assessing lateral pile behaviour

The geotechnical unit stresses will need to be reduced by an appropriate geotechnical strength reduction factor (Φ_g) for pile design in accordance with AS 2159 – 2009 requirements. The resultant strength reduction factor is a function of the level of geotechnical investigation, redundancy in the system, and the pile load testing frequency adopted. For Ultimate Limit State (ULS) design, the geotechnical strength reduction factor should be assessed on a structure-by-structure basis as the value adopted considers the available geotechnical data.

For piles socketed into rock and loaded in vertical compression, it may not be appropriate to rely on skin friction resistance from soils overlying rock due to strain compatibility effects. It may also be necessary to design for circumstances where negative skin friction (downdrag) applies to piles. Examples of where downdrag should be considered in pile design includes where additional surface loading is applied to the compressible units (1A, 1B, 2).

Should driven piles be adopted, Table 7.2 provides the preliminary shaft adhesion design values.

Table 7.2 Driven pile preliminary design values

Unit ID	Unit name	Ultimate shaft resistance (kPa)	Ultimate end bearing (MPa)
1A	Fill (Uncontrolled) Granular	20–50 (30)	-
1B	Fill (Uncontrolled) Cohesive	40–100 (50)	-
2	Estuarine	10–40 (20)	0.5
3A	Alluvium (Granular)	60–150 (75)	2.0
3B	Alluvium (Cohesive)	(50)	0.9
4A	Dolerite (EW-RS)	100-150 (150)	4.0
4B	Dolerite (MW-HW)	-	15.0
4C	Dolerite (FR-SW)	-	50.0

Note:

(1) Refusal of driven piles is anticipated on Unit 4B and 4C on competent bedrock. Cobbles, boulders in the Alluvium as well as large inclusions in the fill may obstruct driving to design depth.

(2) Design end bearing values adopted in strata that is interbedded should consider the material below the pile tip within a minimum of 3 diameters of the pile.

(3) The designer should consider the variable weathering of the dolerite along fracture planes and the potential for variability in founding conditions within close proximity laterally.

(4) No significant embedment of driven piles is anticipated into the more competent Unit 4B and Unit 4C material.

7.2 Shallow footings

It is expected that some of the proposed Macquarie Point structures may be supported on spread footings founded on rock or suitable soil strata. The size of spread footings founded in weathered rock will likely be governed by the serviceability bearing pressures presented in Table 7.3. Site specific data combined with settlement analysis of individual footings may allow higher bearing pressures to be adopted. The designer should also undertake a capacity calculation to confirm the design meets the requirements outlined in Australian Standard AS5100.3 – 2004, 'Foundations and soil-supporting structures'. Appropriate factoring of ULS and SLS geotechnical inputs should be undertaken in accordance with the Standard, when undertaking the design of footing. Design values presented in Table 7.3 assume that footings are loaded vertically and located on a non-sloping ground outside of the zone of influence of other excavations or other structures.

If weathered material is encountered at the founding level, or within the zone of influence, it is unlikely that the indicative ultimate and serviceability bearing resistance values presented in Table 7.3 will be attainable. This will also apply in cases where materials with strengths and stiffnesses lower than those reported in Table 6.1 to Table 6.3 are encountered.

Unfavourable bedding or jointing of rock strata, especially in sloping ground should be carefully considered for the geotechnical assessment of footing capacity. In cases where defects or slopes may control bearing capacity, it may be necessary to locally modify design parameters or otherwise limit permissible loading of the spread footings.

Table 7.3 Bearing pressure design values for spread footings

Unit ID	Unit name	Ultimate bearing resistance for spread footings (kPa)	Serviceability bearing resistance for spread footings (kPa) [1,2]
1A	Fill (Uncontrolled) Granular	100	50
1B	Fill (Uncontrolled) Cohesive	100	50
2	Estuarine	200	75
3A	Alluvium (Granular)	300	100
3B	Alluvium (Cohesive)	300	100
4A	Dolerite (EW)	1,000	250
4B	Dolerite (HW to MW)	15,000	2,500
4C	Dolerite (SW to FR)	50,000	7,500

Notes:

(1) Serviceability bearing resistance assumes settlement magnitudes of less than 1% of the foundation width.

(2) Guide only – to be checked using intrinsic soil strength values for the size, depth, and shape of footing under consideration at any given location.

(3) Spread footings are not recommended to be founded in uncontrolled fill given the variable nature and potential for differential settlement to occur. Spread footings could be used in fill if measures are undertaken to improve the ground stiffness to perform in a consistent manner.

7.3 Excavation support

The requirement for and design of temporary and permanent excavation retention system(s) is dependent on the following:

- extent of excavation (i.e., depth, plan dimensions)
- nature of the materials to be retained
- sequence of construction
- allowable lateral ground movement and associated settlements
- groundwater conditions
- magnitude of any vertical surcharges or loads near or behind the crest of the excavation.

The planning, design, and construction of the excavation and excavation retention system(s) will need to consider appropriate measures to reduce the risk of damaging nearby sensitive assets, which may include but are not limited to buried services, adjacent buildings or structures and existing retention systems (e.g., ground anchors) for adjacent sites. Additionally, ground water control measures will also need to be considered.

The lateral earth pressure coefficients presented in Table 7.4 may be used for the preliminary design of retaining walls or excavation retention systems. These assume dry ground conditions with flat ground behind the crest. The design should also consider additional lateral earth pressures resulting from:

- additional loading/surcharge (including allowance for temporary construction loading) and where sloping ground is retained behind the excavated face
- locked in compaction pressures, where filling works are proposed behind the retained structures
- where excavations are sealed or tanked, hydrostatic ground water pressures should be considered in conjunction with the lateral loads discussed above
- additionally, the contribution of hydrostatic water pressure in the case of an accidental burst water pipe, temporarily elevated groundwater levels due to heavy rainfall or otherwise, should also be considered
- where soils are saturated, the effective (i.e., buoyant) soil unit weights should be considered in lieu of the bulk unit weights when calculating the contribution of the soil to the lateral load on the retaining system. The contribution to the lateral loading of groundwater pressures is additive to the soil-related component.

Table 7.4 Lateral earth pressure coefficients

Unit	Unit Name	Lateral earth pressure coefficients		
		Active, K_a	At Rest, K_0	Passive, K_p
1A	Uncontrolled Fill (Granular)	0.31	0.47	3.25
1B	Uncontrolled Fill (Cohesive)	0.36	0.53	2.77
2	Estuarine	0.33	0.50	3.00
3A	Alluvium (Granular)	0.28	0.44	3.54
3B	Alluvium (Cohesive)	0.35	0.52	2.88

Notes:

(1) All values of K assume level ground behind the retained/excavated face and ignore the contribution of friction along the back of the structure. Higher coefficients would apply where the ground surface slopes behind the crest, or alternatively, this should be modelled as a surcharge load.

(2) We note that generally 0.1%H to 0.4%H movement, where H is the retained height in metres, is required to develop full active pressures. The stability and serviceability performance of retaining structures should be assessed, including the effects on adjacent sensitive assets, structures or receptors.

7.4 Ground anchors

The soil/rock to grout bond stress considered for the design of ground anchors depends largely on the installation methods adopted (i.e., drilling and grouting). Industry practice is typically for ground anchors to be constructed on a design and construct basis, with the specialist contractor being required to demonstrate achievement of the required working loads via acceptance testing.

For the purposes of sizing of ground anchors, the initial maximum soil/rock-grout bond stresses as indicated in Table 7.5 may be considered. These are expected to be achievable with use of air flush drilling and work carried out by a competent contractor with good quality controls in place during installation. Higher values may be achievable, particularly where competent rock is encountered; however, an extensive trial and testing regime would be required to confirm higher allowable unit stresses for the detailed design. For the primary design purposes, ground anchor/soil nail bond lengths should not be less than 3 m and should not exceed 10 m. A geotechnical engineer should also observe the installation, grouting and proof loading for anchors and rock bolts to confirm that the encountered conditions are consistent with design expectations.

Table 7.5 Soil/Rock to grout bond stress values for Anchor Design

Unit ID	Unit name	Ultimate soil-rock grout bond stress (kPa)
4A	Dolerite (EW-RS)	75–150 (100)
4B	Dolerite (HW to MW)	500–1000 (750)
4C	Dolerite (SW to FR)	1000–2000 (1500)

Notes:

(1) Values are for anchors with bond length between 3 m and 10 m

(2) Achievable parameters are heavily dependent on the method of installation and chosen anchor contractor, and reductions in anchor length may be possible through the detailed design phase.

(3) For anchors designed to oppose uplift load: In addition to shaft capacity, uplift capacity should be checked against a cone pull-out failure mode assuming a cone angle of 90° in rock and 60° in soil. For long term loads the submerged weight of the soil or rock should be adopted. For short term or dynamic loading, the total weight of the rock or soil can be adopted. A reduction factor of 0.8 should be applied to the cone mass. If cone pull-out proves to be critical, a more detailed analysis should be undertaken to assess cone pull-out capacity.

(4) The designers are to assess the appropriate reduction factors for the design method adopted.

7.5 Excavation conditions

Following completion of demolition of existing structures including footings and decommissioning of services, some excavation is likely to be required to reach design subgrade levels across the site. Furthermore, excavations may be required for additional structures including buildings or car parks with basements. In the northern zone of the site where top of rock level is encountered between RL 8 m AHD and RL 2 m AHD, excavation into rock is likely to be required.

Excavation progress within the bedrock will depend on the intact rock strength and discontinuity spacing through the bedrock. Figure 7.1 presents an indicative rock excavatability assessment for the proposed excavation works. Generally due to the high and very high strength of the bedrock, anticipated excavation through the bedrock will likely require:

- Mixture of ripping, rock cutting, rock splitting or blasting using hydraulic machinery for Unit 4C.
- Ripping with either D8 dozers or hydraulic hammers for Unit 4B.
- This assessment is a guide only and the contractor should assess the borehole logs and satisfy themselves of the appropriate plant to use. Rock abrasion testing has been undertaken on samples of dolerite and is reported in the geotechnical factual report.

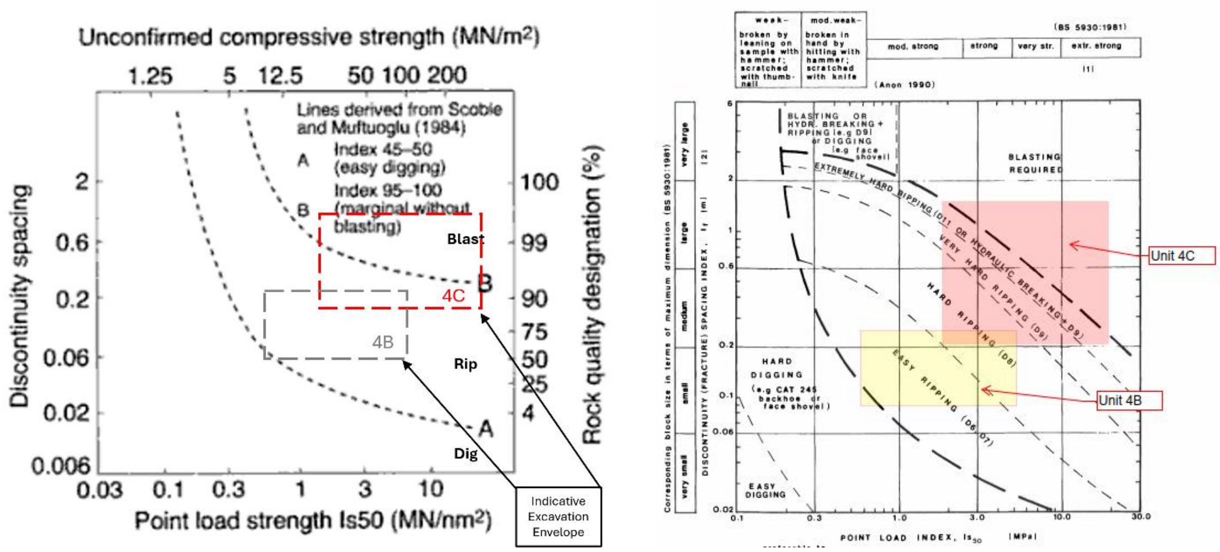


Figure 7.1 Indicative rock excavatability assessment charts (Walton and Wang (1993) and Pettifer and Fookes (1994))

7.6 Vibration

Care should be taken during excavation (and backfilling compaction) to limit the vibration impacts on structures that need to be retained (or new structures). In addition, the potential vibrations from construction may need to be considered with respect to buried services, nearby commercial and residential properties, heritage structures and the ports facility.

We recommend that the following measures are taken to assess and manage vibration risks:

- carry out an assessment of the proximity of vibration-sensitive structures to the site
- carry out dilapidation surveys on vibration sensitive structures before work commences and after work has been completed
- prepare a vibration management plan setting limits on Peak Particle Velocity (PPV) and install, where required, monitoring systems to assess vibrations.

7.7 Cut and batter slopes

Excavations for new structures will need to be designed to control ground movement. This may require shoring and/or a retaining system prior to excavation, especially in the loose sand deposits. Alternatively, if space allows, batter slopes can be adopted. Table 7.6 provides the recommended batter slopes for different materials.

Table 7.6 Recommended maximum batter slopes (excavations/slopes up to 2 m vertical depth/height)

Unit ID	Unit name	Permanent batter slope	Temporary batter slope
1A, 1B	Fill (Uncontrolled) Granular, Fill (Uncontrolled) Cohesive	1(v):3(h)	1(v):2(h)
4A	Dolerite (EW)	1(v):2(h)	1(v):1.5(h)
4B	Dolerite (HW to MW)[1]	1(v):1(h)	1(v):1(h)
4C	Dolerite (SW to FR) [1]	1(v):1(h)	1(v):1(h)

Notes:

(1) where discontinuities are present, stability should be assessed by a Geotechnical Engineer or Engineering Geologist.

(2) no advice has been provided for the Estuarine and Alluvium Units as these materials are situated below the groundwater table. Excavations in these materials should be done on a case-by-case basis.

In all cases, excavations should be assessed for stability by a Geotechnical Engineer or Engineering Geologist and all batter slopes within granular and cohesive soils should be protected from erosion. Allowable surcharge loads from construction plant or spoil placed in close proximity to the excavation crest should be assessed by a geotechnical engineer. If slopes or excavations other than those in the table above are to be used, additional slope stability assessments should be completed.

8 Limitations

Your attention is drawn to the limitations statement, which is included in Appendix H of this report. The statements presented in that document are intended to inform a reader of the report about its proper use. There are important limitations as to who can use the report and how it can be used. It is important that a reader of the report understands and has realistic expectations about those matters. The limitations statement does not alter the obligations WSP has under the contract between it and its client.

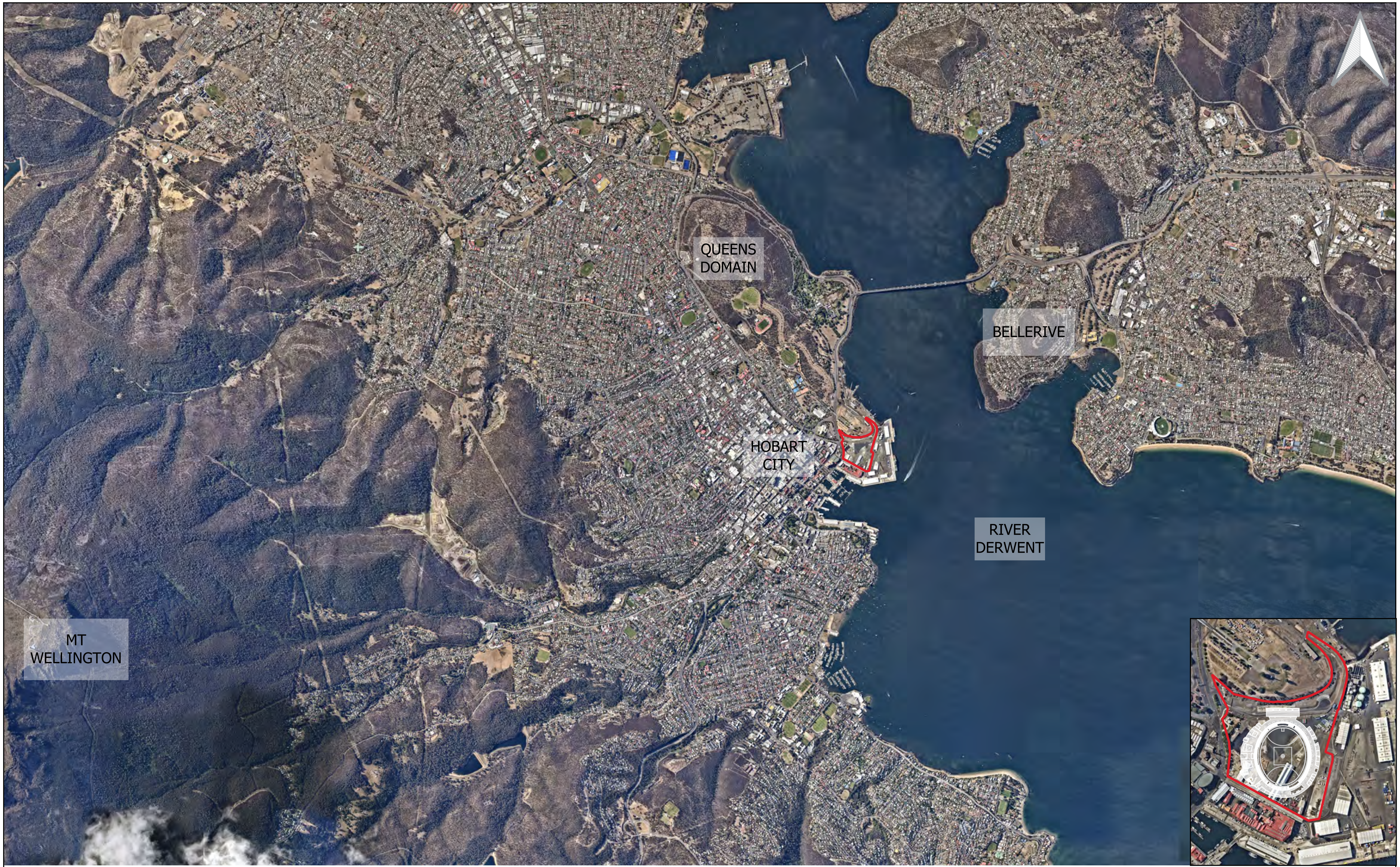
9 Reference list

- Australian Standard AS1170.4 – 2007: Structural Design Actions – Earthquake Actions in Australia, Standards Australia.
- Australian Standard AS2159 – 2009: Pile Footings Design and Installation, Standards Australia.
- Australian Standard AS5100.3 – 2004: Foundations and Soil-Supporting Structures, Standards Australia.
- Burland, J.B. and Coatsworth, S.R. (1987) *Estimating the Settlement of Foundations on Sands and Gravels*, Proc. Int. Conference on Foundations and Tunnels, 24-26 March 1987, University of London.
- Harvey, N. and Caton, B. (2010). Coastal Management in Australia, University of Adelaide Press.
- Kulhawy, F.H., Mayne, P.W. (1990). *Manual on Estimating Soil Properties for Foundation Design*. Report EL-6800, Electric Power Research Institute, Washington DC.
- Koppula, S. D. (1981). Statistical evaluation of compression index. Geotechnical Testing Journal ASTM Vol 4(2) p68–73
- Lewis, S.E., Sloss, C.R., Murray-Wallace, C.V., Woodroffe, C.D. and Smithers, S.G., (2012) *Post-glacial sea-level changes around the Australian margin: a review.* Quaternary Science Reviews Vol. 74, 15 August 2012 p115-138
- Mesri, G., Godlewski. P., M. (1977). *Time and stress compressibility interrelationship*. Journal of Geotechnical Engineering Div. ASCE 103 (GT5), p417–430
- Ministry of Forests. (1996). *Community Watershed Guidebook*. Retrieved December 4, 2013, from Community Watershed Guidebook: http://www.for.gov.bc.ca/TASB/LEGSREGS/FPC/FPCGUIDE/WATRSHED/Watertoc.htm
- Robertson, P. K., Cabal, K. (2022). *Guide to Cone Penetration Testing:* 7th Edition. Gregg Drilling, California.
- Pettifer G.S., Fookes P.G. (1994). *A revision of the graphical method of assessment the excavatability of rock.* Quarterly Journal of Engineering Geology and Hydrogeology, p 145–164
- Terzaghi. K. (1927). *Principles of final soil classification*. Public Roads Vol. 8, p41–53
- Terzaghi. K., Peck. R. (1967). *Soil mechanics in engineering practice*, 2nd edition. Wiley, New York
- USACE. (1990). *Engineering and design – settlement analysis*, EM1110-1-1904, Department of the Army, US Army Corps of Engineers
- Walton, G & Wang H. (1993) Technical Note. *Aspects of Diggability in Weak Rocks.* (Presented at Meeting of the Engineering Group of the Geological Society on 9 November 1993 and submitted to Leeds University Mining Association Journal)
- Wroth. C .P., Wood. D. M. (1978). *The correlation of index properties with some basic engineering properties of soils.* Canadian Geotechnical Journal. Vol 15 (2), p137–145

Appendix A

Figures

Ground model figures



Legend

- Stadium Footprint
- Site Boundary

1:35,000

300 0 300 600 900 1,200 m

CLIENT	Macquarie Point Development Corporation
PROJECT	Macquarie Point Stadium Development
CONSULTANT	wsp
YYYY-MM-DD	2024-07-02
PREPARED	AG / JVC
REVIEWED	TM
APPROVED	RM
TITLE	Regional Site Plan
PROJECT NO	PS212776
REV.	0
FIGURE	A1

Legend

- Site Boundary
- Stadium Footprint

1:2,500

20 0 20 40 60 80 m

CLIENT	Macquarie Point Development Corporation
PROJECT	Macquarie Point Stadium Development
CONSULTANT	wsp
YYYY-MM-DD	2024-07-02
PREPARED	AG / JVC
REVIEWED	TM
APPROVED	RM
TITLE	**Development Plan**
PROJECT NO	**PS212776**
REV.	**0**
FIGURE	**A2**

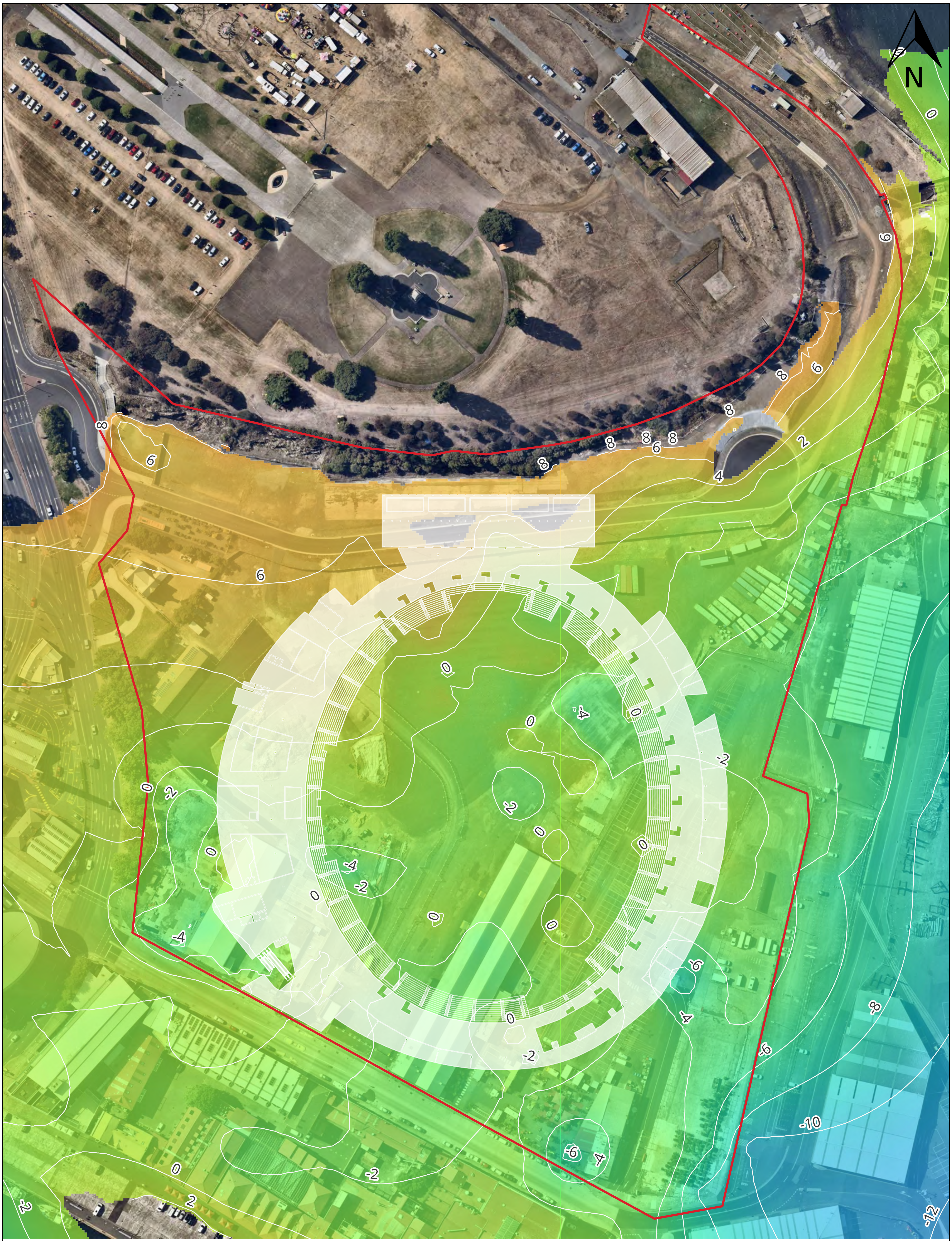
Legend

- Acid Sulfate Borehole
- Borehole
- Cone Penetration Test
- Dilatometer Test
- Site Boundary
- Stadium Footprint

1:1,500

10 0 10 20 30 40 m

CLIENT	Macquarie Point Development Corporation
PROJECT	Macquarie Point Stadium Development
CONSULTANT	wsp
YYYY-MM-DD	2024-07-02
PREPARED	AG / JVC
REVIEWED	TM
APPROVED	RM
TITLE	Ground Investigation Plan
PROJECT NO	PS212776
REV.	0
FIGURE	A3



Legend

- Stadium Footprint
- Site Boundary
- Contours 2m AHD

Inferred Elevation

m AHD

-22 22

1:1,450

10 0 10 20 30 40 m

CLIENT Macquarie Point Development Cooperation		PROJECT Macquarie Point Stadium Development	
CONSULTANT wsp	YYYY-MM-DD: 2024-07-02	TITLE Inferred Elevation Base of Fill	
	PREPARED: AG / JVC		
	REVIEWED: TW		
	APPROVED: RM	PROJECT NO: PS212776	DOC: · REV.: 0 · FIGURE: A4

QGIS v.3.28.4-Firenze - printed by: Gooding_Amy - path: U:\Projects\AU\212xxx\212776_Macquarie_Point_Geo4_WIP\3_Interp\SS\CPT\Standing_Groundwater_For_CPT.qgz

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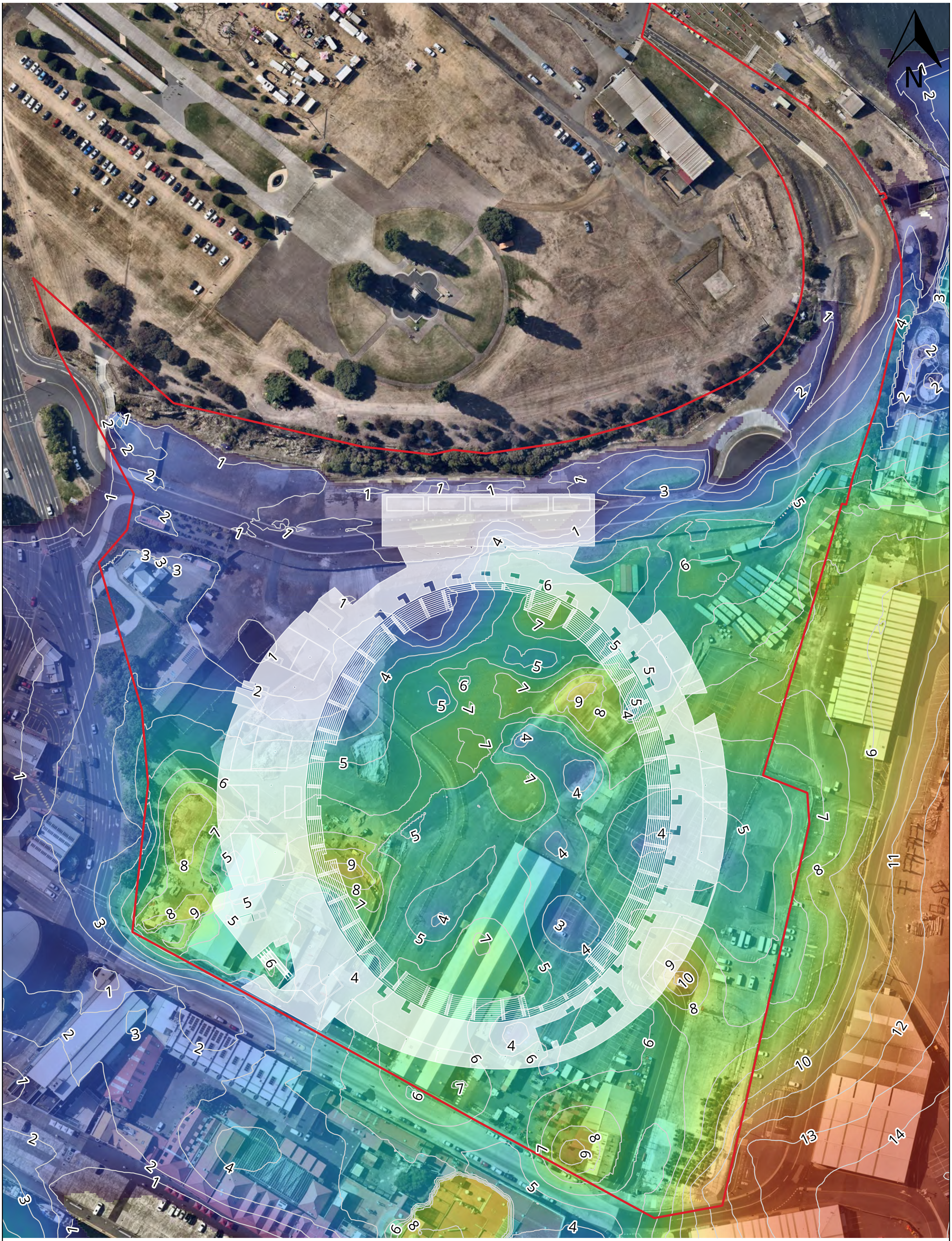

Legend

- Stadium Footprint
- Site Boundary
- Contours 1m

Inferred Thickness

m

0 16

1:1,450

10 0 10 20 30 40 m

CLIENT	Macquarie Point Development Cooperation
PROJECT	Macquarie Point Stadium Development
CONSULTANT	wsp
YYYY-MM-DD	2024-27-02
PREPARED	AG / JVC
REVIEWED	TW
APPROVED	RM
TITLE	Inferred Thickness of Fill
PROJECT NO	PS212776
DOC	
REV.	0
FIGURE	A5

QGIS v.3.28.4-Firenze - printed by: Gooding, Amy - path: U:\Projects\AU\212xxx\212776_Macquarie_Point_Geo4_WIP\3_Interp\SS\CPT\Standing_Grounwater_For_CPT.qgz

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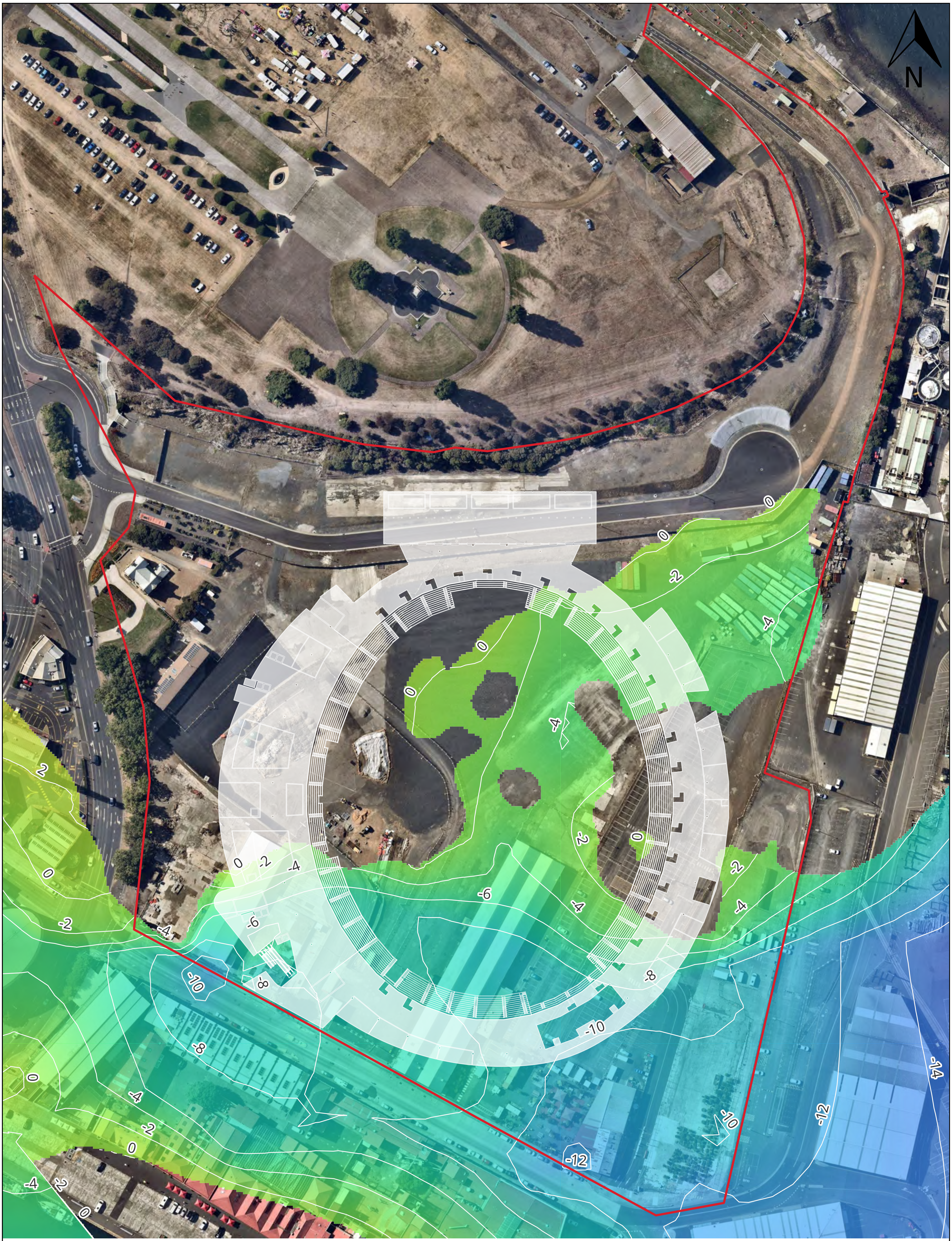

Legend

Stadium Footprint

Site Boundary

Contours 2m AHD

Inferred Elevation

m AHD

-22 | 22

1:1,450

10 0 10 20 30 40 m

CLIENT	Macquarie Point Development Cooperation
PROJECT	Macquarie Point Stadium Development
CONSULTANT	wsp
YYYY-MM-DD	2024-07-02
PREPARED	AG / JVC
REVIEWED	TW
APPROVED	RM
TITLE	Inferred Elevation Base of Estuarine
PROJECT NO	PS212776
DOC	
REV.	0
FIGURE	A6

QGIS v.3.28.4-Firenze - printed by: Gooding, Amy - path: U:\ProjectsAU\212xxx\212776_Macquarie_Point_Geo4_WIP\3_Interp\SS\CPT\Standing_Groundwater_For_CPT.qgz

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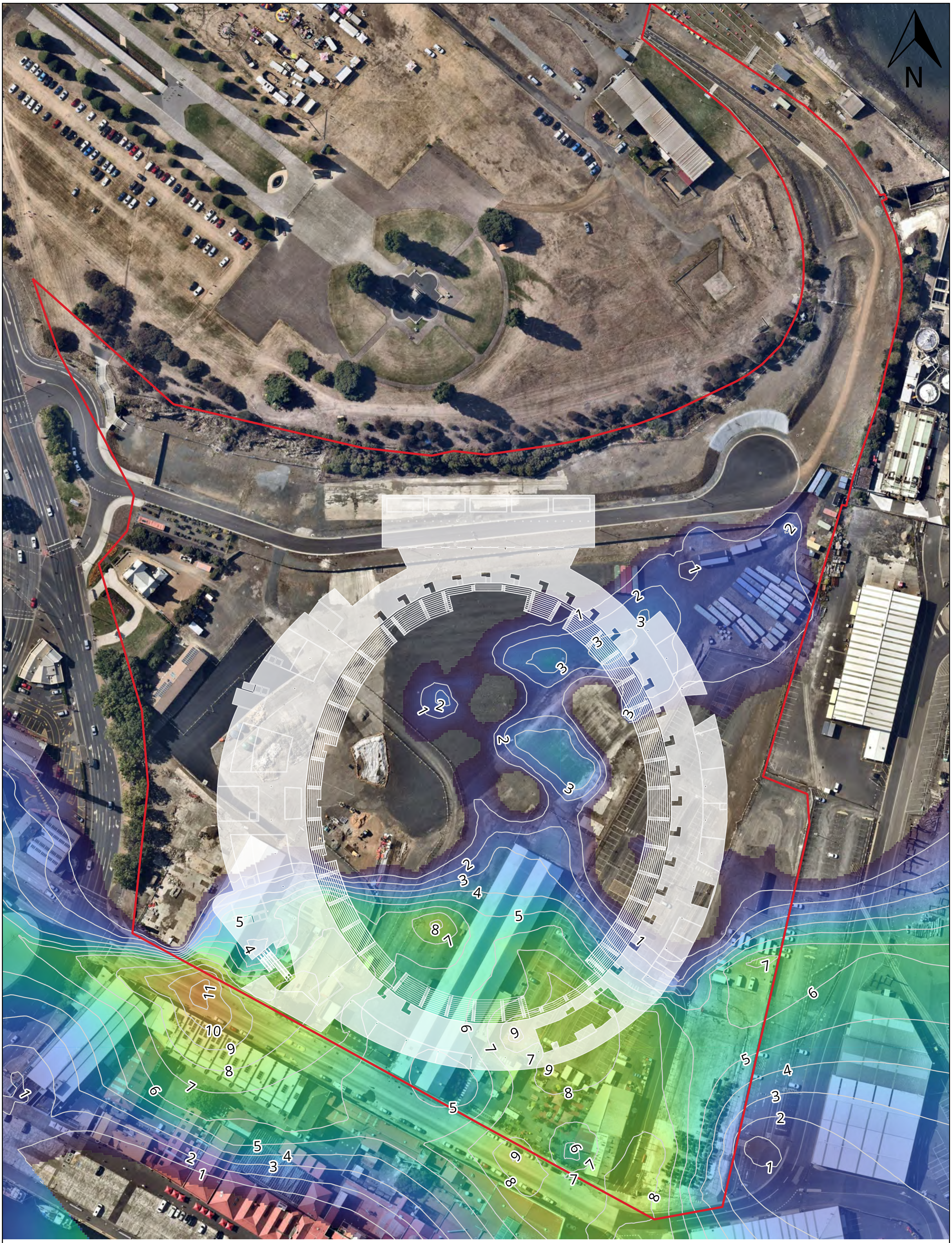

Legend

- Stadium Footprint
- Site Boundary
- Contours 1m

Inferred Thickness

m

0 16

1:1,450

10 0 10 20 30 40 m

CLIENT	Macquarie Point Development Cooperation
PROJECT	Macquarie Point Stadium Development
CONSULTANT	wsp
YYYY-MM-DD	2024-07-02
PREPARED	AG / JVC
REVIEWED	TW
APPROVED	RM
TITLE	Inferred Thickness of Estuarine
PROJECT NO	PS212776
DOC	
REV.	0
FIGURE	A7

QGIS v.3.28.4-Firenze - printed by: Gooding, Amy - path: U:\ProjectsAU\212xxx\212776_Macquarie_Point_Geo\4_WIP\3_Interp\SS\CPT\Standing_Groundwater_For_CPT.qgz

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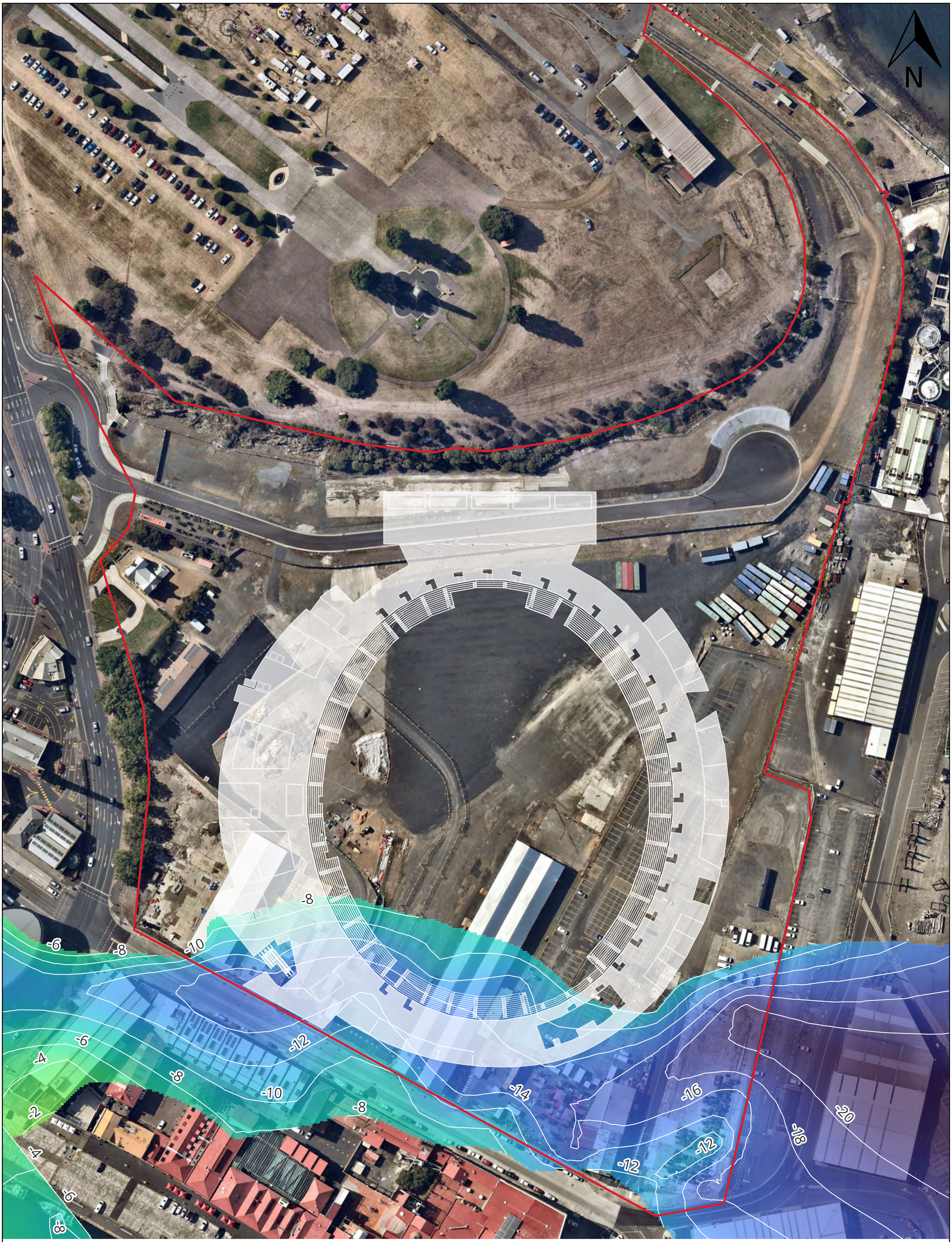

Legend

Stadium Footprint

Site Boundary

Contours 2m AHD

Inferred Elevation

m AHD

-22 22

1:1,450

10 0 10 20 30 40 m

CLIENT	Macquarie Point Development Cooperation
PROJECT	Macquarie Point Stadium Development
CONSULTANT	wsp
YYYY-MM-DD	2024-07-02
PREPARED	AG / JVC
REVIEWED	TW
APPROVED	RM
TITLE	Inferred Elevation Base of Alluvium
PROJECT NO	PS212776
DOC	
REV.	0
FIGURE	A8

QGIS v.3.28.4-Firenze - printed by: Gooding, Amy - path: U:\ProjectsAU\212xxx\212776_Macquarie_Point_Geo4_WIP\3_Interp\SS\CPT\Standing_Grounwater_For_CPT.qgz

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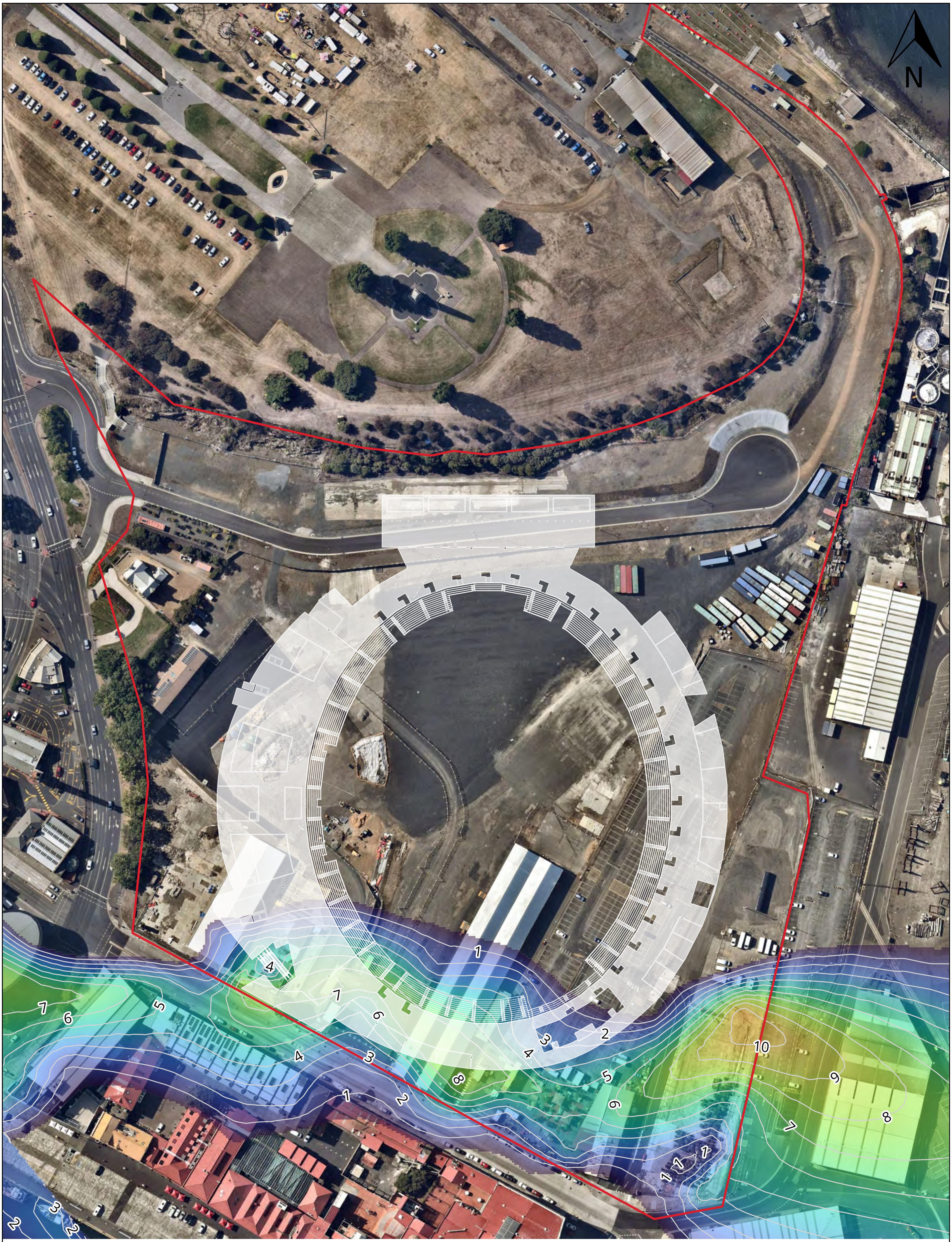

Legend

- Stadium Footprint
- Site Boundary
- Contours 1m

Inferred Thickness

m

0 16

1:1,450

10 0 10 20 30 40 m

CLIENT	PROJECT
Macquarie Point Development Cooperation	Macquarie Point Stadium Development

CONSULTANT			TITLE
wsp	YYYY-MM-DD	2024-07-02	Inferred Thickness of Alluvium
	PREPARED	AG / JVC	
	REVIEWED	TW	
	APPROVED	RM	

PROJECT NO	DOC	REV.	FIGURE
PS212776		0	A9

QGIS v.3.28.4-Firenze - printed by: Gooding, Amy - path: U:\Projects\AU\212xxx\212776_Macquarie_Point_Geo4_WIP\3_Interp\SS\CPT\Standing_Groundwater_For_CPT.qgz

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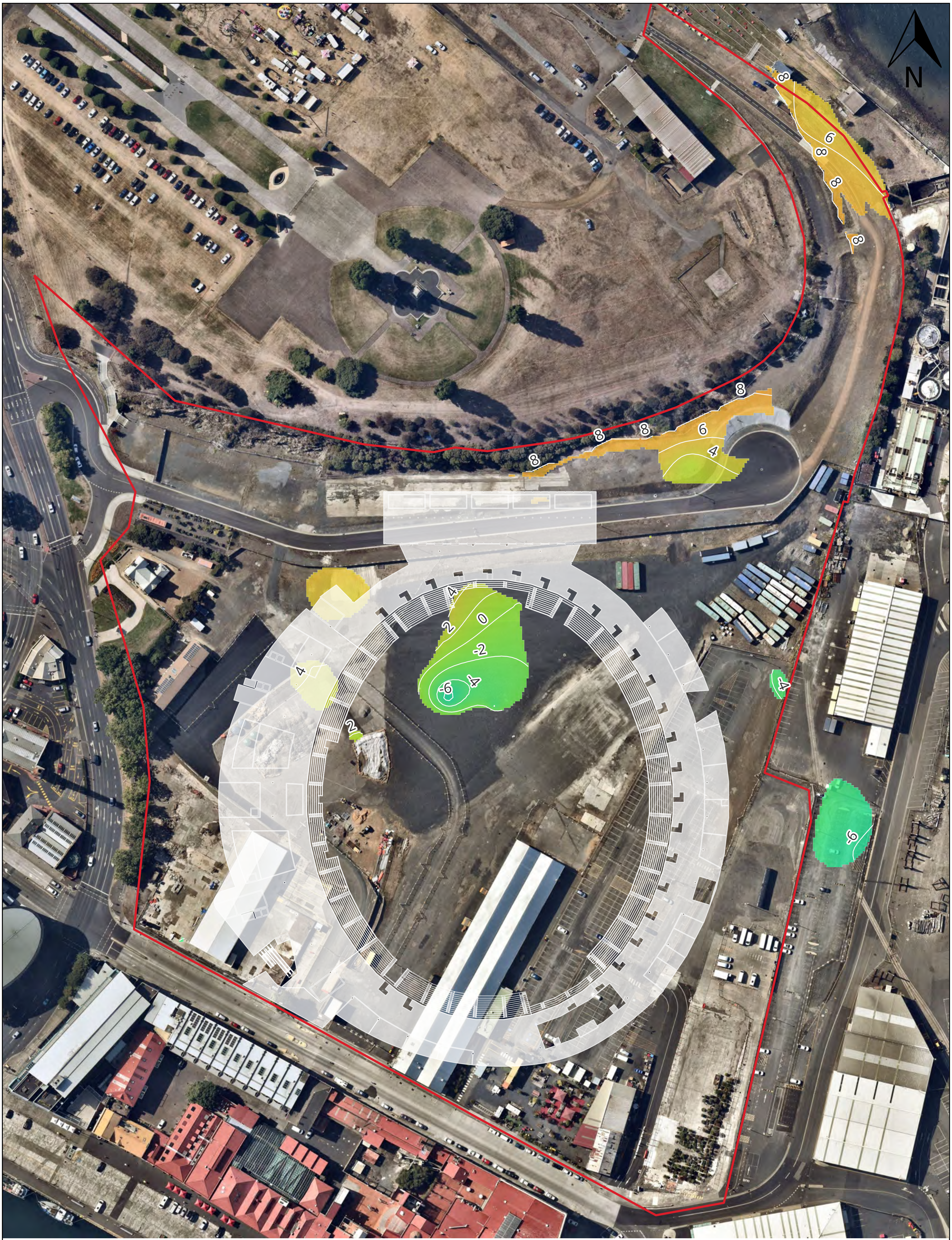

Legend

Stadium Footprint

Site Boundary

Contours 2m AHD

Inferred Elevation

m AHD

-22 22

1:1,450

10 0 10 20 30 40 m

CLIENT Macquarie Point Development Cooperation		PROJECT Macquarie Point Stadium Development	
CONSULTANT	YYYY-MM-DD 2024-25-06	TITLE **Inferred Elevation Base of Extremely Weathered Dolerite**	
	PREPARED AG / JVC		
	REVIEWED TW	PROJECT NO **PS212776**	DOC
	APPROVED RM	REV. **0**	FIGURE **A10**

QGIS v.3.28.4-Firenze - printed by: Gooding, Amy - path: U:\ProjectsAU\212xxx\212776_Macquarie_Point_Geo\4_WIP\3_Interp\SS\CPT\Standing_Groumwater_For_CPT.qgz

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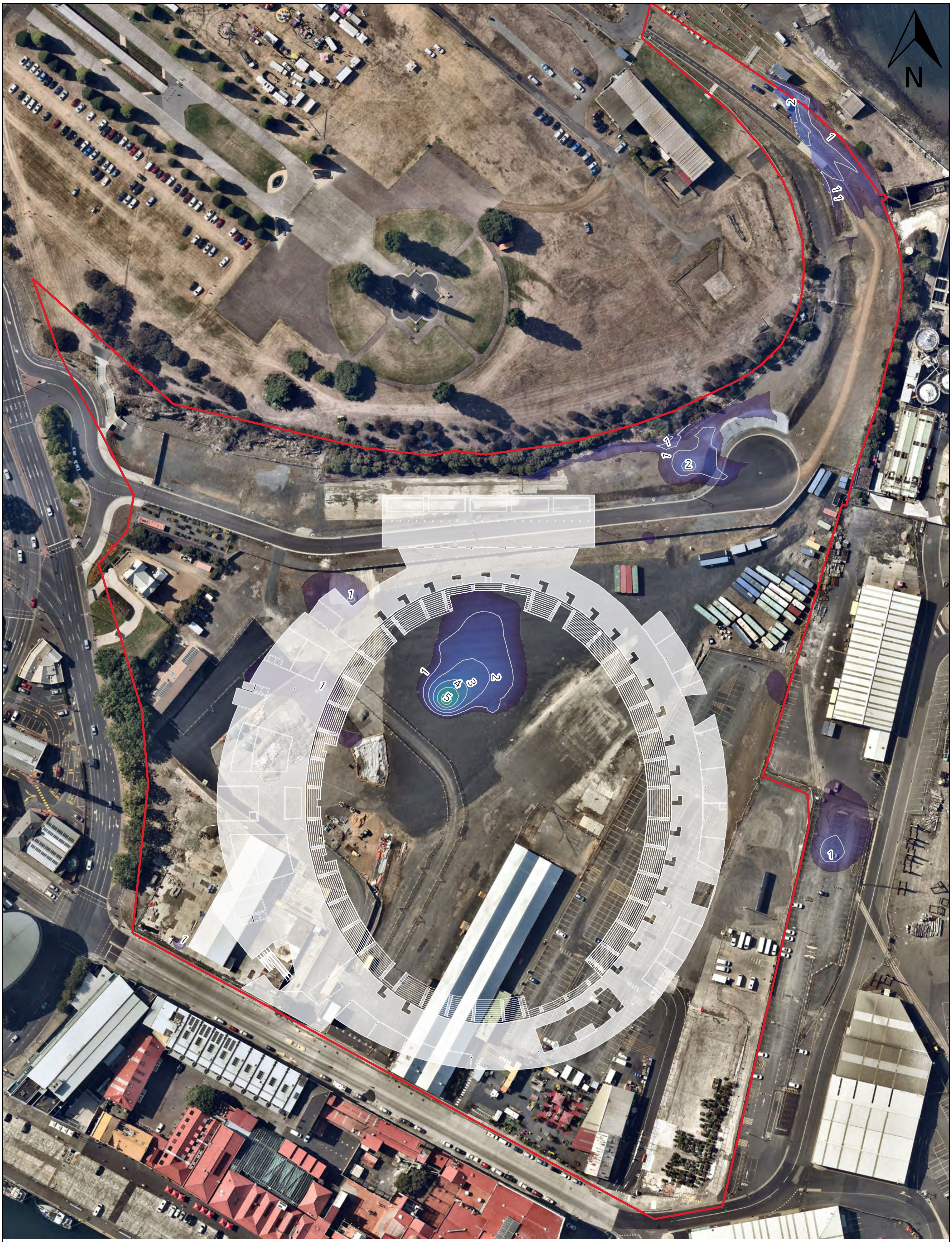

Legend

- Stadium Footprint
- Site Boundary
- Contours 1m

Inferred Thickness

m

0 16

1:1,450

10 0 10 20 30 40 m

CLIENT	Macquarie Point Development Cooperation
PROJECT	Macquarie Point Stadium Development
CONSULTANT	
YYYY-MM-DD	2024-07-02
PREPARED	AG / JVC
REVIEWED	TW
APPROVED	RM
TITLE	Inferred Thickness of Extremely Weathered Dolerite
PROJECT NO	PS212776
DOC	
REV.	0
FIGURE	A11

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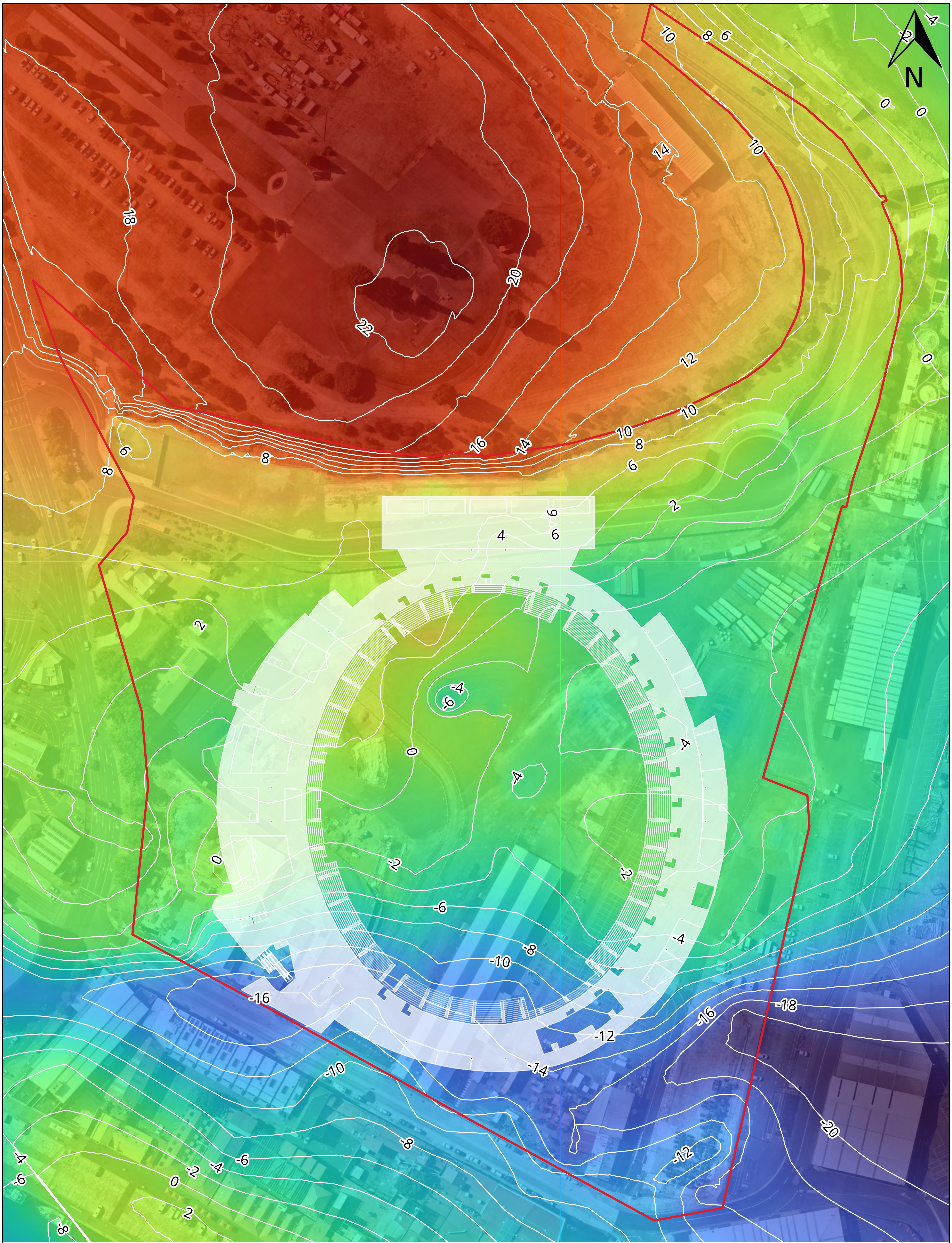

Legend

- Stadium Footprint
- Site Boundary
- Contours 2m AHD

Inferred Elevation

m AHD

-22 22

1:1,450

10 0 10 20 30 40 m

CLIENT	Macquarie Point Development Cooperation
PROJECT	Macquarie Point Stadium Development
CONSULTANT	wsp
YYYY-MM-DD	2024-25-06
PREPARED	JVC
REVIEWED	TW
APPROVED	RM
TITLE	Inferred top of rock
PROJECT NO	PS212776
DOC	
REV.	0
FIGURE	A12

QGIS v.3.28.4-Firenze - printed by: Gooding, Amy - path: U:\Projects\AU\212xxx\212776_Macquarie_Point_Geo4_WIP\3_Interp\SS\CPT\Standing_Groundwater_For_CPT.qgz

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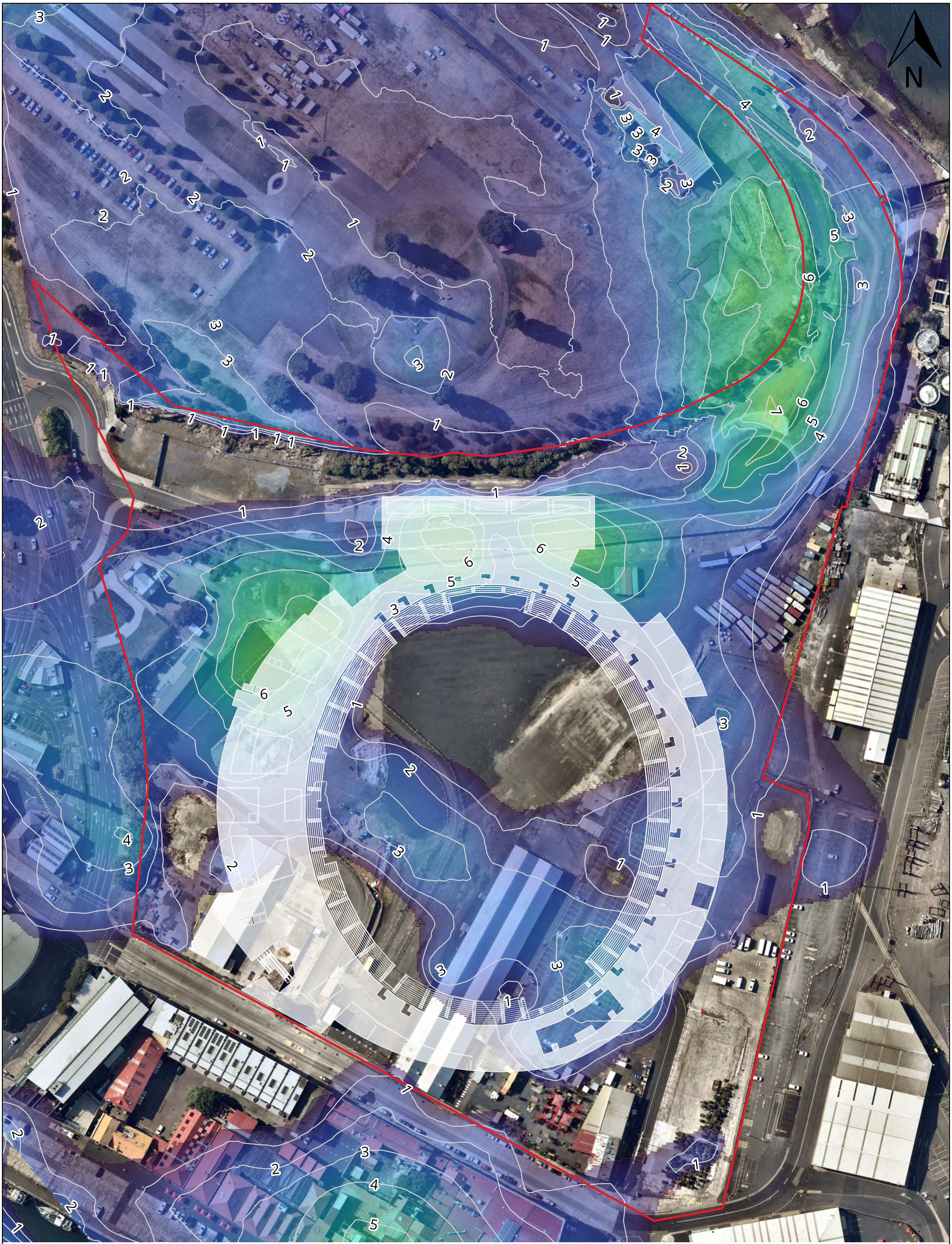

Legend

- Stadium Footprint
- Site Boundary
- Contours 1m

Inferred Thickness

m

0 16

1:1,450

10 0 10 20 30 40 m

CLIENT	Macquarie Point Development Cooperation
PROJECT	Macquarie Point Stadium Development
CONSULTANT	wsp
YYYY-MM-DD	2024-07-02
PREPARED	AG / JVC
REVIEWED	TW
APPROVED	RM
TITLE	Inferred Thickness of Highly and Moderately Weathered Dolerite
PROJECT NO	PS212776
DOC	
REV.	0
FIGURE	A13

QGIS v.3.28.4-Firenze - printed by: Gooding_Amy - path: U:\Projects\AU\212xxx\212776_Macquarie_Point_Geo4_WIP\3_Interp\SS\CPT\Standing_Groundwater_For_CPT.qgz

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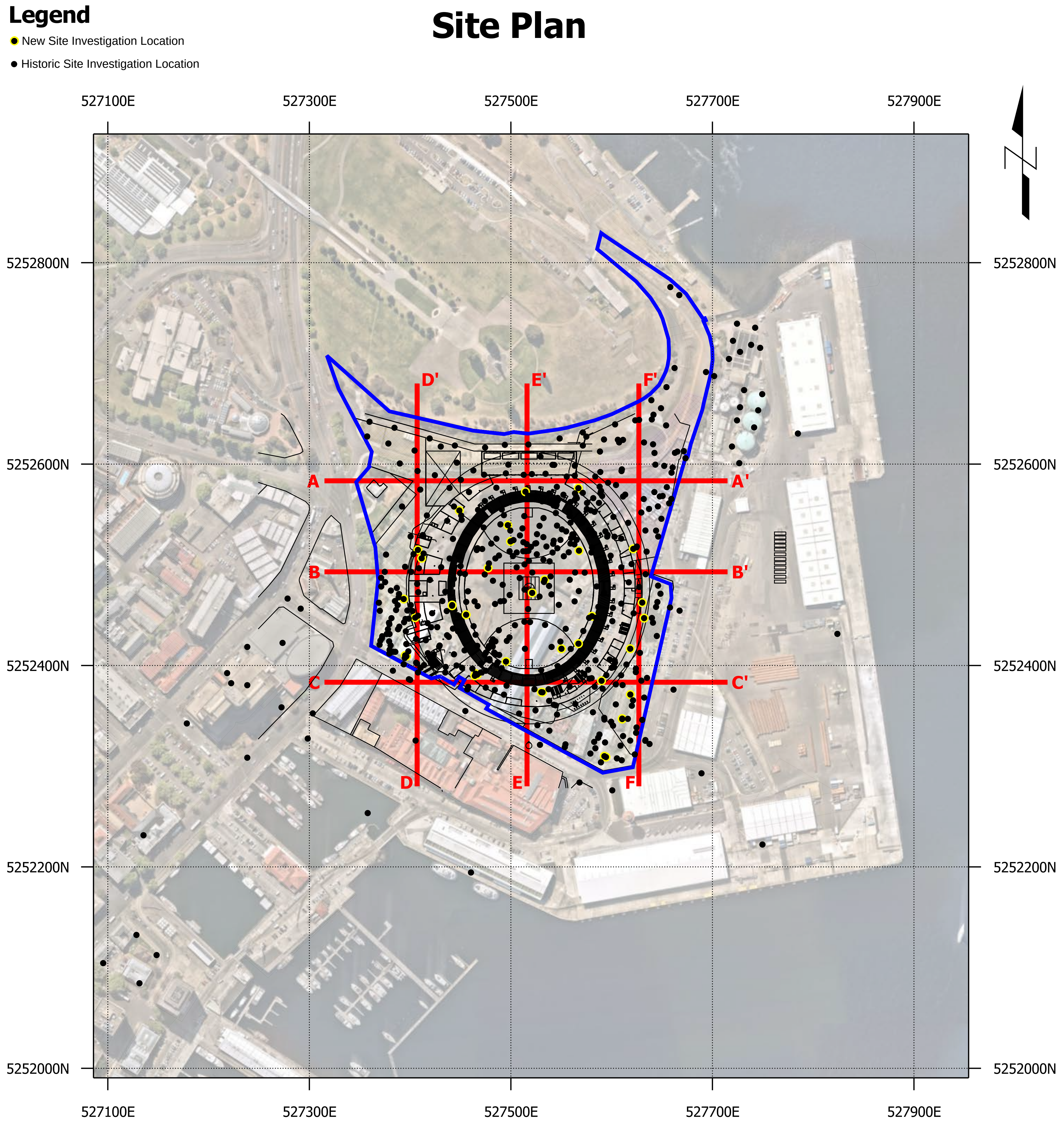
Appendix B

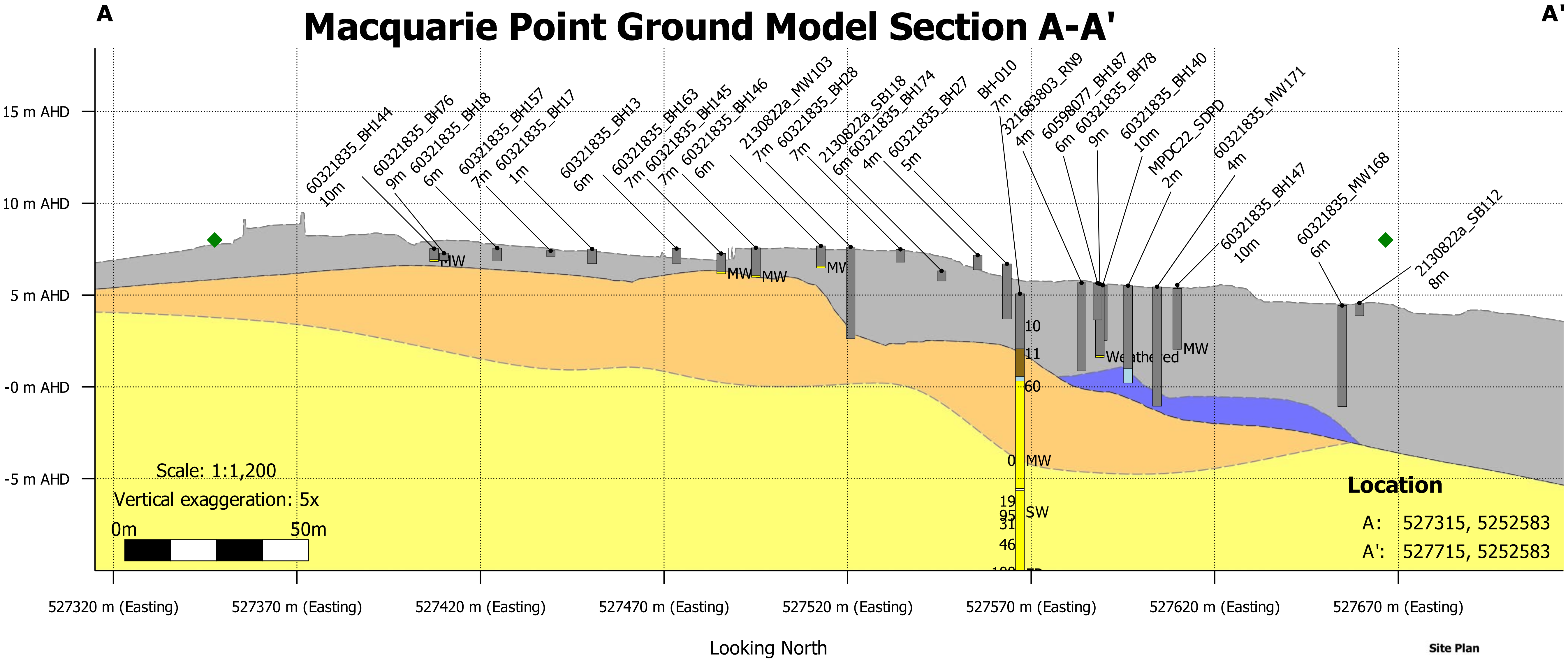
Long sections

List of sections

Site Plan


Legend
New Site Investigation Location
Historic Site Investigation Location
527100E
527300E
527500E
527700E
527900E
5252800N
5252600N
5252400N
5252200N
5252000N
A
A'
B
B'
C
C'
D
D'
E
E'
F
F'
N

Macquarie Point Ground Model Section A-A'



Legend

Gound Model

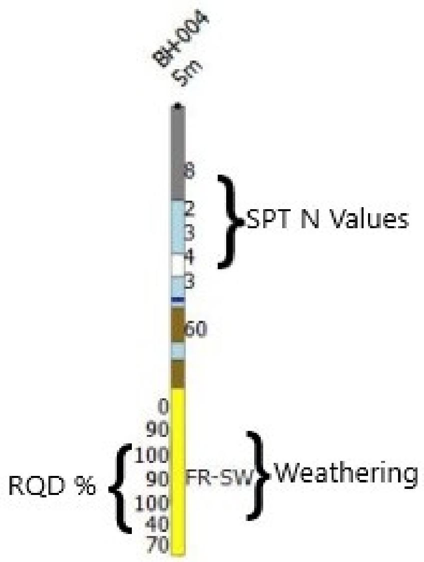
- FILL
- Estuarine
- Dolerite, MW-HW
- Dolerite, Fr-SW

Major Material Logged

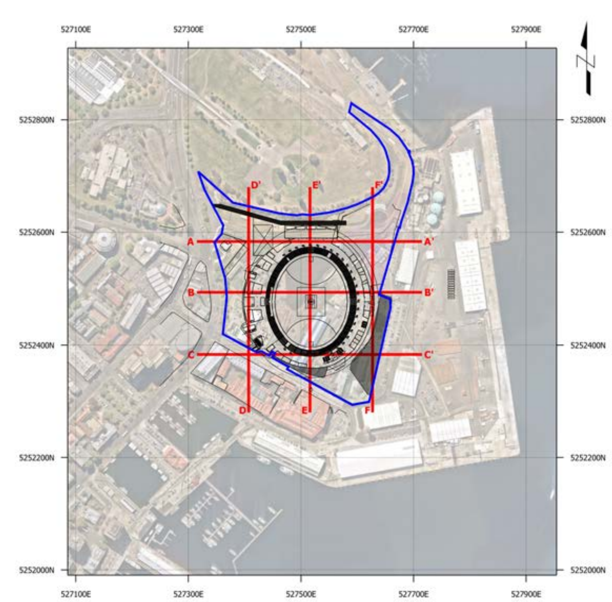
- Coreloss
- Cobbles
- Clay
- Sandstone
- Fill
- Sand
- Boulders
- Dolerite
- Gravel
- Silt
- Mudstone

◆ Mac Point Precinct Plan_20231106 - 02 Site Boundary_Boundary

Graphic Log Legend



Site Plan



		Creator	Approved by
		JRG	TDM

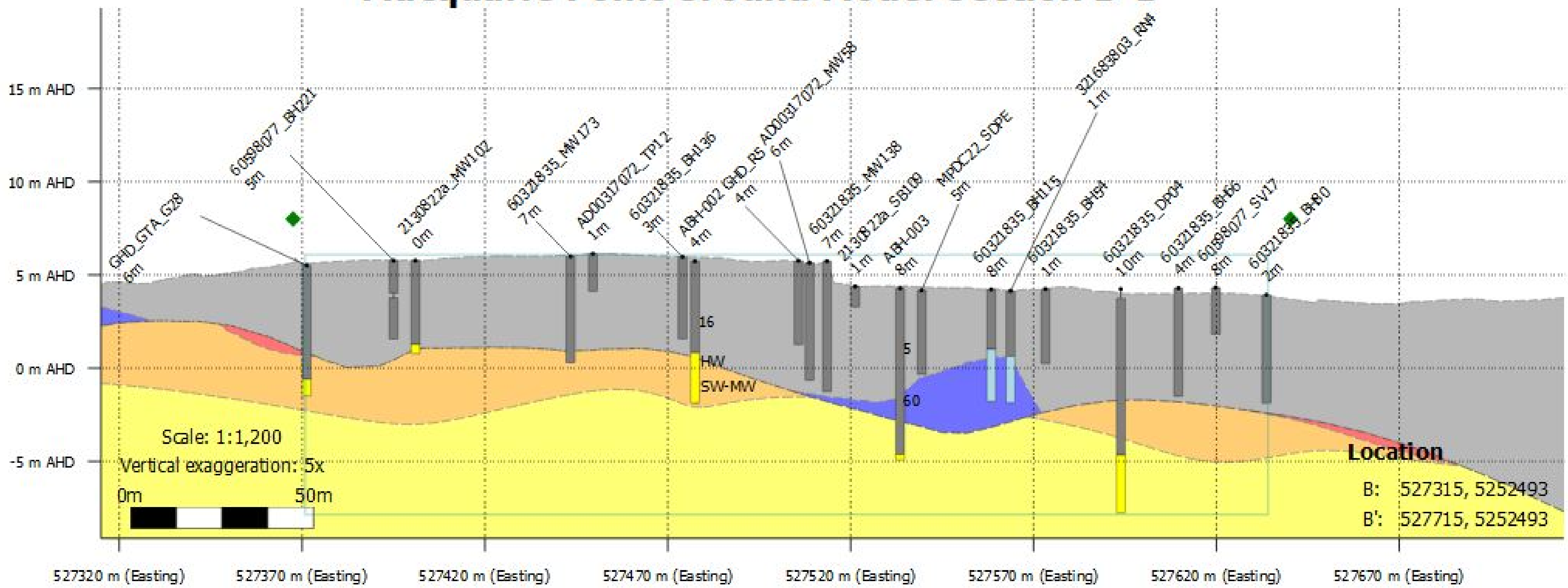
Document type	Document status
Section	DRAFT
Title	Identification number
Macquarie Point Ground Model Section A-A'	PS212776

Rev.	Date of issue	Sheet
A01	12/06/2024	1/6

B

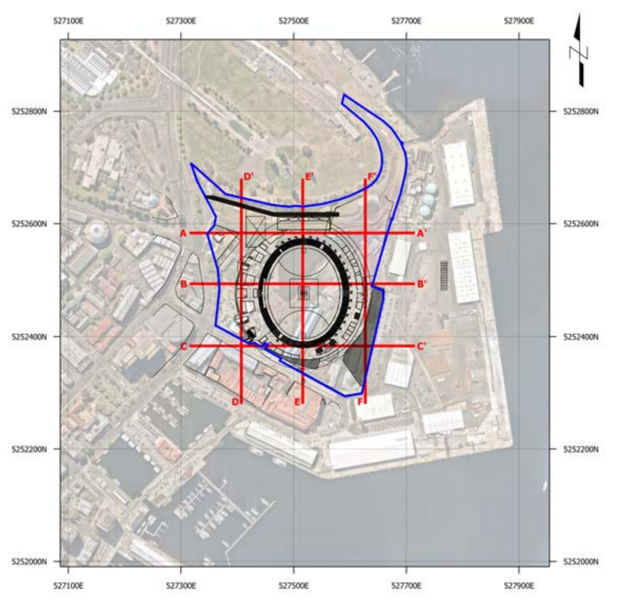
Macquarie Point Ground Model Section B-B'

B'



Looking North

Site Plan



Legend

Gound Model

FILL · Estuarine

Dolerite, MW-HW · Dolerite, Fr-SW

Major Material Logged

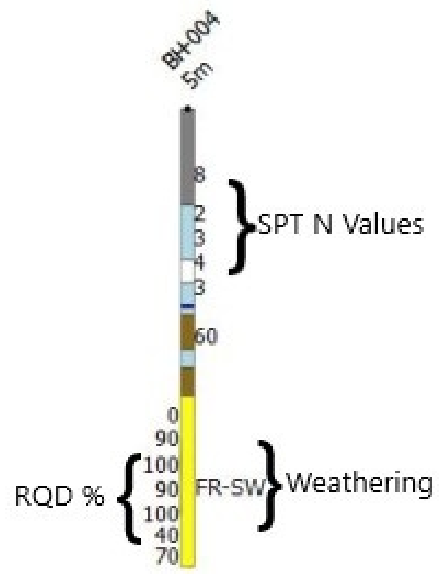
Coreloss · Cobbles · Clay · Sandstone

Fill · Sand · Boulders · Dolerite

Gravel · Silt · Mudstone

◆ Mac Point Precinct Plan_20231106 - 02 Site Boundary_Boundary

Graphic Log Legend



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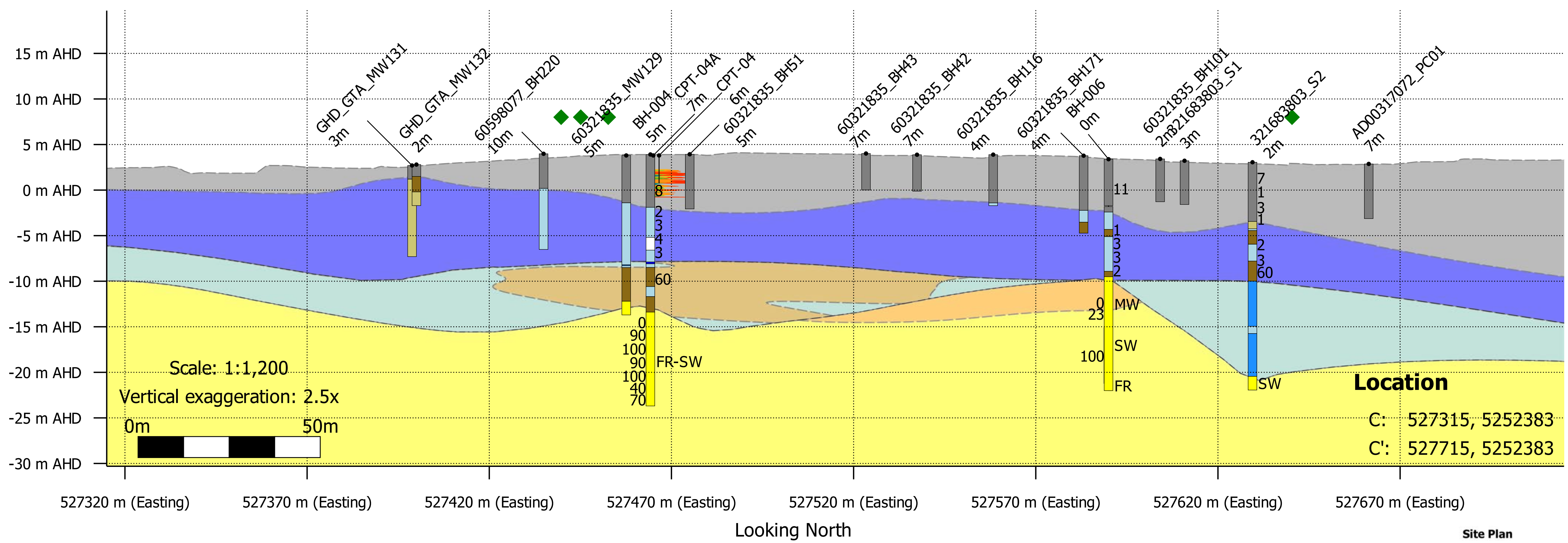
wsp		Document type Section	Document status DRAFT
		Title Macquarie Point Ground Model Section B-B'	Identification number PS212776
			Rev. A01 · Date of issue 12/06/2024 · Sheet 2/6

Macquarie Point Ground Model Section C-C'



Legend

Gound Model

- FILL
- Estuarine
- ALV, Cohesive
- ALV, Granular
- Dolerite, MW-HW
- Dolerite, Fr-SW

Major Material Logged

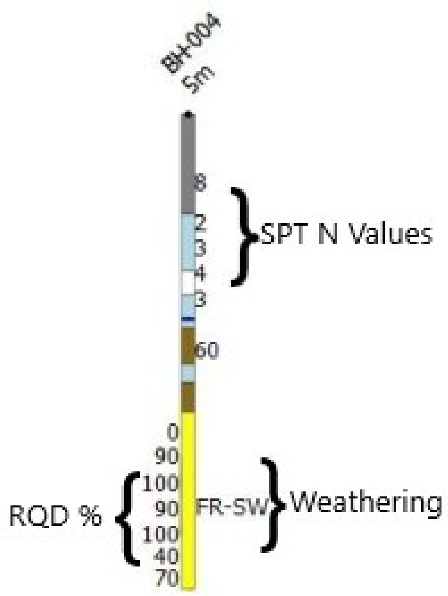
- coreloss
- Gravel
- SAND
- clay
- mudstone
- dolerite
- Fill
- COBBLES
- silt
- BOULDERS
- sandstone

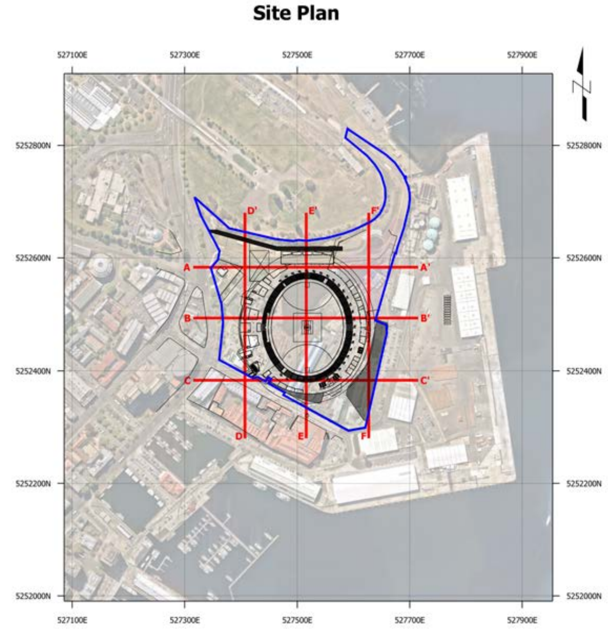
Conetip_Qt

0 1.8 3.5 5.2 7

- Mac Point Precinct Plan_20231106 - 02 Site Boundary_Boundary

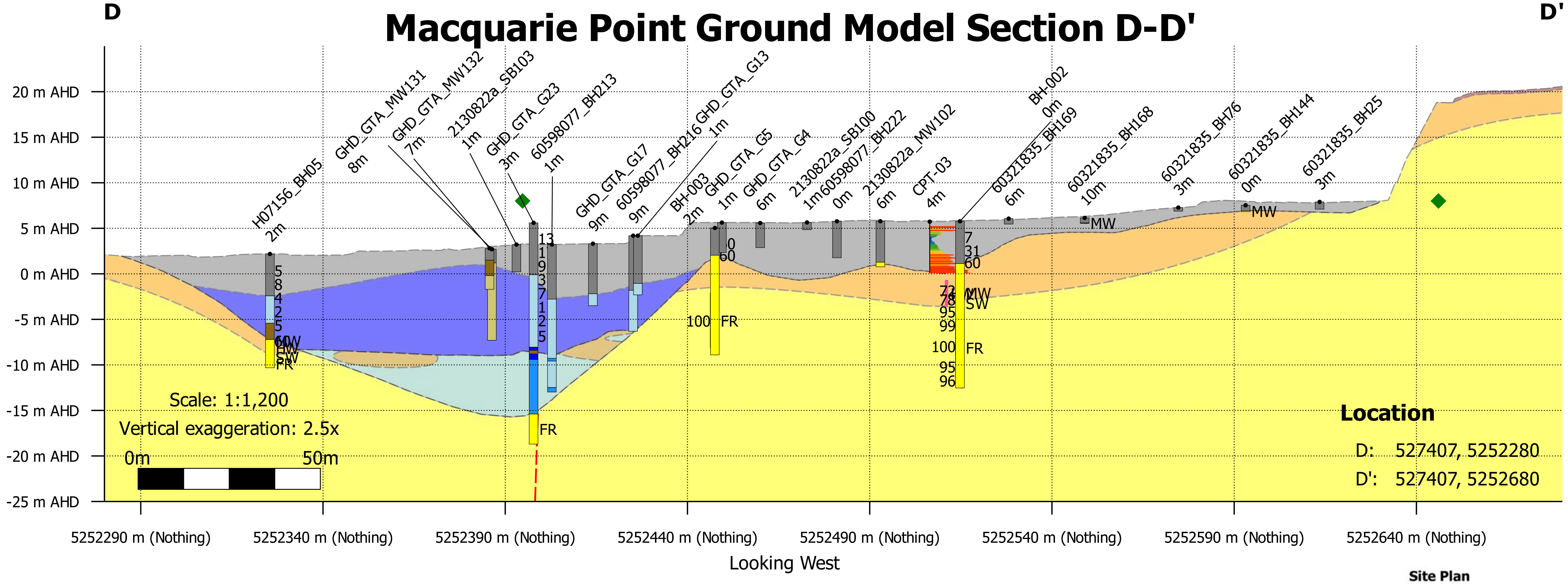
Graphic Log Legend



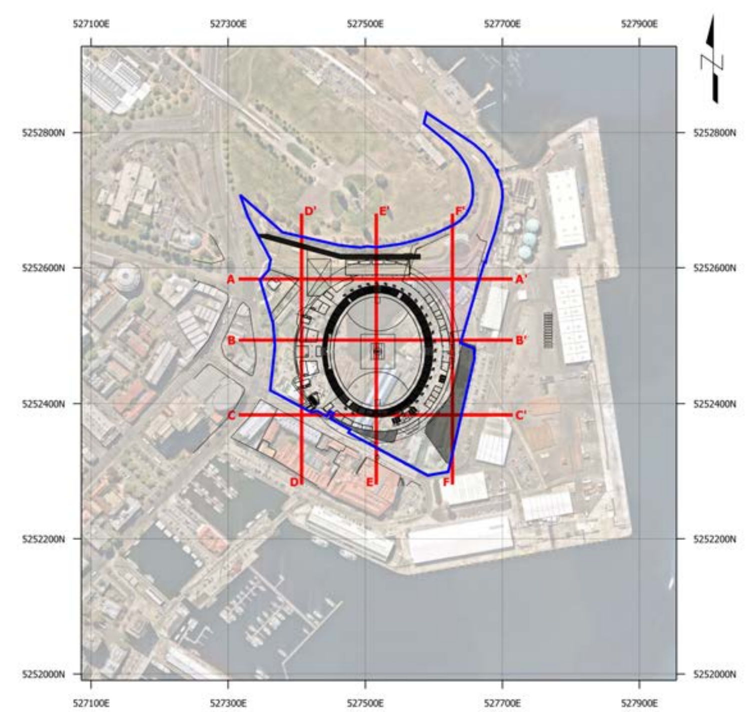


		Creator	Approved by
		JRG	TDM
wsp		Document type: Section	Document status: DRAFT
		Title: Macquarie Point Ground Model Section C-C'	Identification number: PS212776
			Rev.: A01 / Date of issue: 12/06/2024 / Sheet: 3/6

Macquarie Point Ground Model Section D-D'



Site Plan



Legend

Gound Model

FILL, Estuarine

ALV, Cohesive, ALV, Granular

Dolerite, RS, Dolerite, MW-HW, Dolerite, Fr-SW

Major Material Logged

coreloss, Gravel, SAND, clay, mudstone, dolerite

Fill, COBBLES, silt, BOULDERS, sandstone

Conetip_Qt

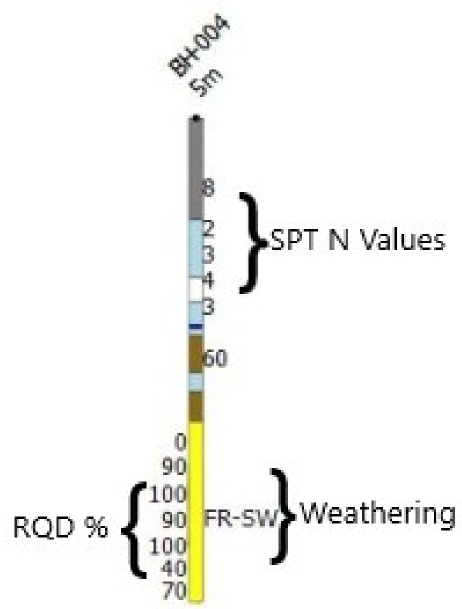
0 1.8 3.5 5.2 7

Surfaces

normal fault inside Dolerite

Mac Point Precinct Plan_20231106 - 02 Site Boundary_Boundary

Graphic Log Legend



		Creator	Approved by
		JRG	TDM

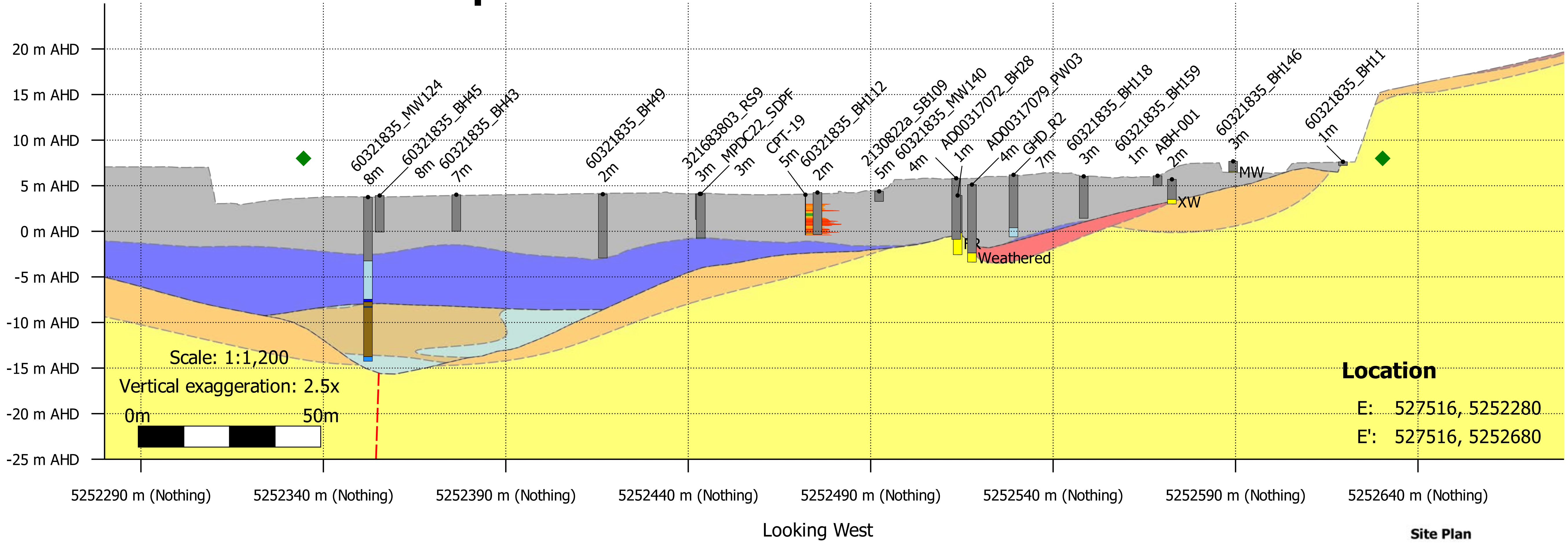
Document type	Document status
Section	DRAFT
Title	Identification number
Macquarie Point Ground Model Section D-D'	PS212776

Rev.	Date of issue	Sheet
A01	12/06/2024	4/6

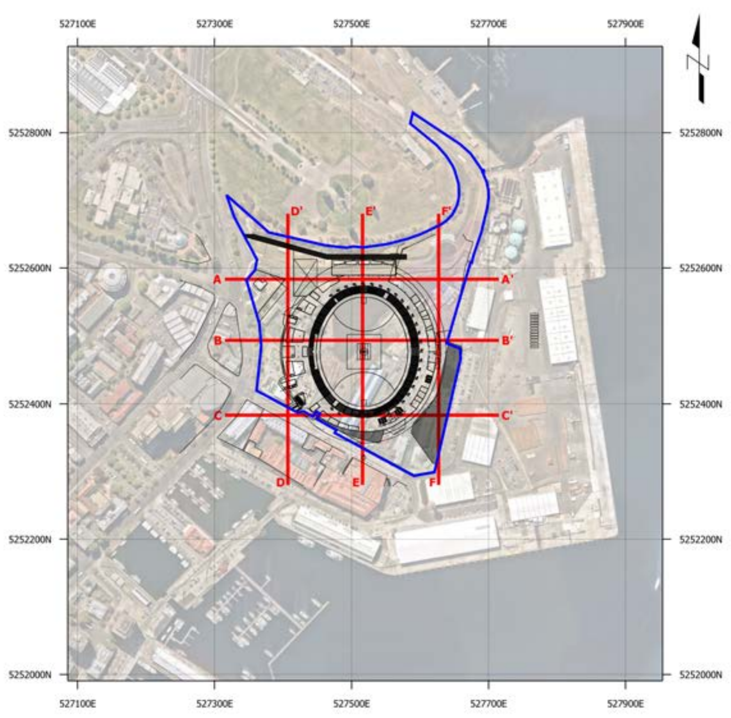
E

Macquarie Point Ground Model Section E-E'

E'



Site Plan



Legend

Gound Model

FILL Estuarine

ALV, Cohesive ALV, Granular

Dolerite, RS Dolerite, XW Dolerite, MW-HW Dolerite, Fr-SW

Major Material Logged

coreloss Gravel SAND clay mudstone dolerite

Fill COBBLES silt BOULDERS sandstone

Conetip_Qt

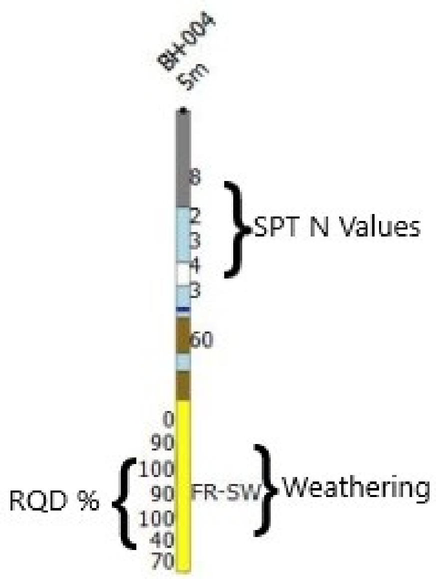
0 1.8 3.5 5.2 7

Surfaces

normal fault inside Dolerite

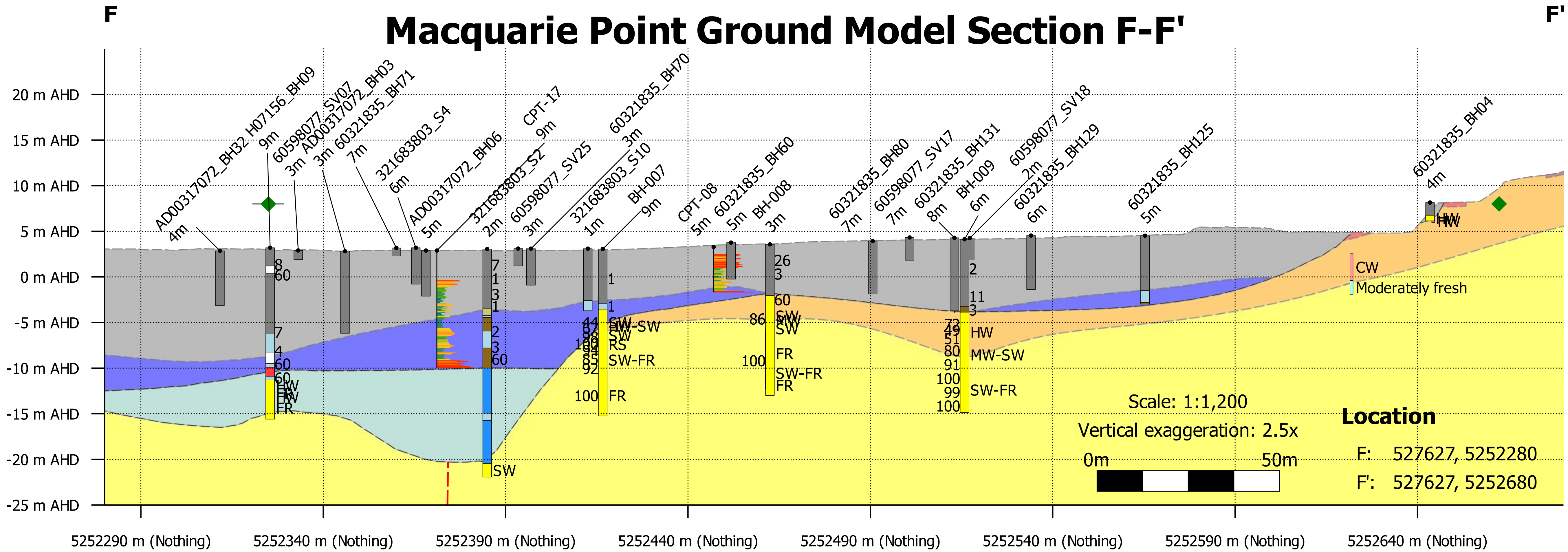
Mac Point Precinct Plan_20231106 - 02 Site Boundary_Boundary

Graphic Log Legend



		Creator JRG	Approved by TDM		
wsp		Document type Section	Document status DRAFT		
		Title Macquarie Point Ground Model Section F-F'	Identification number PS212776		
			Rev. A01	Date of issue 12/06/2024	Sheet 5/6

Macquarie Point Ground Model Section F-F'



Looking West

Legend

Gound Model

FILL Estuarine

ALV, Granular

Dolerite, RS Dolerite, XW Dolerite, MW-HW Dolerite, Fr-SW

Major Material Logged

coreloss Gravel SAND clay mudstone dolerite

Fill COBBLES silt BOULDERS sandstone

Conetip_Qt

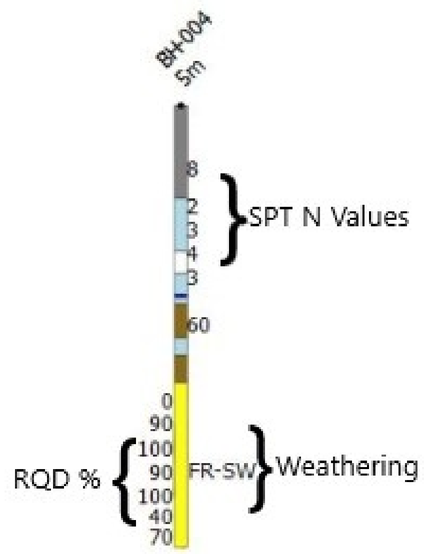
0 1.8 3.5 5.2 7

Surfaces

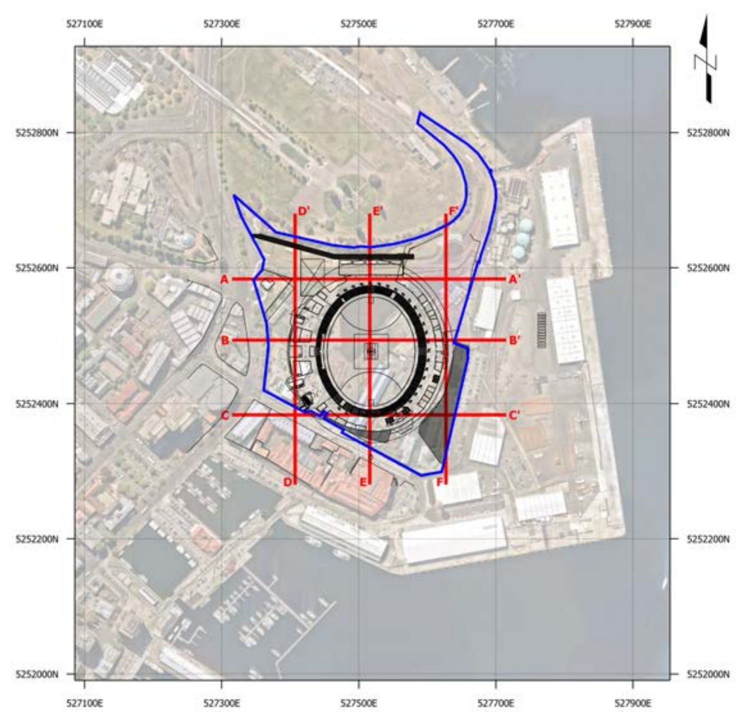
normal fault inside Dolerite

Mac Point Precinct Plan_20231106 - 02 Site Boundary_Boundary

Graphic Log Legend



Site Plan



		Creator	Approved by
		JRG	TDM
wsp		Document type: Section	Document status: DRAFT
		Title: Macquarie Point Ground Model Section F-F'	Identification number: PS212776
			Rev.: A01 / Date of issue: 12/06/2024 / Sheet: 6/6

Appendix C

Borehole plots – CPT

SPT figures

Fig	Title
C1	SPT N Value vs Elevation (All Lithologies)
C2	SPT N Value vs Elevation for Fill
C3	SPT N Value vs Elevation for Estuarine
C4	SPT N Value vs Elevation for Alluvium

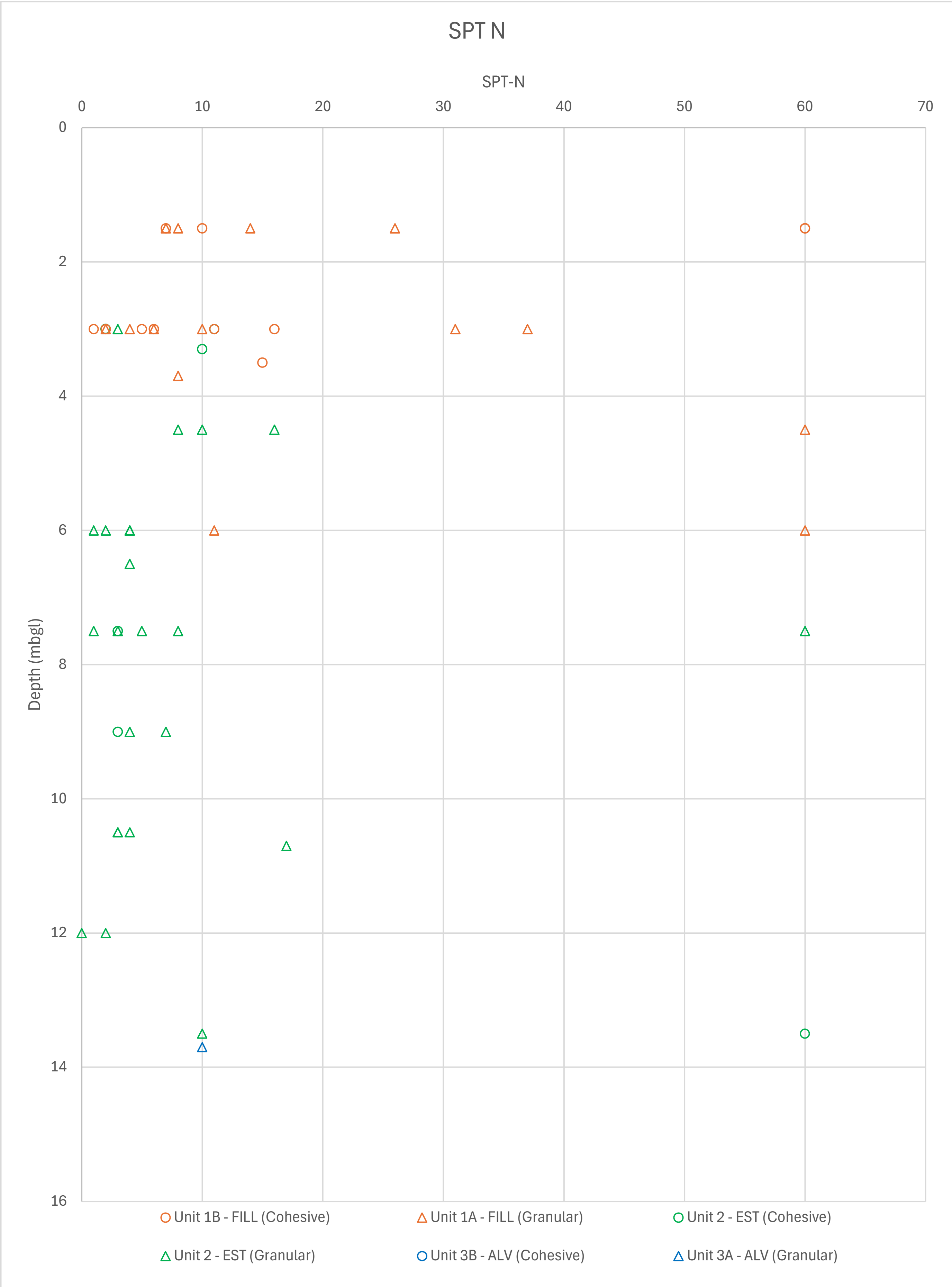


Fig. C1

Macquarie Point Stadium, Hobart
SPT N Values with Depth

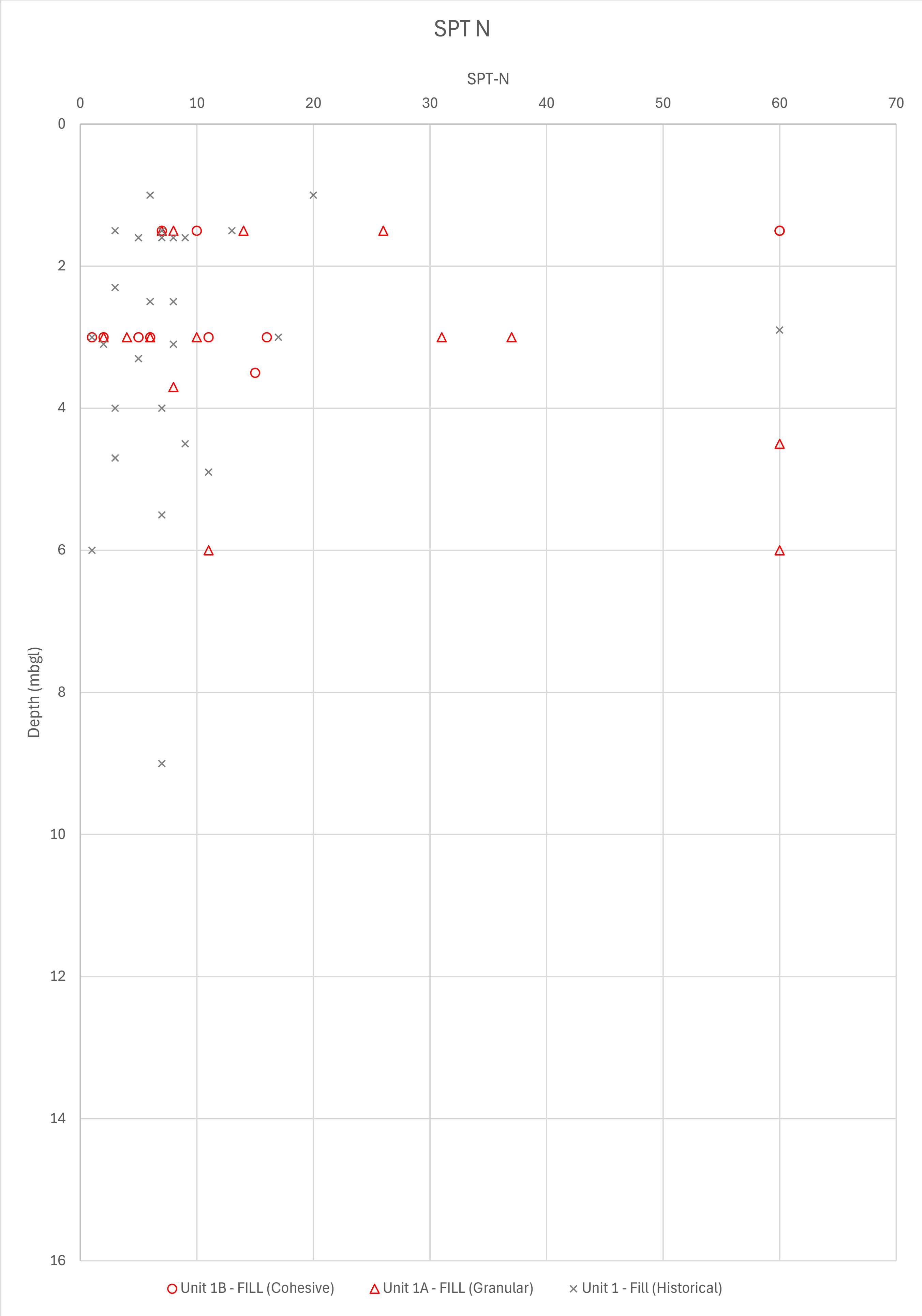


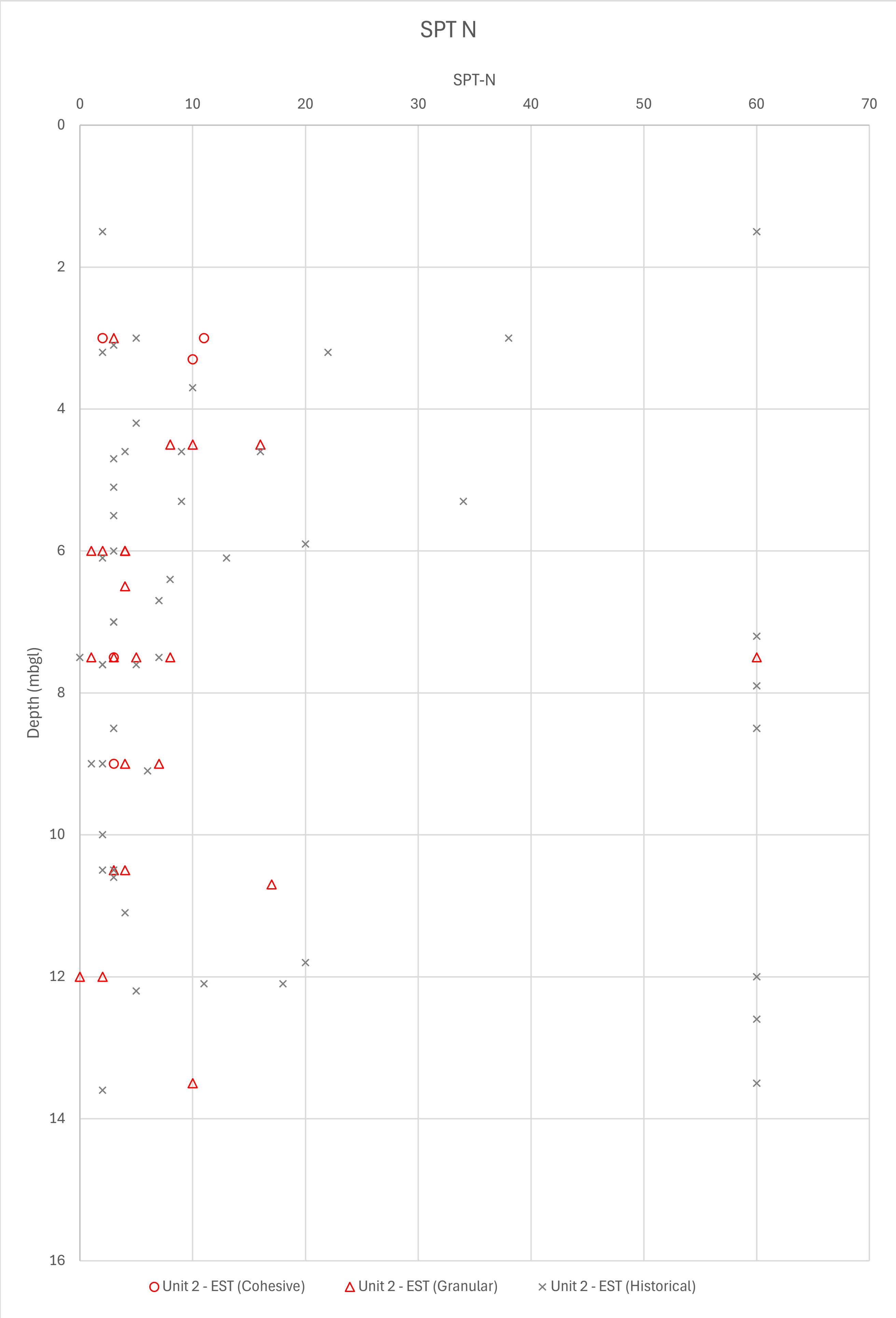
Fig. C2

Macquarie Point Stadium, Hobart

SPT N Values with Depth

Fill

N = 60 indicates refusal



wsp

Fig. C3

Macquarie Point Stadium, Hobart

SPT N Values with Depth

Estuarine

N = 60 indicates refusal

MAC POINT

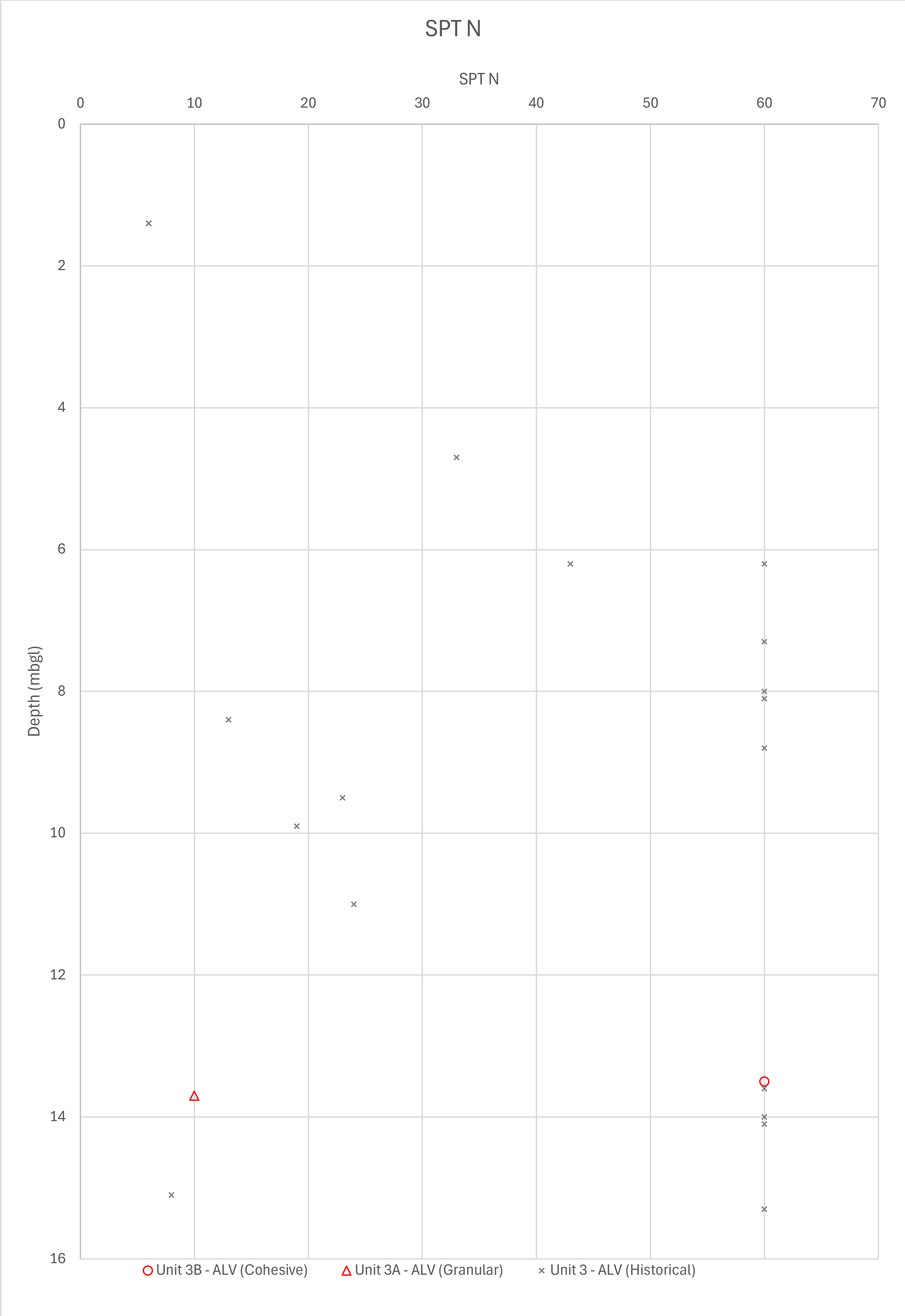


Fig. C4

Macquarie Point Stadium, Hobart

SPT N Values with Depth

Alluvium

N = 60 indicates refusal

Appendix D

CPT interpretation

CPT Interpretation

CPT-03

Total depth: 5.64 m, Date: 19/04/2024

Surface Elevation: 5.73 m

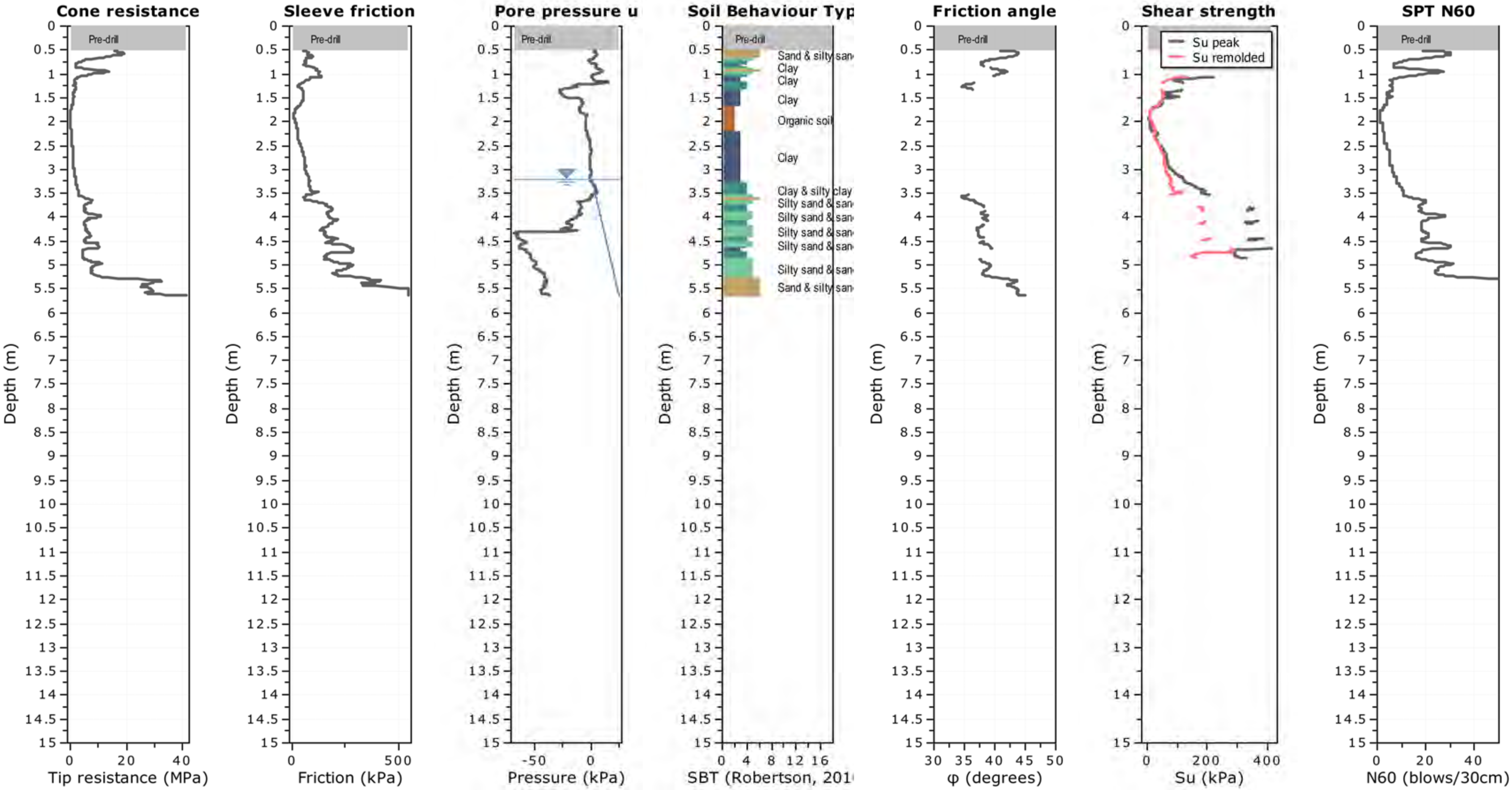
Coords: X:527411.04, Y:5252505.03

Cone Type: C15CFIIPT.C21201

Cone Operator: IGS

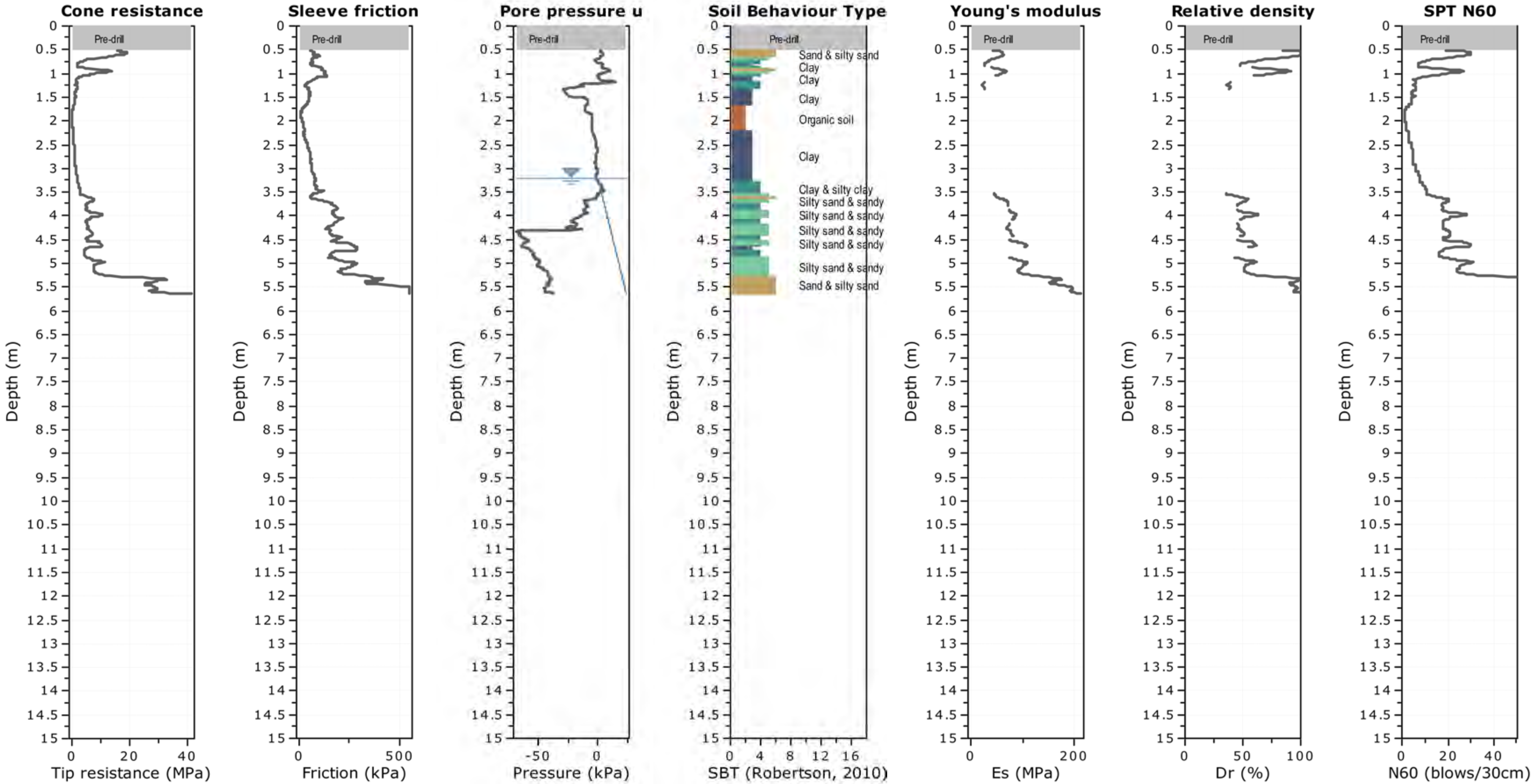
Project: PS212776 Macquarie Point

Location: Hobart, Tasmania



Project: PS212776 Macquarie Point
Location: Hobart, Tasmania

CPT-03
Total depth: 5.64 m, Date: 19/04/2024
Surface Elevation: 5.73 m
Coords: X:527411.04, Y:5252505.03
Cone Type: C15CFIIPT.C21201
Cone Operator: IGS



CPT-04

Total depth: 3.07 m, Date: 19/04/2024

Surface Elevation: 3.80 m

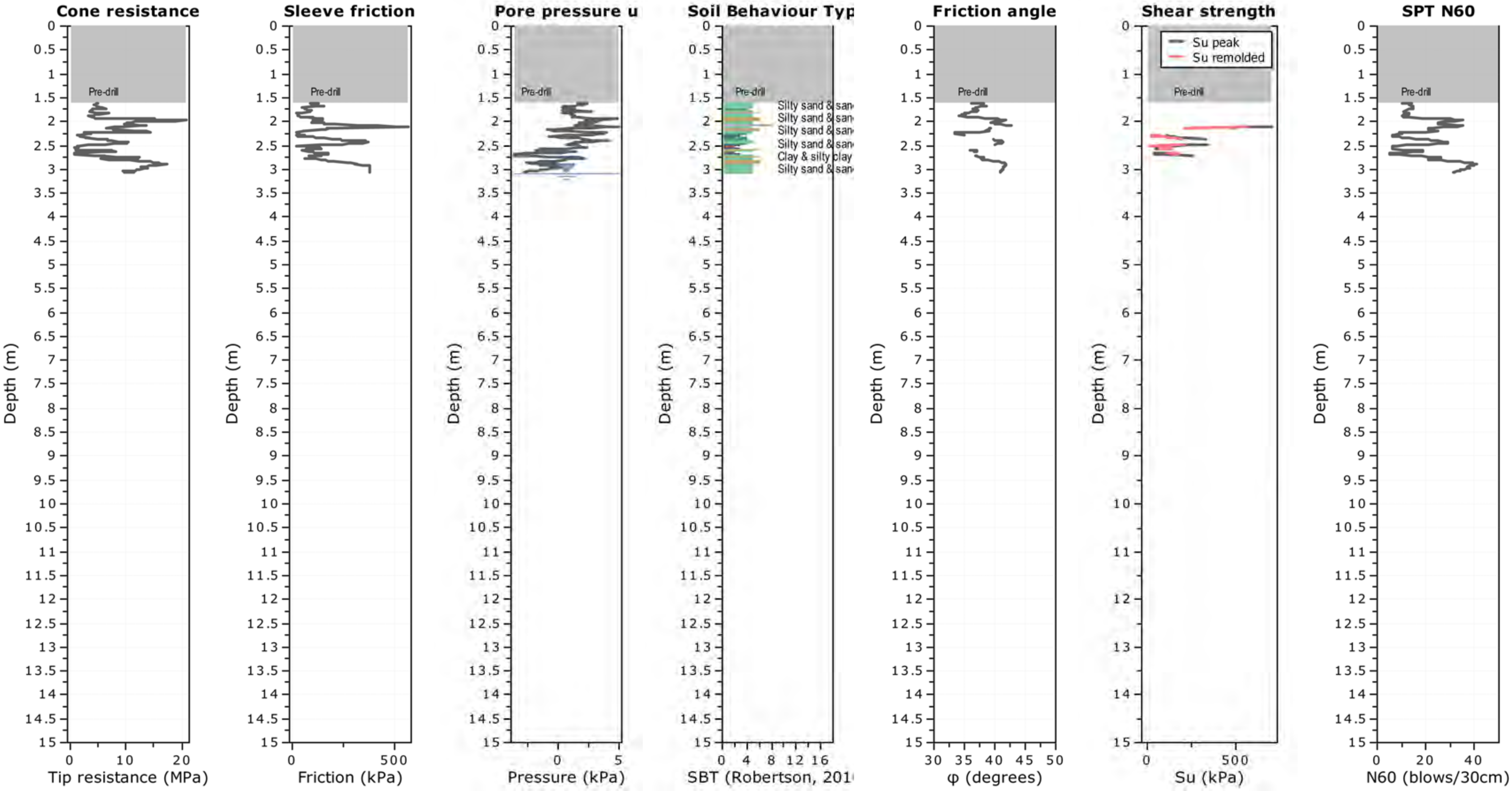
Coords: X:527466.20, Y:5252389.46

Cone Type: C15CFIIPT.C21201

Cone Operator: IGS

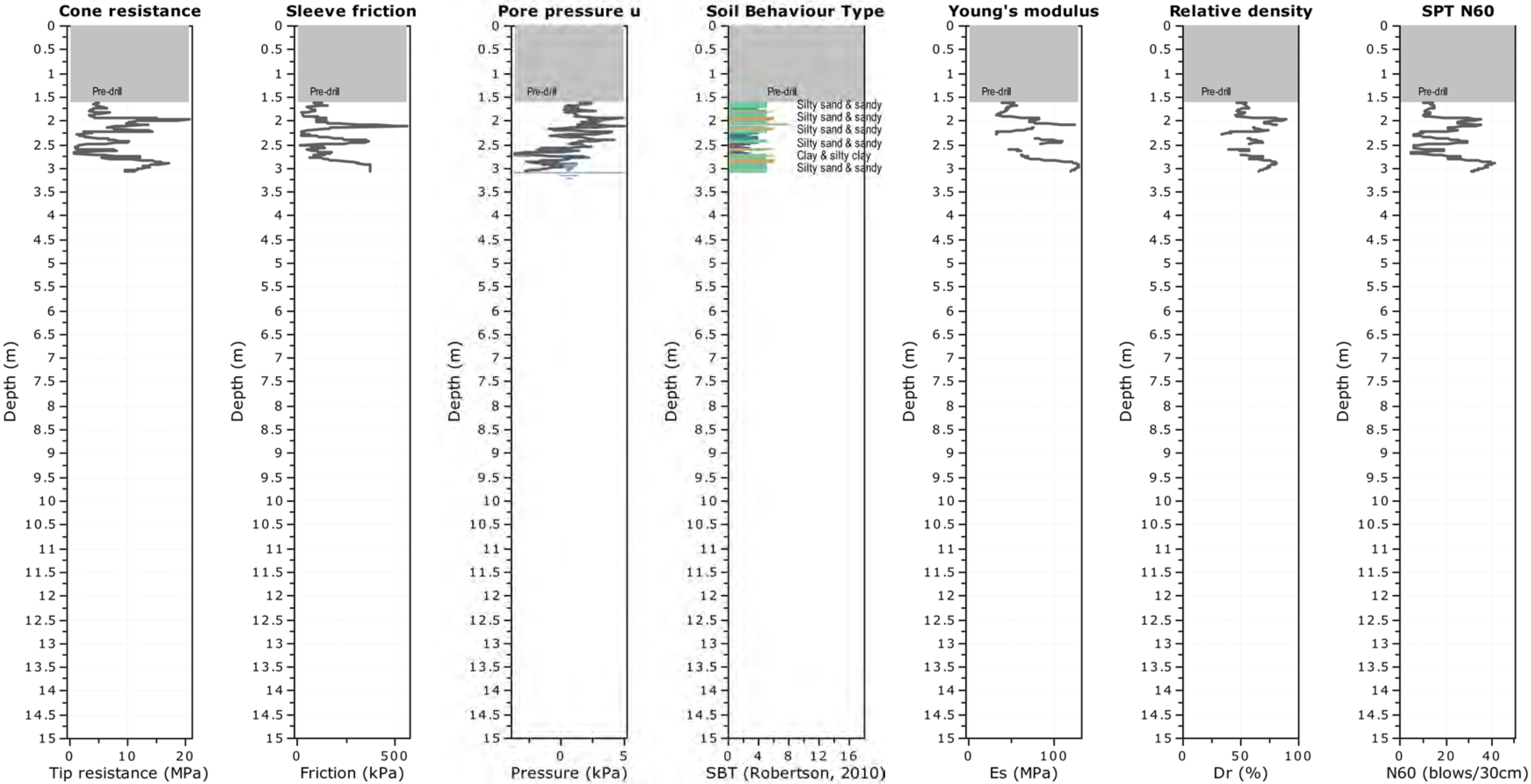
Project: PS212776 Macquarie Point

Location: Hobart, Tasmania



Project: PS212776 Macquarie Point
Location: Hobart, Tasmania

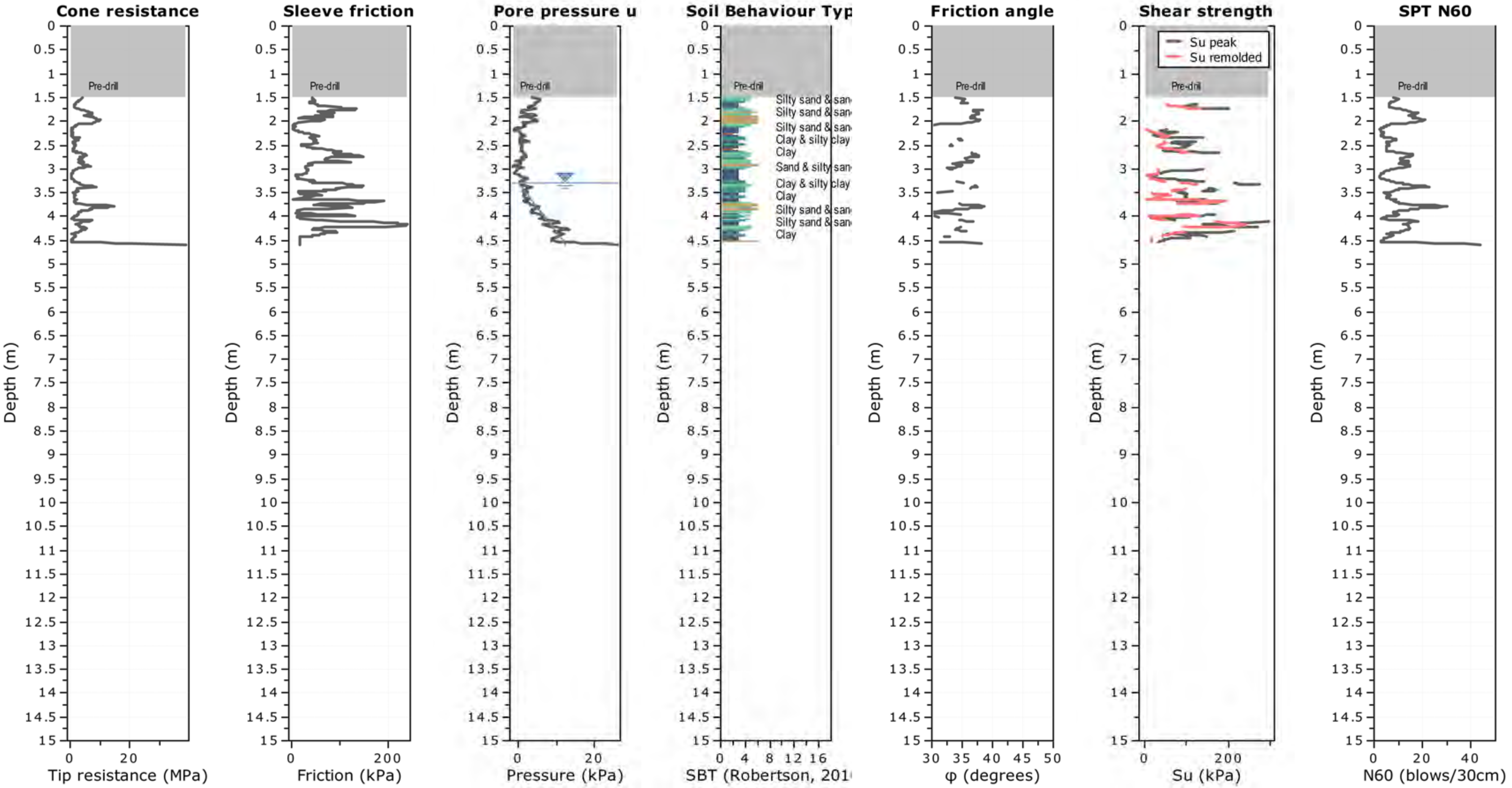
CPT-04
Total depth: 3.07 m, Date: 19/04/2024
Surface Elevation: 3.80 m
Coords: X:527466.20, Y:5252389.46
Cone Type: C15CFIIPT.C21201
Cone Operator: IGS



CPT-04a

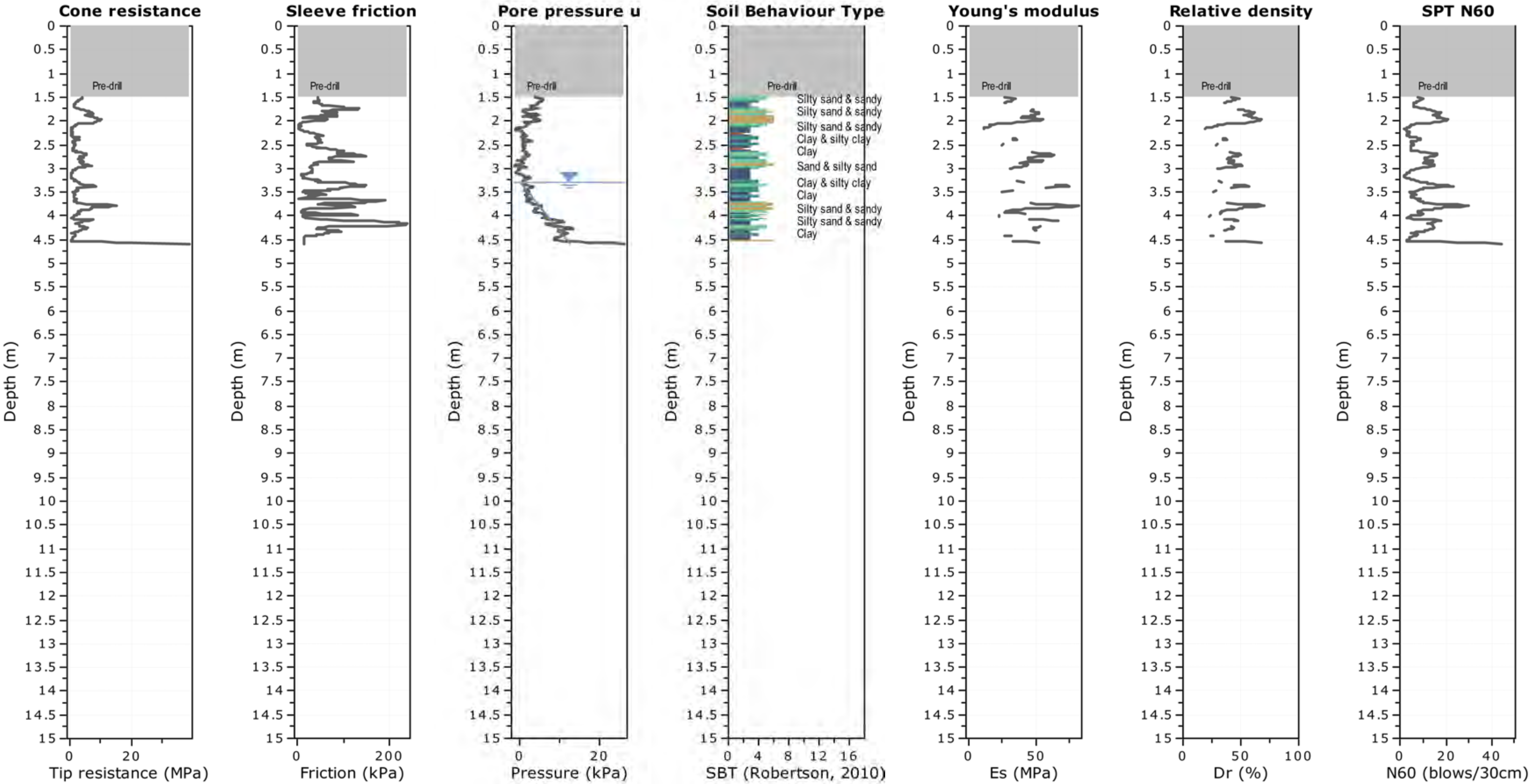
Total depth: 4.58 m, Date: 19/04/2024
Surface Elevation: 3.80 m
Coords: X:527466.20, Y:5252389.46
Cone Type: C15CFIIPT.C21201
Cone Operator: IGS

Project: PS212776 Macquarie Point
Location: Hobart, Tasmania



Project: PS212776 Macquarie Point
Location: Hobart, Tasmania

CPT-04a
Total depth: 4.58 m, Date: 19/04/2024
Surface Elevation: 3.80 m
Coords: X:527466.20, Y:5252389.46
Cone Type: C15CFIIPT.C21201
Cone Operator: IGS



Project: PS212776 Macquarie Point

Location: Hobart, Tasmania

CPT-05

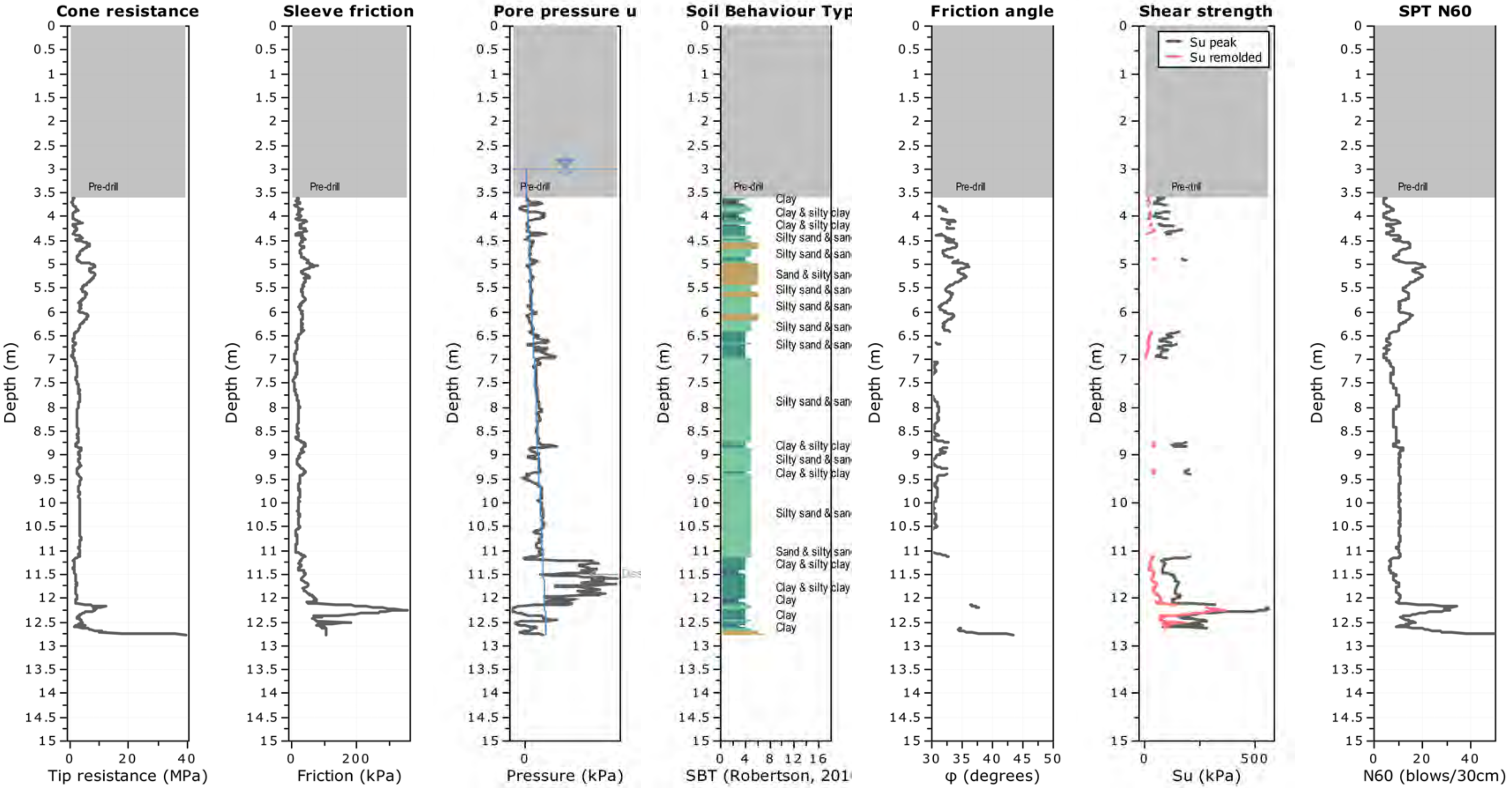
Total depth: 12.77 m, Date: 10/05/2024

Surface Elevation: 3.80 m

Coords: X:527531.01, Y:5252371.94

Cone Type: C10CFIIP.C181094

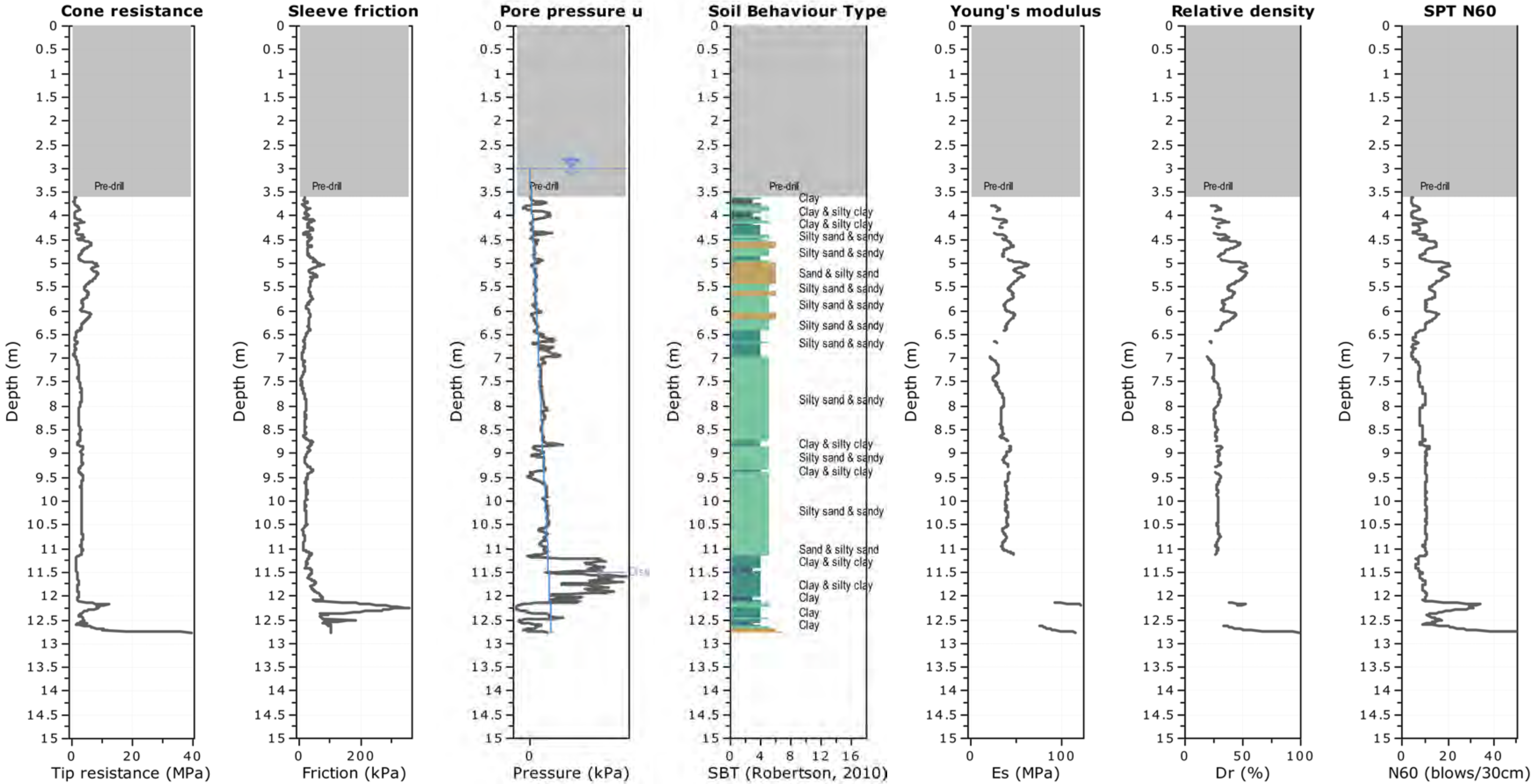
Cone Operator: IGS



CPT-05

Total depth: 12.77 m, Date: 10/05/2024
Surface Elevation: 3.80 m
Coords: X:527531.01, Y:5252371.94
Cone Type: C10CFIIP.C181094
Cone Operator: IGS

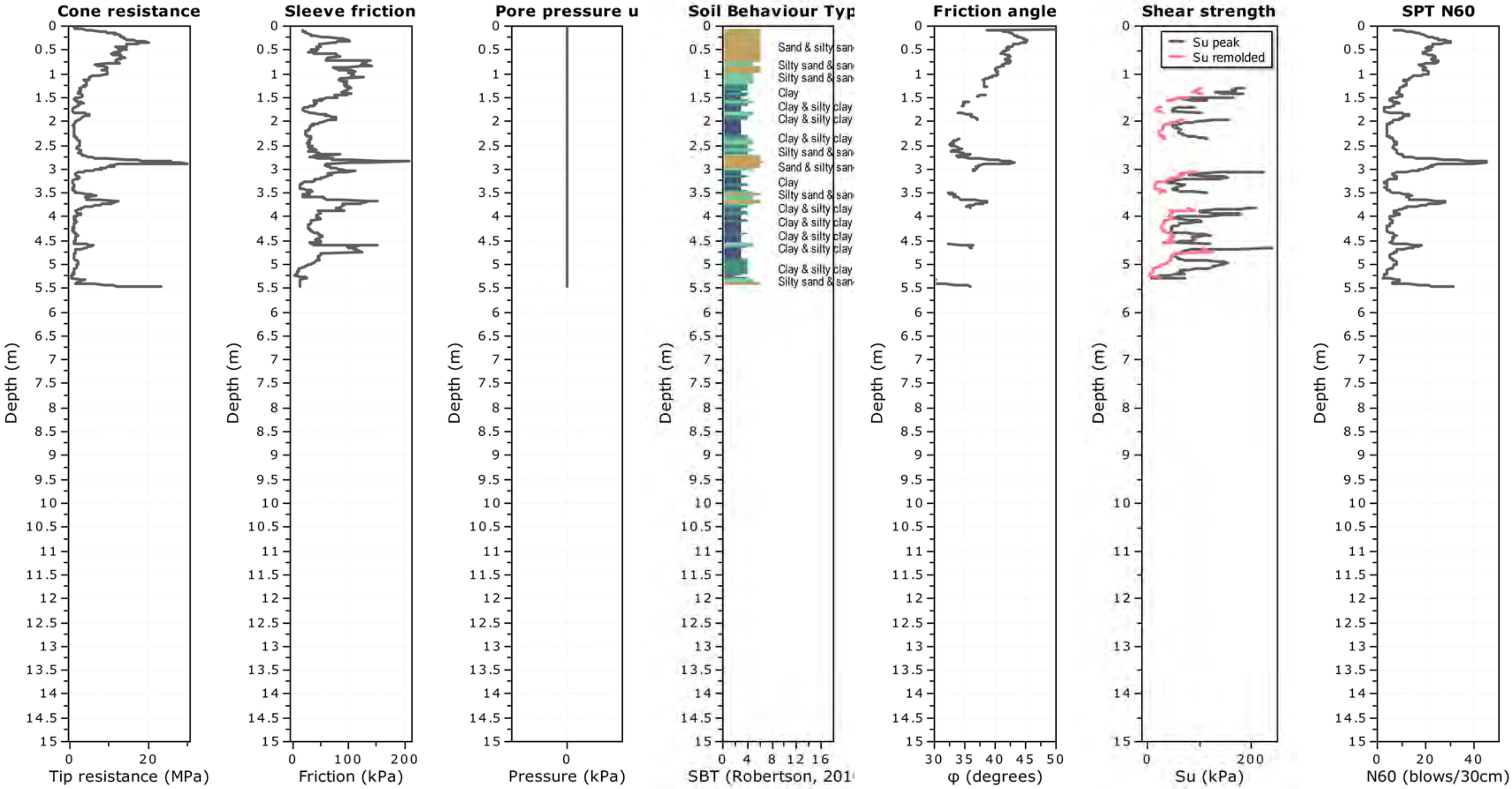
Project: PS212776 Macquarie Point
Location: Hobart, Tasmania



CPT-06

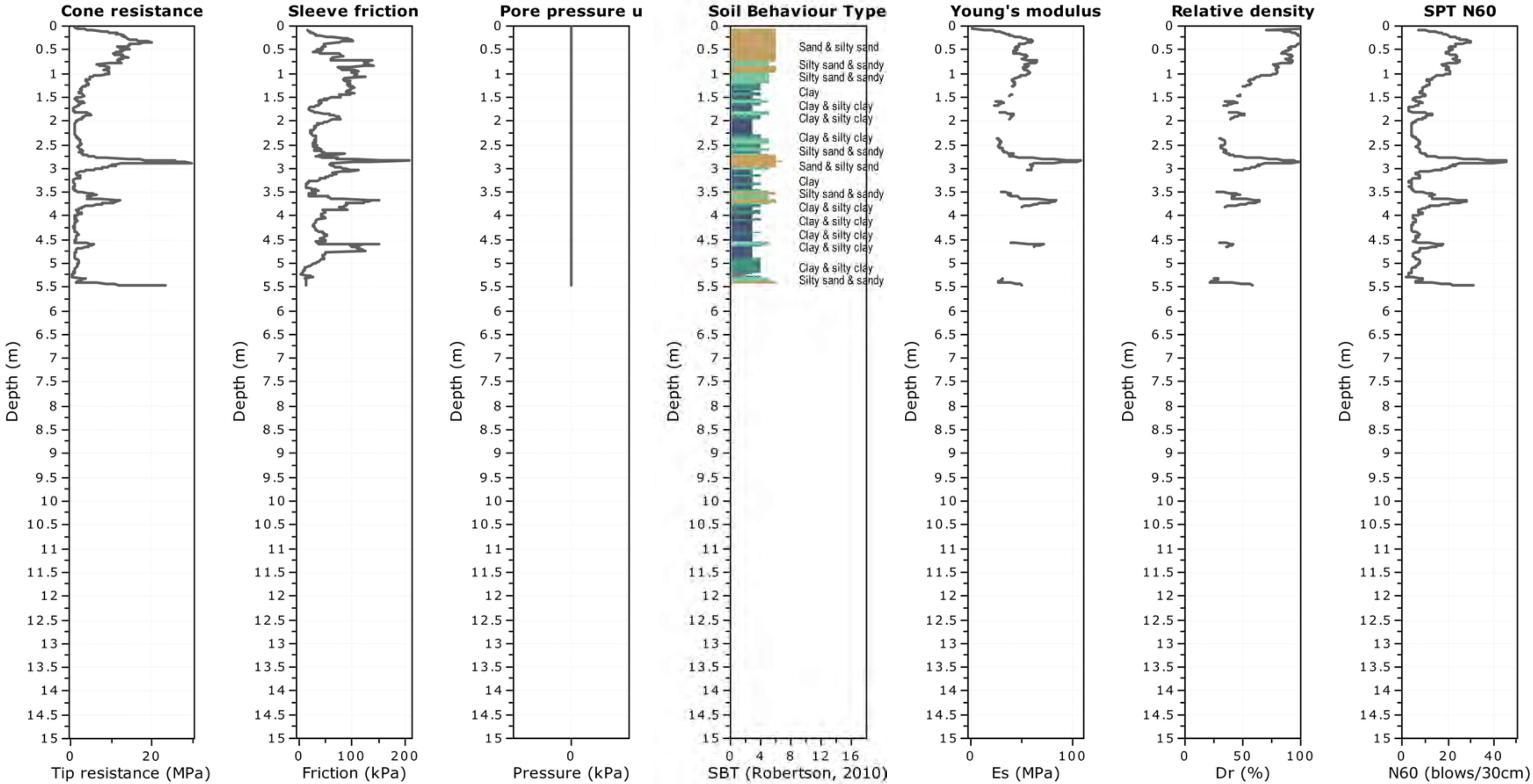
Total depth: 5.47 m, Date: 18/04/2024
Surface Elevation: 5.23 m
Coords: X:527393.24, Y:5252464.57
Cone Type: C15CFIIPT.C21200
Cone Operator: IGS

Project: PS212776 Macquarie Point
Location: Hobart, Tasmania



Project: PS212776 Macquarie Point
Location: Hobart, Tasmania

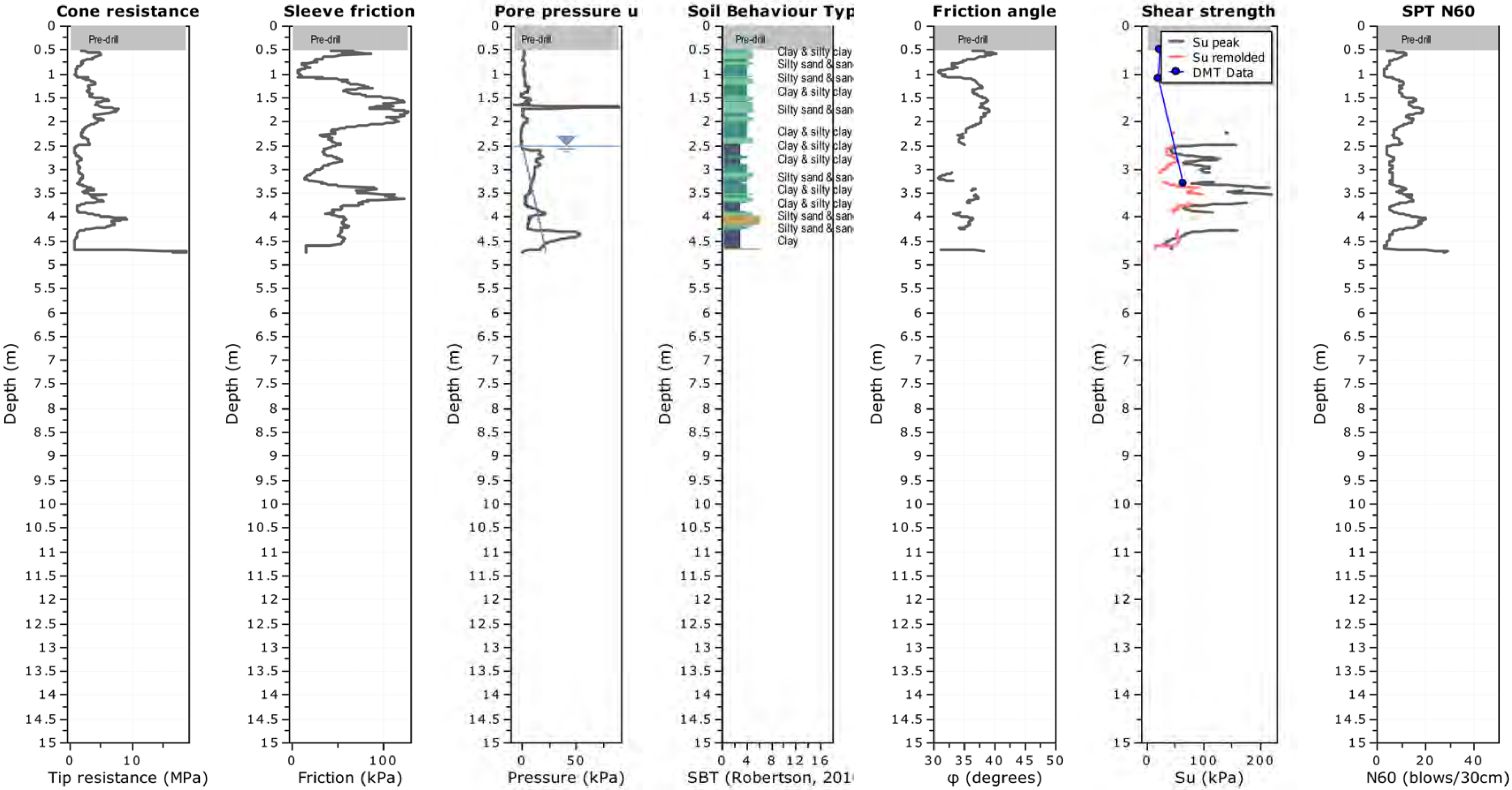
CPT-06
Total depth: 5.47 m, Date: 18/04/2024
Surface Elevation: 5.23 m
Coords: X:527393.24, Y:5252464.57
Cone Type: C15CFIIPT.C21200
Cone Operator: IGS



CPT-07

Total depth: 4.74 m, Date: 16/04/2024
Surface Elevation: 5.10 m
Coords: X:527395.92, Y:5252407.64
Cone Type: C15CFIIPT.C21200
Cone Operator: IGS

Project: PS212776 Macquarie Point
Location: Hobart, Tasmania



CPT-07

Total depth: 4.74 m, Date: 16/04/2024

Surface Elevation: 5.10 m

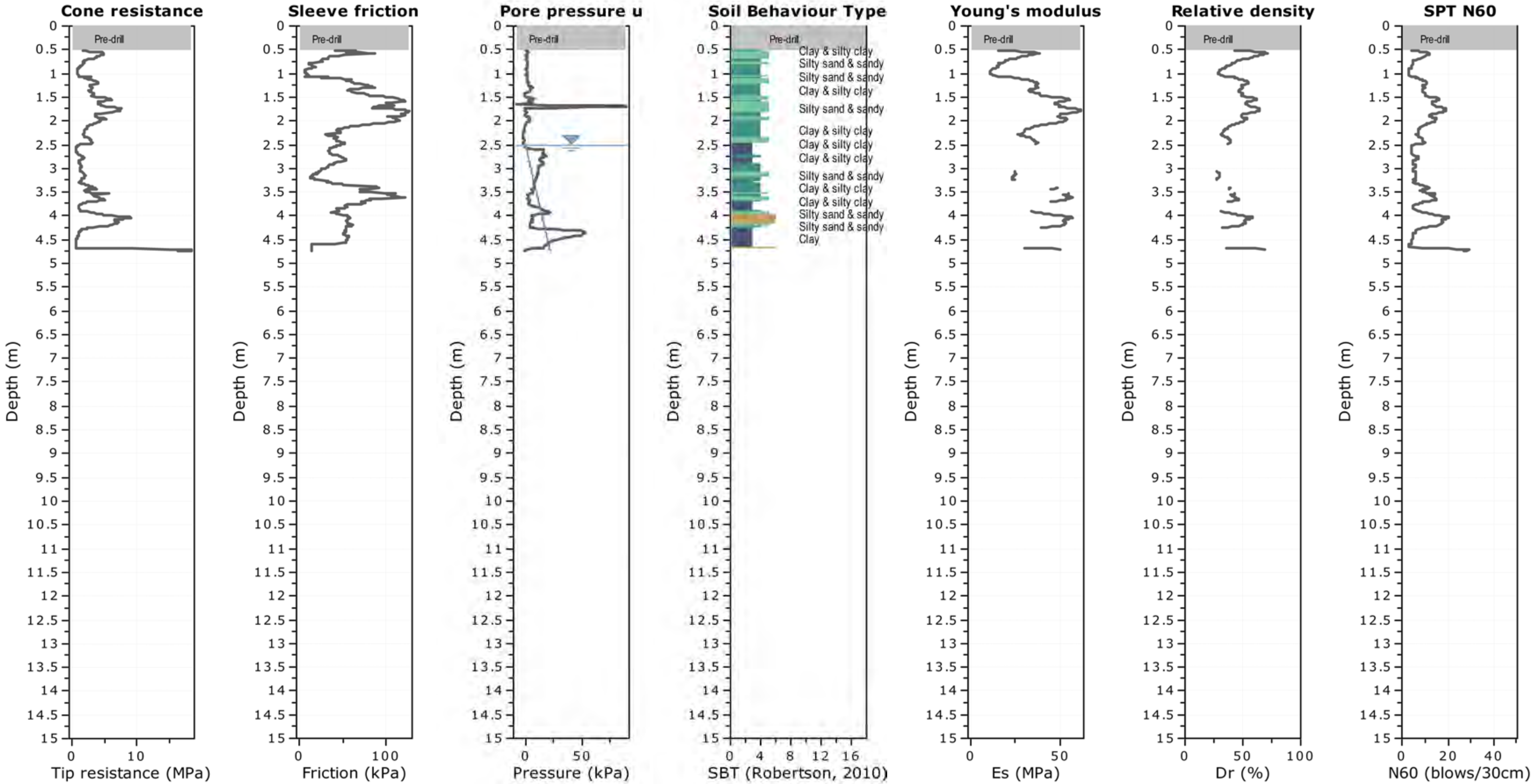
Coords: X:527395.92, Y:5252407.64

Cone Type: C15CFIIPT.C21200

Cone Operator: IGS

Project: PS212776 Macquarie Point

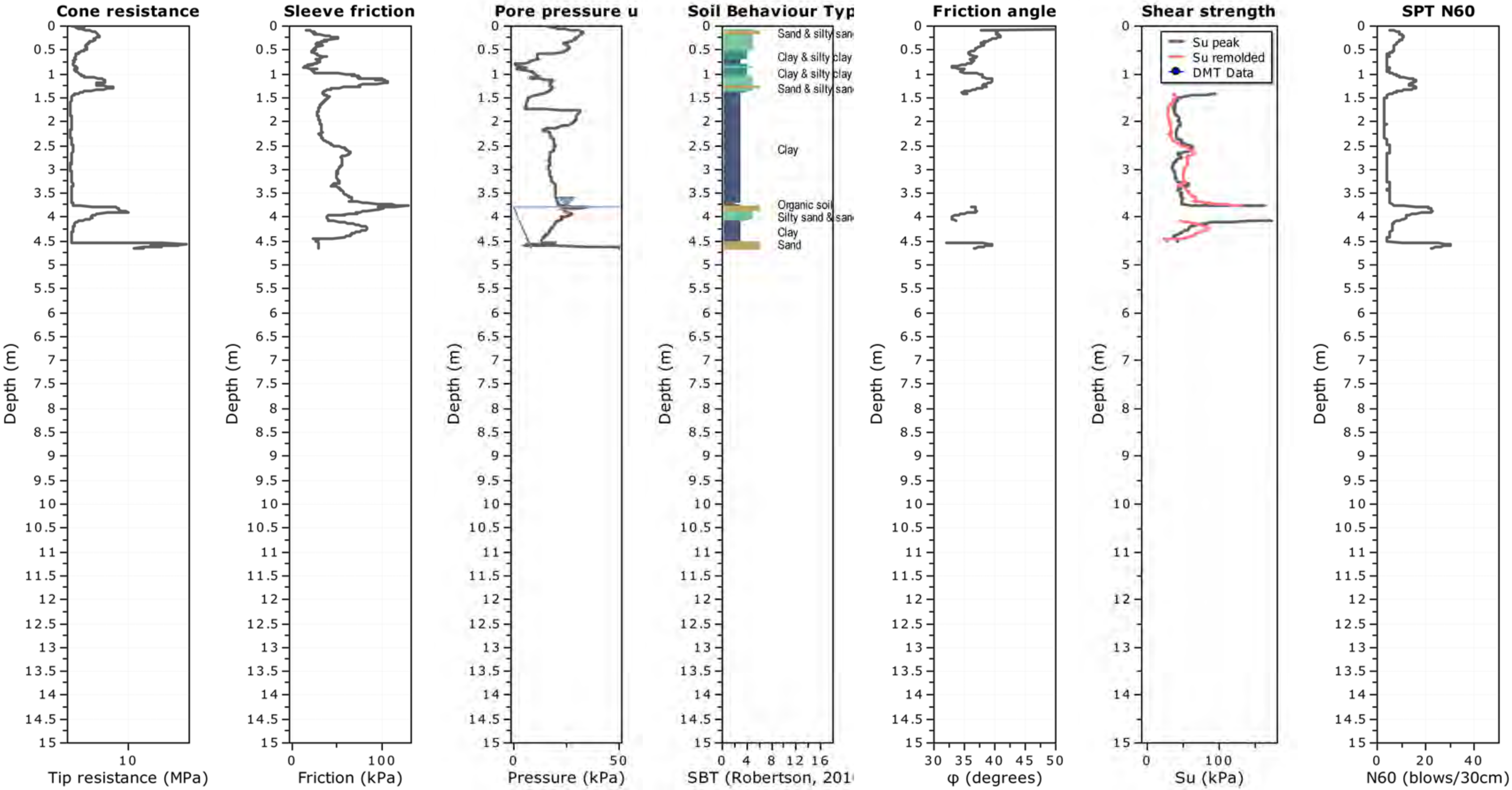
Location: Hobart, Tasmania



CPT-07a

Total depth: 4.67 m, Date: 16/04/2024
Surface Elevation: 5.10 m
Coords: X:527394.73, Y:5252408.59
Cone Type: C15CFIIPT.C21200
Cone Operator: IGS

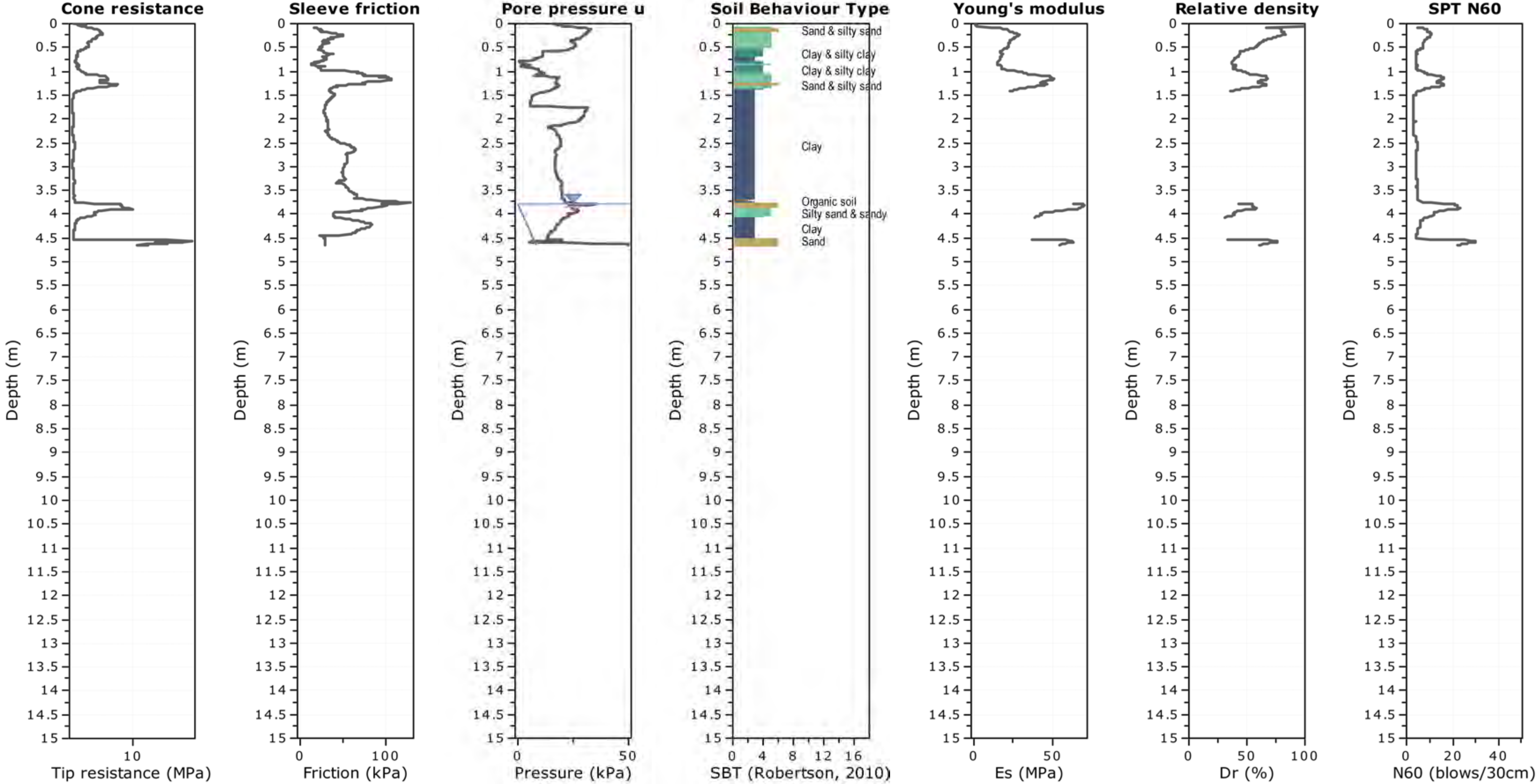
Project: PS212776 Macquarie Point
Location: Hobart, Tasmania



CPT-07a

Total depth: 4.67 m, Date: 16/04/2024
Surface Elevation: 5.10 m
Coords: X:527394.73, Y:5252408.59
Cone Type: C15CFIIPT.C21200
Cone Operator: IGS

Project: PS212776 Macquarie Point
Location: Hobart, Tasmania



CPT-08

Total depth: 4.97 m, Date: 9/05/2024

Surface Elevation: 3.29 m

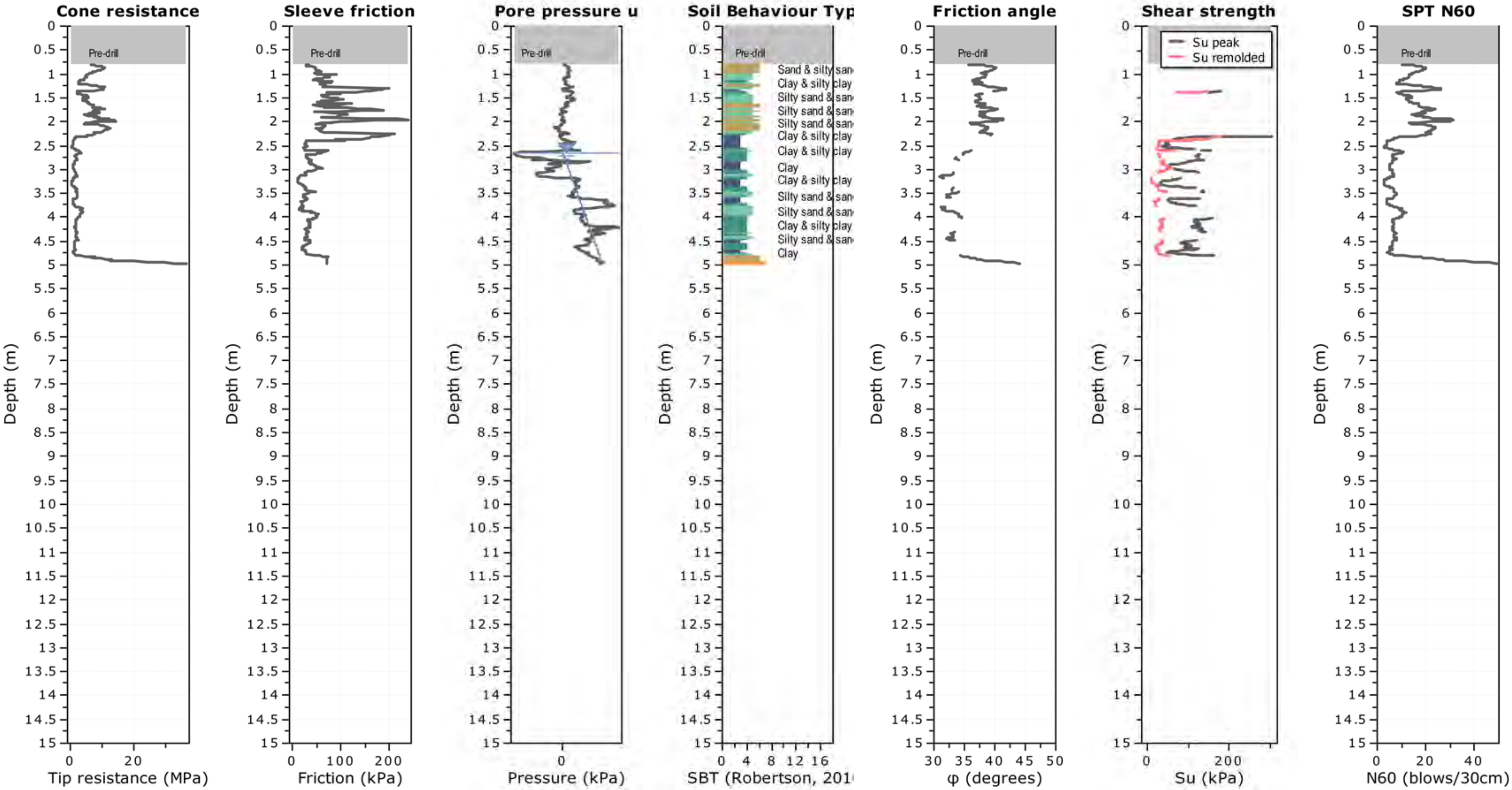
Coords: X:527631.78, Y:5252445.60

Cone Type: C10CFIIP.C181094

Cone Operator: IGS

Project: PS212776 Macquarie Point

Location: Hobart, Tasmania



Project: PS212776 Macquarie Point

Location: Hobart, Tasmania

CPT-08

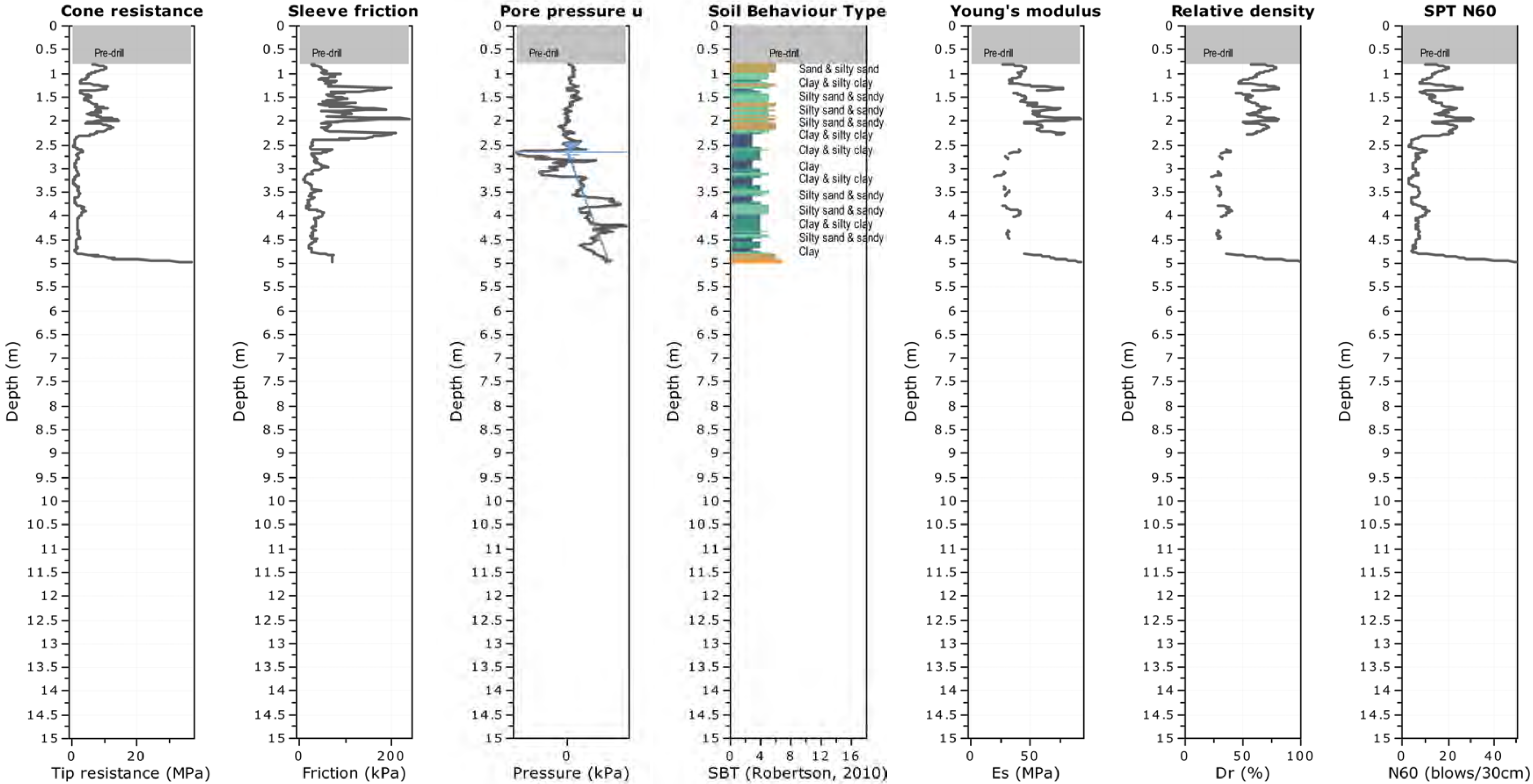
Total depth: 4.97 m, Date: 9/05/2024

Surface Elevation: 3.29 m

Coords: X:527631.78, Y:5252445.60

Cone Type: C10CFIIP.C181094

Cone Operator: IGS



CPT-11

Total depth: 4.51 m, Date: 8/05/2024

Surface Elevation: 5.19 m

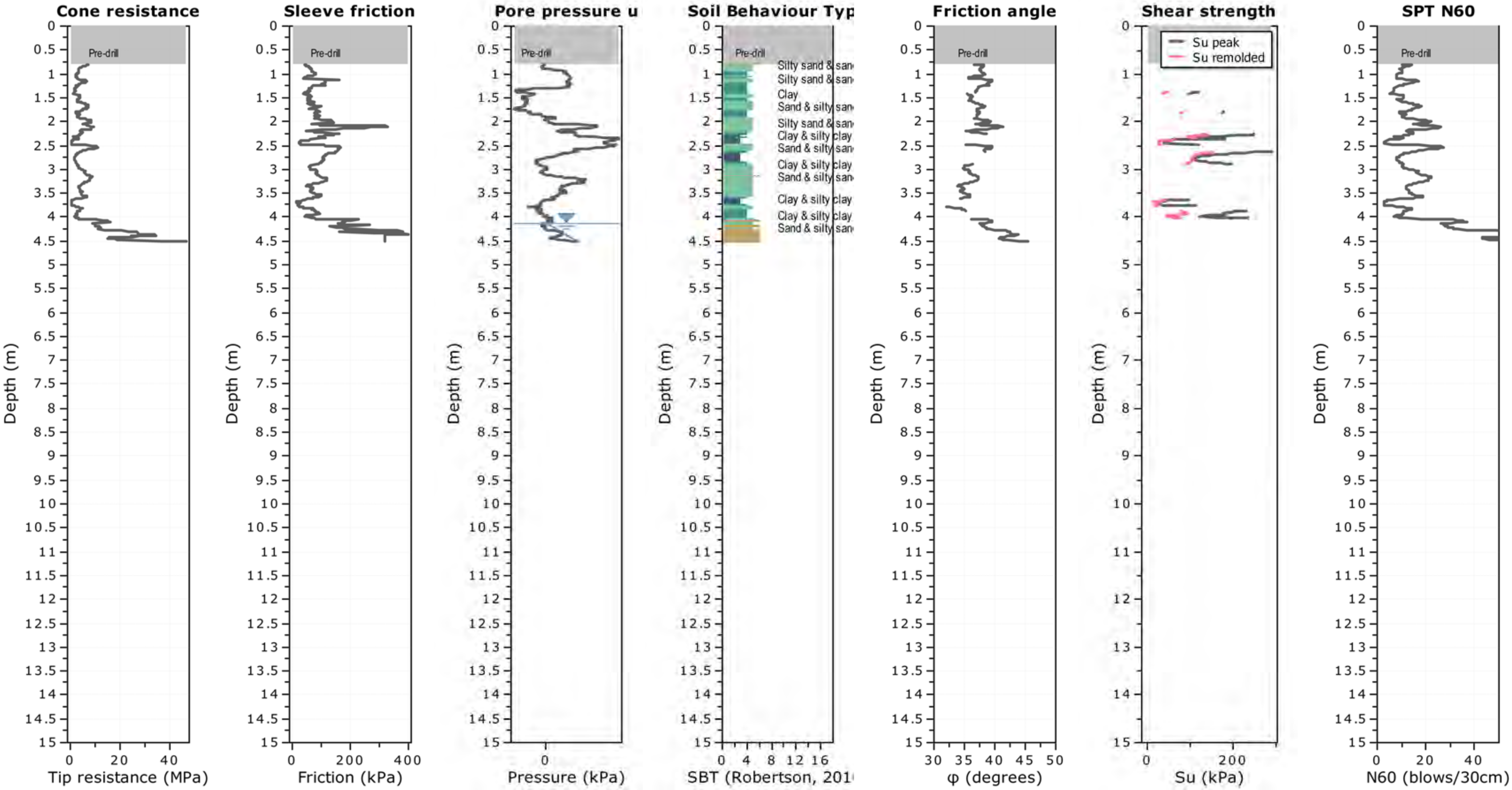
Coords: X:527496.56, Y:5252537.88

Cone Type: C10CFIIP.C181094

Cone Operator: IGS

Project: PS212776 Macquarie Point

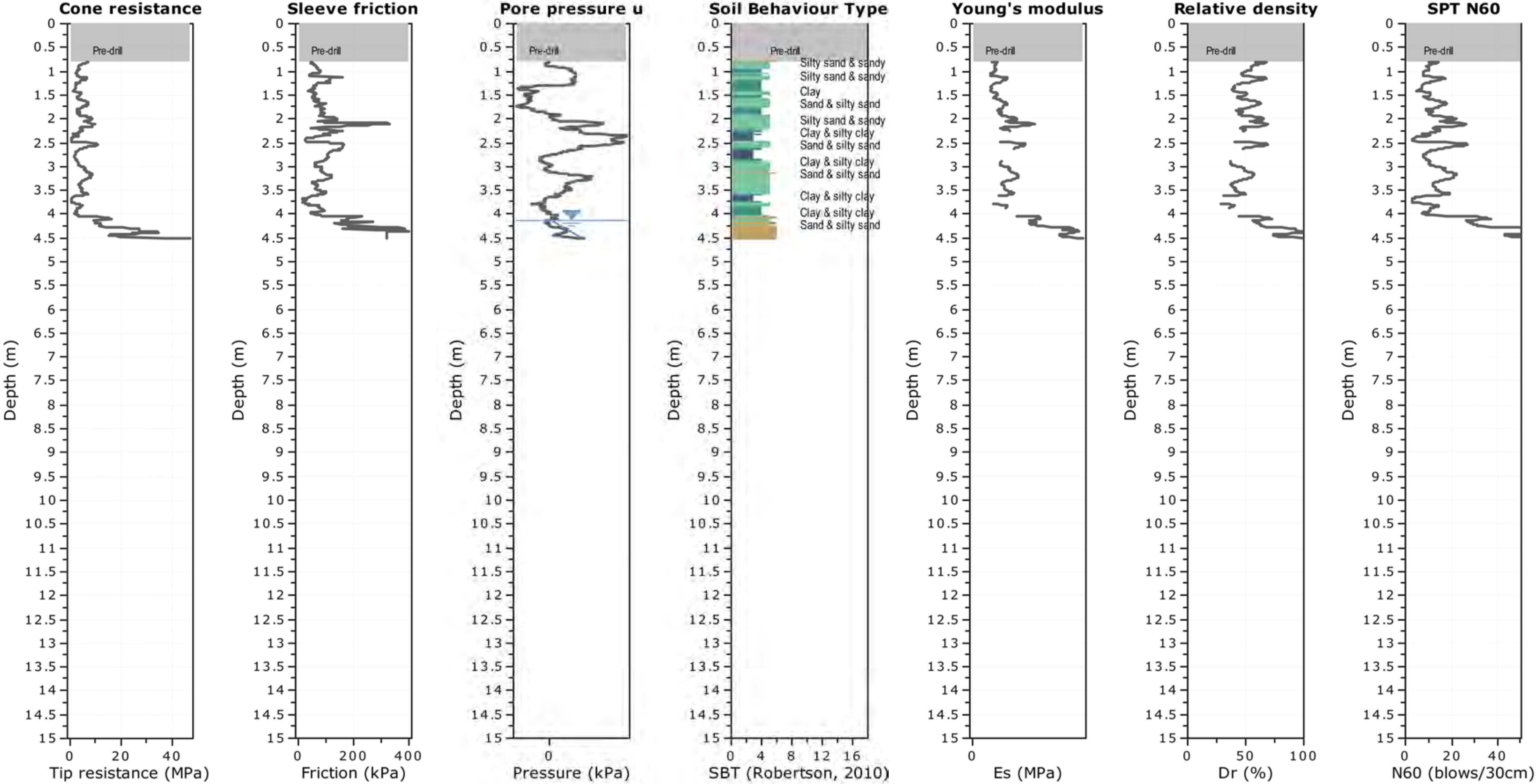
Location: Hobart, Tasmania



CPT-11

Total depth: 4.51 m, Date: 8/05/2024
Surface Elevation: 5.19 m
Coords: X:527496.56, Y:5252537.88
Cone Type: C10CFIIP.C181094
Cone Operator: IGS

Project: PS212776 Macquarie Point
Location: Hobart, Tasmania



CPT-12

Total depth: 1.82 m, Date: 18/04/2024

Surface Elevation: 5.50 m

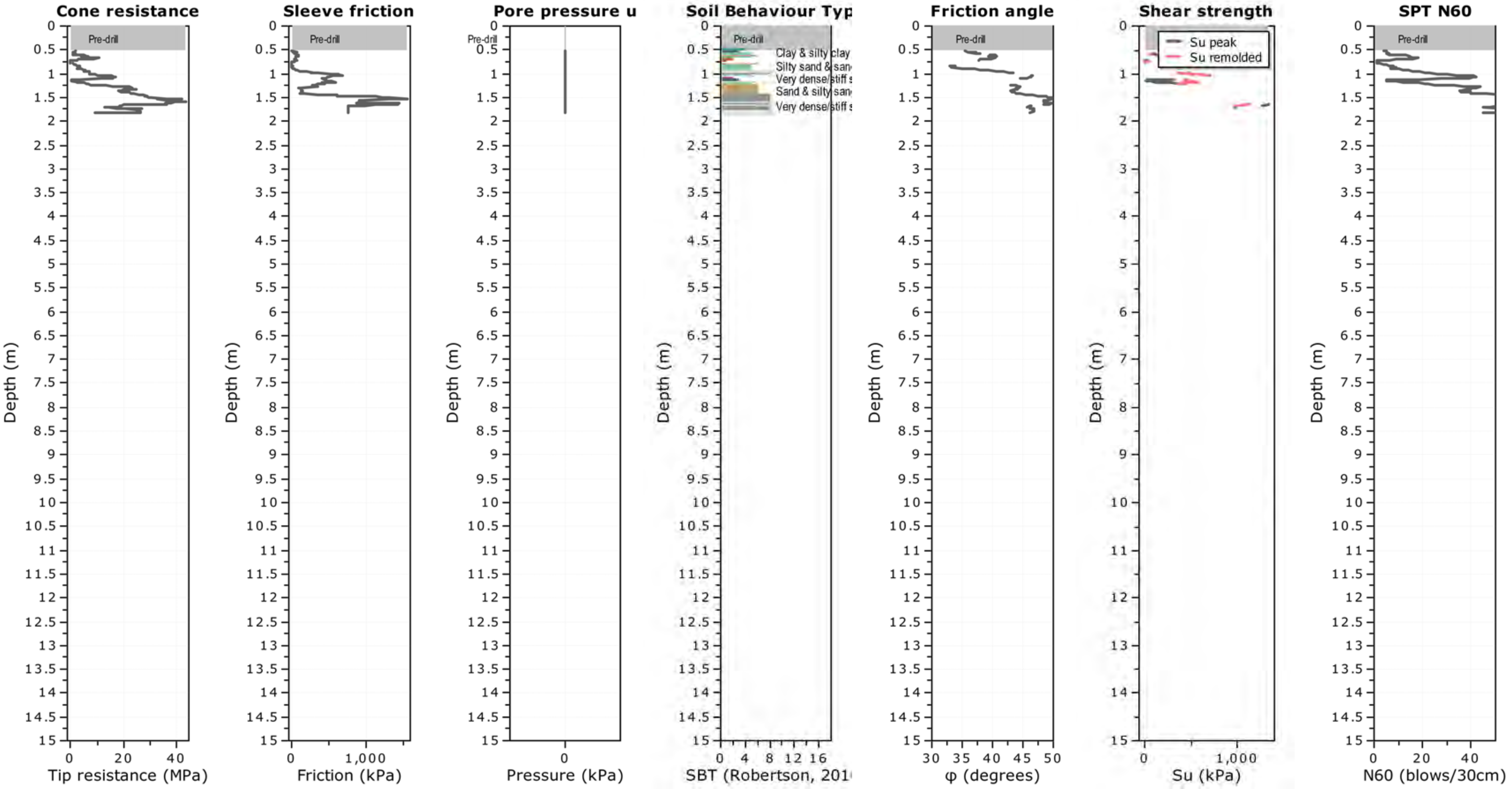
Coords: X:527441.31, Y:5252458.53

Cone Type: C15CFIIPT.C21200

Cone Operator: IGS

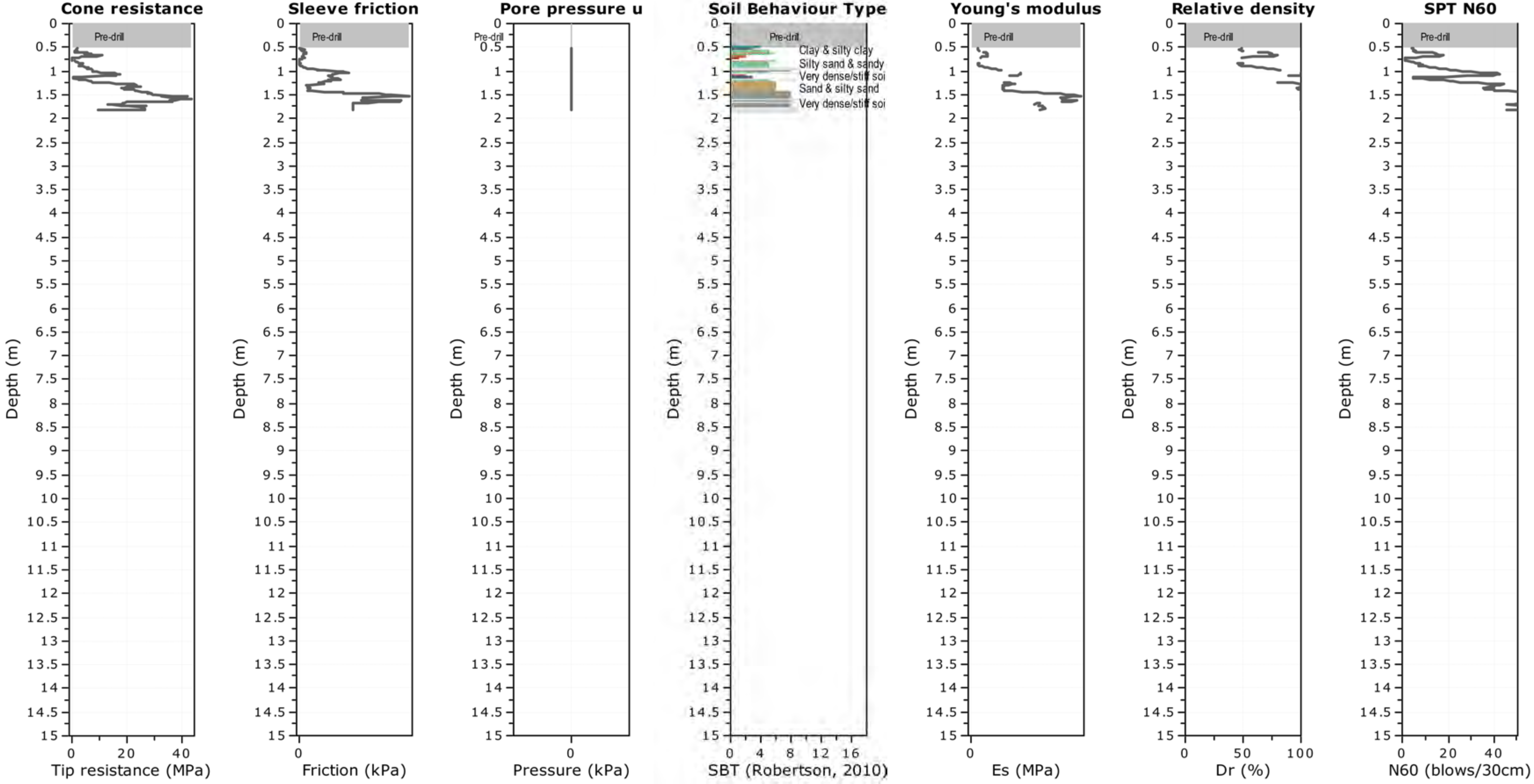
Project: PS212776 Macquarie Point

Location: Hobart, Tasmania



Project: PS212776 Macquarie Point
Location: Hobart, Tasmania

Total depth: 1.82 m, Date: 18/04/2024
Surface Elevation: 5.50 m
Coords: X:527441.31, Y:5252458.53
Cone Type: C15CFIIPT.C21200
Cone Operator: IGS



CPT-12a

Total depth: 5.77 m, Date: 18/04/2024

Surface Elevation: 5.48 m

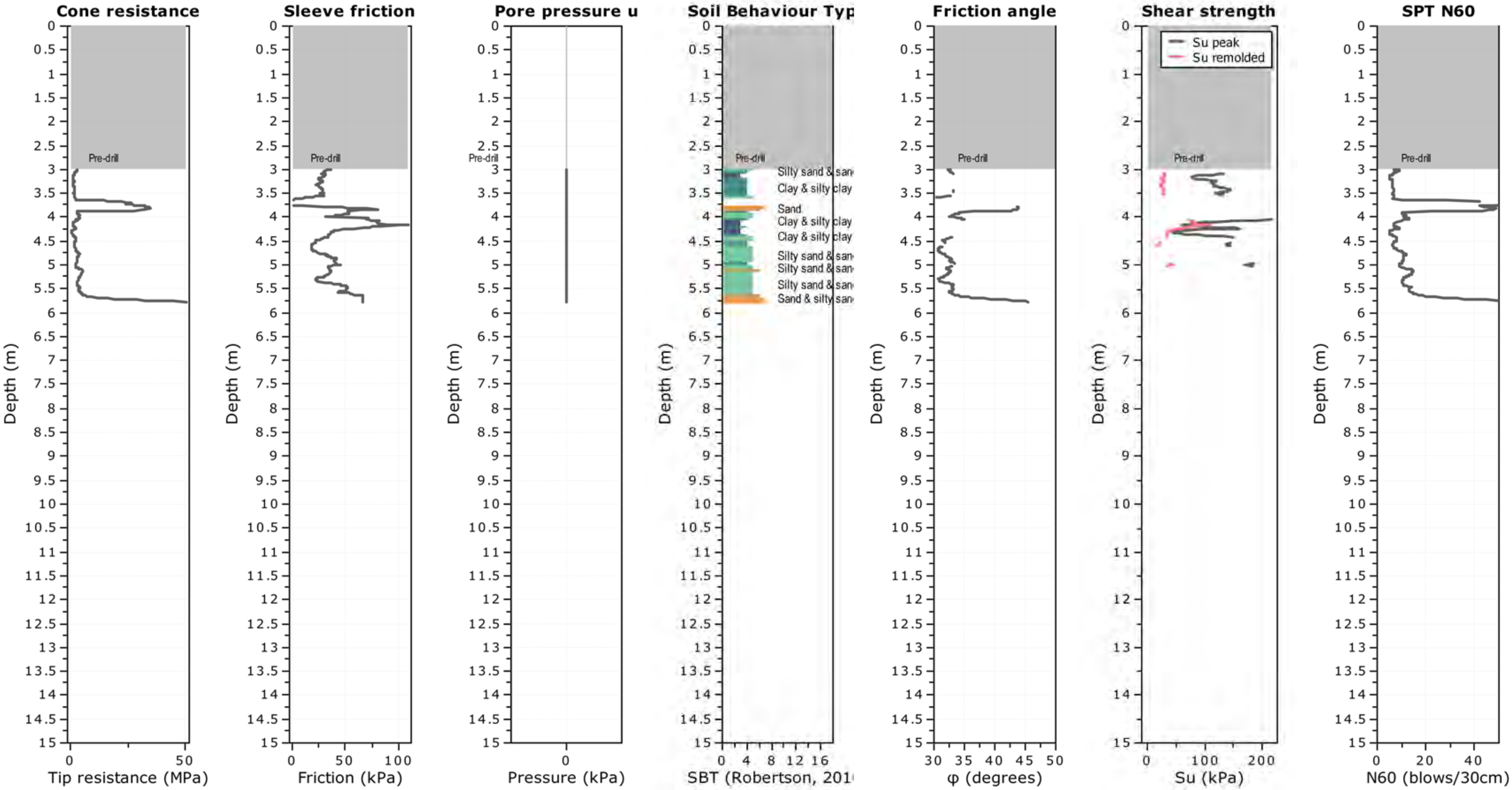
Coords: X:527441.25, Y:5252457.91

Cone Type: C15CFIIPT.C21200

Cone Operator: IGS

Project: PS212776 Macquarie Point

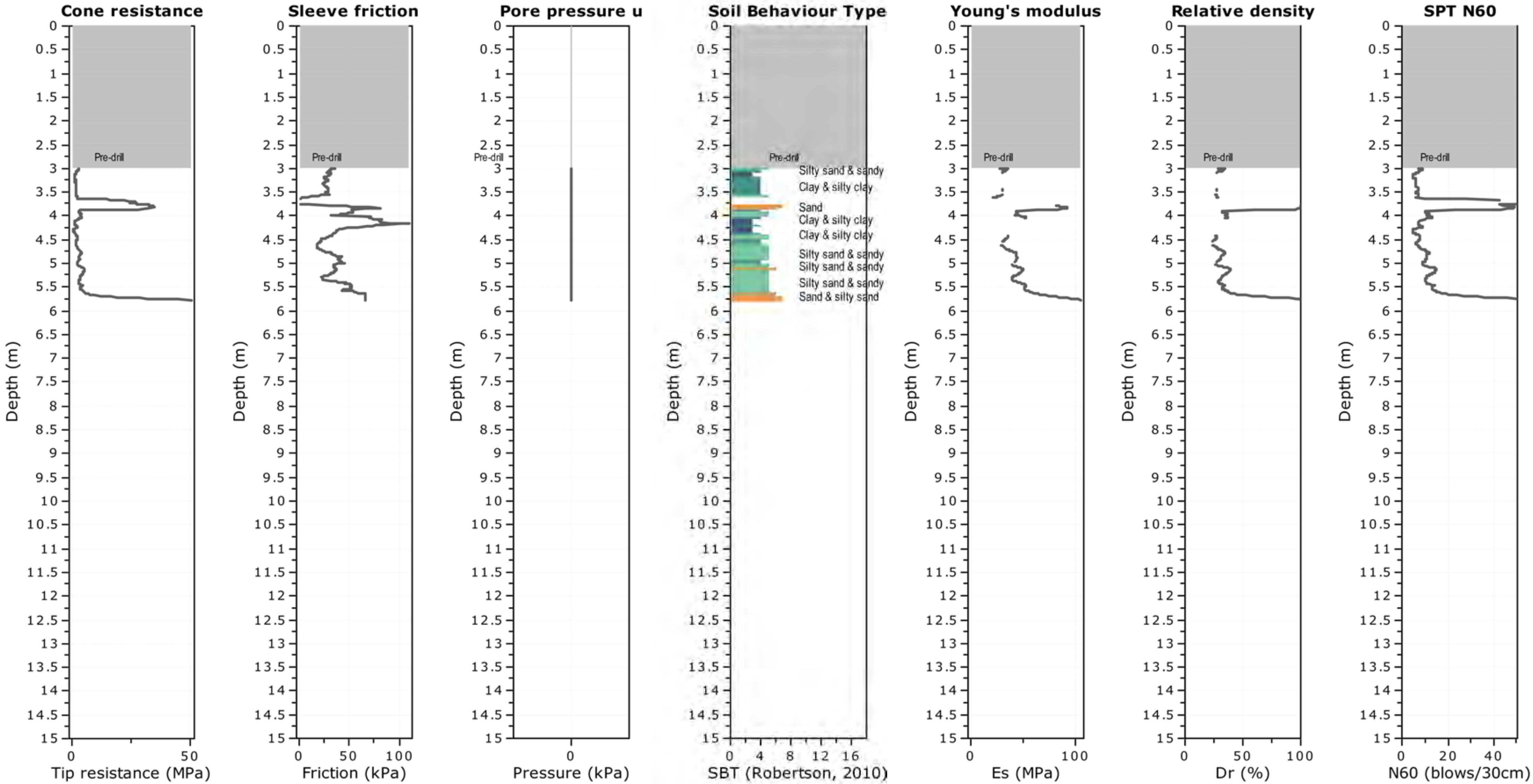
Location: Hobart, Tasmania



CPT-12a

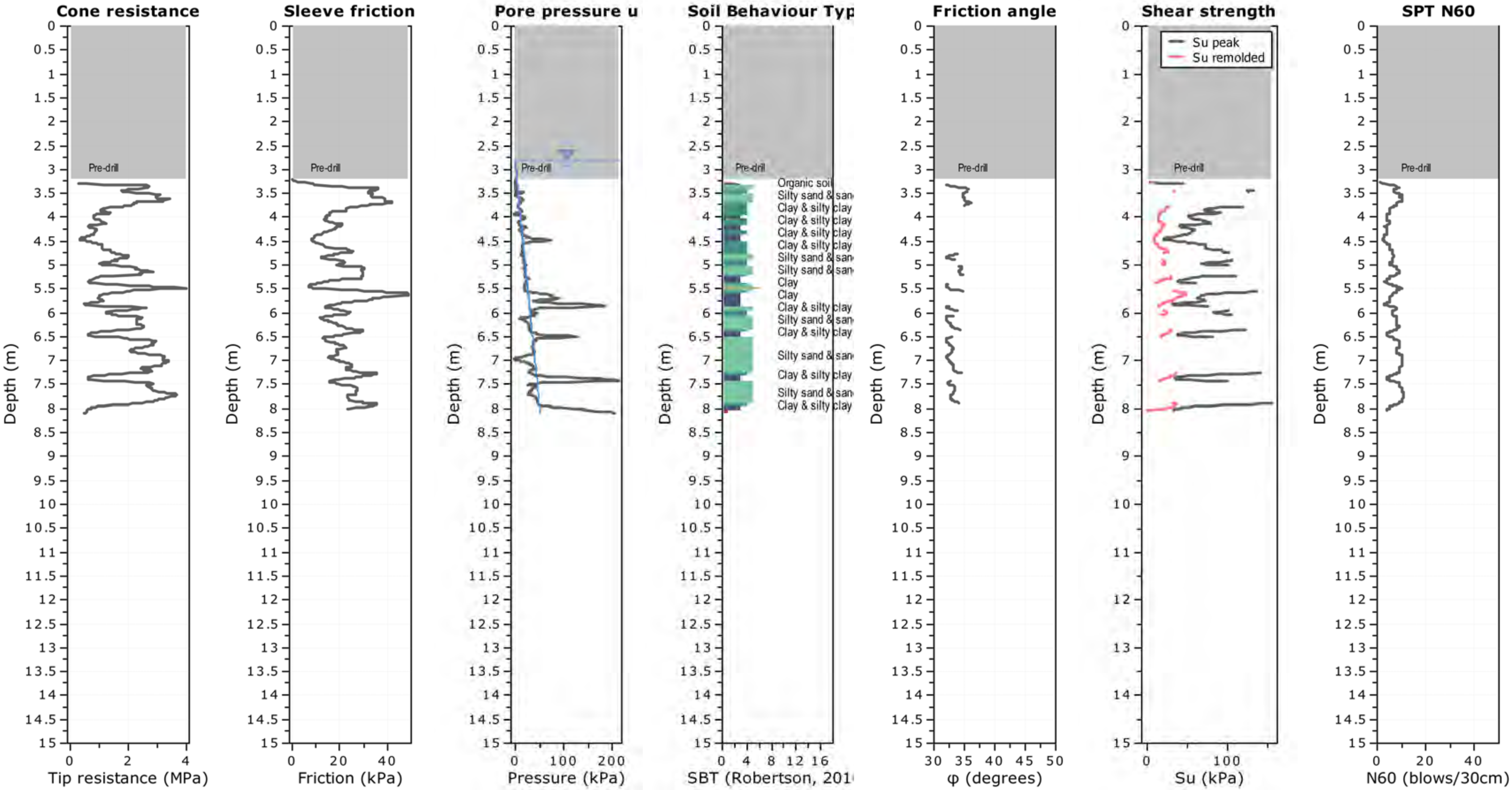
Total depth: 5.77 m, Date: 18/04/2024
Surface Elevation: 5.48 m
Coords: X:527441.25, Y:5252457.91
Cone Type: C15CFIIPT.C21200
Cone Operator: IGS

Project: PS212776 Macquarie Point
Location: Hobart, Tasmania



Project: PS212776 Macquarie Point
Location: Hobart, Tasmania

CPT-13
Total depth: 8.09 m, Date: 9/05/2024
Surface Elevation: 2.96 m
Coords: X:527610.06, Y:5252345.37
Cone Type: C10CFIIP.C181094
Cone Operator: IGS



CPT-13

Total depth: 8.09 m, Date: 9/05/2024

Surface Elevation: 2.96 m

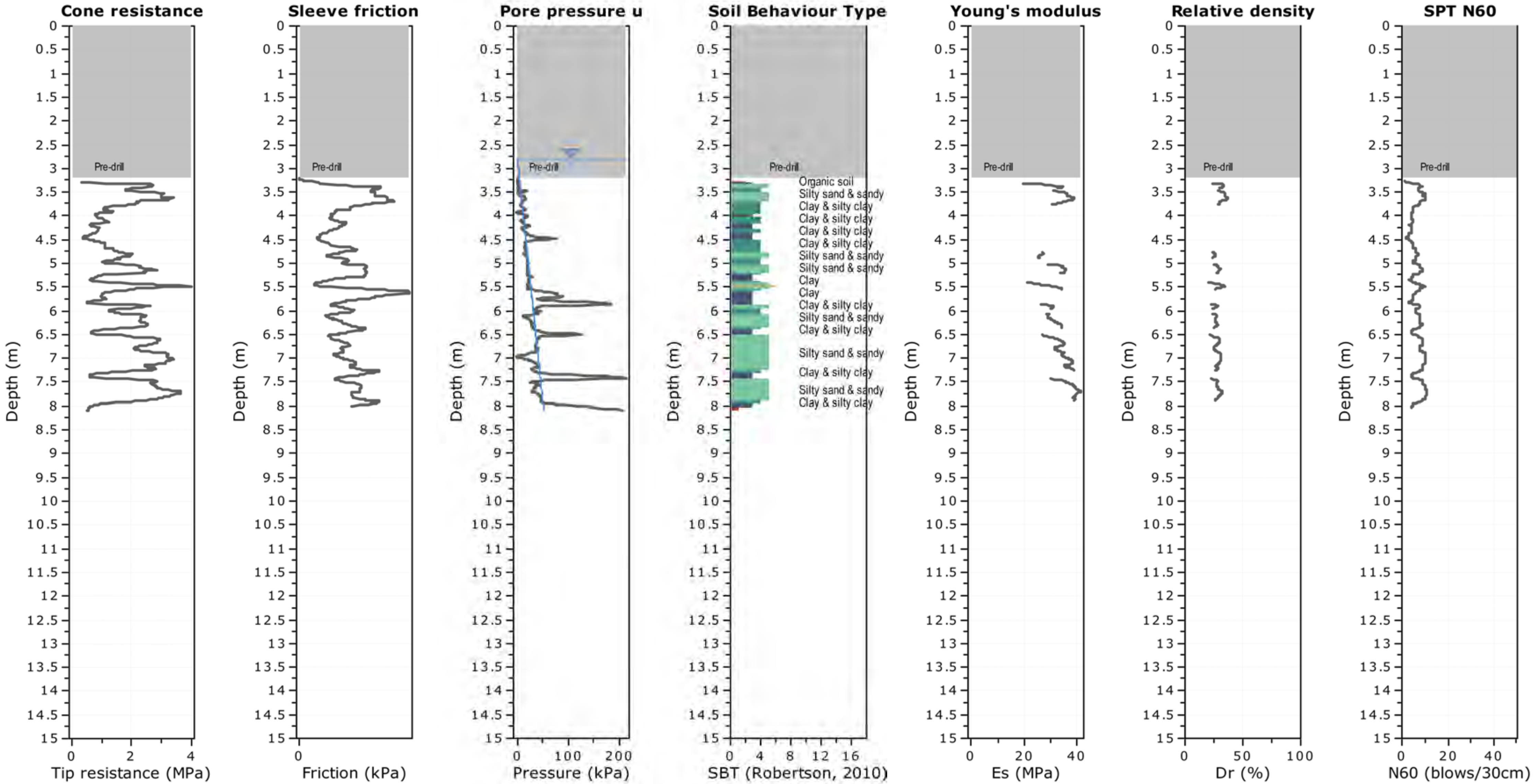
Coords: X:527610.06, Y:5252345.37

Cone Type: C10CFIIP.C181094

Cone Operator: IGS

Project: PS212776 Macquarie Point

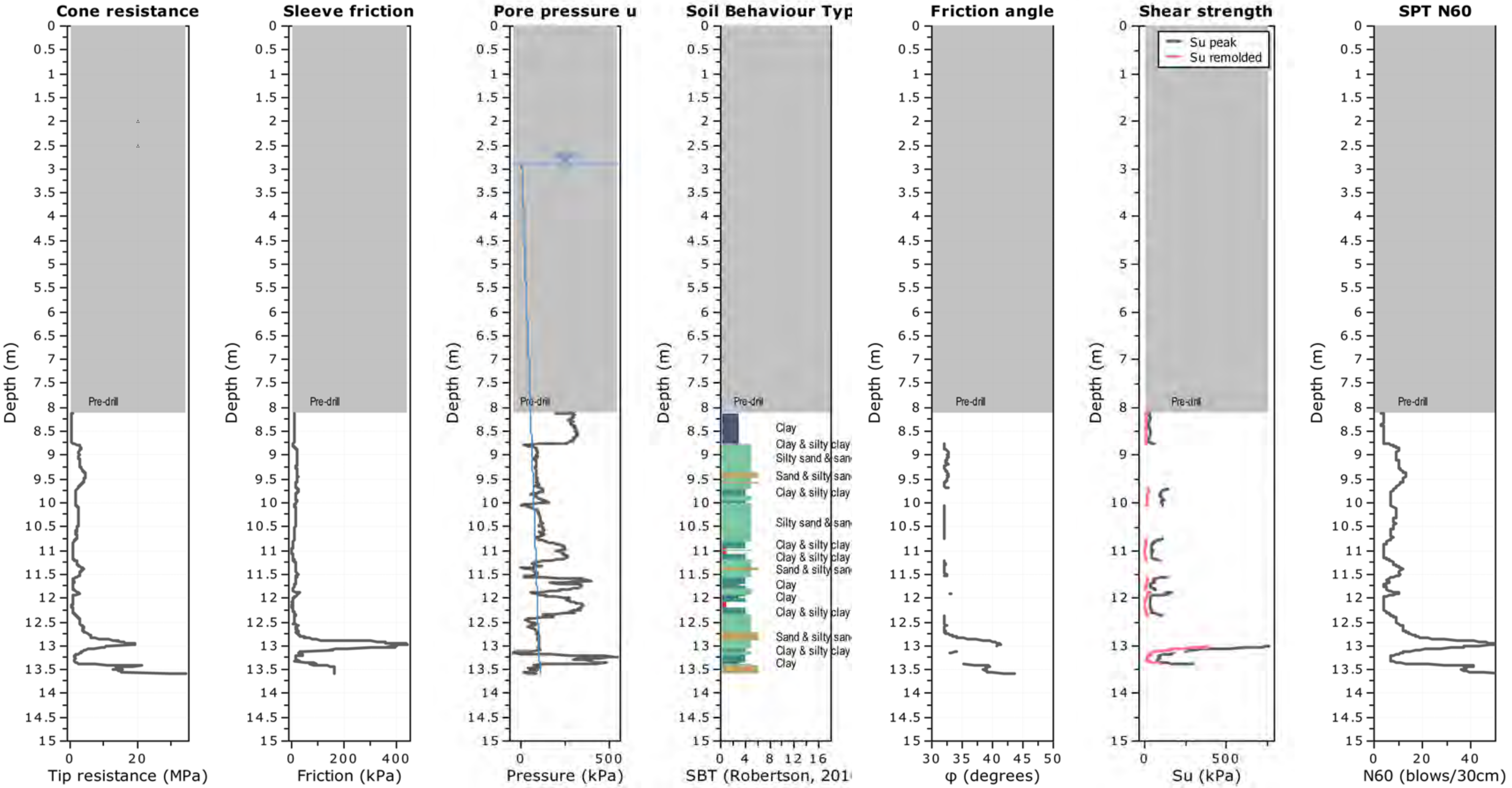
Location: Hobart, Tasmania



CPT-13a

Total depth: 13.59 m, Date: 31/05/2024
Surface Elevation: 2.96 m
Coords: X:527610.06, Y:5252345.37
Cone Type:
Cone Operator:

Project: PS212776 Macquarie Point
Location: Hobart, Tasmania



CPT-13a

Total depth: 13.59 m, Date: 31/05/2024

Surface Elevation: 2.96 m

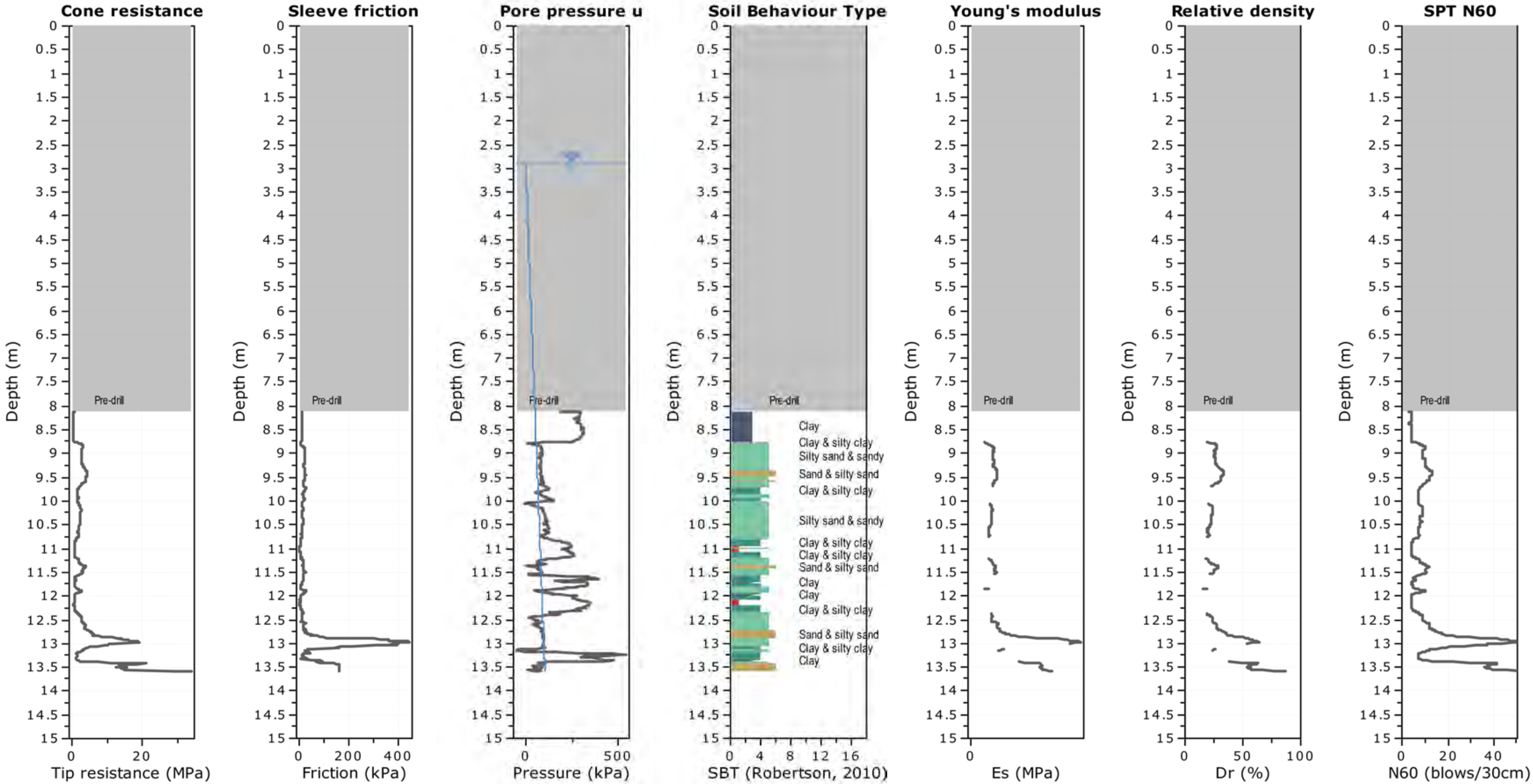
Coords: X:527610.06, Y:5252345.37

Cone Type:

Cone Operator:

Project: PS212776 Macquarie Point

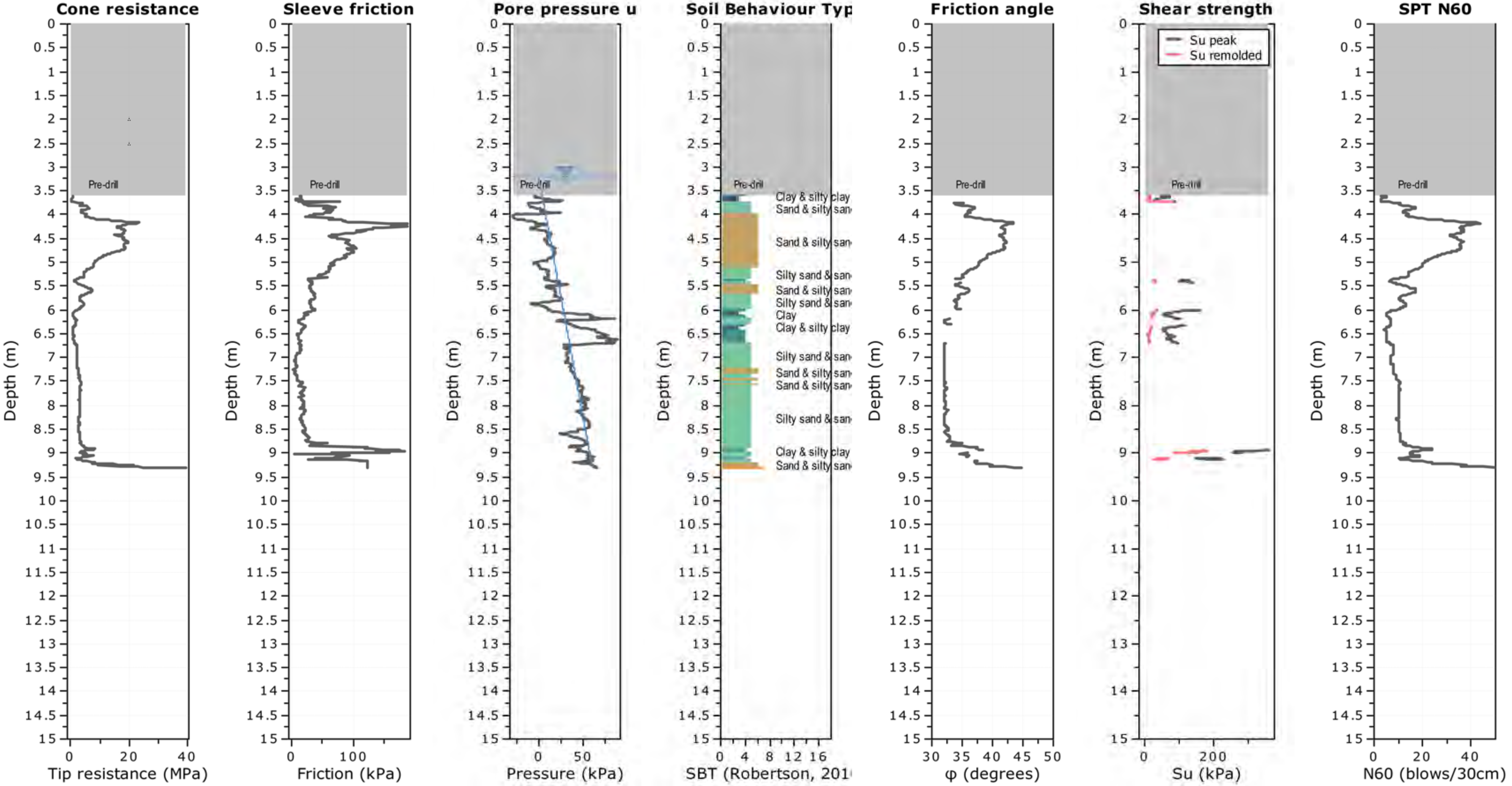
Location: Hobart, Tasmania



CPT-14

Total depth: 9.31 m, Date: 31/05/2024
Surface Elevation: 2.89 m
Coords: X:527549.77, Y:5252415.20
Cone Type:
Cone Operator:

Project: PS212776 Macquarie Point
Location: Hobart, Tasmania



CPT-14

Total depth: 9.31 m, Date: 31/05/2024

Surface Elevation: 2.89 m

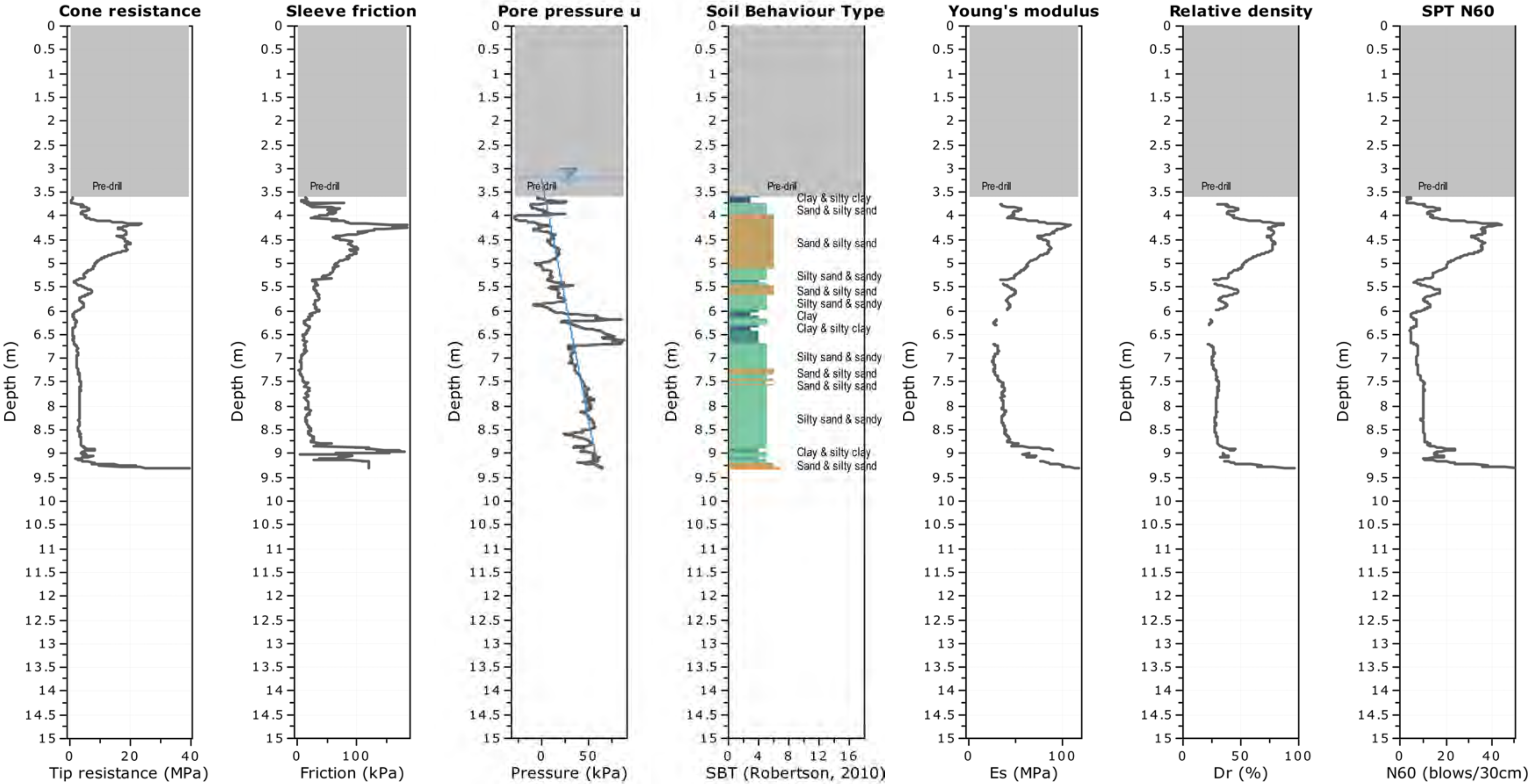
Coords: X:527549.77, Y:5252415.20

Cone Type:

Cone Operator:

Project: PS212776 Macquarie Point

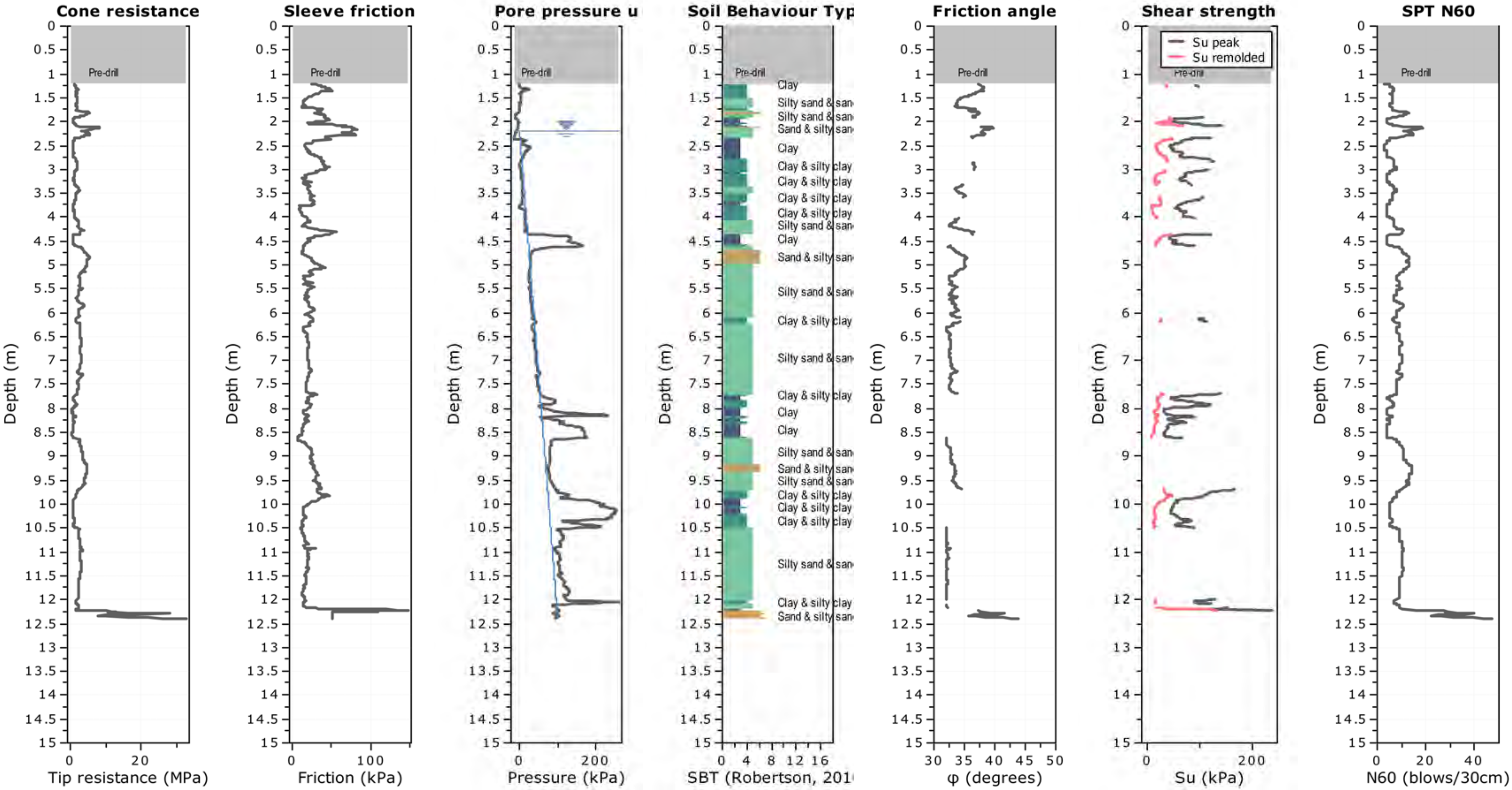
Location: Hobart, Tasmania



CPT-16

Total depth: 12.40 m, Date: 31/05/2024
Surface Elevation: 4.01 m
Coords: X:527593.95, Y:5252307.65
Cone Type:
Cone Operator:

Project: PS212776 Macquarie Point
Location: Hobart, Tasmania



CPT-16

Total depth: 12.40 m, Date: 31/05/2024

Surface Elevation: 4.01 m

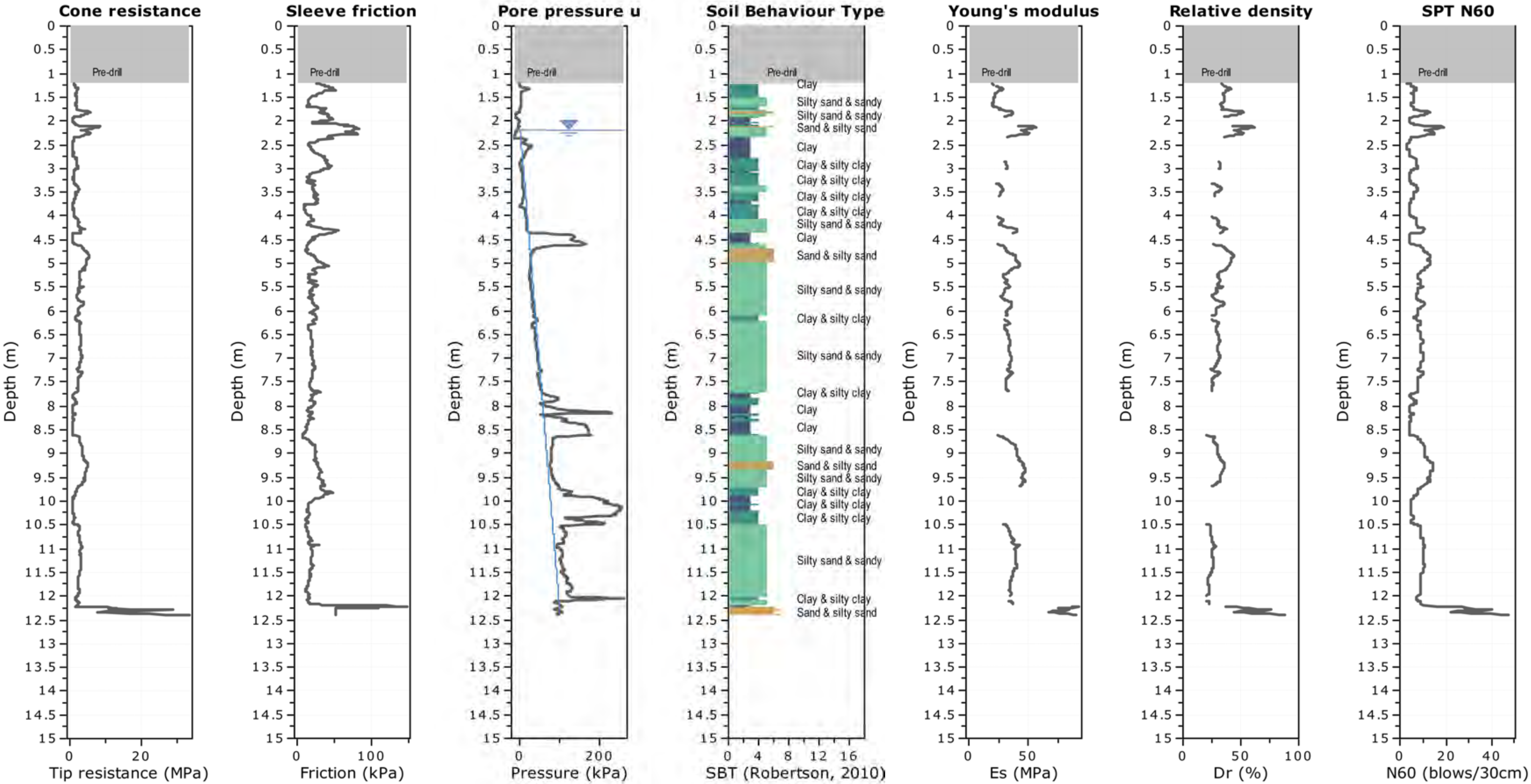
Coords: X:527593.95, Y:5252307.65

Cone Type:

Cone Operator:

Project: PS212776 Macquarie Point

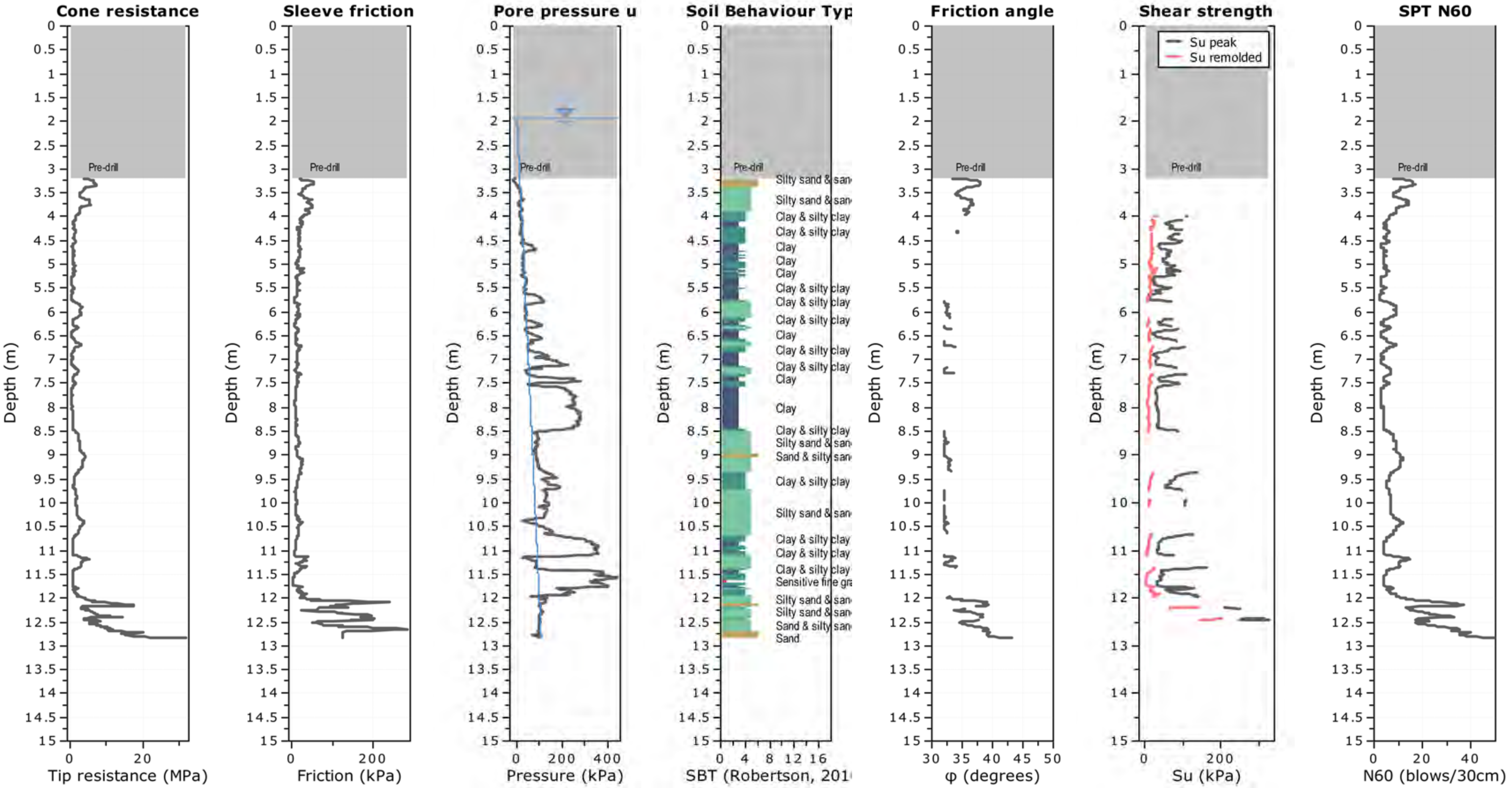
Location: Hobart, Tasmania



CPT-17

Total depth: 12.84 m, Date: 31/05/2024
Surface Elevation: 2.89 m
Coords: X:527618.13, Y:5252369.71
Cone Type:
Cone Operator:

Project: PS212776 Macquarie Point
Location: Hobart, Tasmania



CPT-17

Total depth: 12.84 m, Date: 31/05/2024

Surface Elevation: 2.89 m

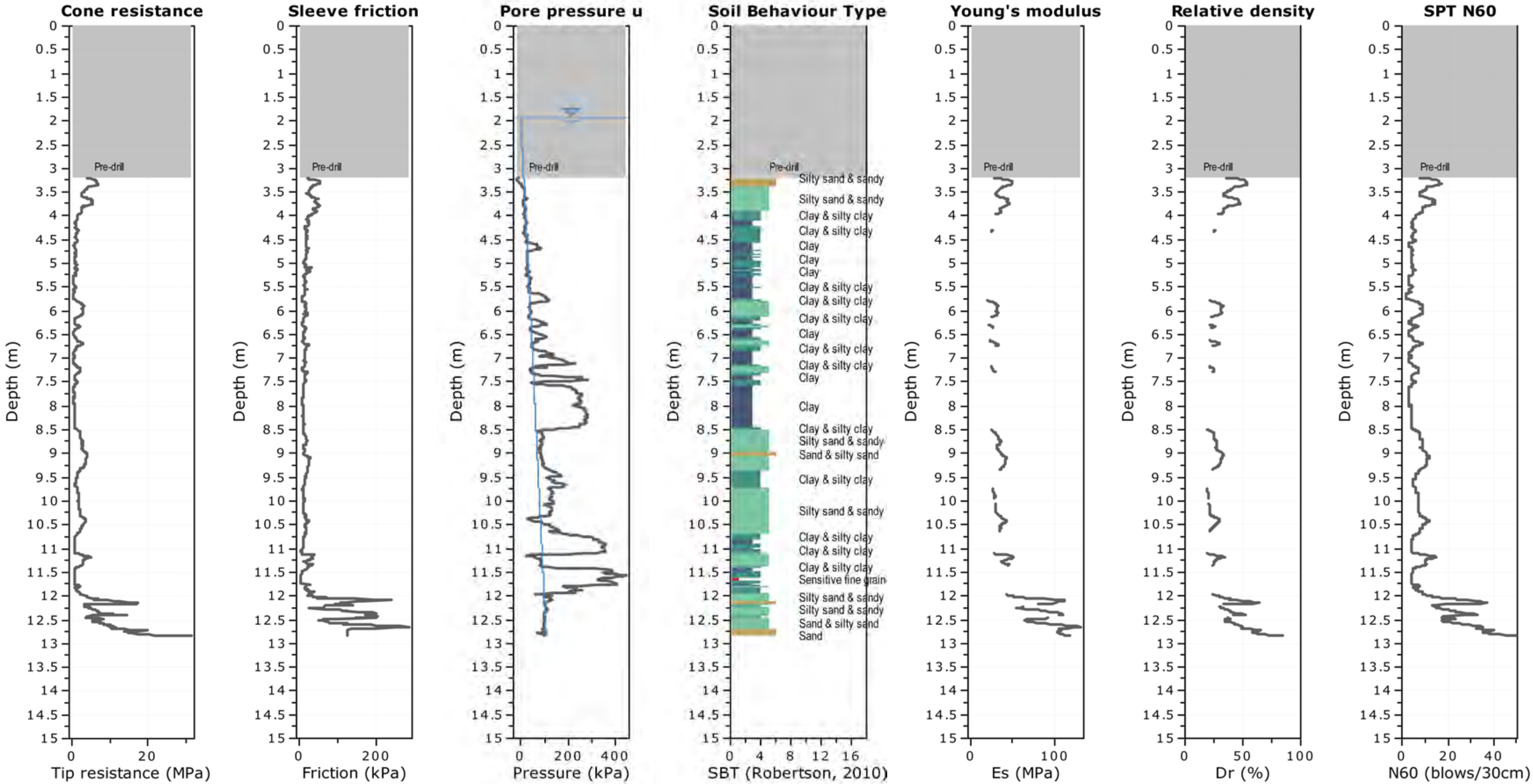
Coords: X:527618.13, Y:5252369.71

Cone Type:

Cone Operator:

Project: PS212776 Macquarie Point

Location: Hobart, Tasmania



CPT-019

Total depth: 4.43 m, Date: 31/05/2024

Surface Elevation: 4.01 m

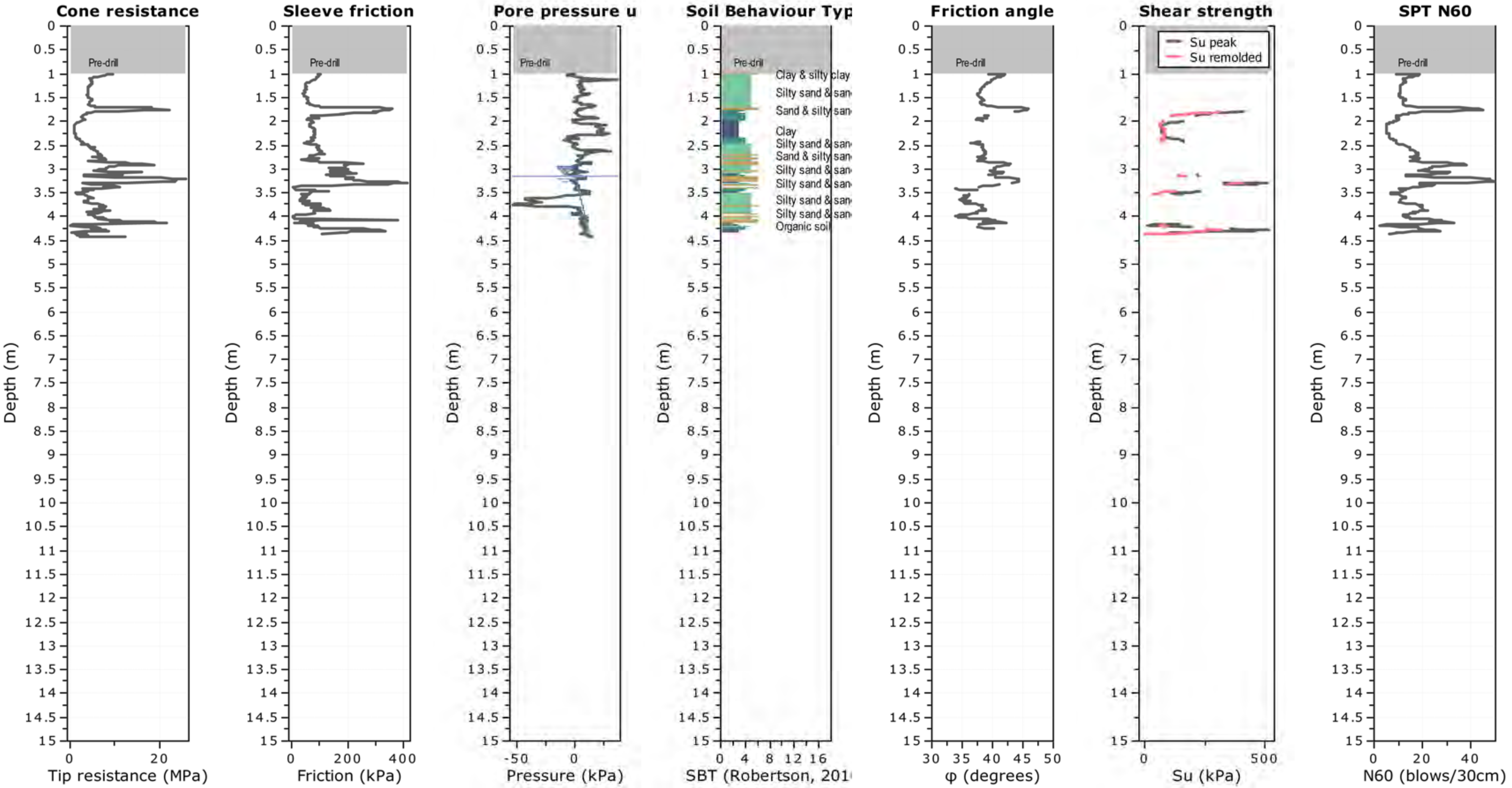
Coords: X:527520.73, Y:5252470.68

Cone Type:

Cone Operator:

Project: PS212776 Macquarie Point

Location: Hobart, Tasmania



CPT-019

Total depth: 4.43 m, Date: 31/05/2024

Surface Elevation: 4.01 m

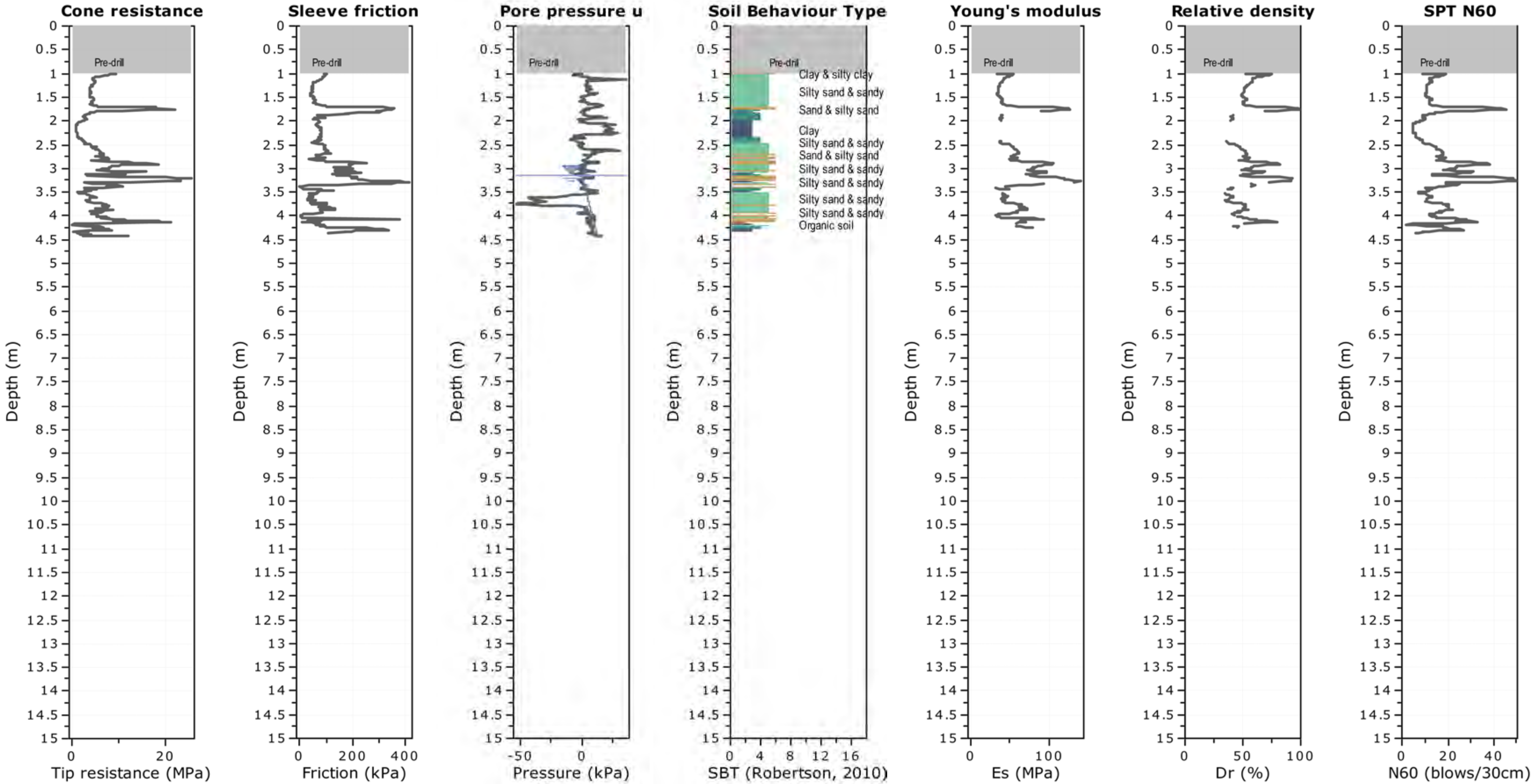
Coords: X:527520.73, Y:5252470.68

Cone Type:

Cone Operator:

Project: PS212776 Macquarie Point

Location: Hobart, Tasmania



Appendix E

Laboratory test figures

Laboratory test interpretation

Fig	Title
E1	Atterberg Limits and Moisture Content vs depth
E2	Plasticity Chart
E3	Particle Size Distribution – All Units
E4	Particle Size Distribution – Fill
E5	Particle Size Distribution – Estuarine
E6	Particle Size Distribution – Alluvium
E7	UCS vs PLT Axial
E8	UCS / PLT Axial / PLT Diametral
E9	RQD bands by Weathering

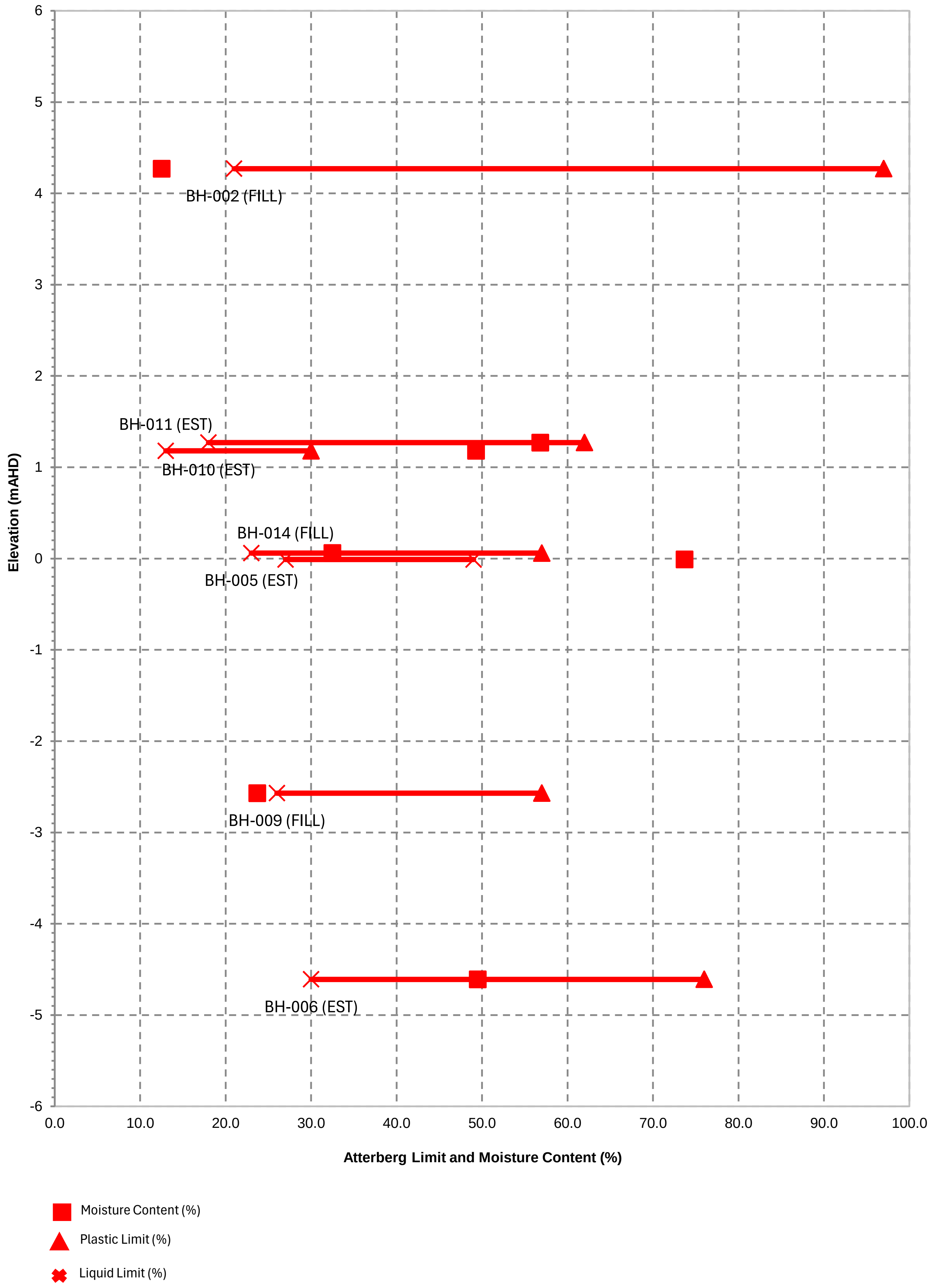


Fig. E1	Macquarie Point Stadium, Hobart **Atterberg Limits and Moisture Content (%) vs Elevation**	

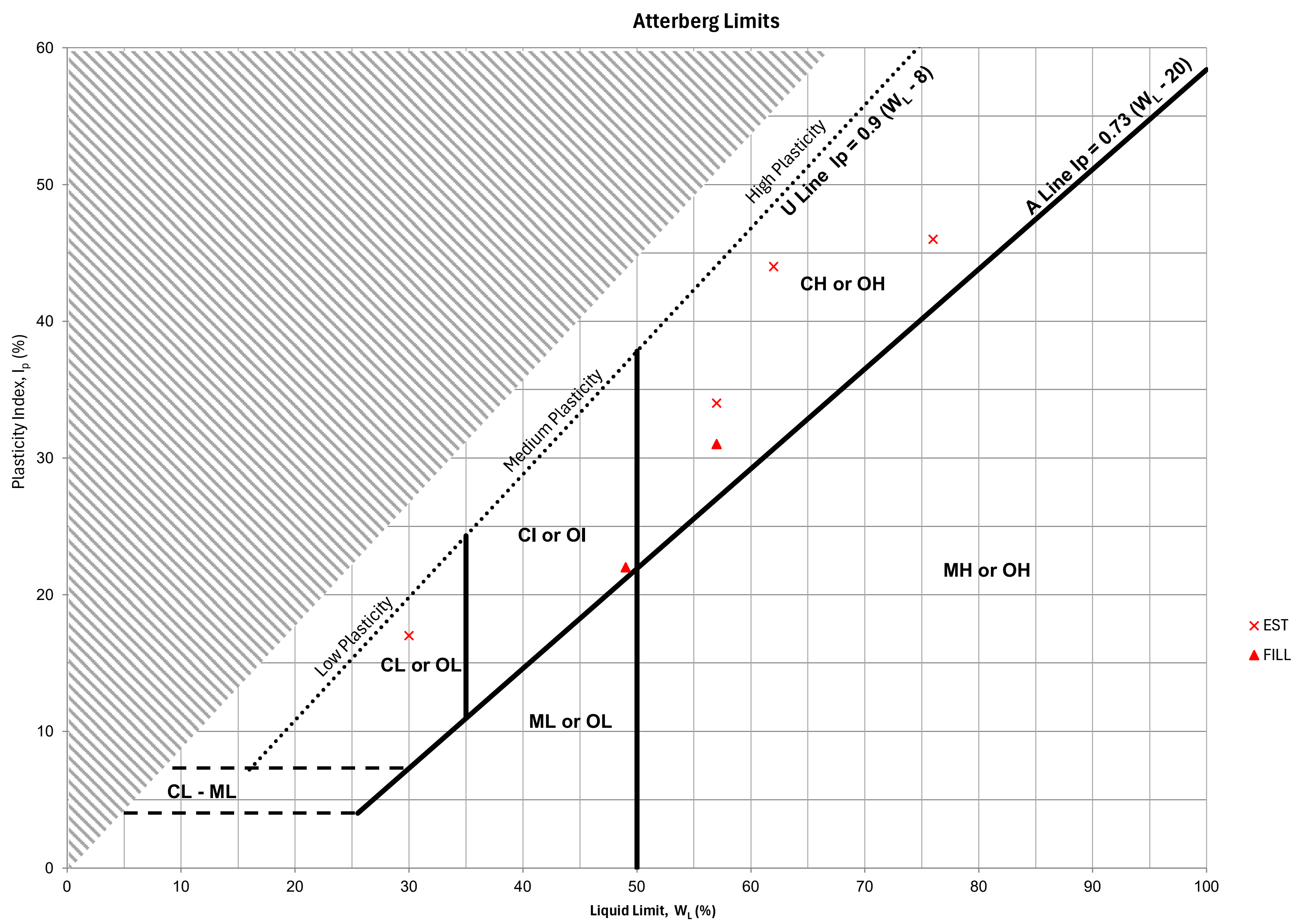


Fig. E2

Macquarie Point Stadium, Hobart
Atterberg Limits

FILL / EST

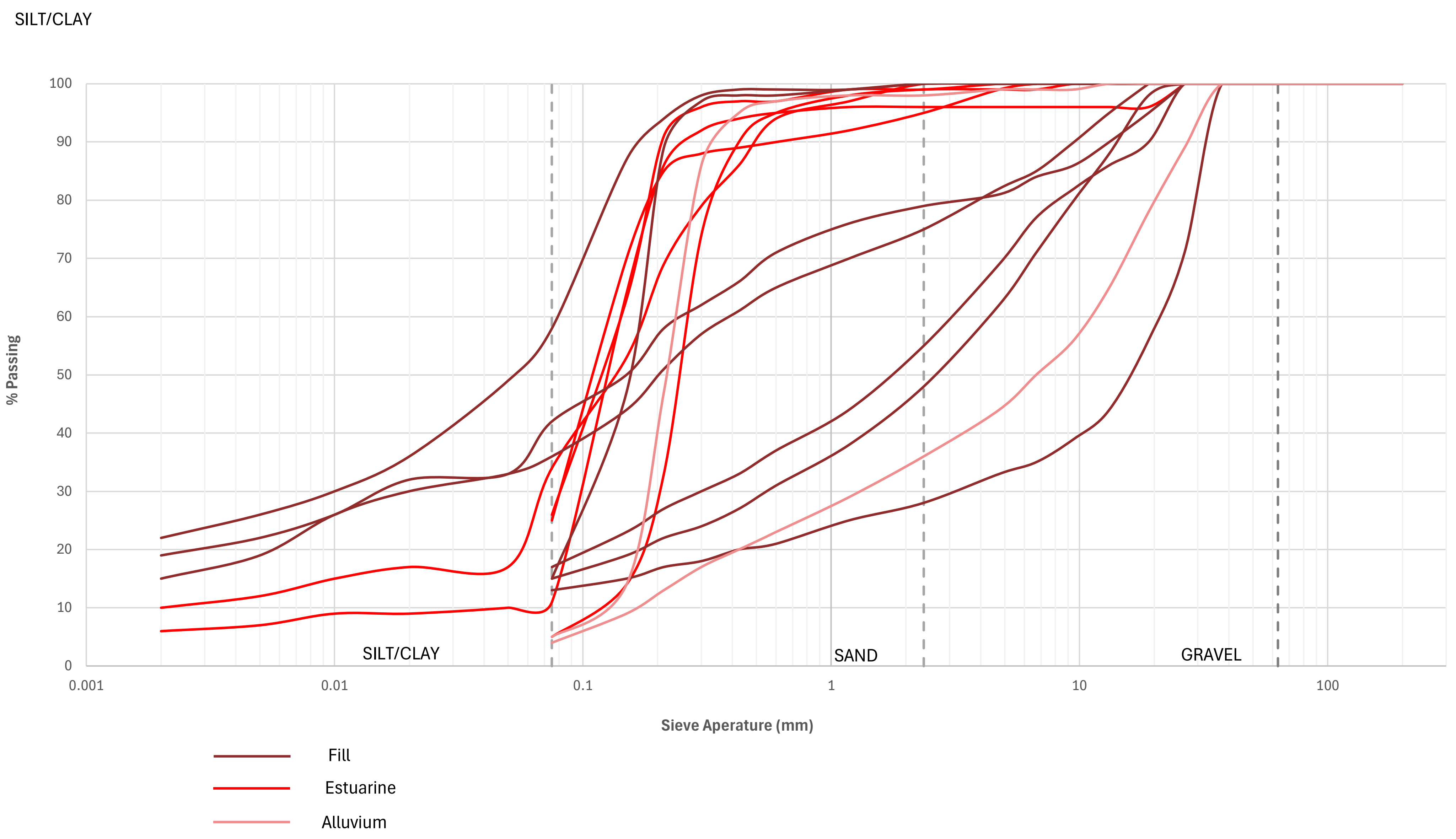


Fig. E3

Macquarie Point Stadium, Hobart

Particle Size Distribution Envelope - All

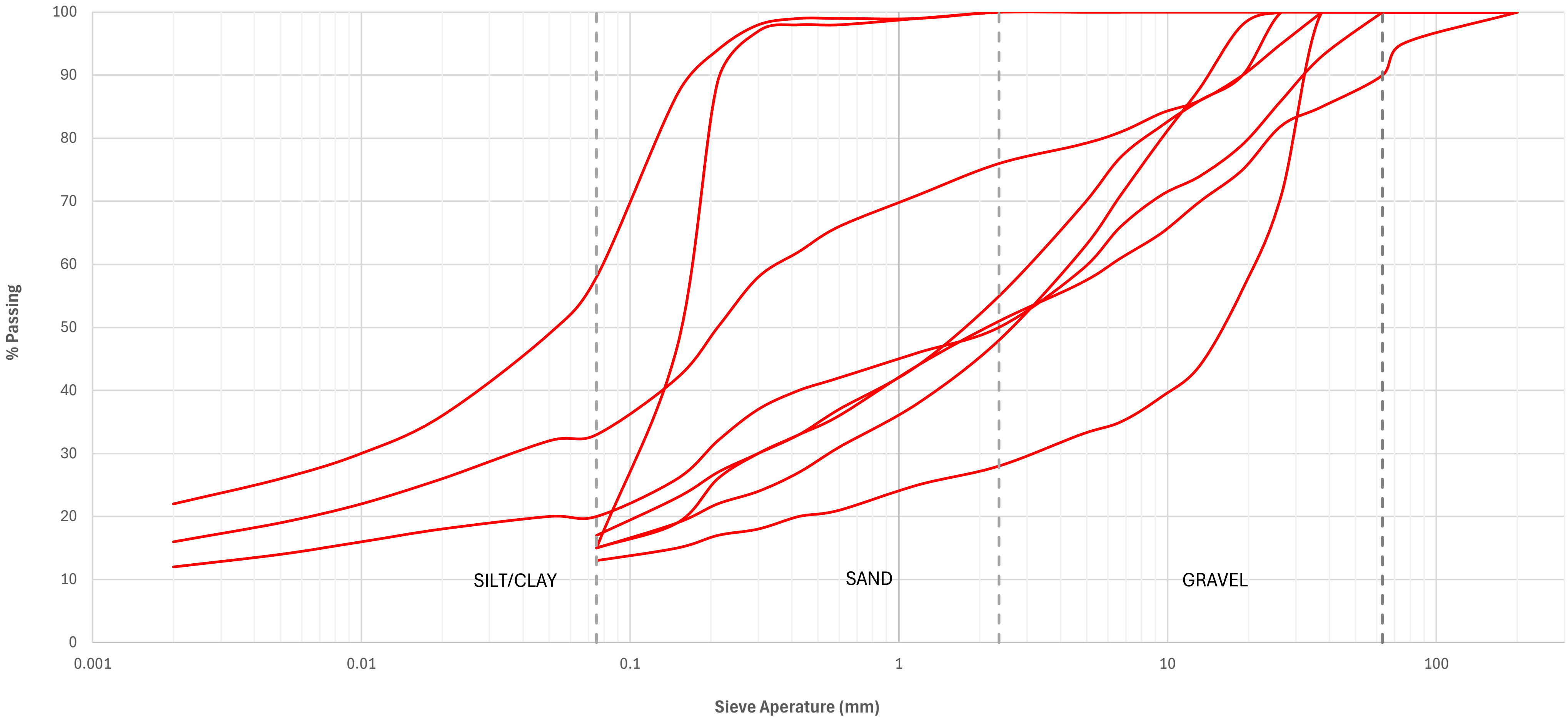


Fig. E4

Macquarie Point Stadium, Hobart

Particle Size Distribution Envelope - Fill

Fill

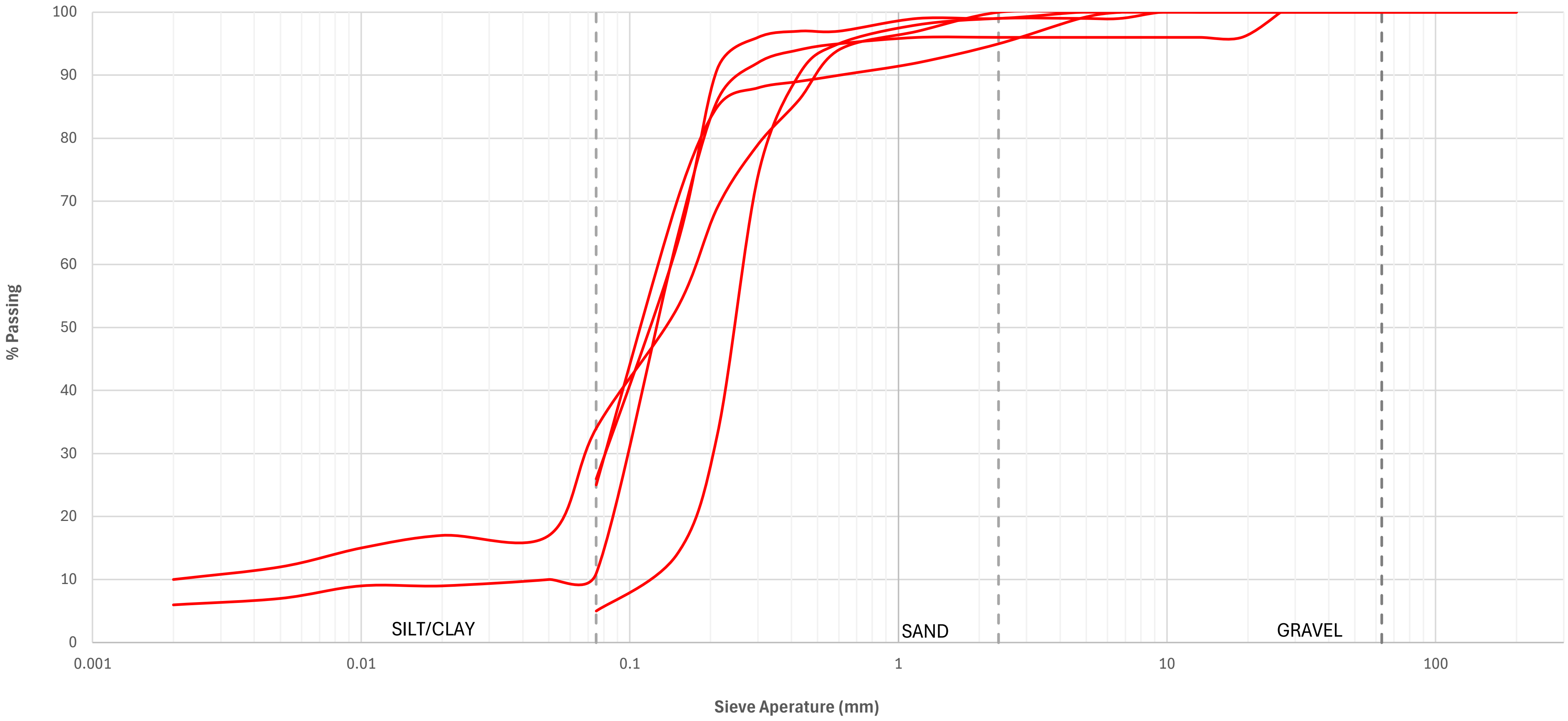


Fig. E5

Macquarie Point Stadium, Hobart

Particle Size Distribution Envelope - Estuarine

Estuarine

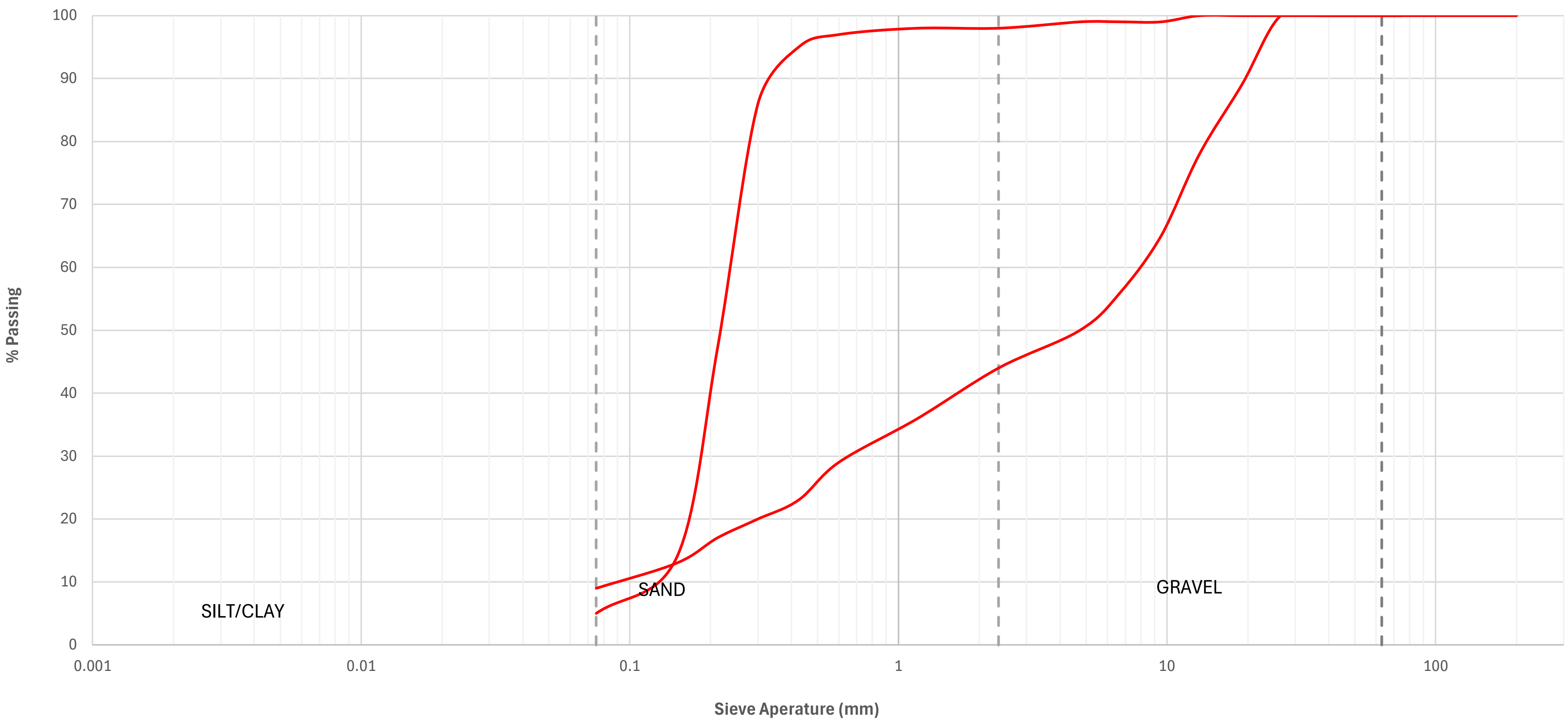


Fig. E6

Macquarie Point Stadium, Hobart

Particle Size Distribution Envelope - Alluvium

Alluvium

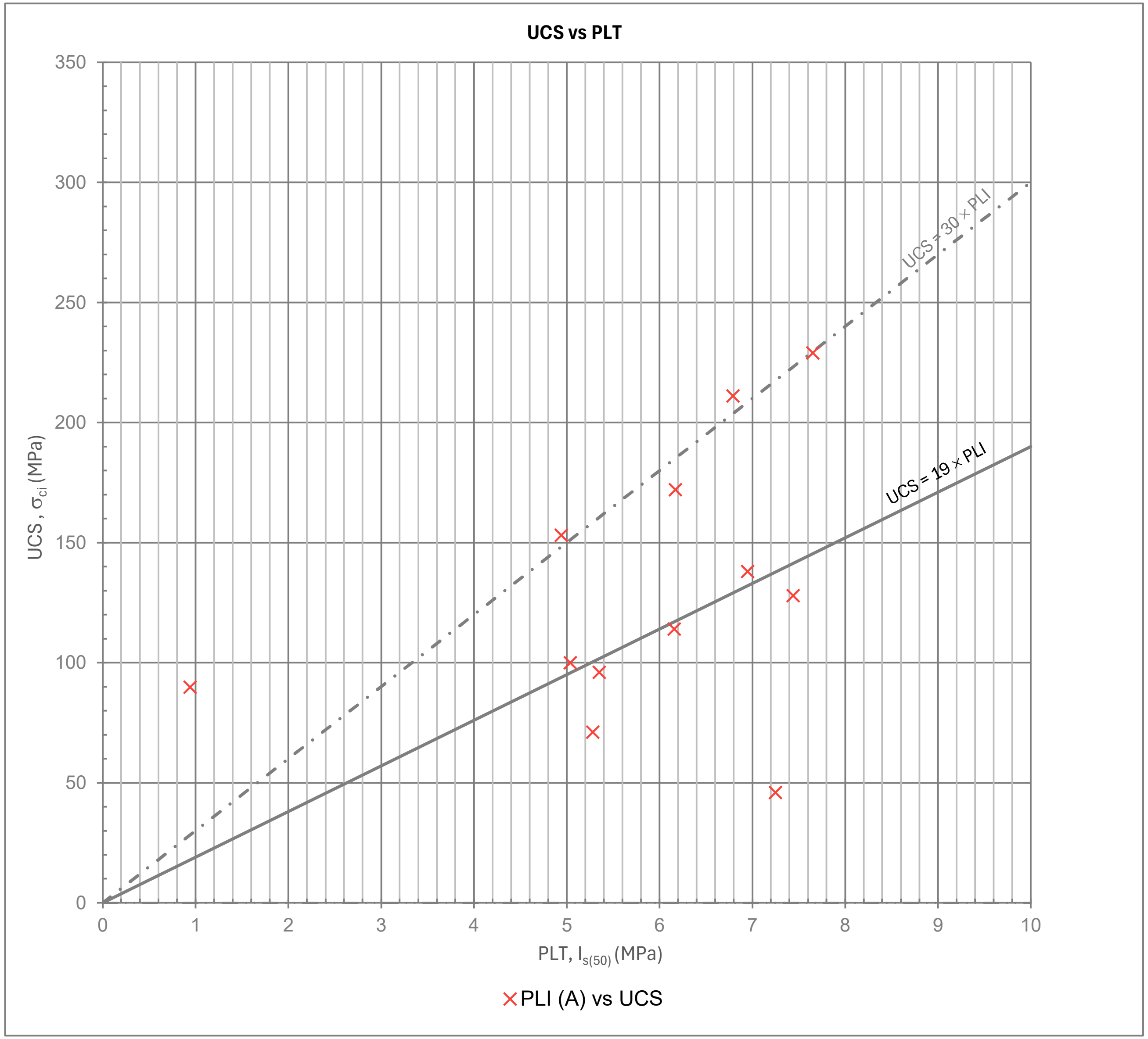


Fig. E7

Macquarie Point Stadium, Hobart

UCS vs PLT (Axial)

Dolerite

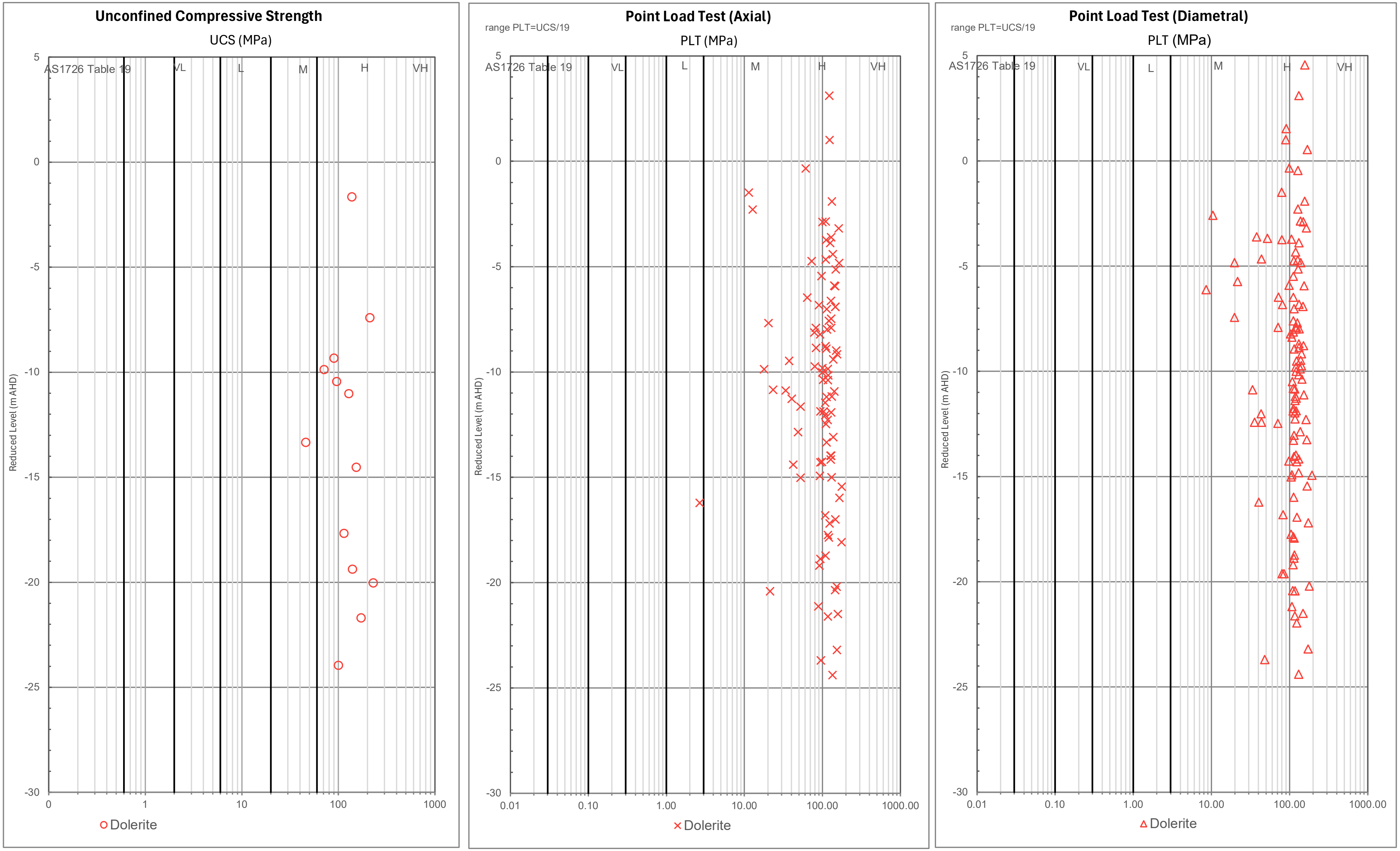
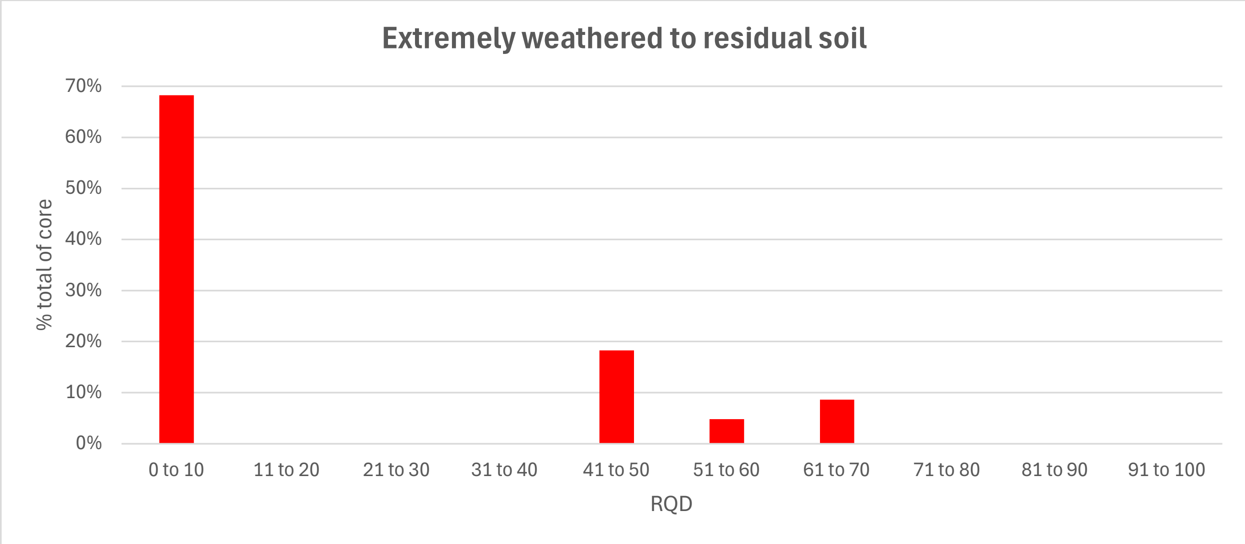


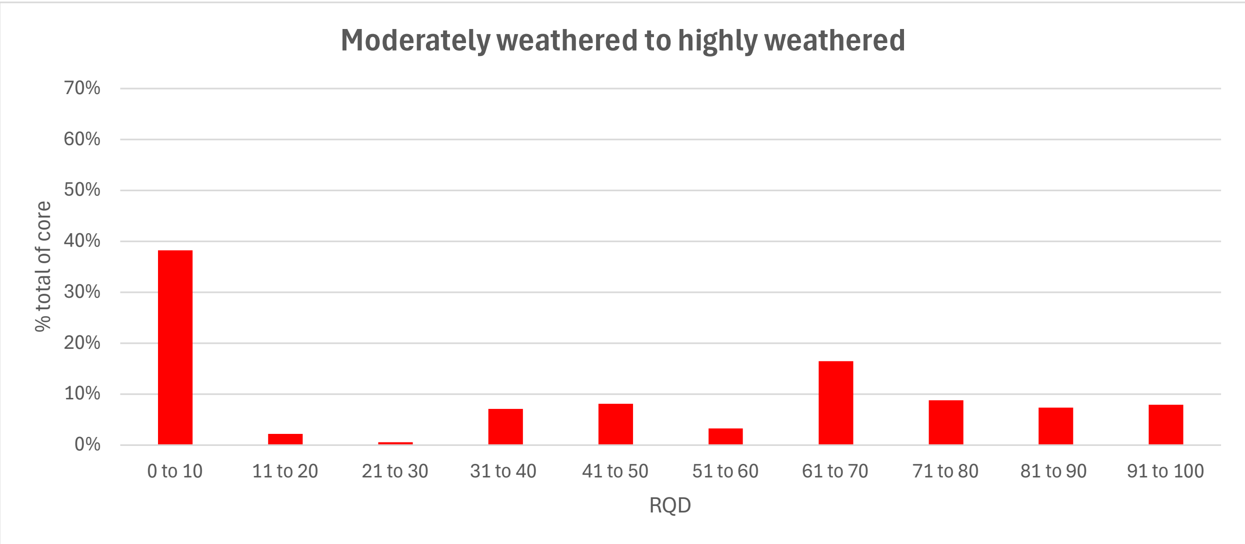
Fig. E8

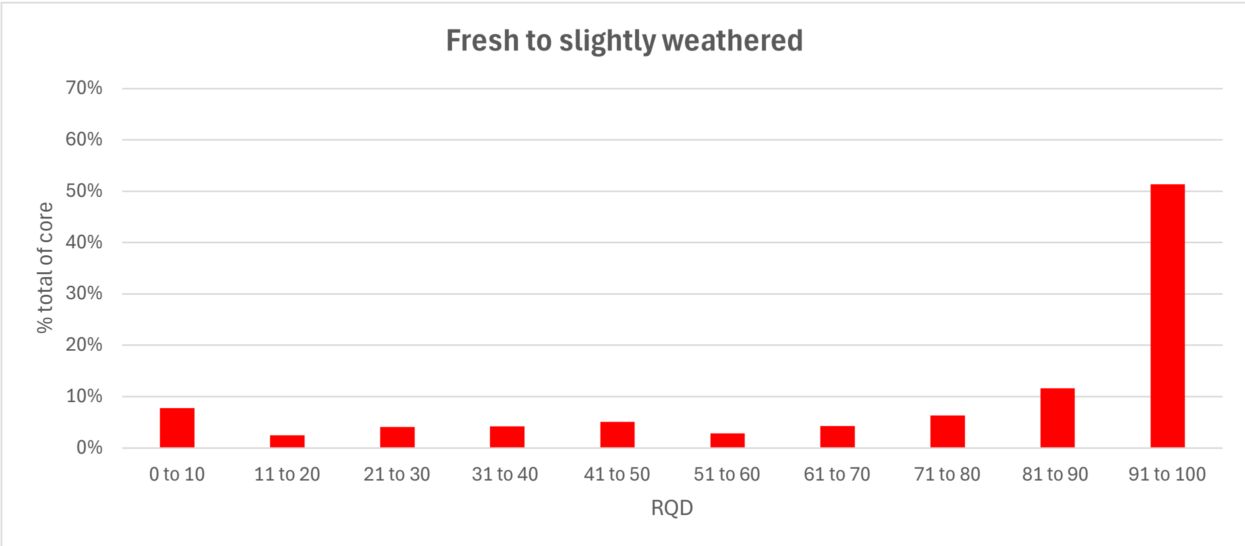
Macquarire Point Stadium

UCS, PLT (Axial and Diametral)

Dolerite







Plot E9	Macquarie Point Stadium, Hobart **RQD bands by Weathering**	

Appendix F

Correlations – SPT N and CPT

CPT and SPT – correlations

Fig	Title
F1	CPT q_c profiles combined
F2	CPT q_c profiles combined (0 MPa to 10 MPa)
F3	SPT N Value vs Young's Modulus E (Unit 1)
F4	SPT N Value vs Young's Modulus E (Unit 2)
F5	SPT/CPT N Value vs friction angle – Fill
F6	SPT/CPT N Value vs friction angle – Estuarine
F7	Inferred undrained shear strength – Fill
F8	Inferred undrained shear strength – Estuarine

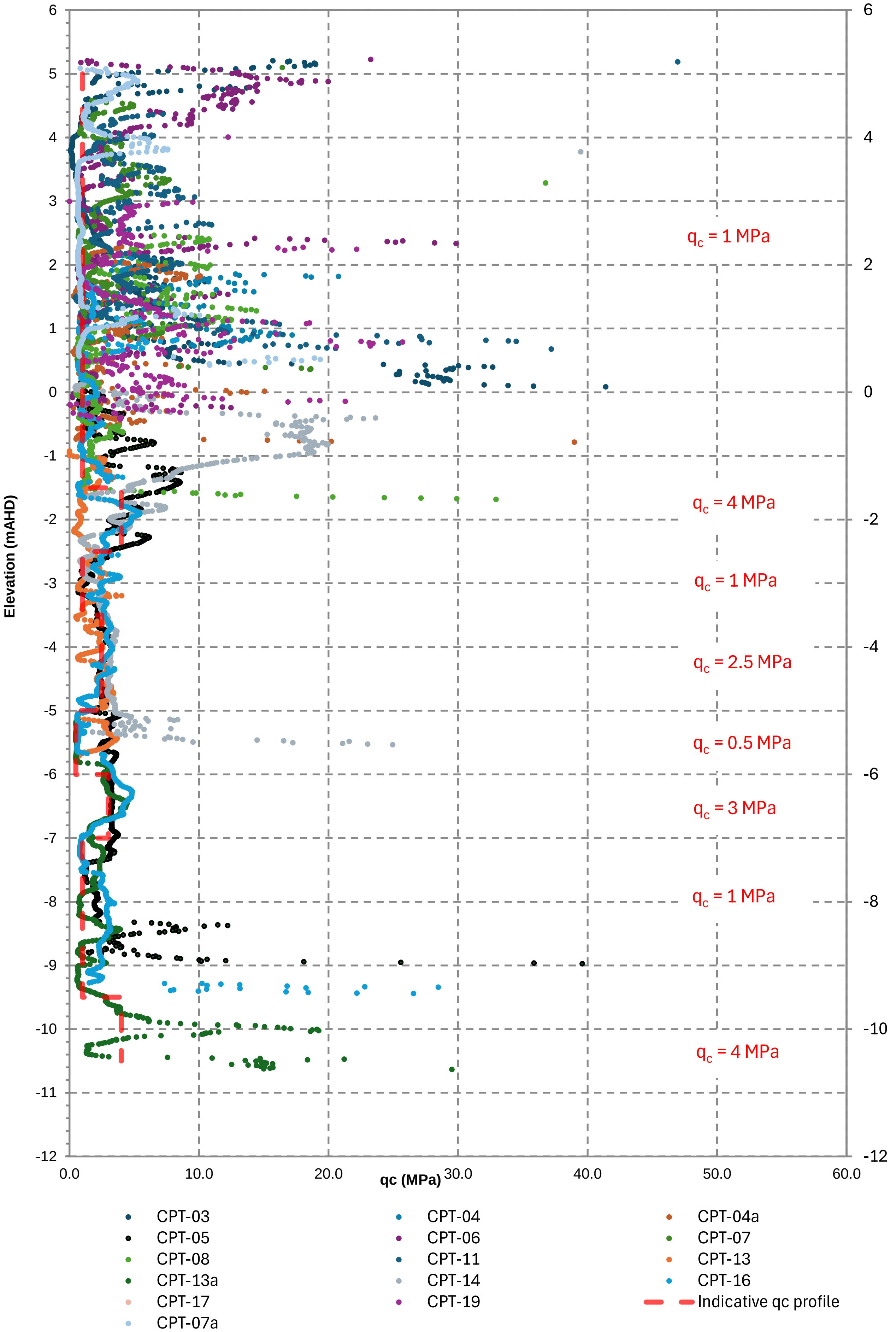
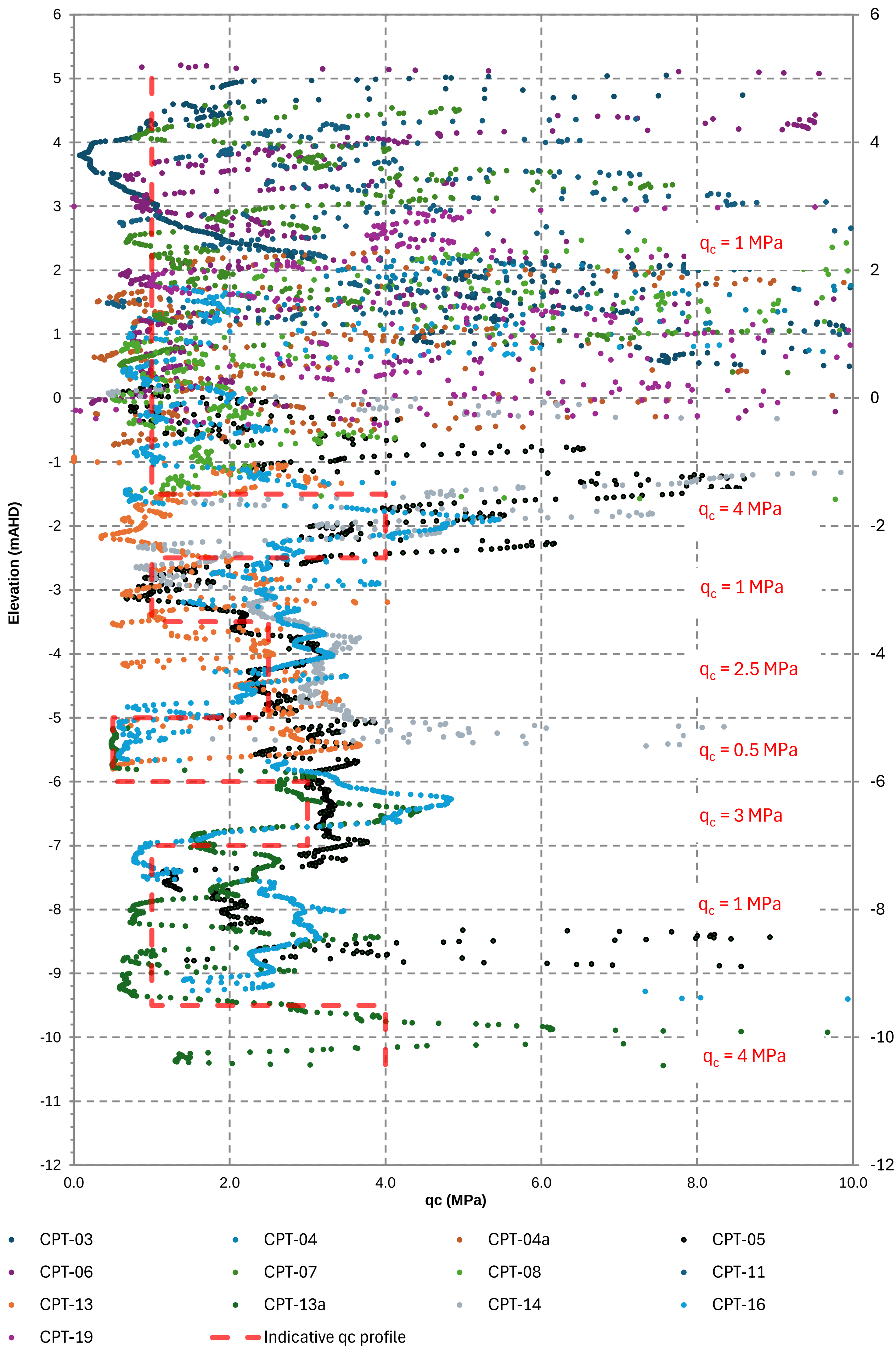


Fig. F1

Macquarie Point Stadium, Hobart

q_c vs elevation

All



wsp

Fig. F2

Macquarie Point Stadium, Hobart

q_c vs elevation

All traces 0 MPa to 10 MPa

MAC POINT

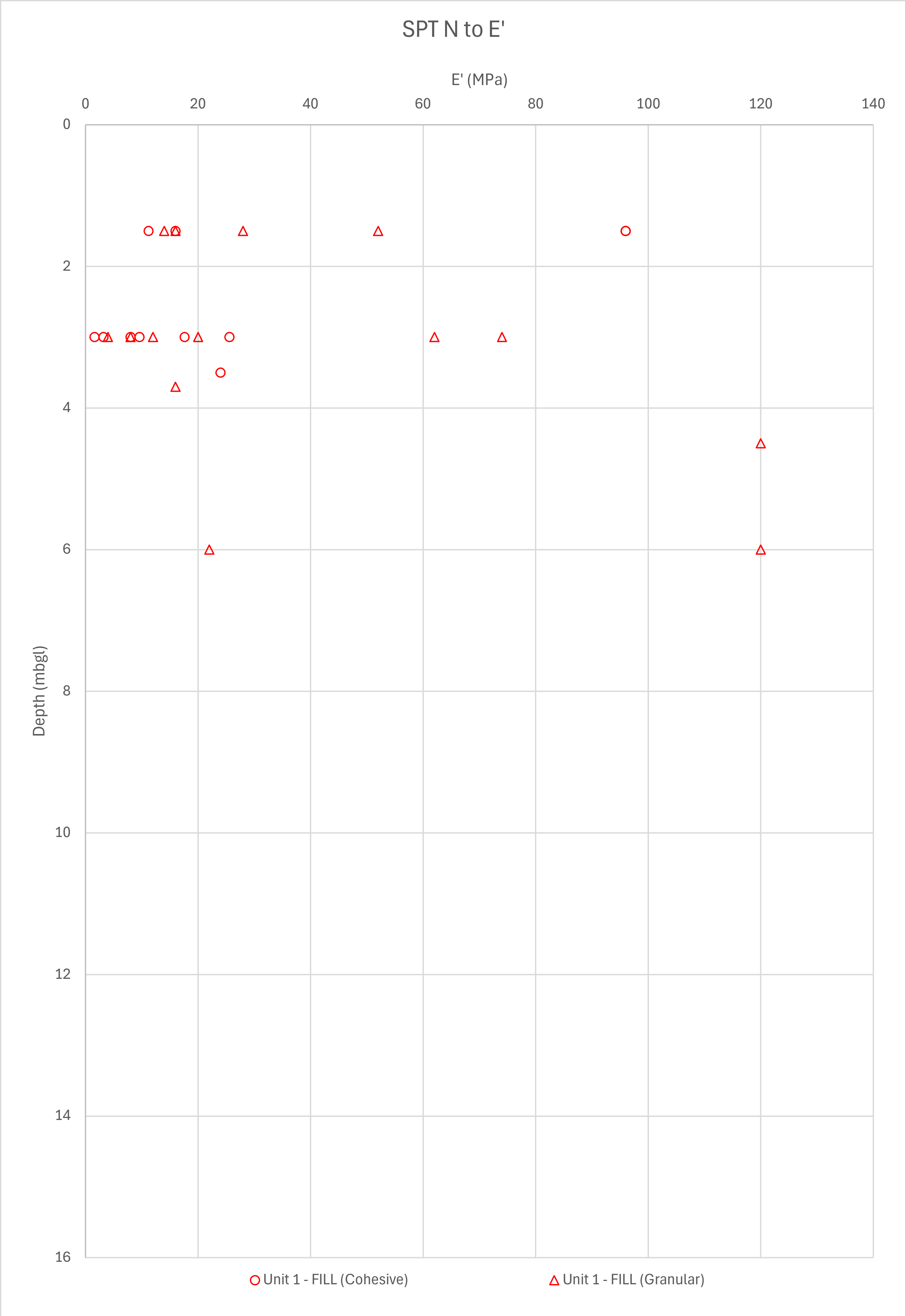


Fig. F3

Macquarie Point Stadium, Hobart
Inferred E' based on SPT N values
Fill

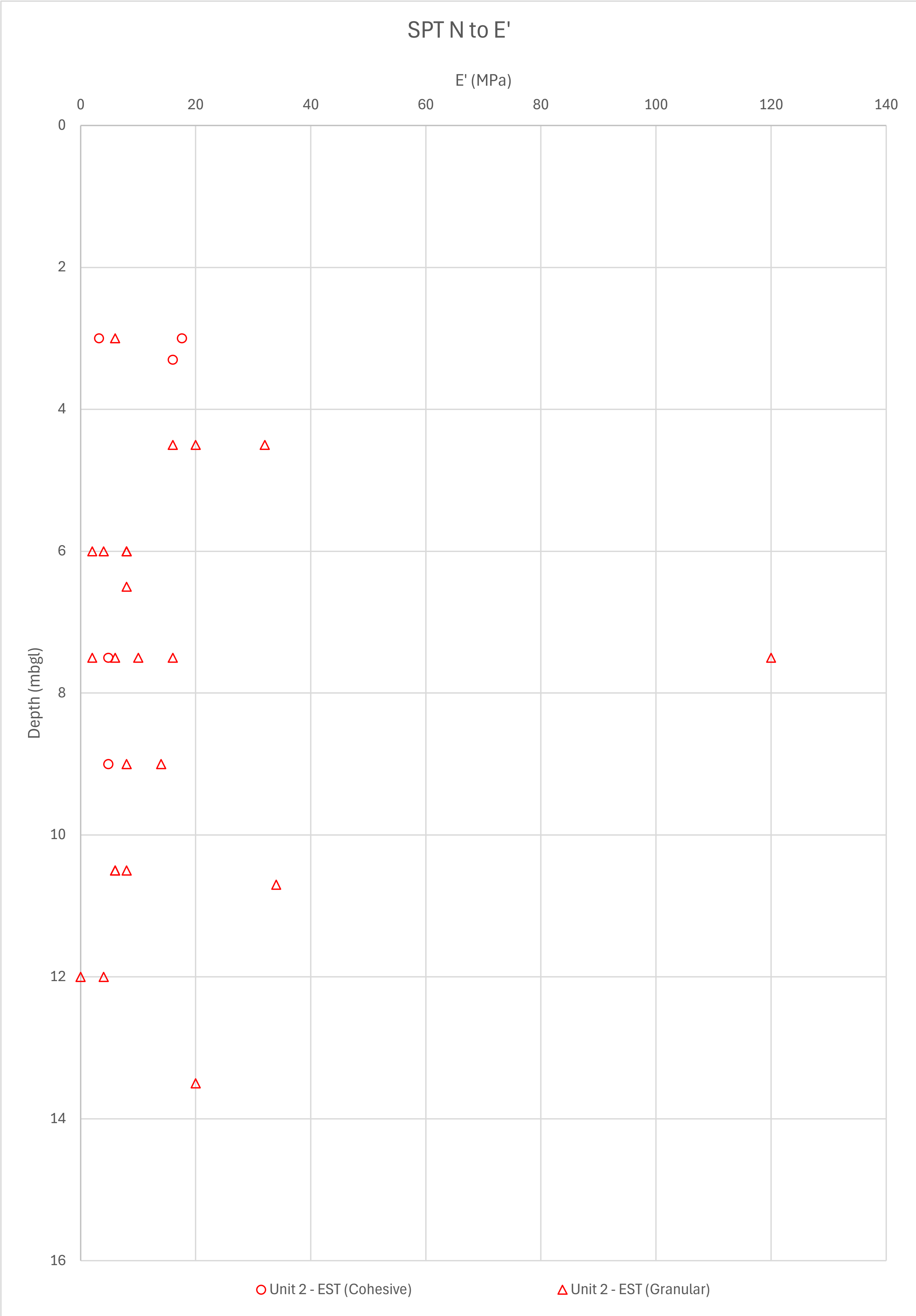
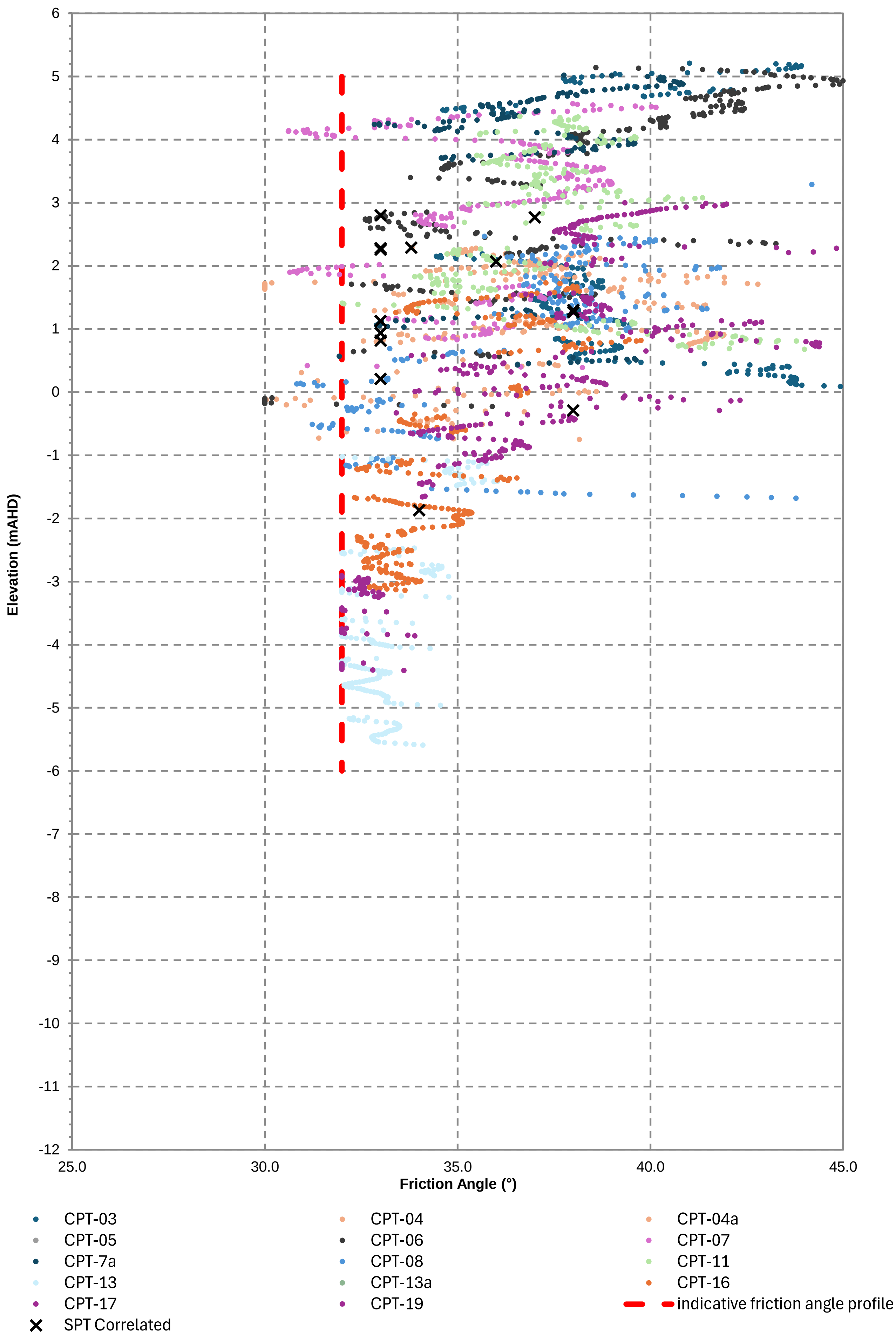


Fig. F4

Macquarie Point Stadium, Hobart
Inferred E' based on SPT N values
Estuarine



wsp	**Fig. F5**	Macquarie Point Stadium, Hobart **Friction angle vs elevation** **Fill**

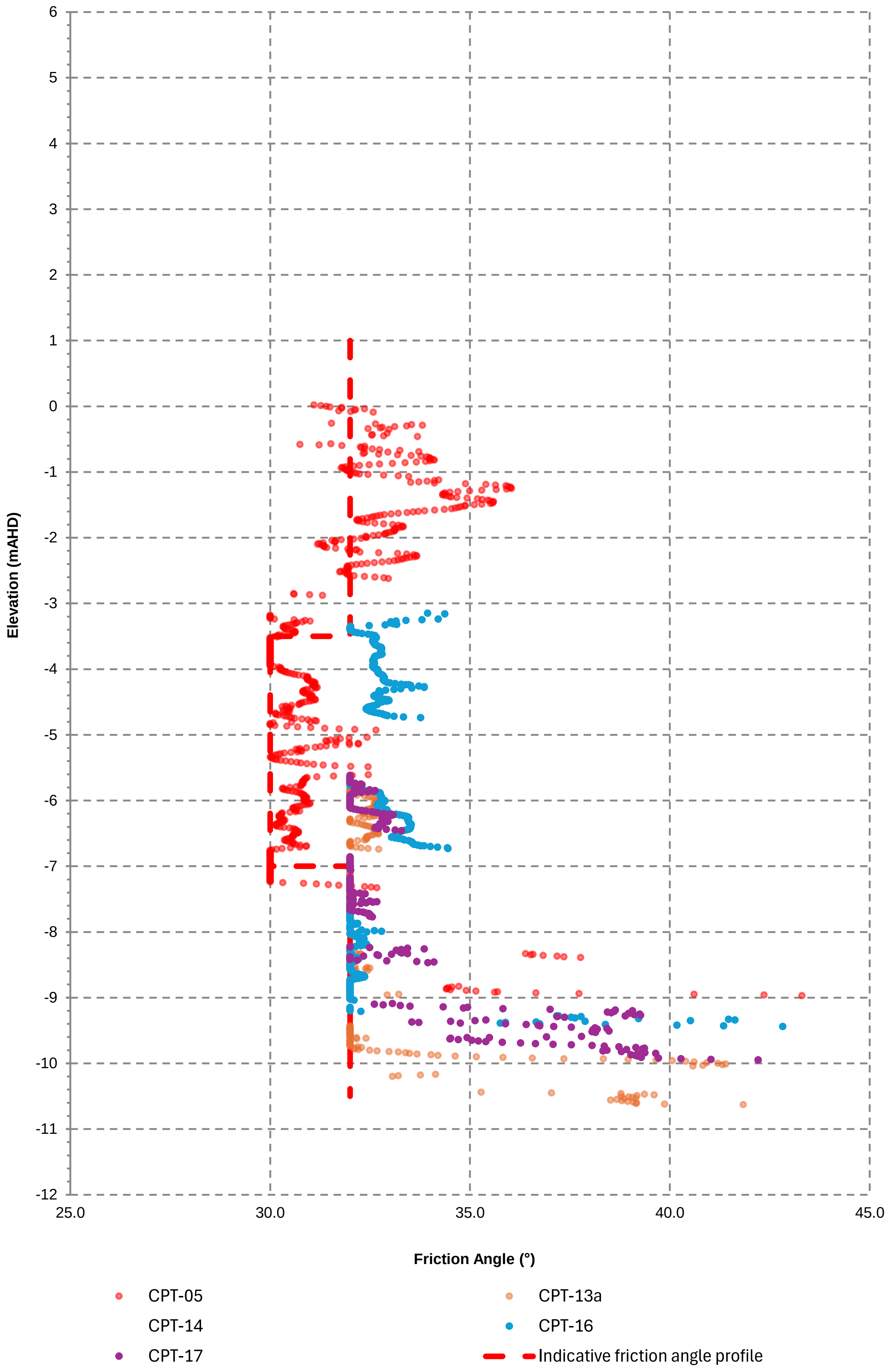
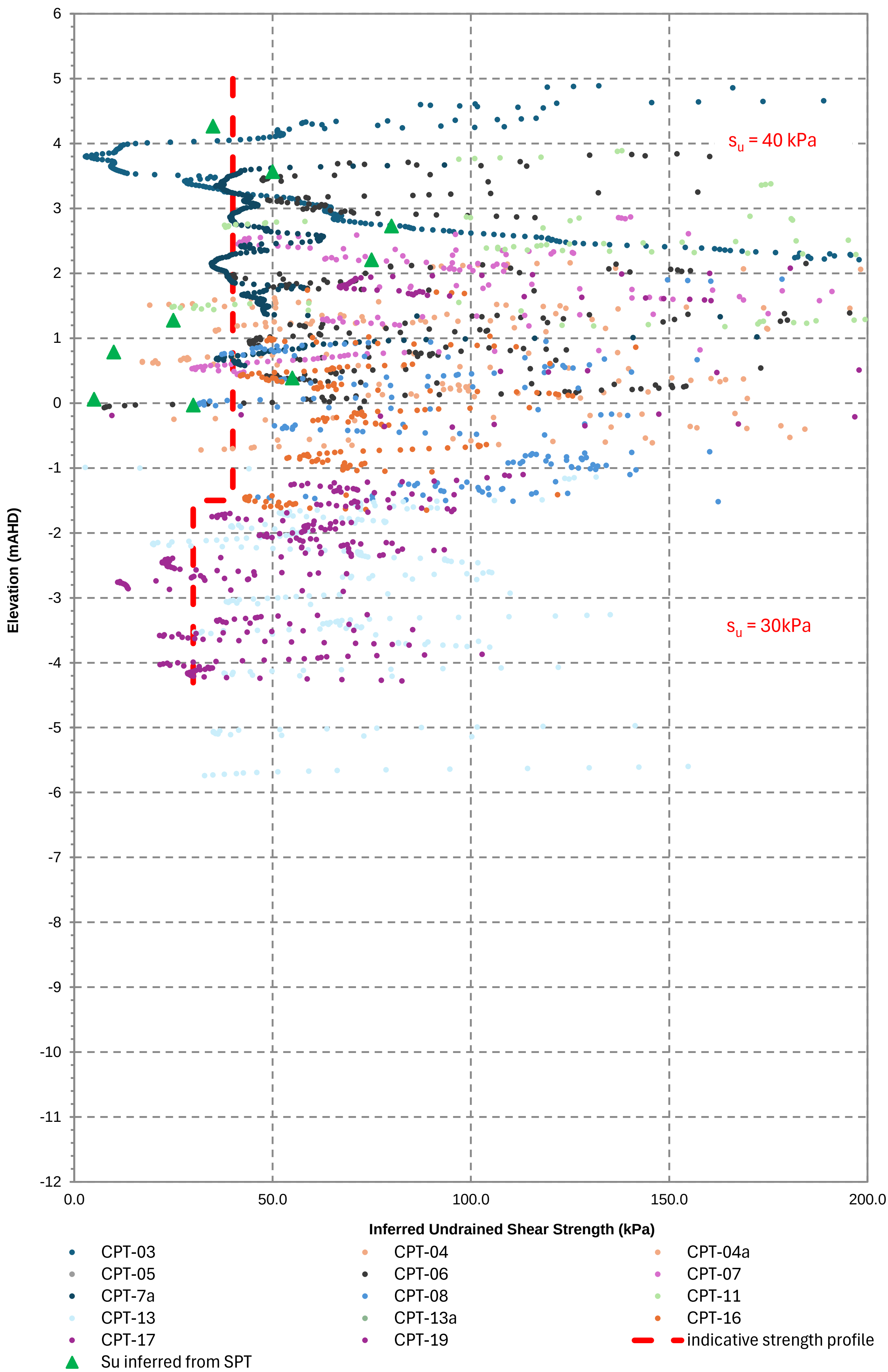


Fig. F6	Macquarie Point Stadium, Hobart **Friction angle vs elevation** **Estuarine**	MAC POINT



wsp	Fig. F7	Macquarie Point Stadium, Hobart **Inferred Undrained shear strength vs elevation Fill**	MAC POINT

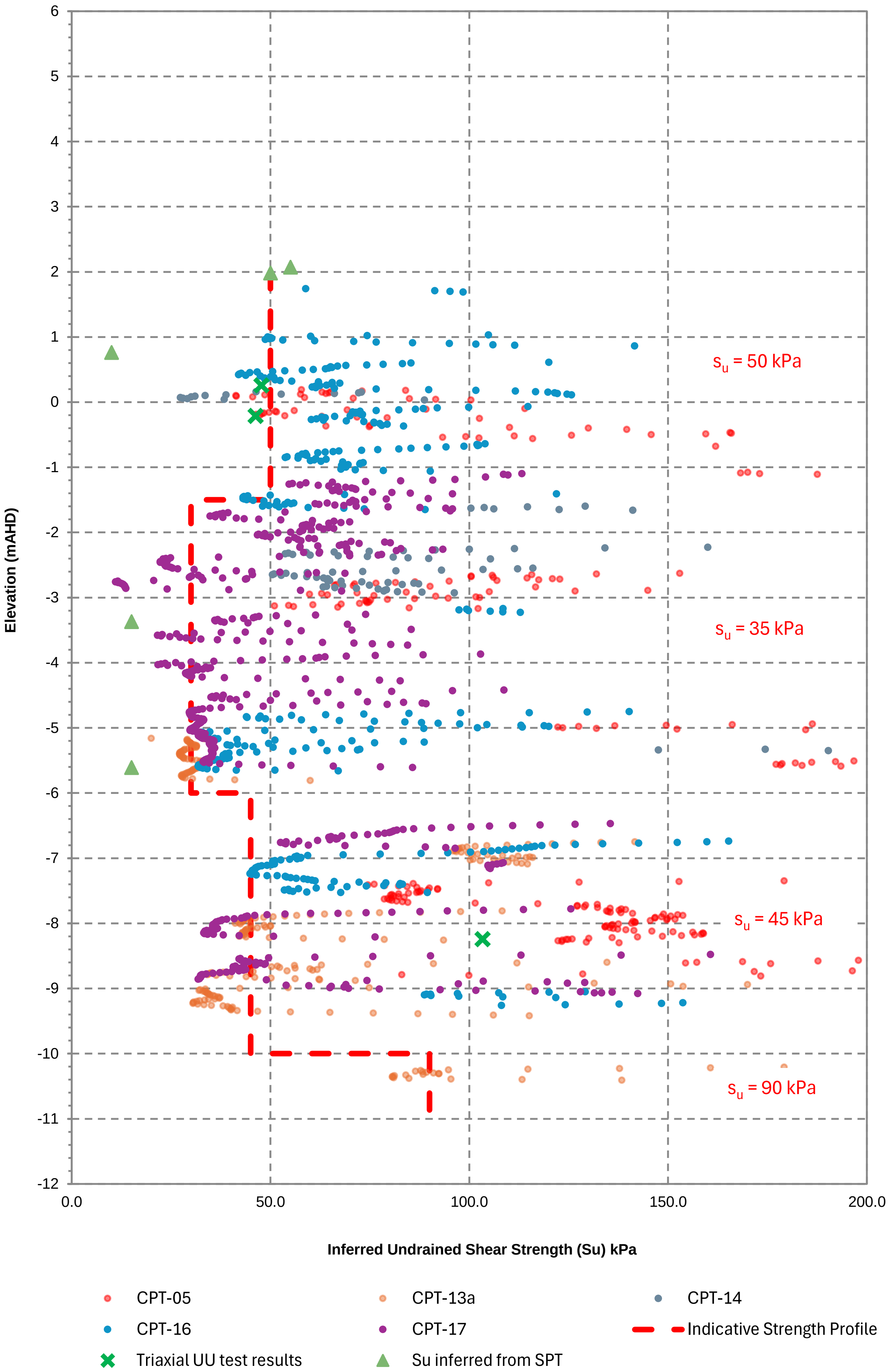


Fig F8

Macquarie Point Stadium, Hobart

Inferred Undrained shear strength vs elevation Estuarine

Appendix G

Liquefaction assessment

LIQUEFACTION ANALYSIS REPORT

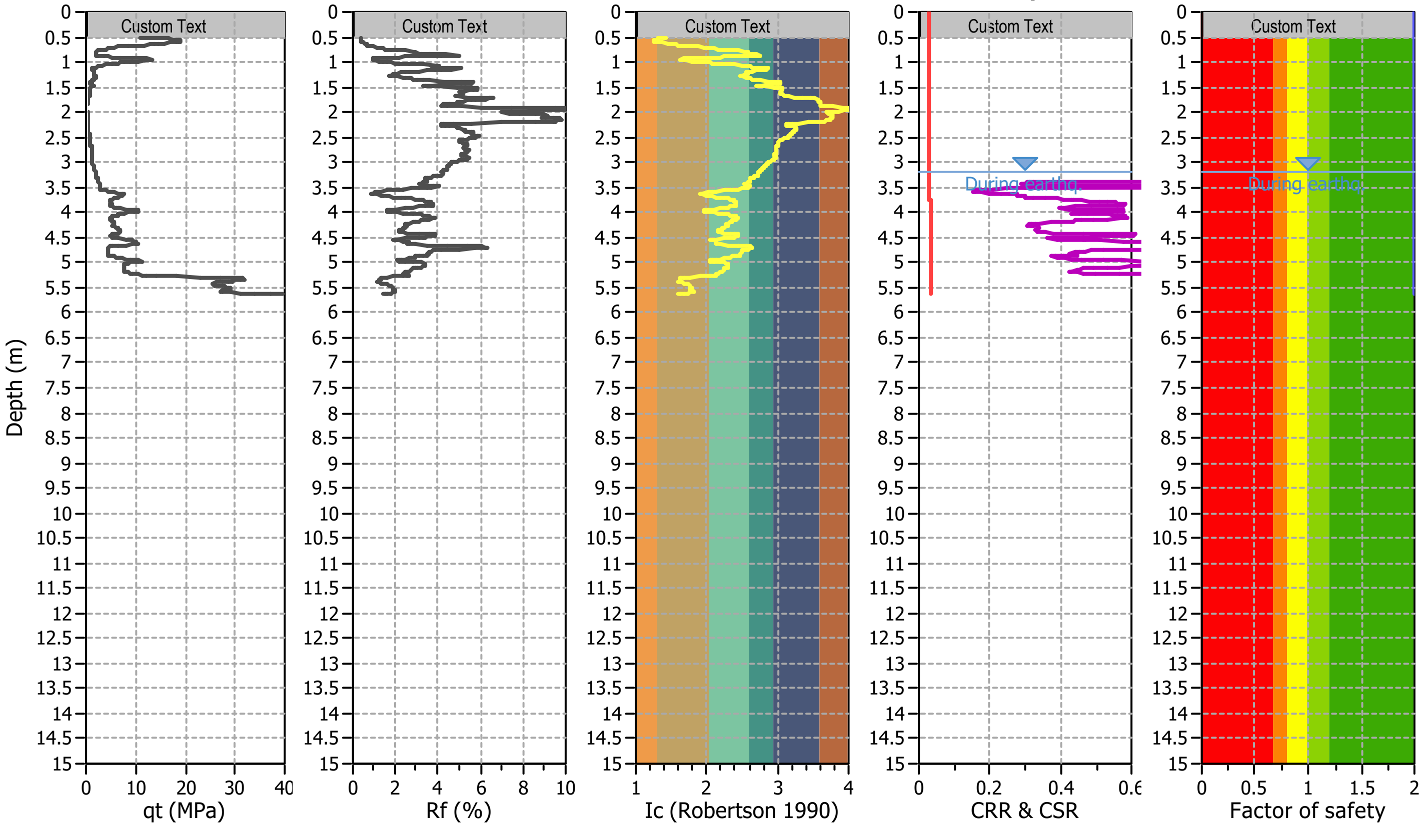
Project title : PS212776 Macquarie Point **Location : Hobart, Tasmania**

CPT file : CPT-03

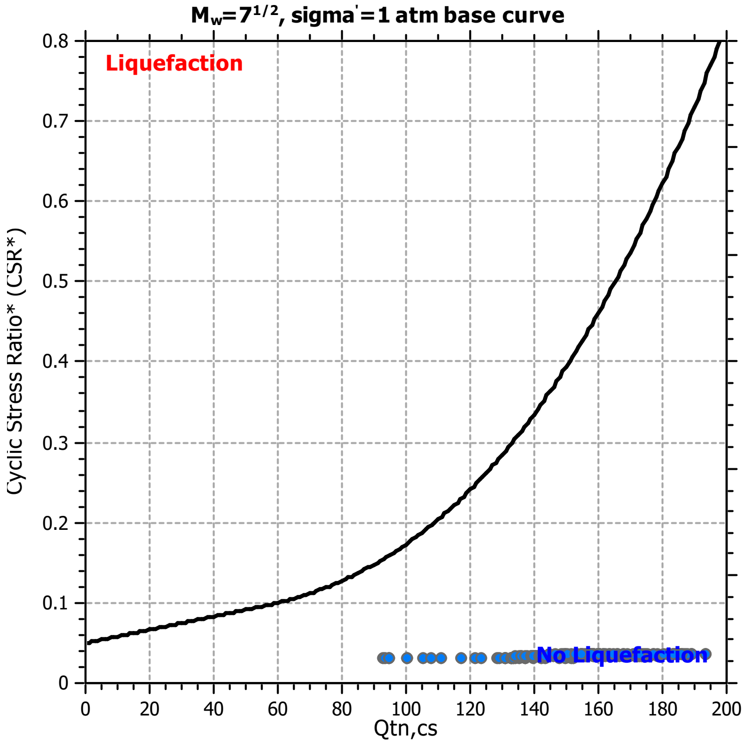
Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	3.20 m	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	3.20 m	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	5.50	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.10	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

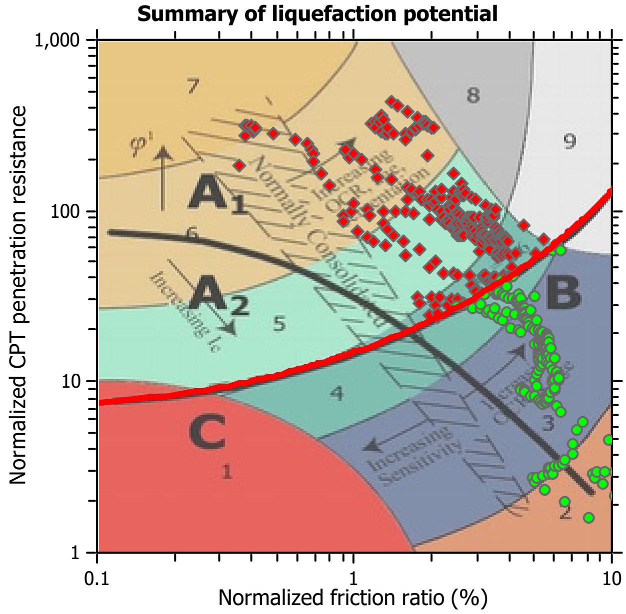
Cone resistance **Friction Ratio** **SBTn Plot** **CRR plot** **FS Plot**



$M_w=7^{1/2}$, sigma'=1 atm base curve



Summary of liquefaction potential



Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

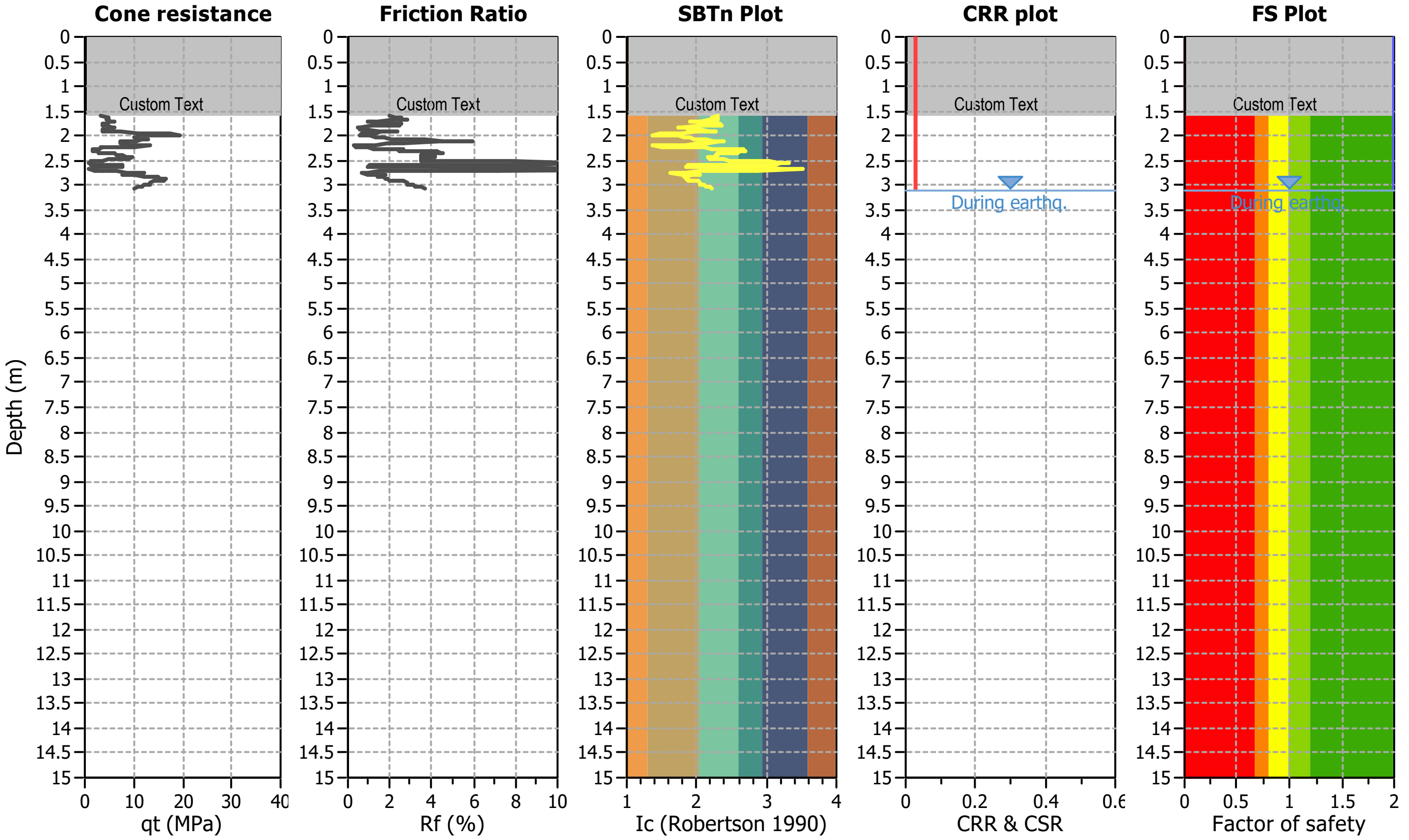
LIQUEFACTION ANALYSIS REPORT

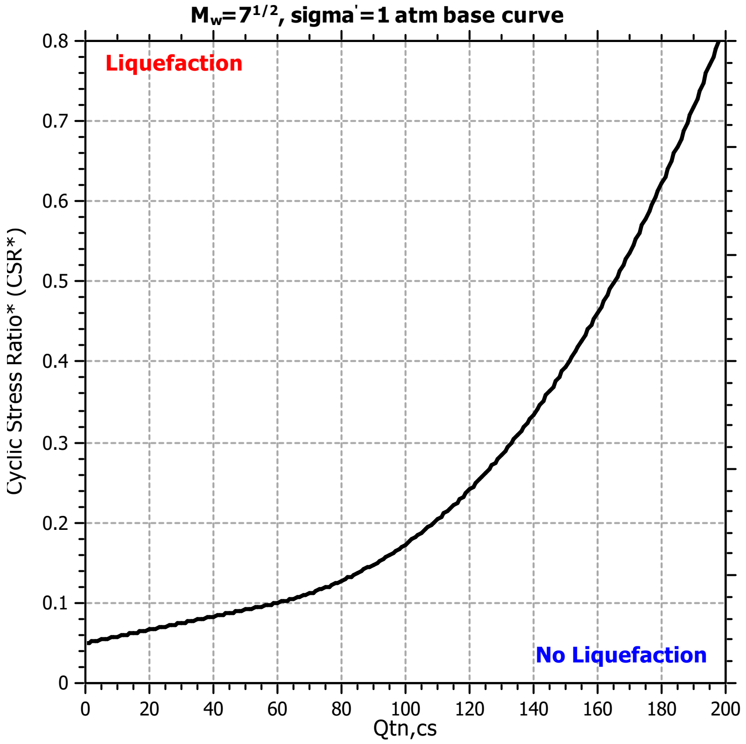
Project title : PS212776 Macquarie Point **Location : Hobart, Tasmania**

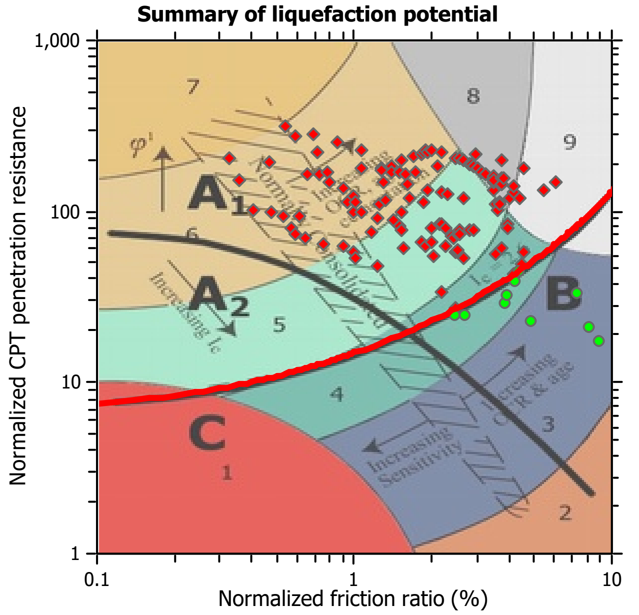
CPT file : CPT-04

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	3.10 m	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	3.10 m	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	5.50	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.10	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		







Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

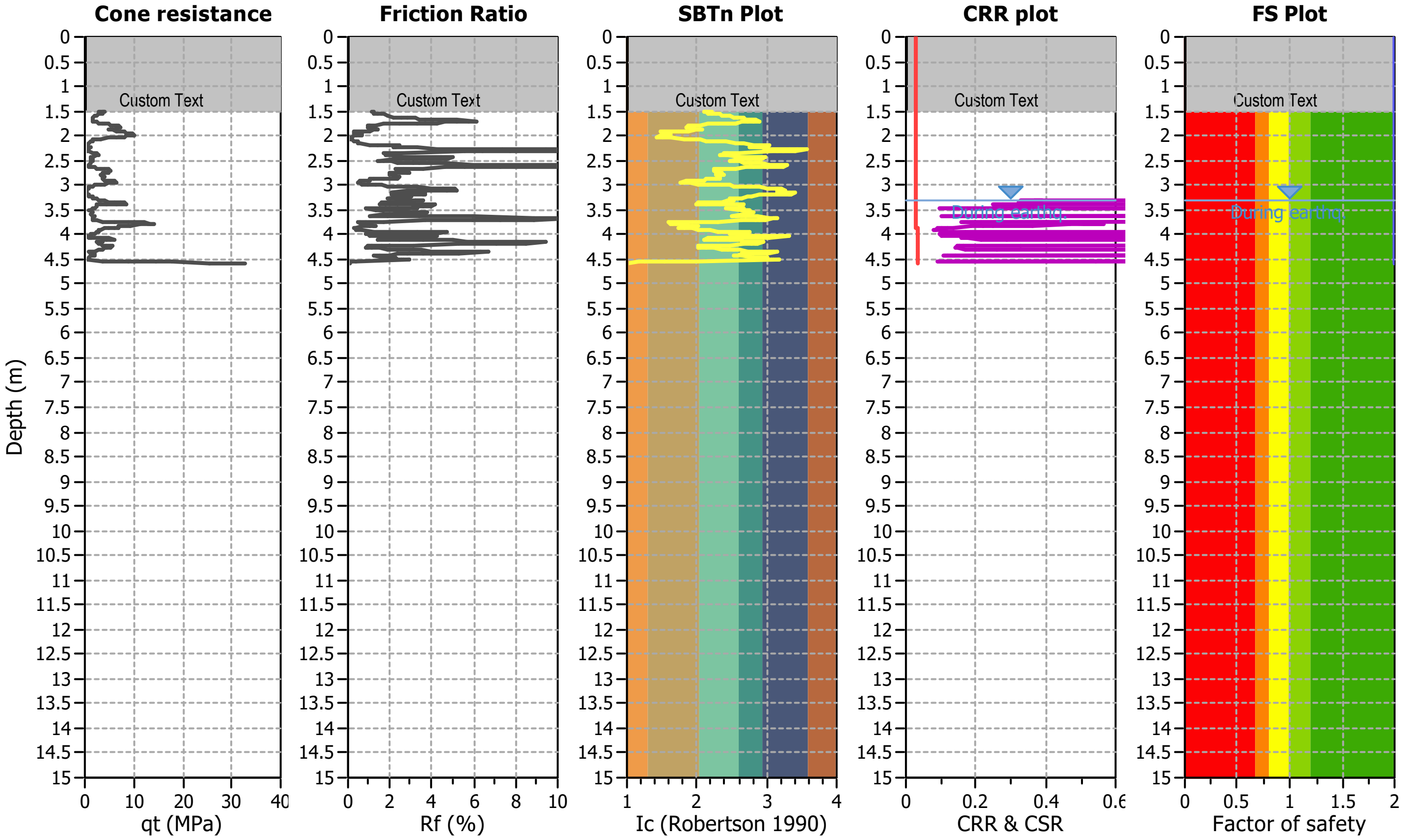
LIQUEFACTION ANALYSIS REPORT

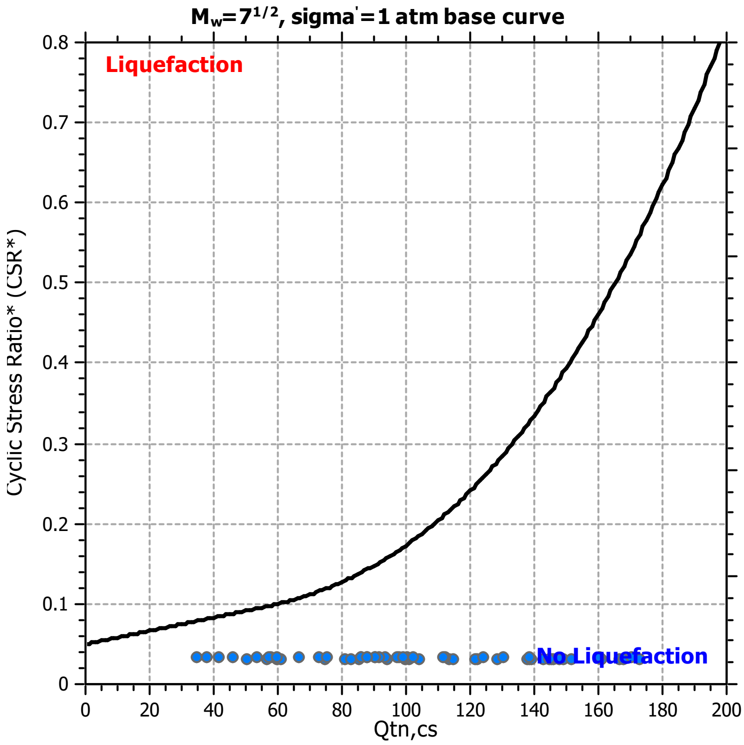
Project title : PS212776 Macquarie Point **Location : Hobart, Tasmania**

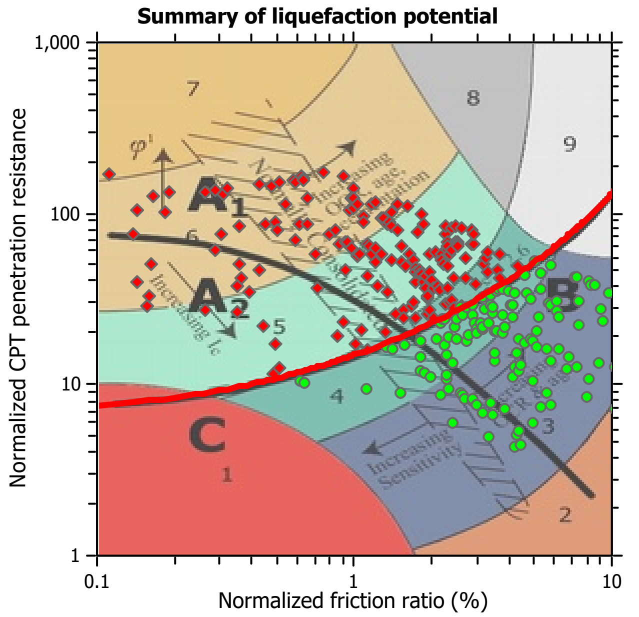
CPT file : CPT-04a

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	3.30 m	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	3.30 m	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	5.50	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.10	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		







Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

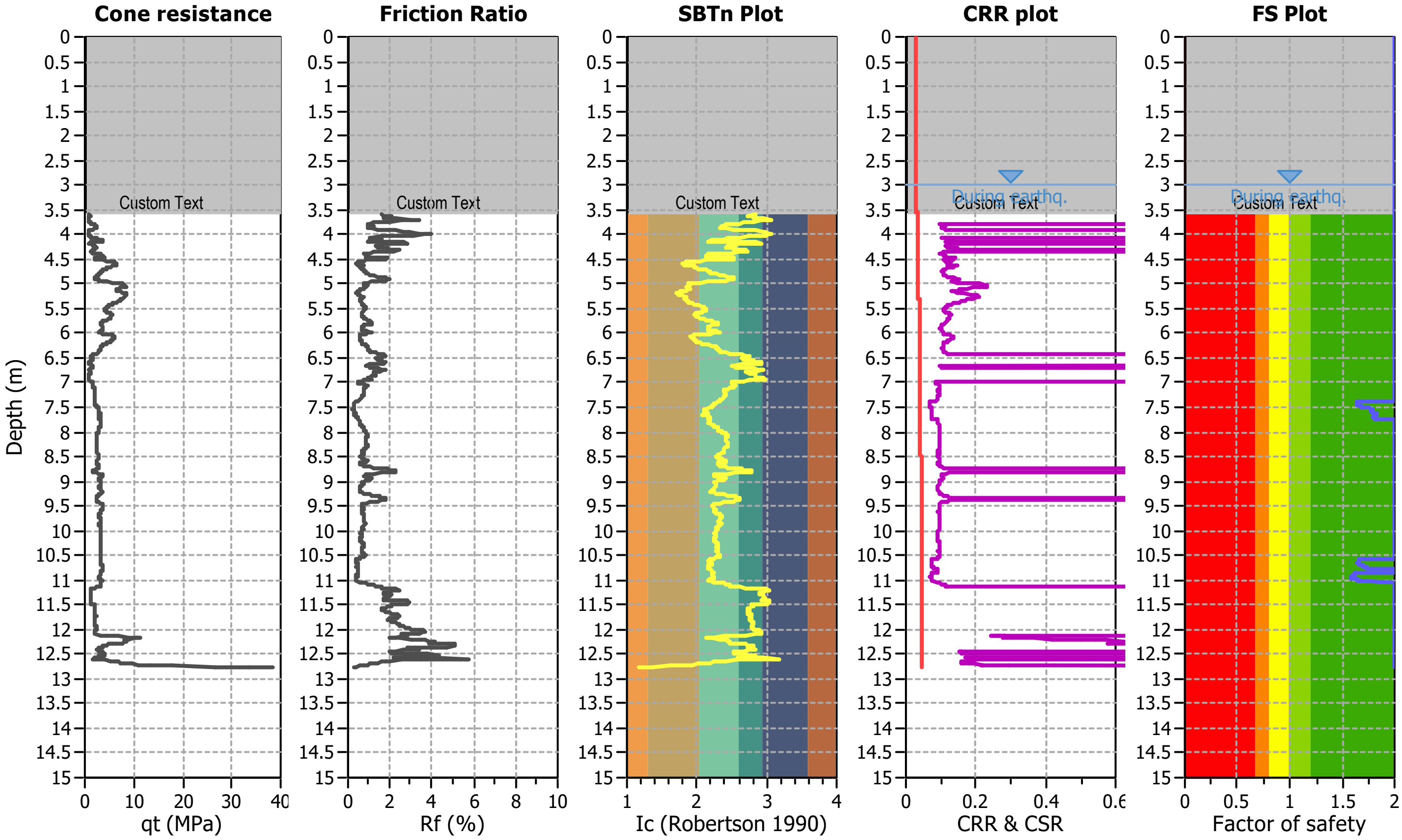
LIQUEFACTION ANALYSIS REPORT

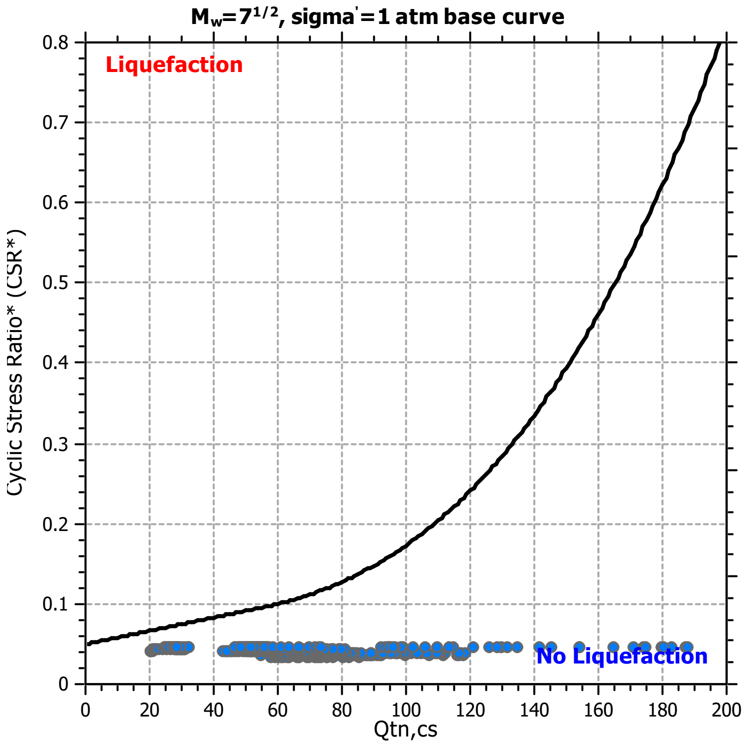
Project title : PS212776 Macquarie Point **Location : Hobart, Tasmania**

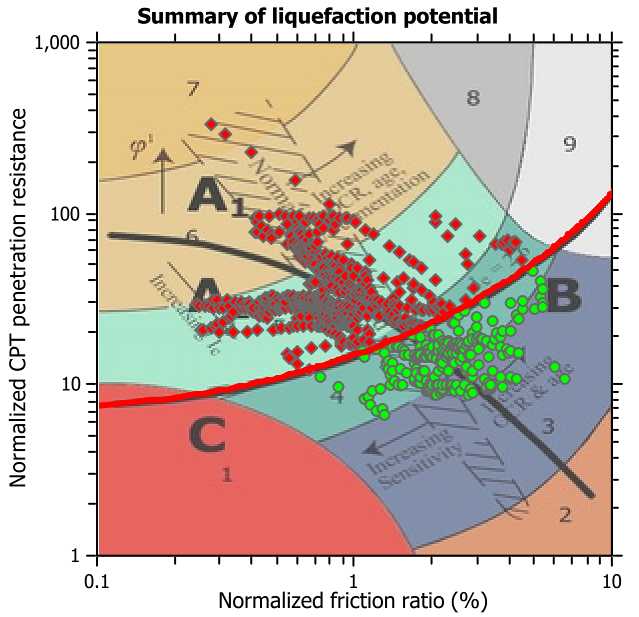
CPT file : CPT-05

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	3.00 m	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	3.00 m	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	5.50	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.10	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		







Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

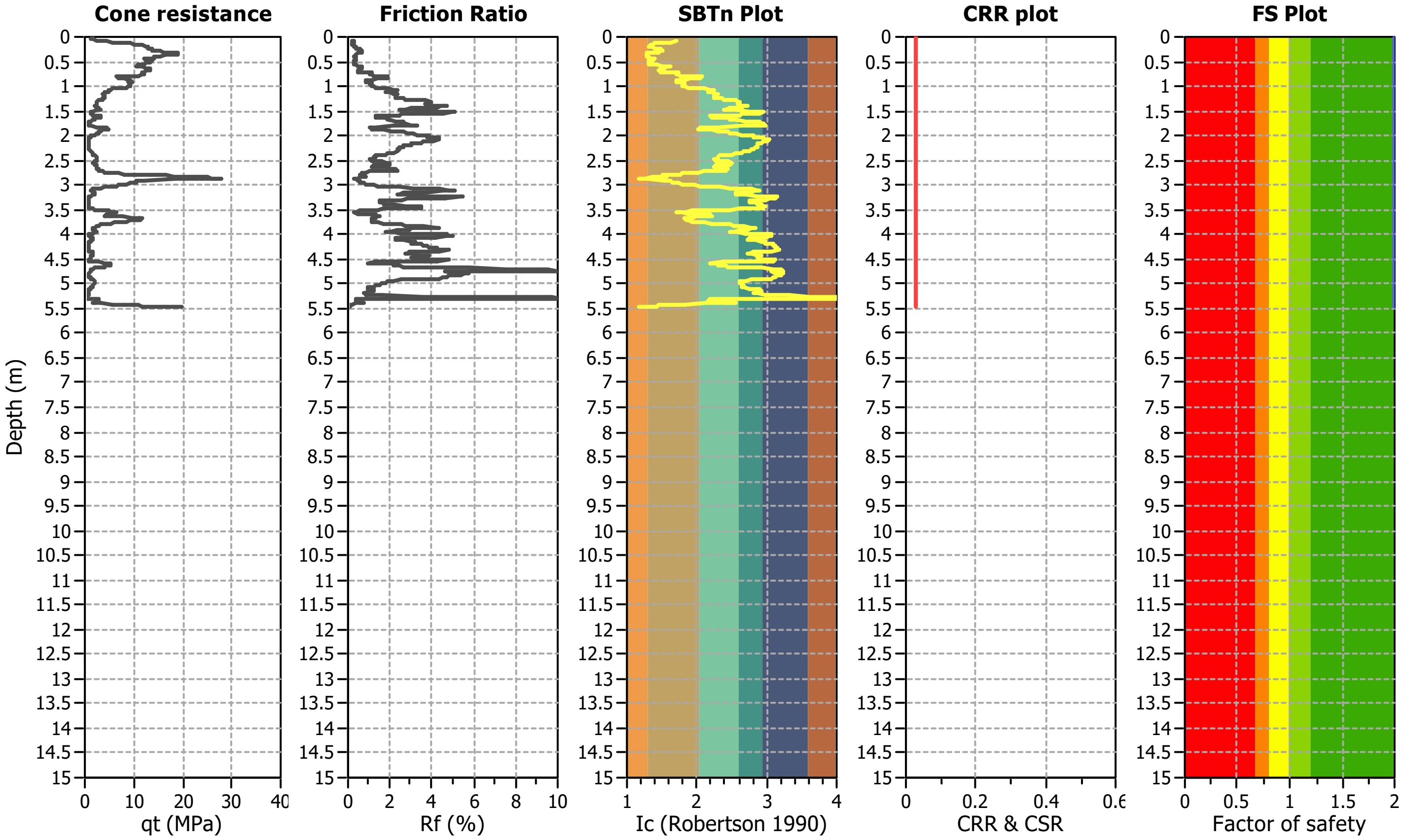
LIQUEFACTION ANALYSIS REPORT

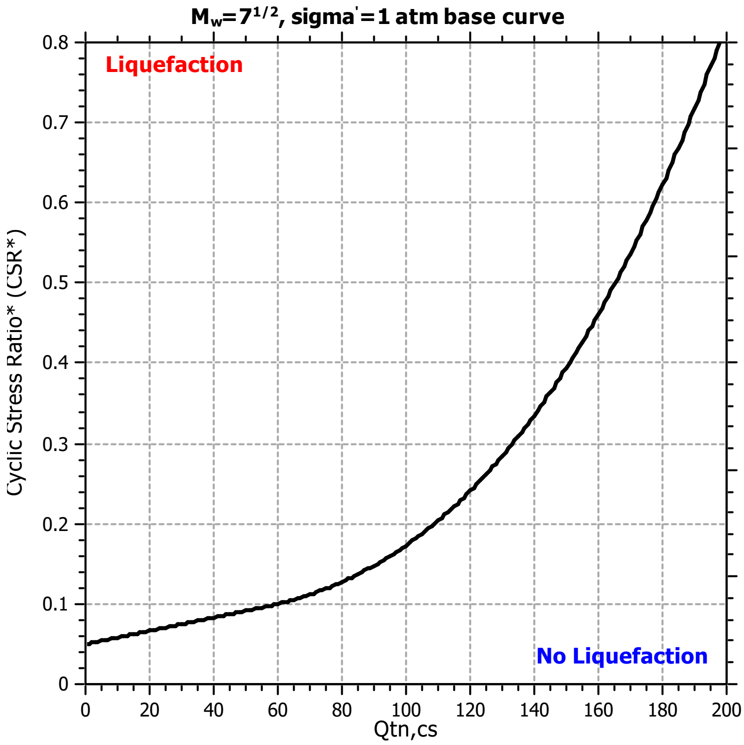
Project title : PS212776 Macquarie Point **Location : Hobart, Tasmania**

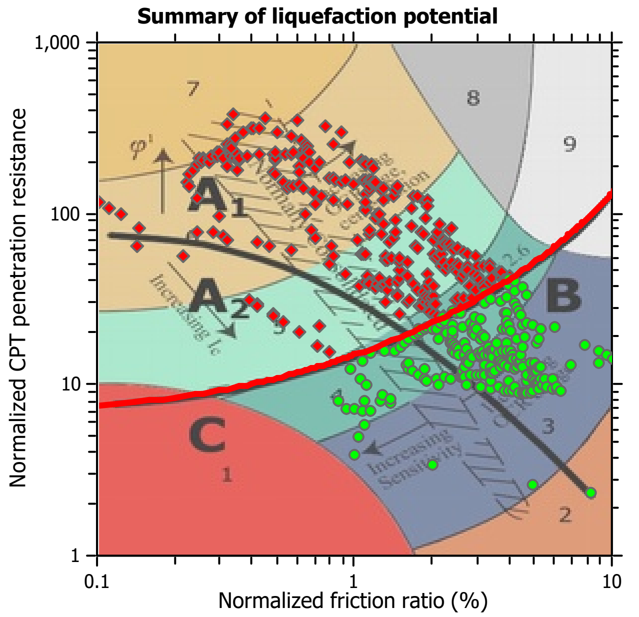
CPT file : CPT-06

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	25.00 m	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	25.00 m	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	5.50	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.10	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		







Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

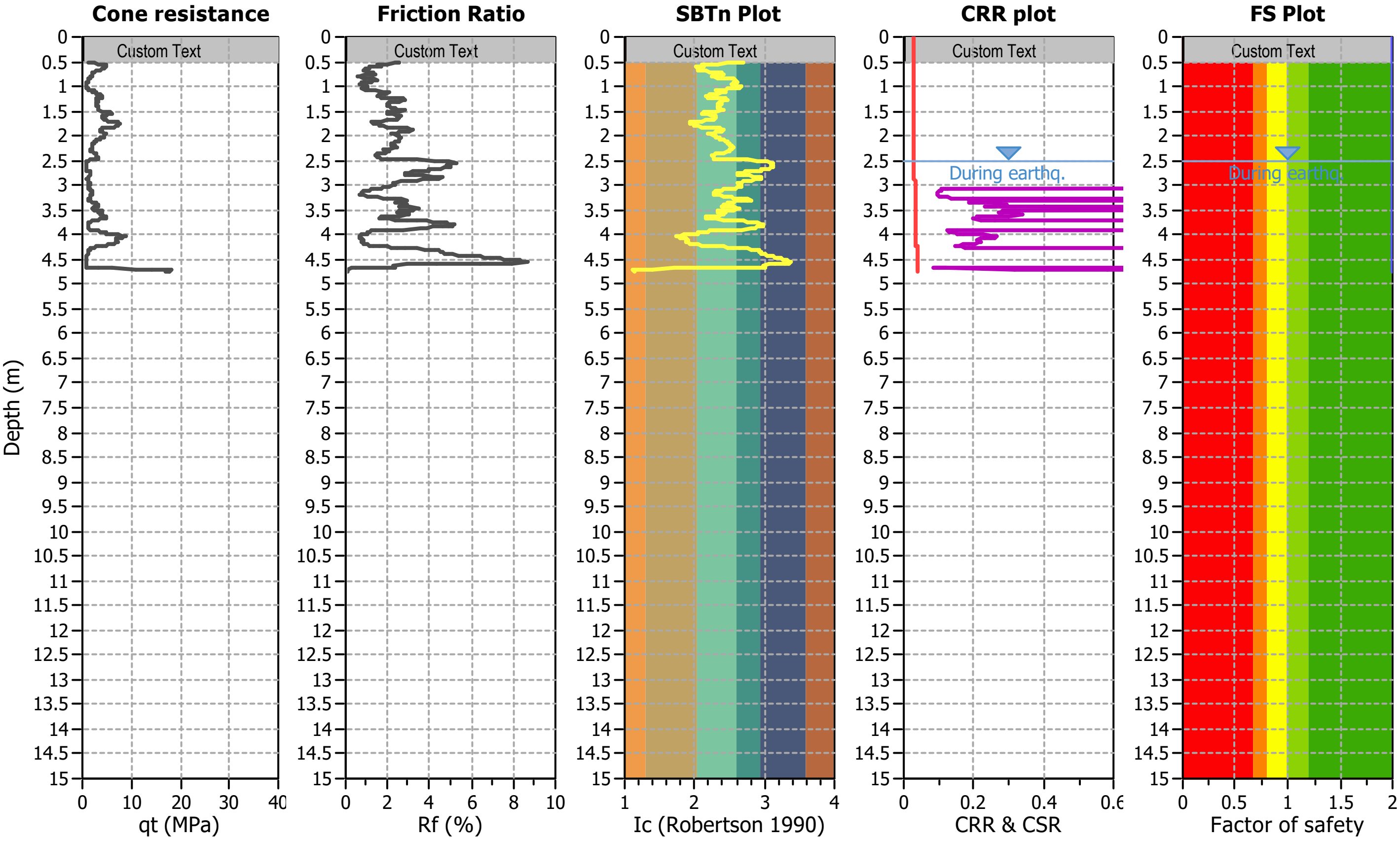
LIQUEFACTION ANALYSIS REPORT

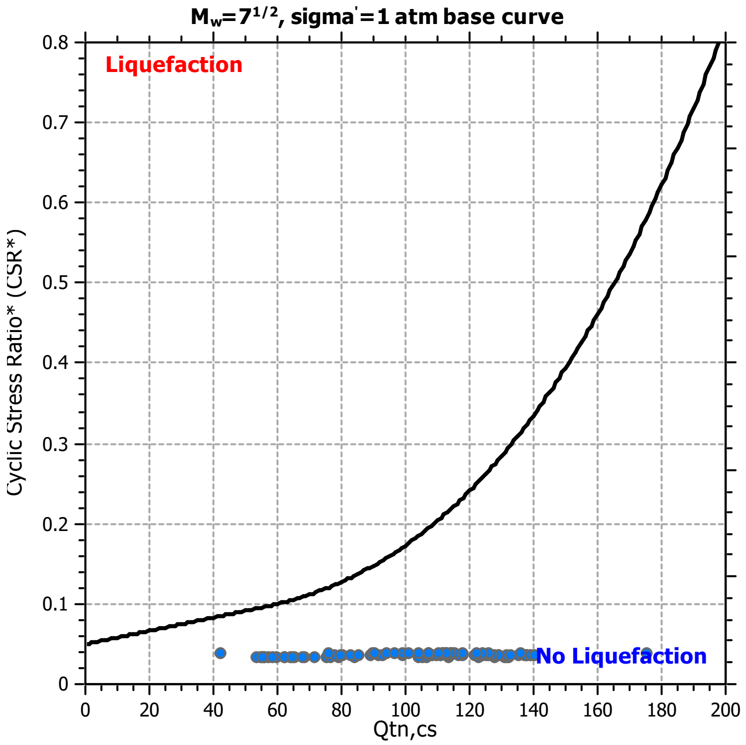
Project title : PS212776 Macquarie Point **Location : Hobart, Tasmania**

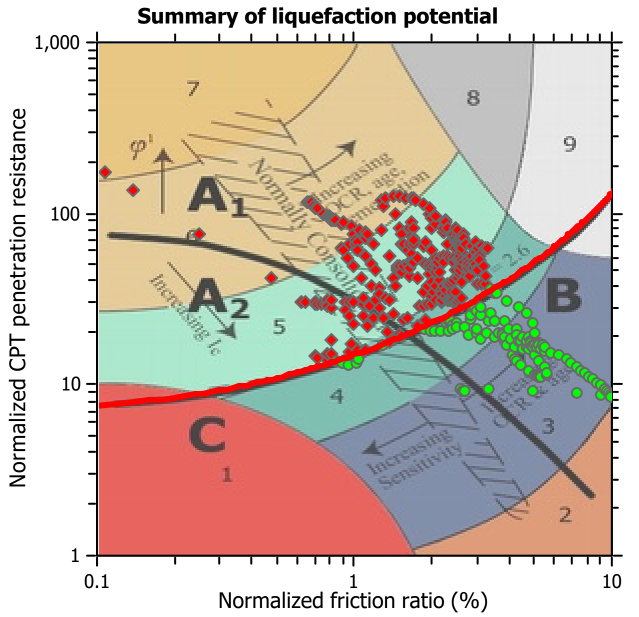
CPT file : CPT-07

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	2.50 m	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	2.50 m	Fill height:	N/A		
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth applied:	No
Earthquake magnitude M_w:	5.50	Ic cut-off value:	2.60	Trans. detect. applied:	No	Limit depth:	N/A
Peak ground acceleration:	0.10	Unit weight calculation:	Based on SBT	K_σ applied:	Yes	MSF method:	Method based







Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

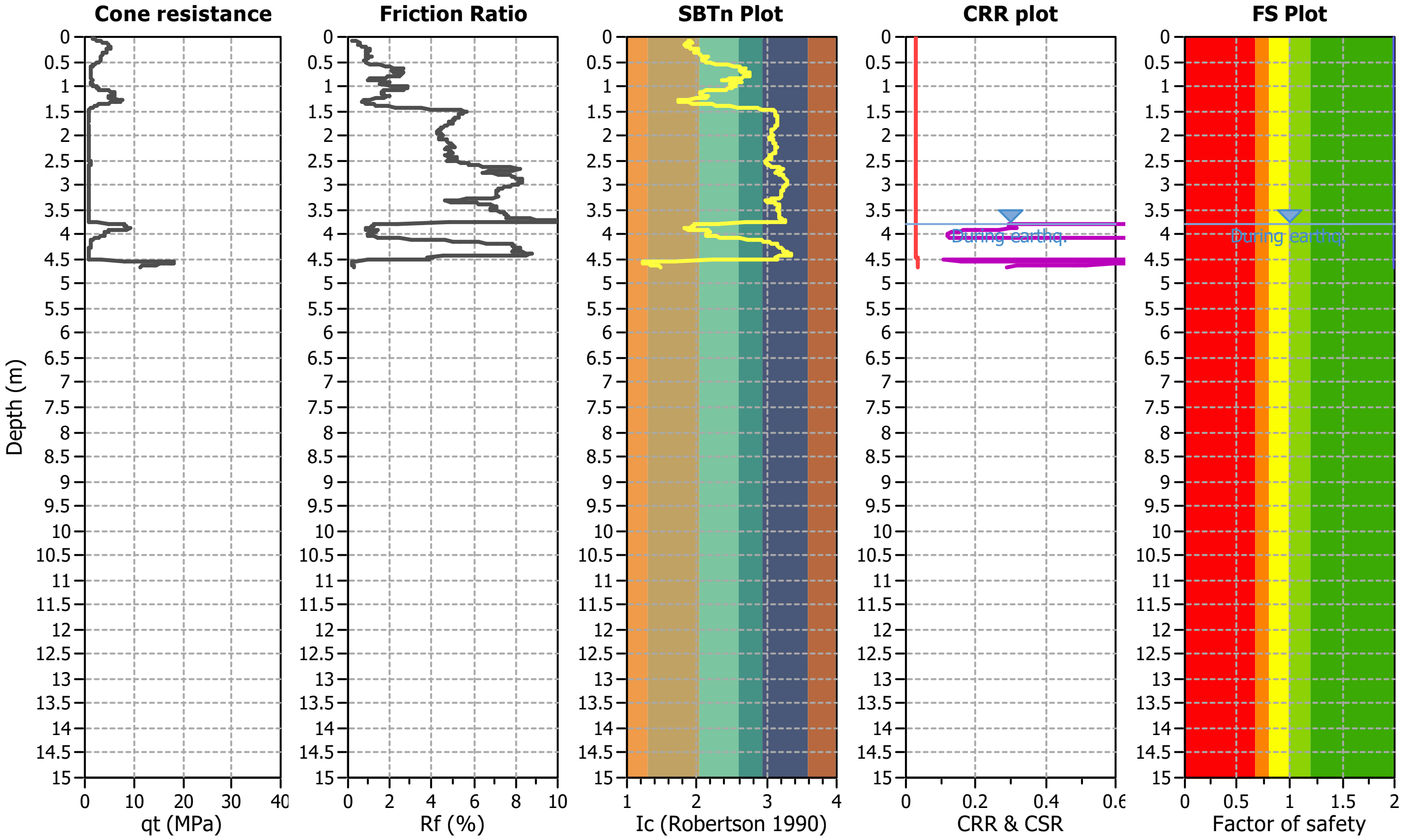
LIQUEFACTION ANALYSIS REPORT

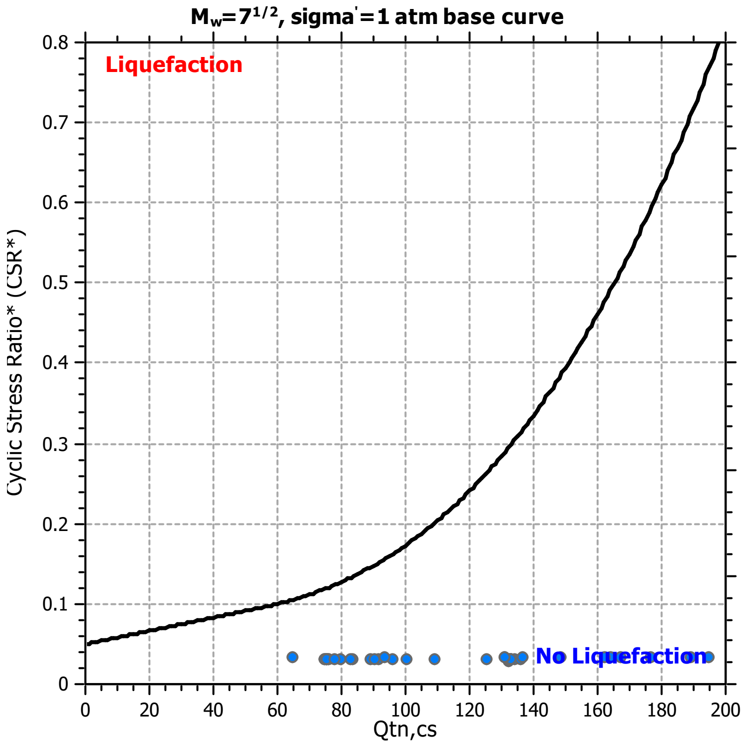
Project title : PS212776 Macquarie Point **Location : Hobart, Tasmania**

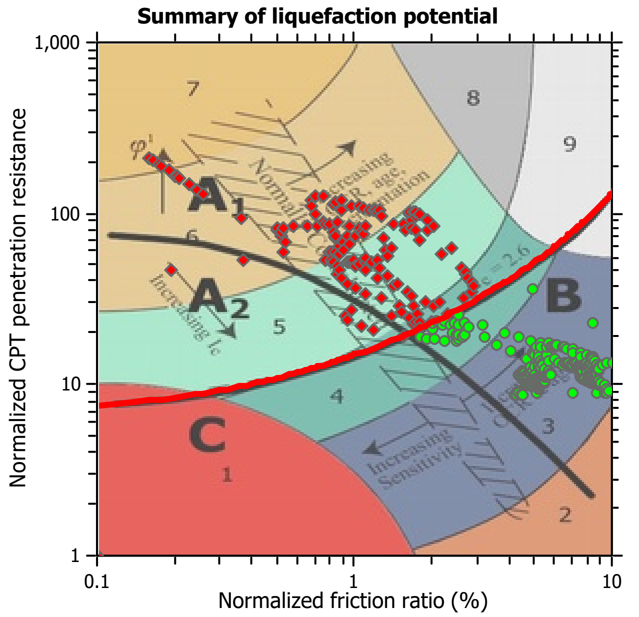
CPT file : CPT-07a

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	3.80 m	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	3.80 m	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	5.50	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.10	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		







Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

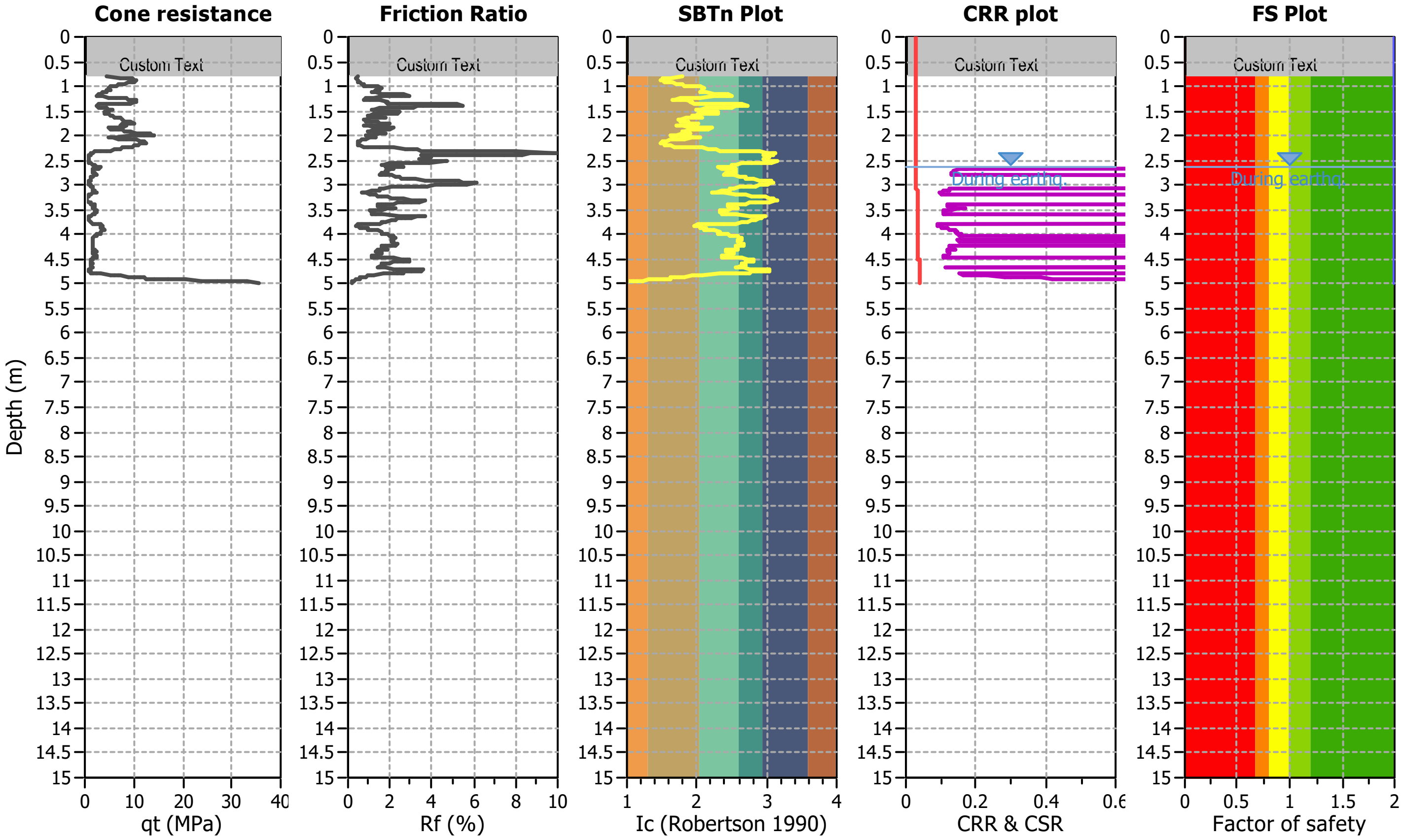
LIQUEFACTION ANALYSIS REPORT

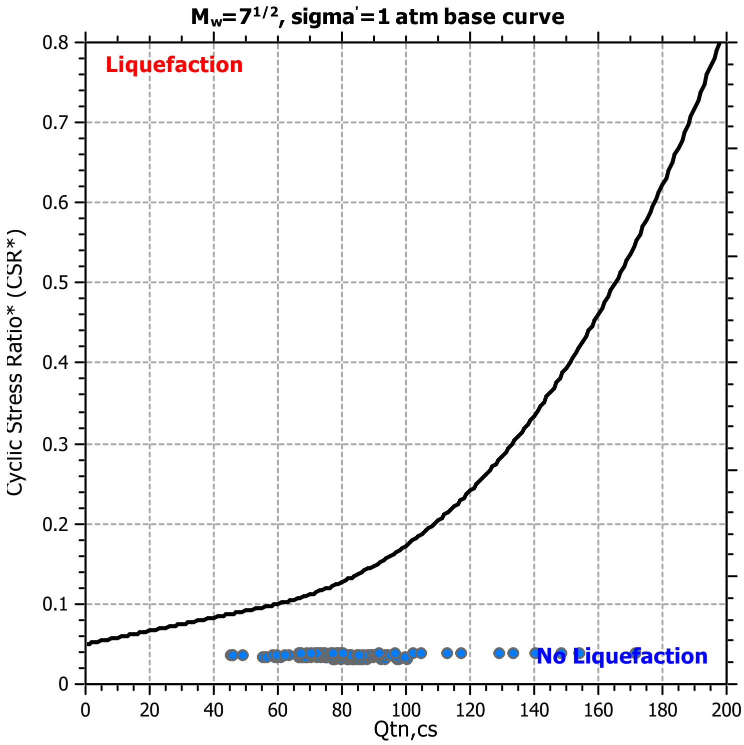
Project title : PS212776 Macquarie Point **Location : Hobart, Tasmania**

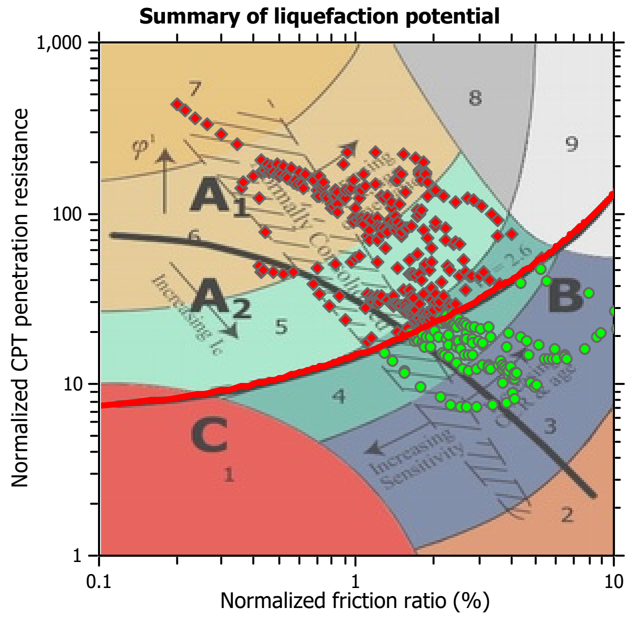
CPT file : CPT-08

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	2.65 m	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	2.65 m	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	5.50	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.10	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		







Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

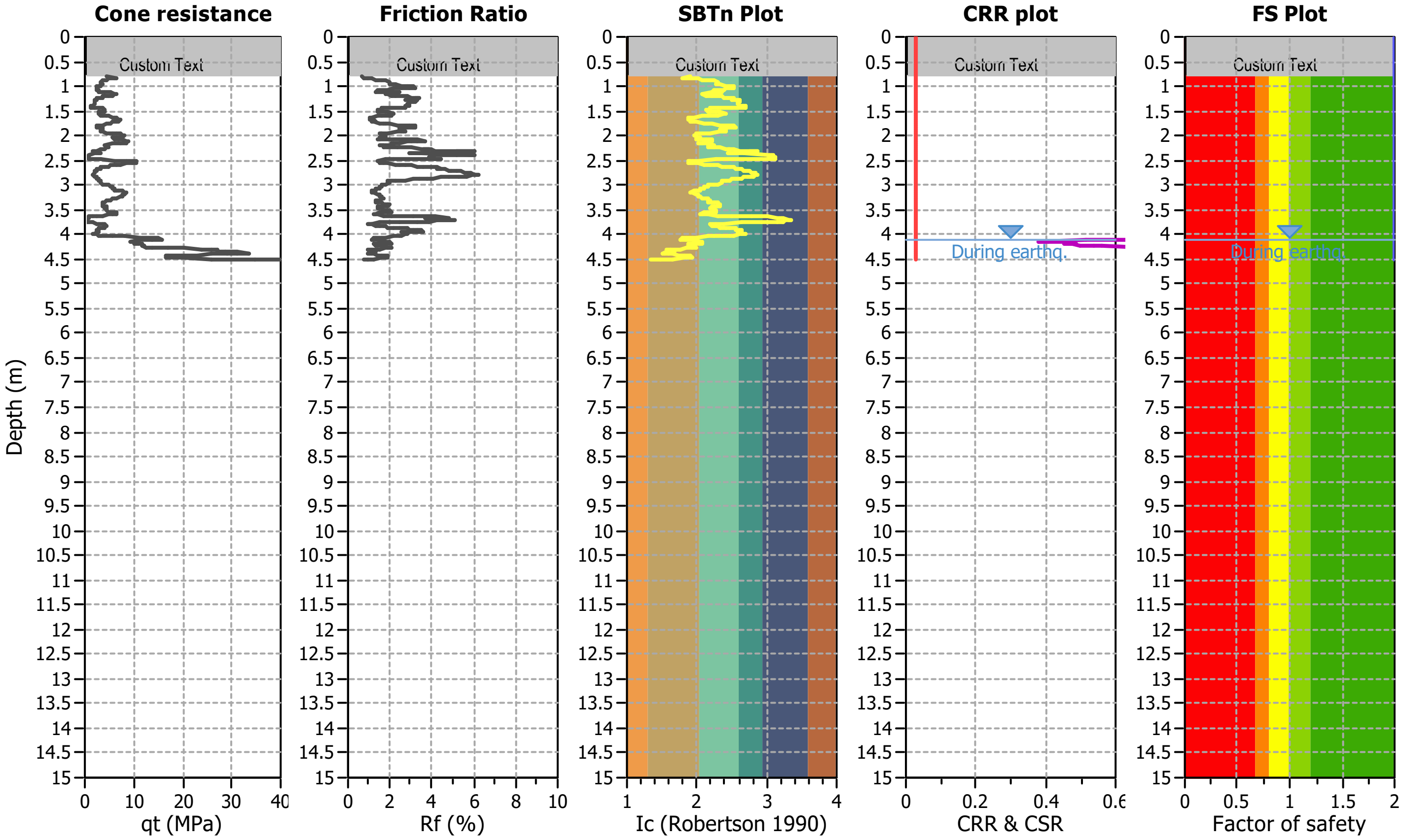
LIQUEFACTION ANALYSIS REPORT

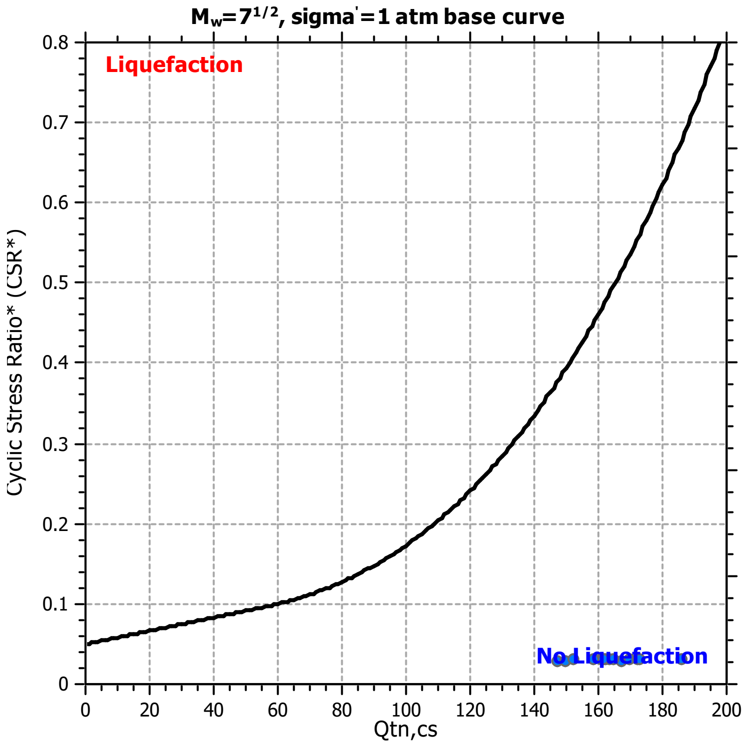
Project title : PS212776 Macquarie Point **Location : Hobart, Tasmania**

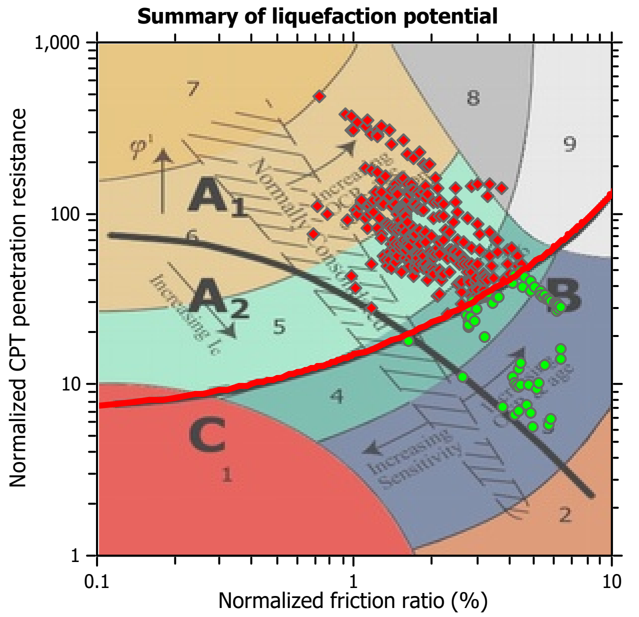
CPT file : CPT-11

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	4.12 m	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	4.12 m	Fill height:	N/A		
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth applied:	No
Earthquake magnitude M_w:	5.50	Ic cut-off value:	2.60	Trans. detect. applied:	No	Limit depth:	N/A
Peak ground acceleration:	0.10	Unit weight calculation:	Based on SBT	K_σ applied:	Yes	MSF method:	Method based







Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

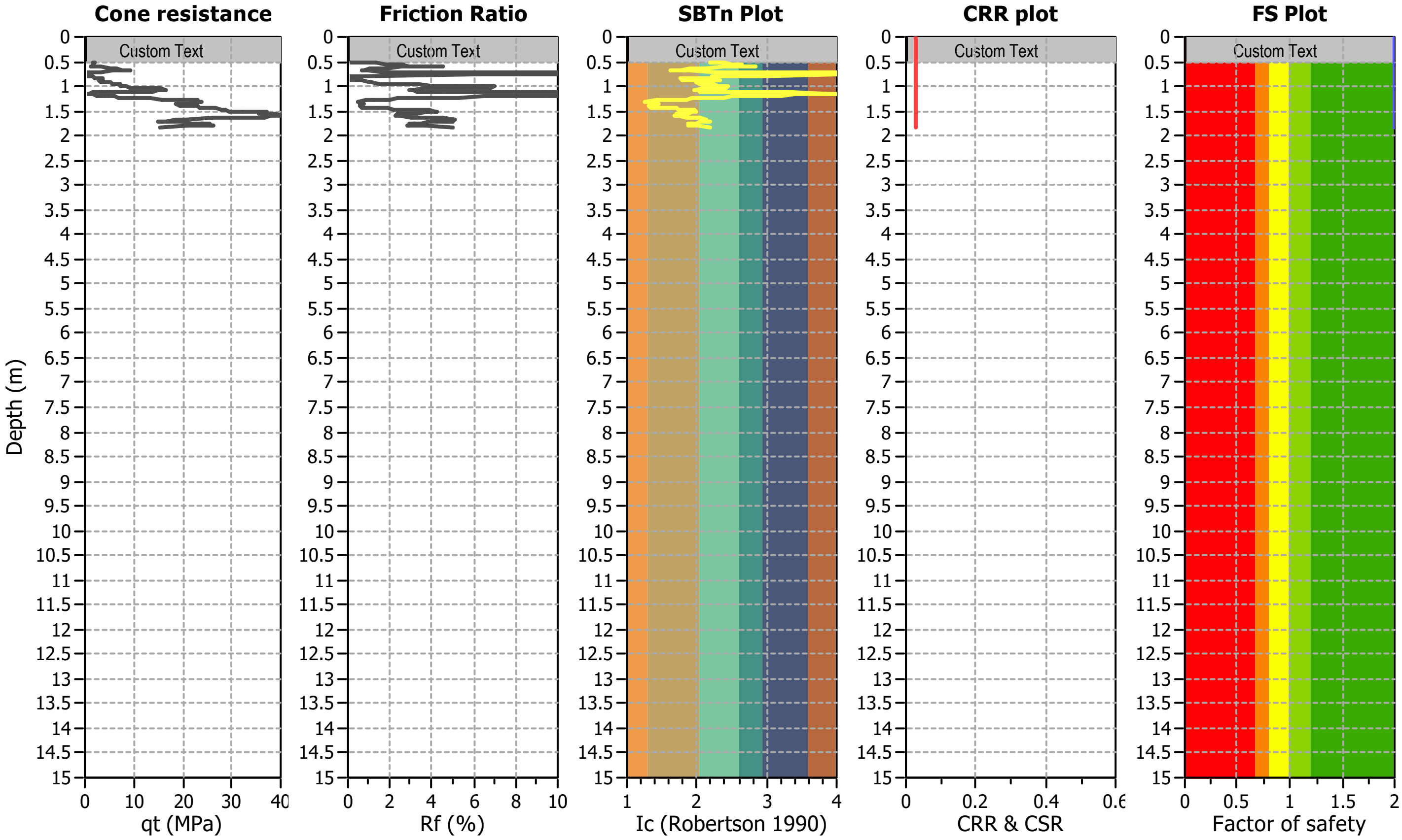
LIQUEFACTION ANALYSIS REPORT

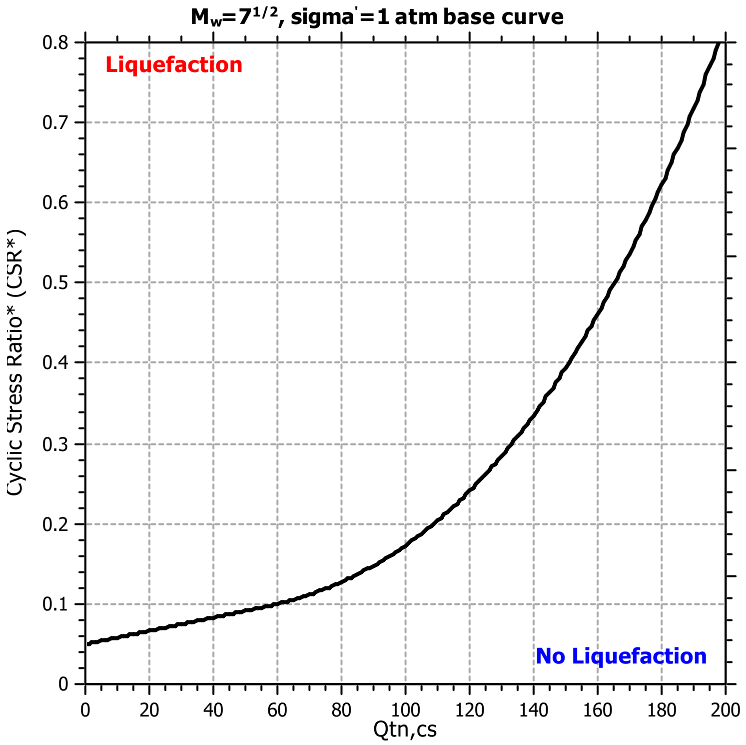
Project title : PS212776 Macquarie Point **Location : Hobart, Tasmania**

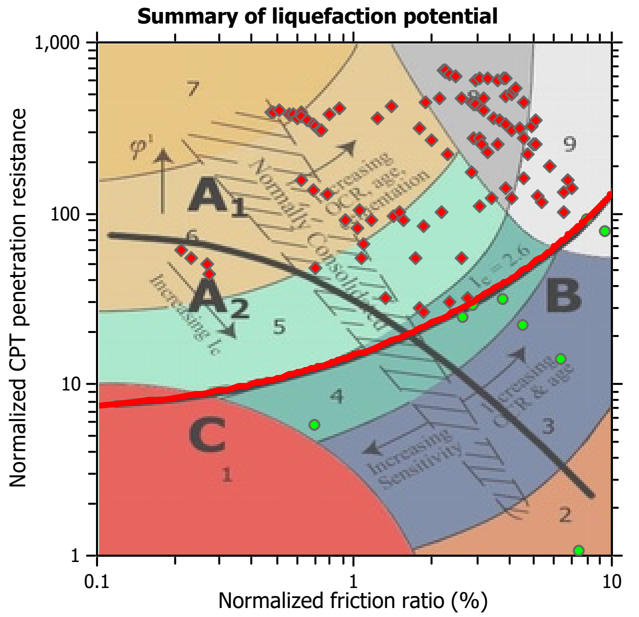
CPT file : CPT-12

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	25.00 m	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	25.00 m	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	5.50	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.10	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		







Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

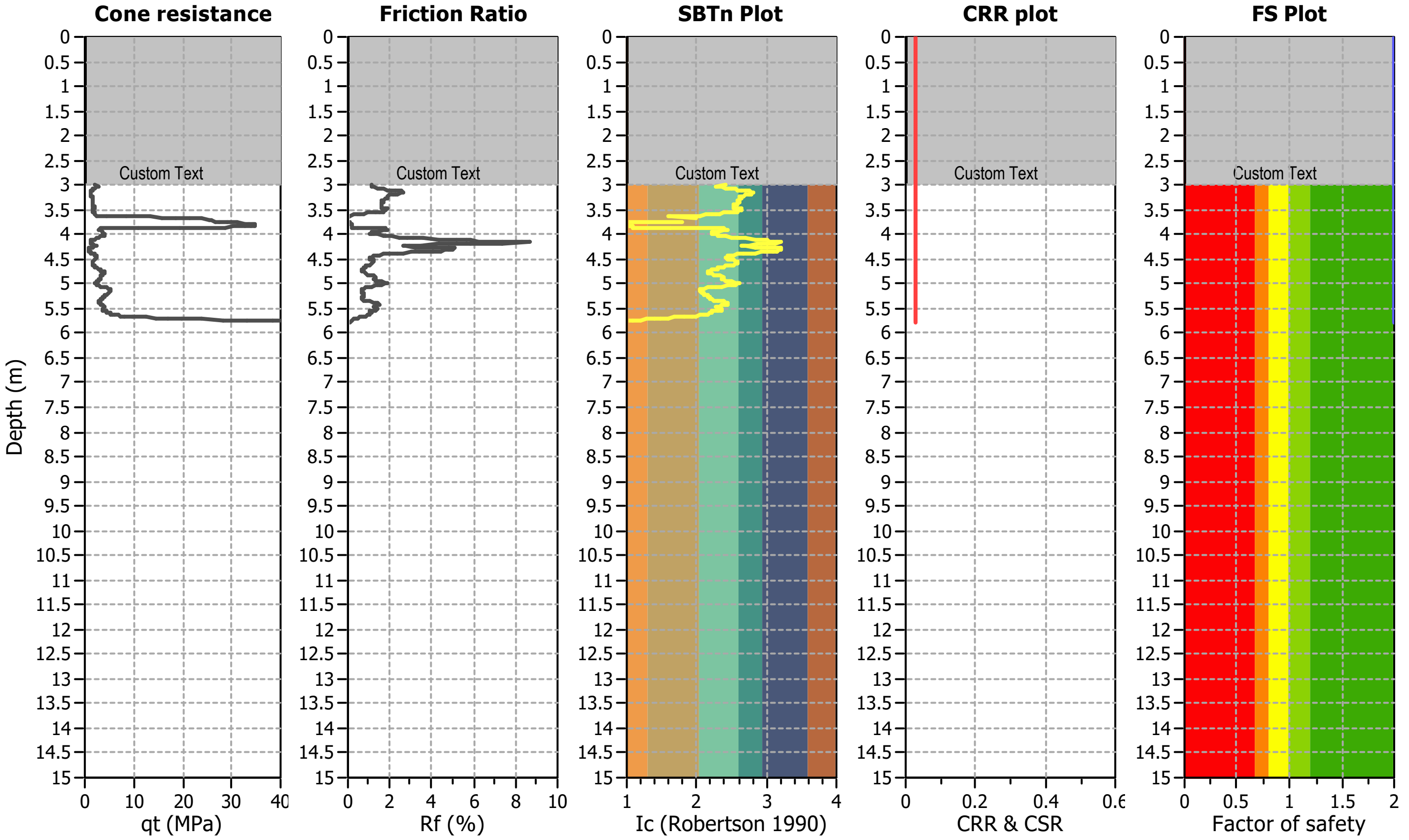
LIQUEFACTION ANALYSIS REPORT

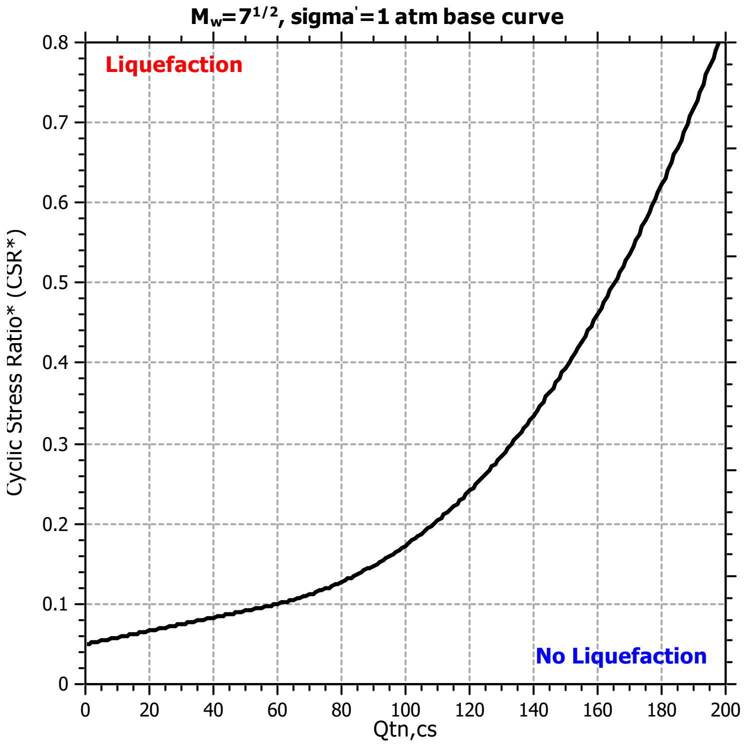
Project title : PS212776 Macquarie Point **Location : Hobart, Tasmania**

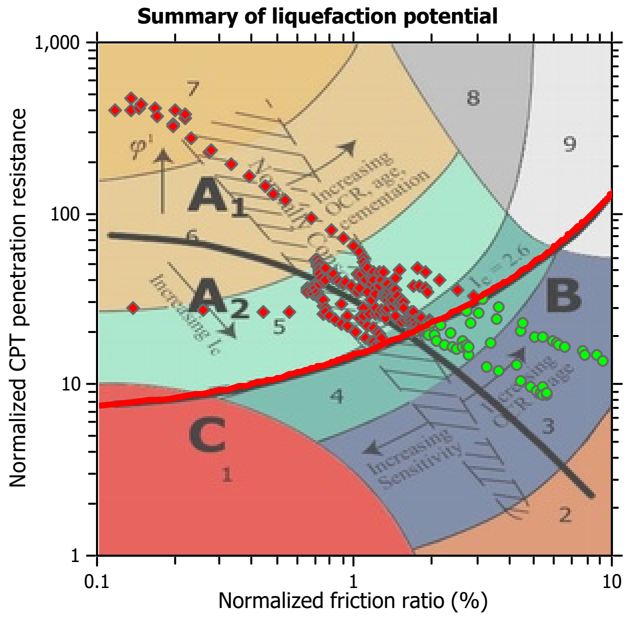
CPT file : CPT-12a

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	25.00 m	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	25.00 m	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	5.50	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.10	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		







Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

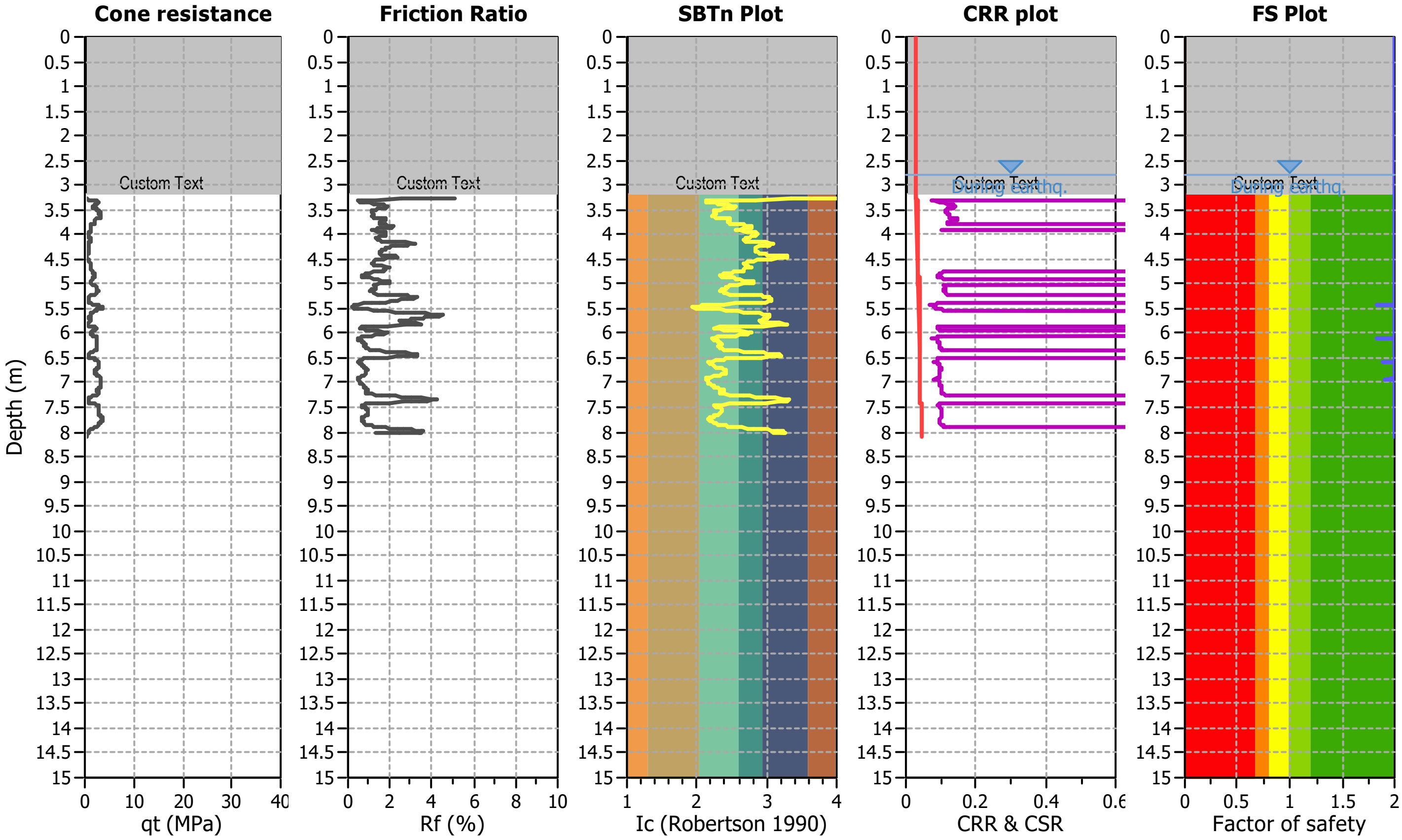
LIQUEFACTION ANALYSIS REPORT

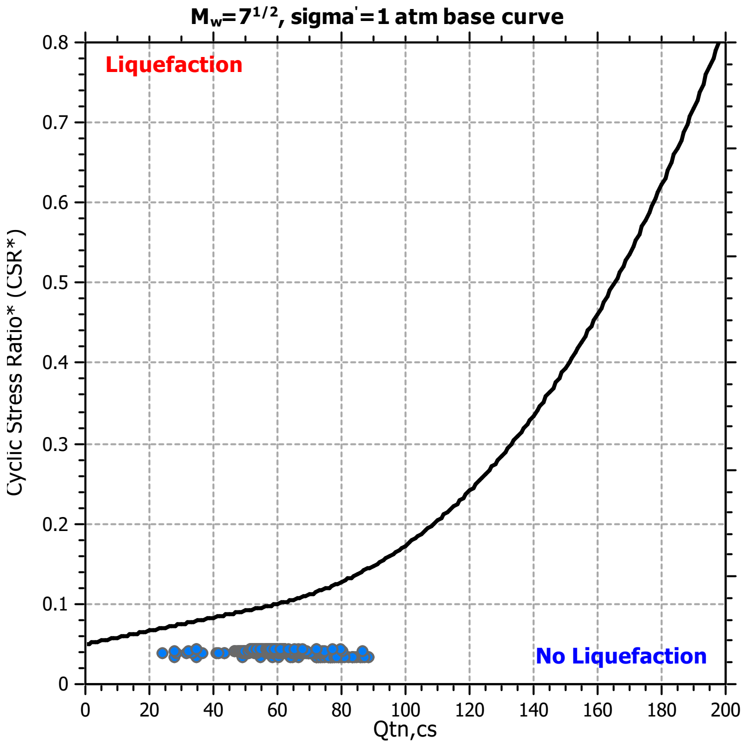
Project title : PS212776 Macquarie Point **Location : Hobart, Tasmania**

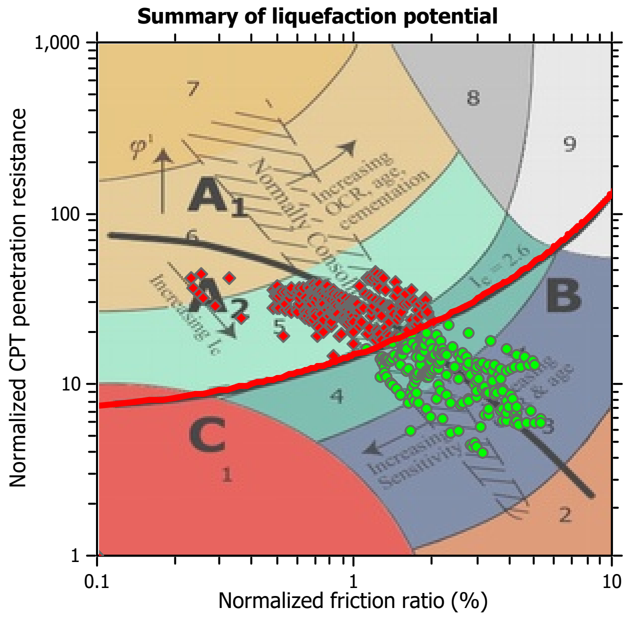
CPT file : CPT-13

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	2.80 m	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	2.80 m	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	5.50	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.10	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		







Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

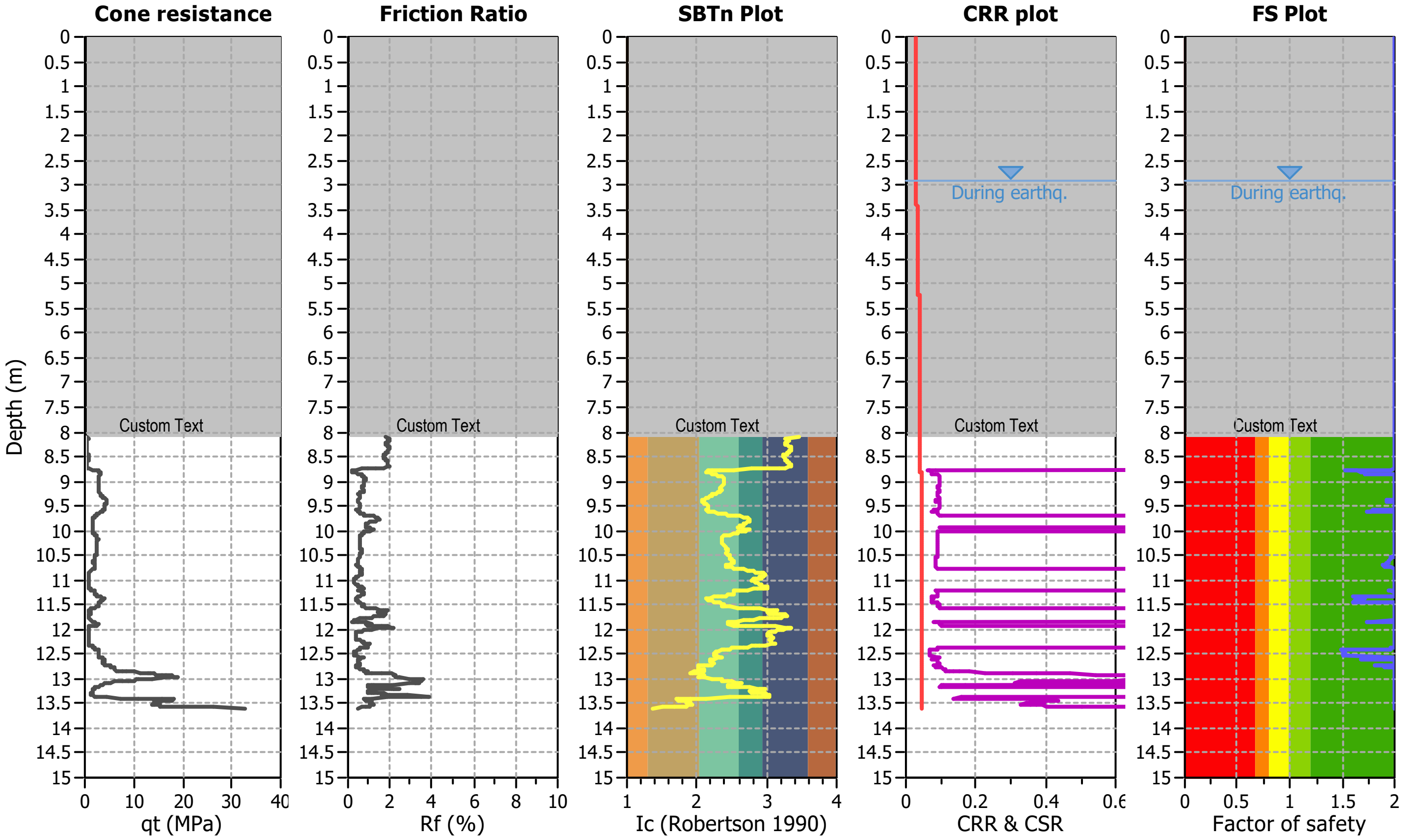
LIQUEFACTION ANALYSIS REPORT

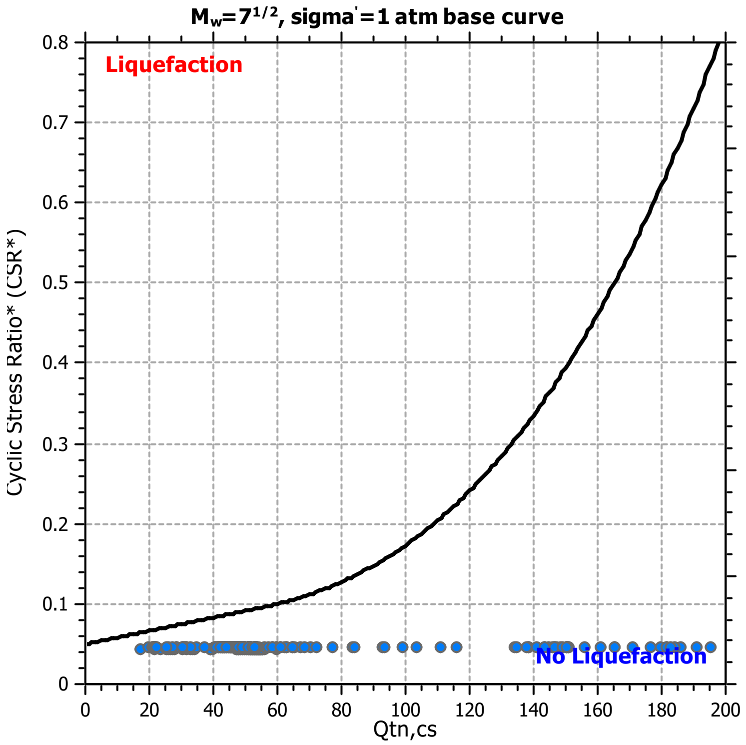
Project title : PS212776 Macquarie Point **Location : Hobart, Tasmania**

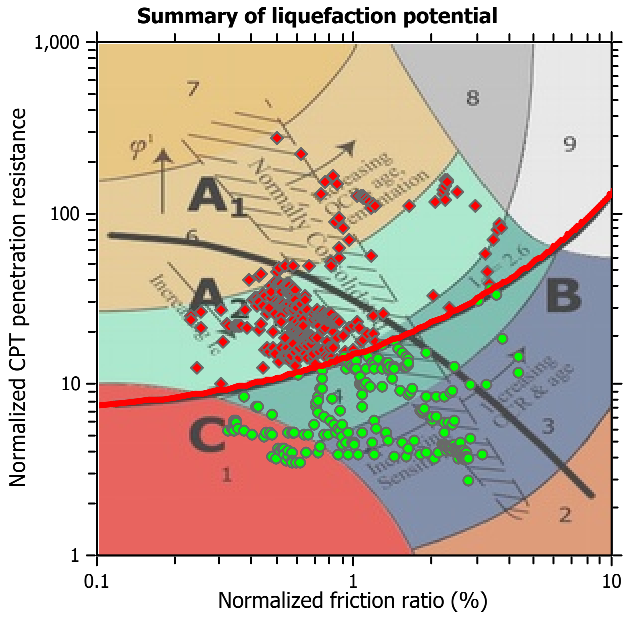
CPT file : CPT-13a

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	2.90 m	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	2.90 m	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	5.50	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.10	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		







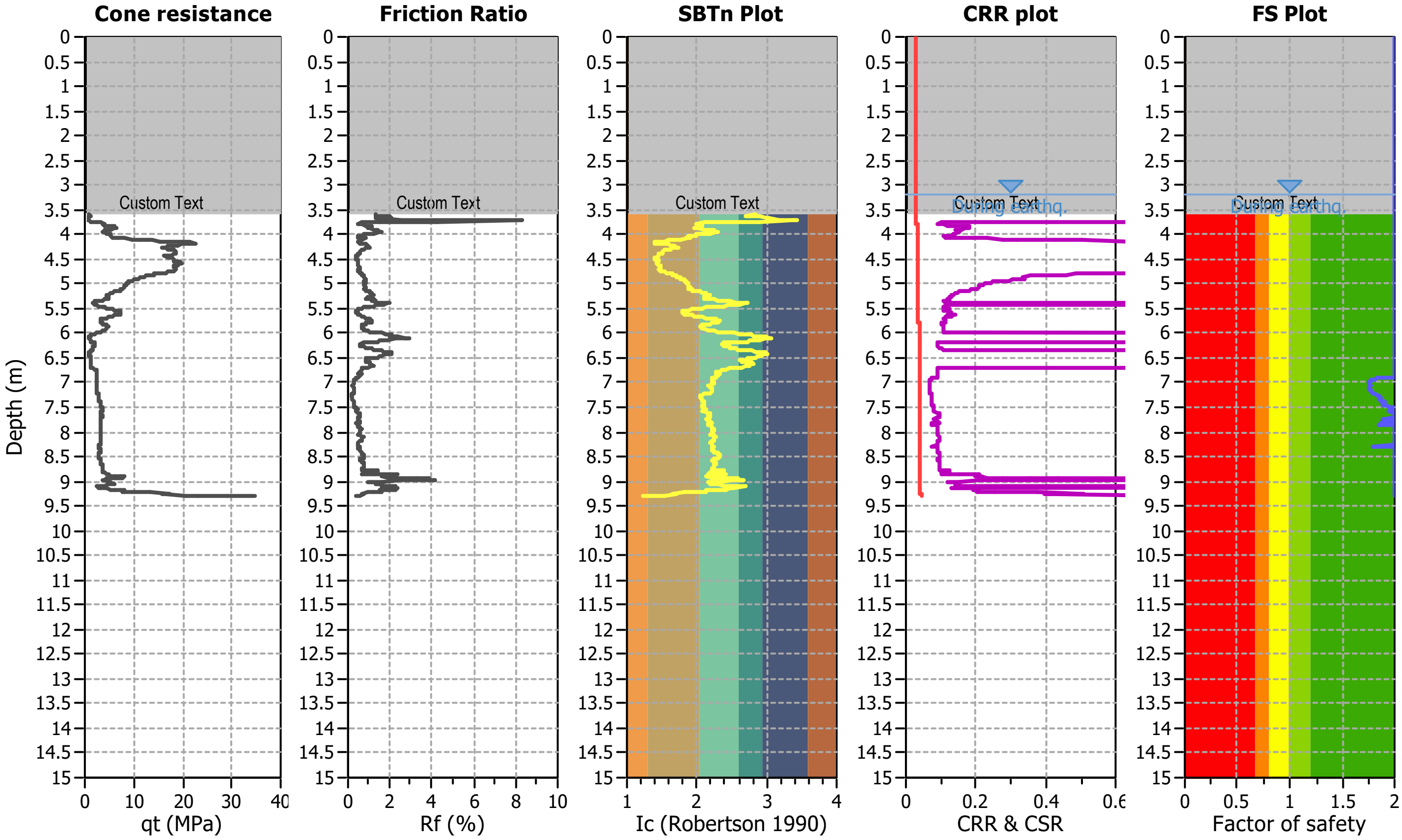
Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

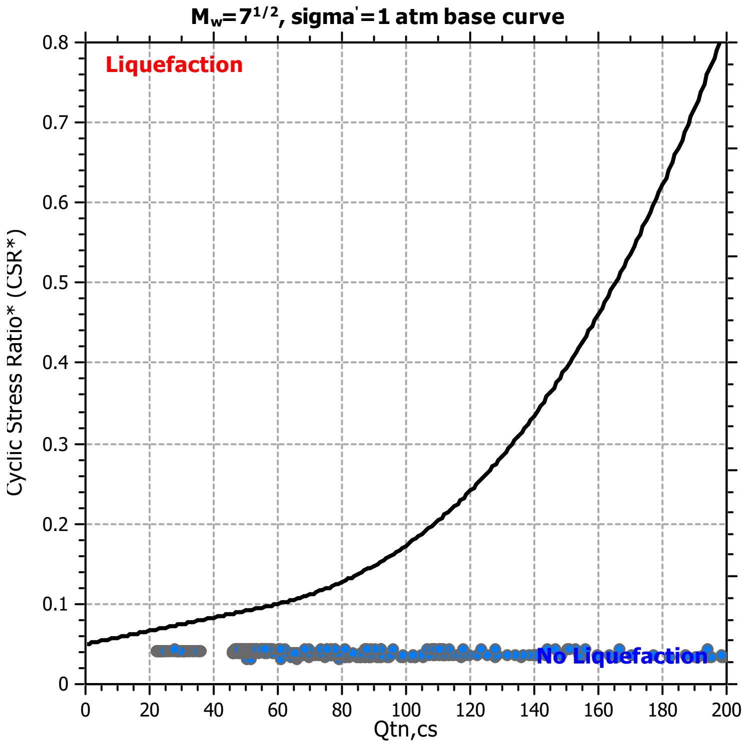
Project title : PS212776 Macquarie Point **Location : Hobart, Tasmania**

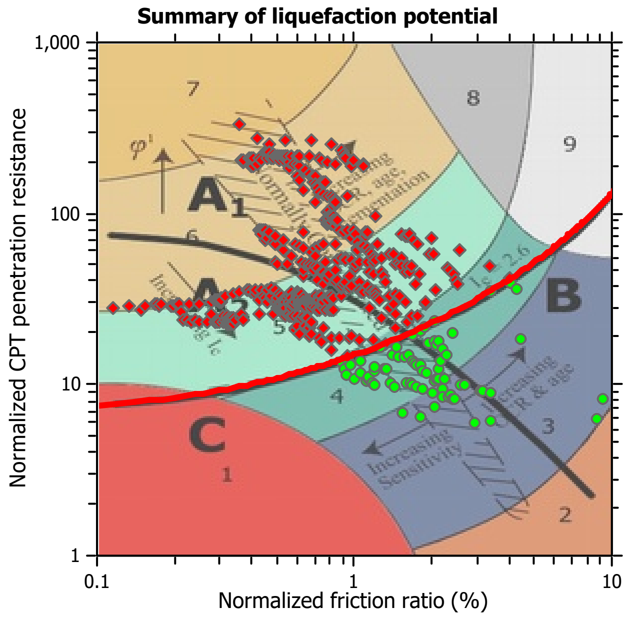
CPT file : CPT-14

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	3.20 m	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	3.20 m	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	5.50	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.10	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		







Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

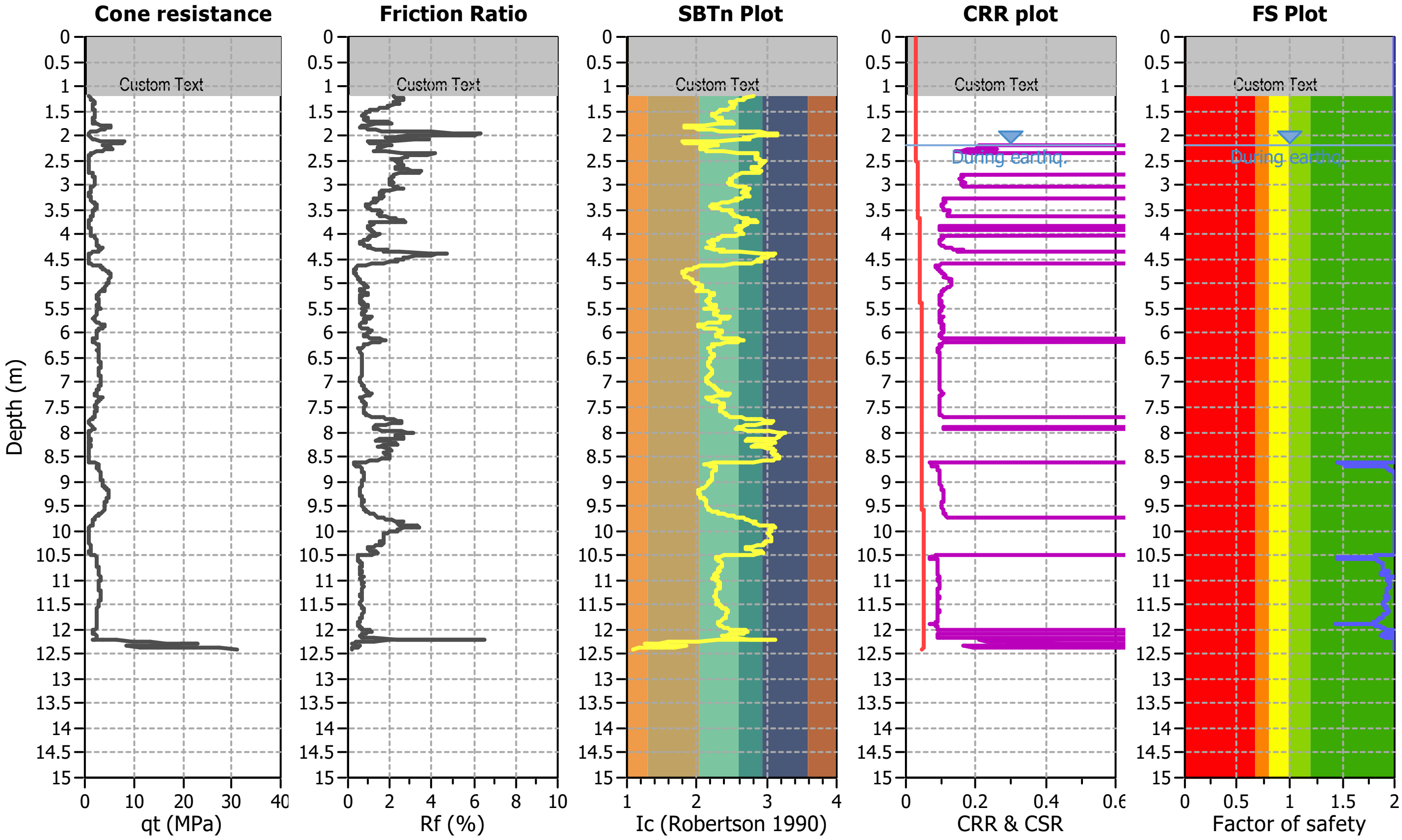
LIQUEFACTION ANALYSIS REPORT

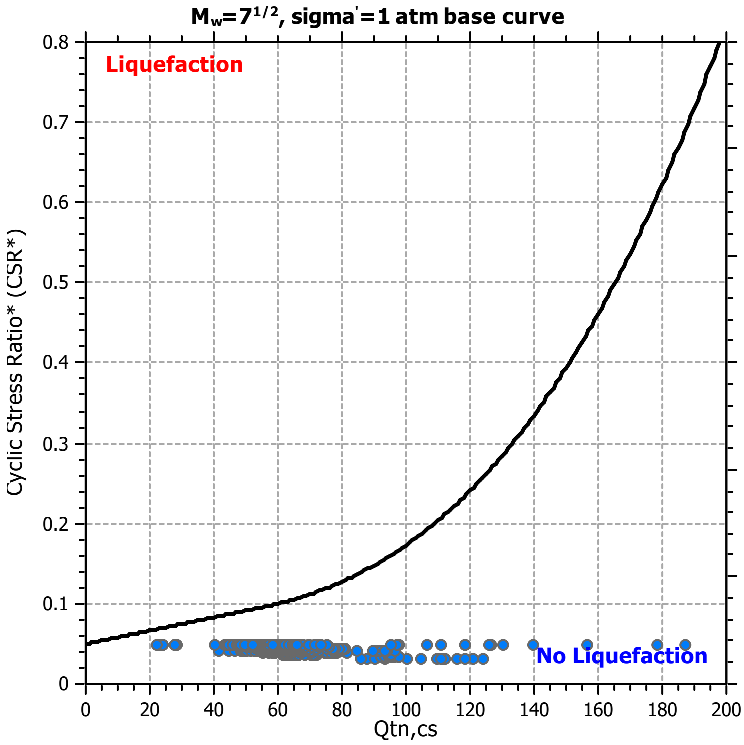
Project title : PS212776 Macquarie Point **Location : Hobart, Tasmania**

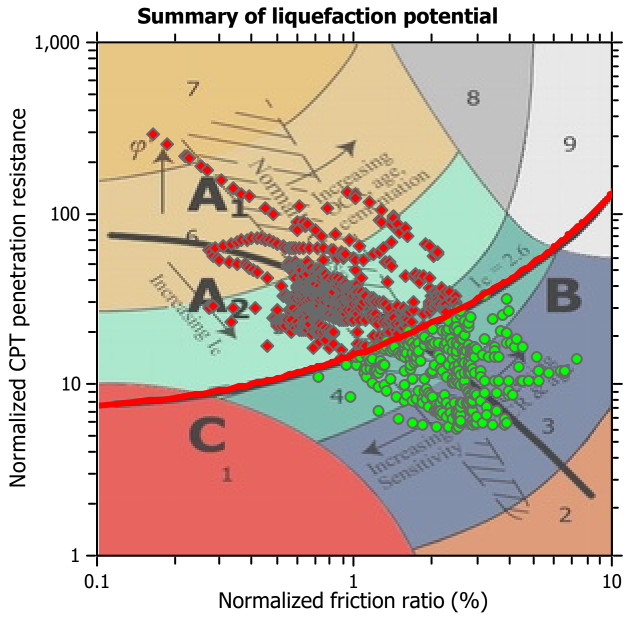
CPT file : CPT-16

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	2.20 m	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	2.20 m	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	5.50	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.10	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		







Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

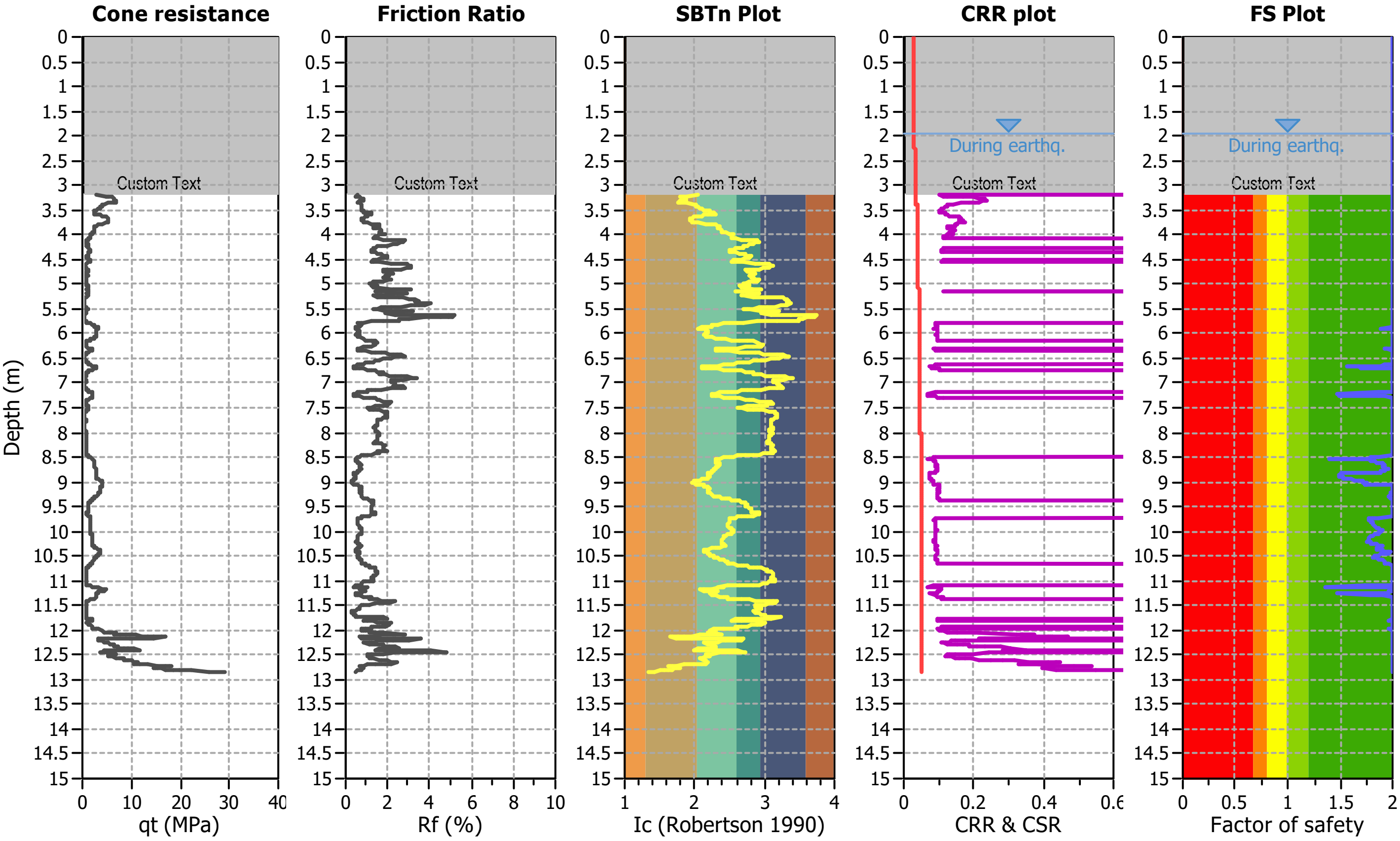
LIQUEFACTION ANALYSIS REPORT

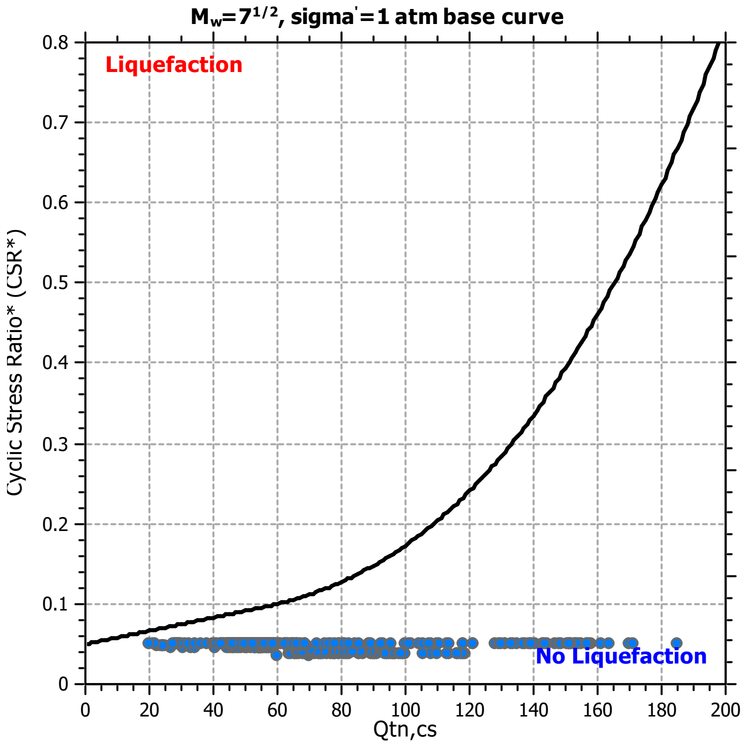
Project title : PS212776 Macquarie Point **Location : Hobart, Tasmania**

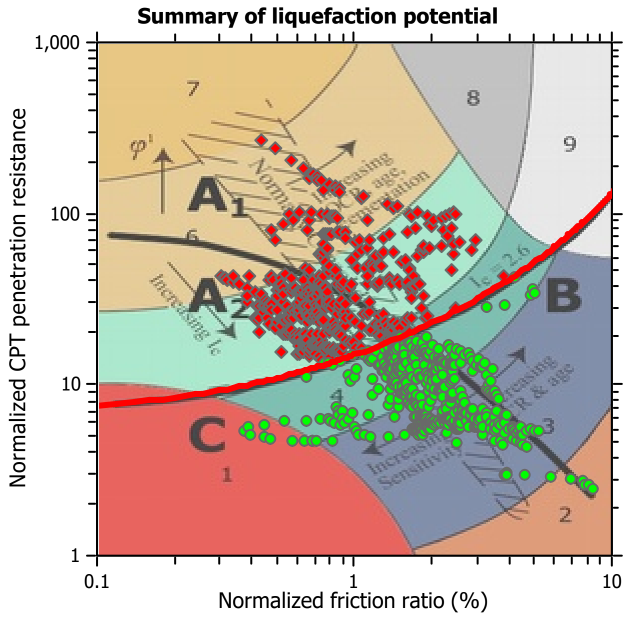
CPT file : CPT-17

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	1.95 m	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	1.95 m	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	5.50	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.10	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		







Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

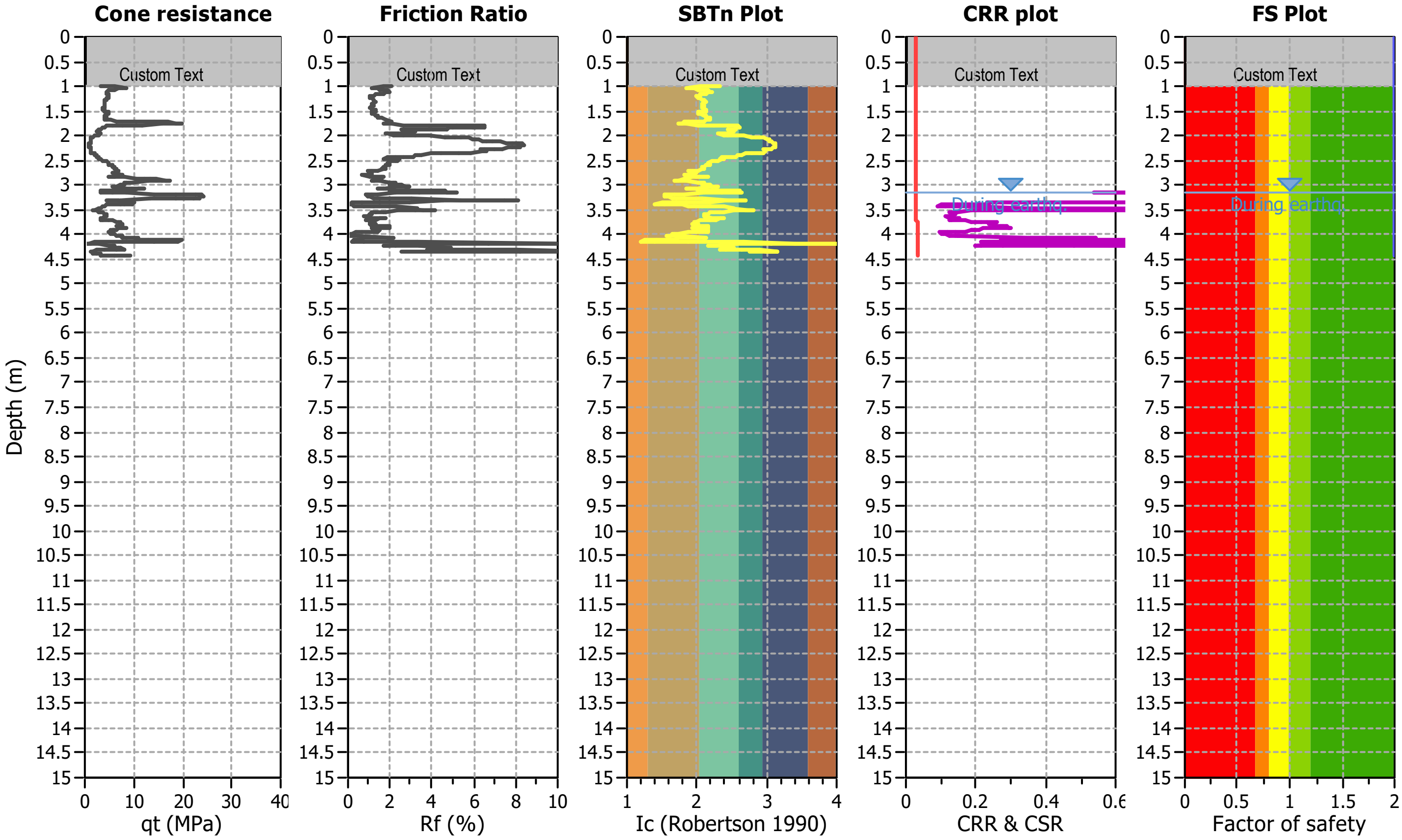
LIQUEFACTION ANALYSIS REPORT

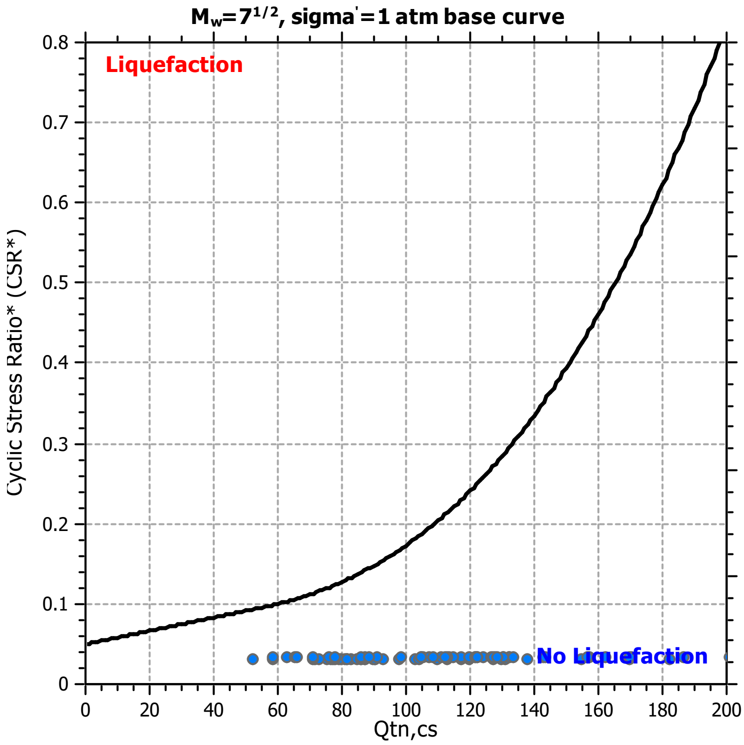
Project title : PS212776 Macquarie Point **Location : Hobart, Tasmania**

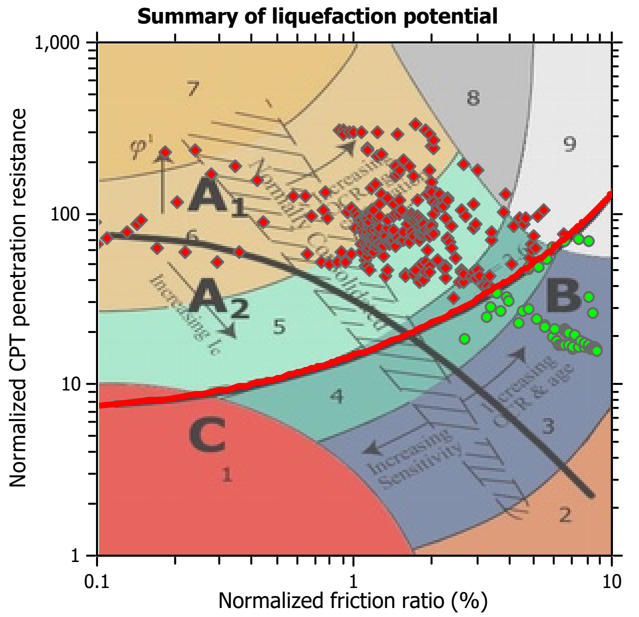
CPT file : CPT-19

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	3.16 m	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	3.16 m	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	5.50	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.10	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		







Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading

Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry

Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening

Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

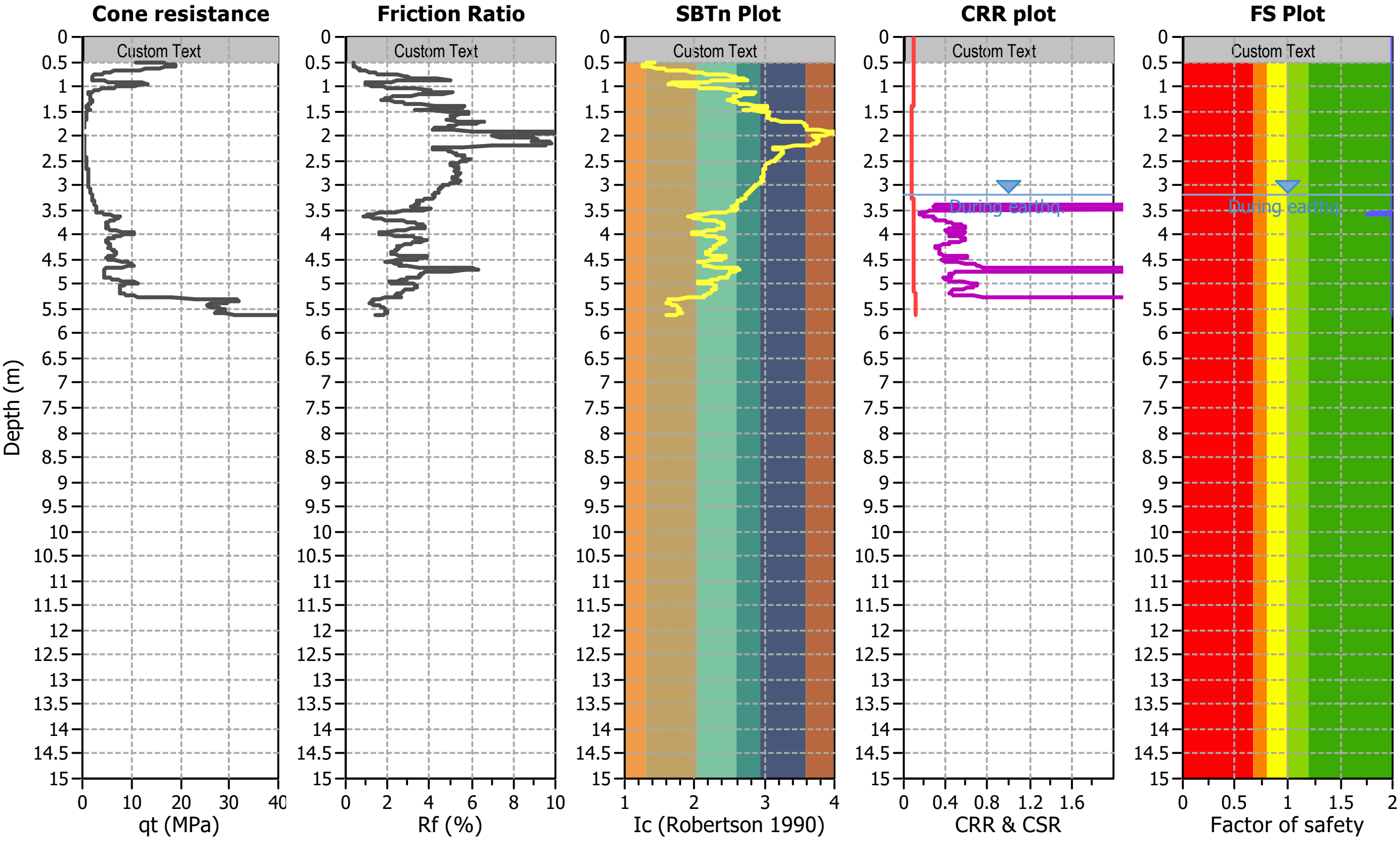
LIQUEFACTION ANALYSIS REPORT

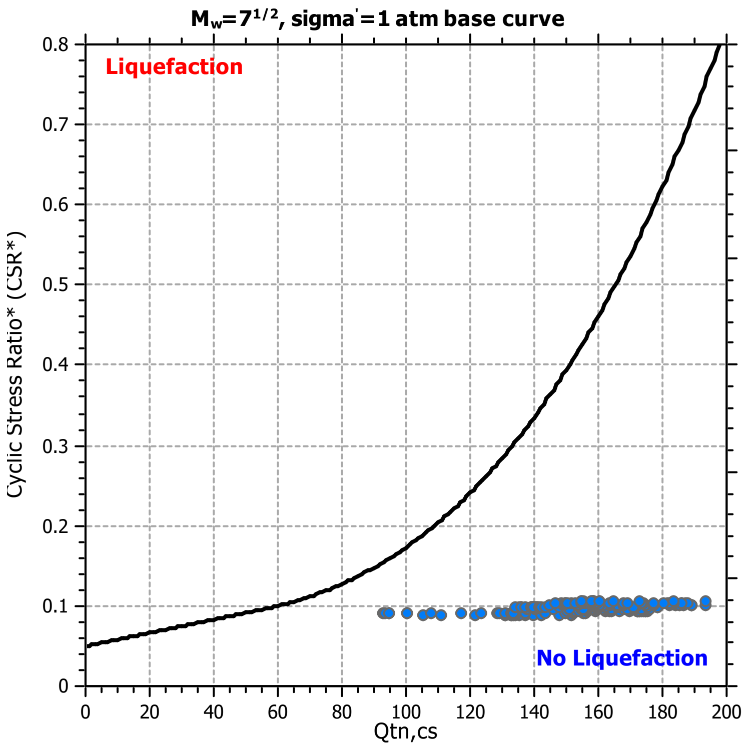
Project title : PS212776 Macquarie Point **Location : Hobart, Tasmania**

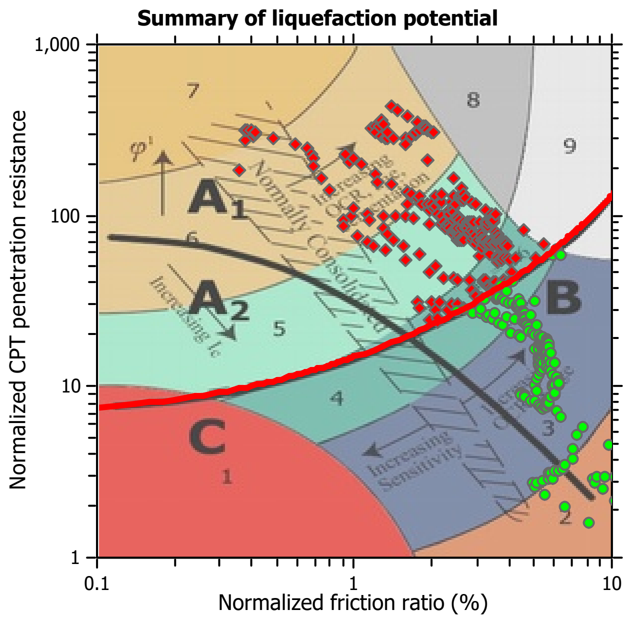
CPT file : CPT-03

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	3.20 m	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	3.20 m	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.50	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.19	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		







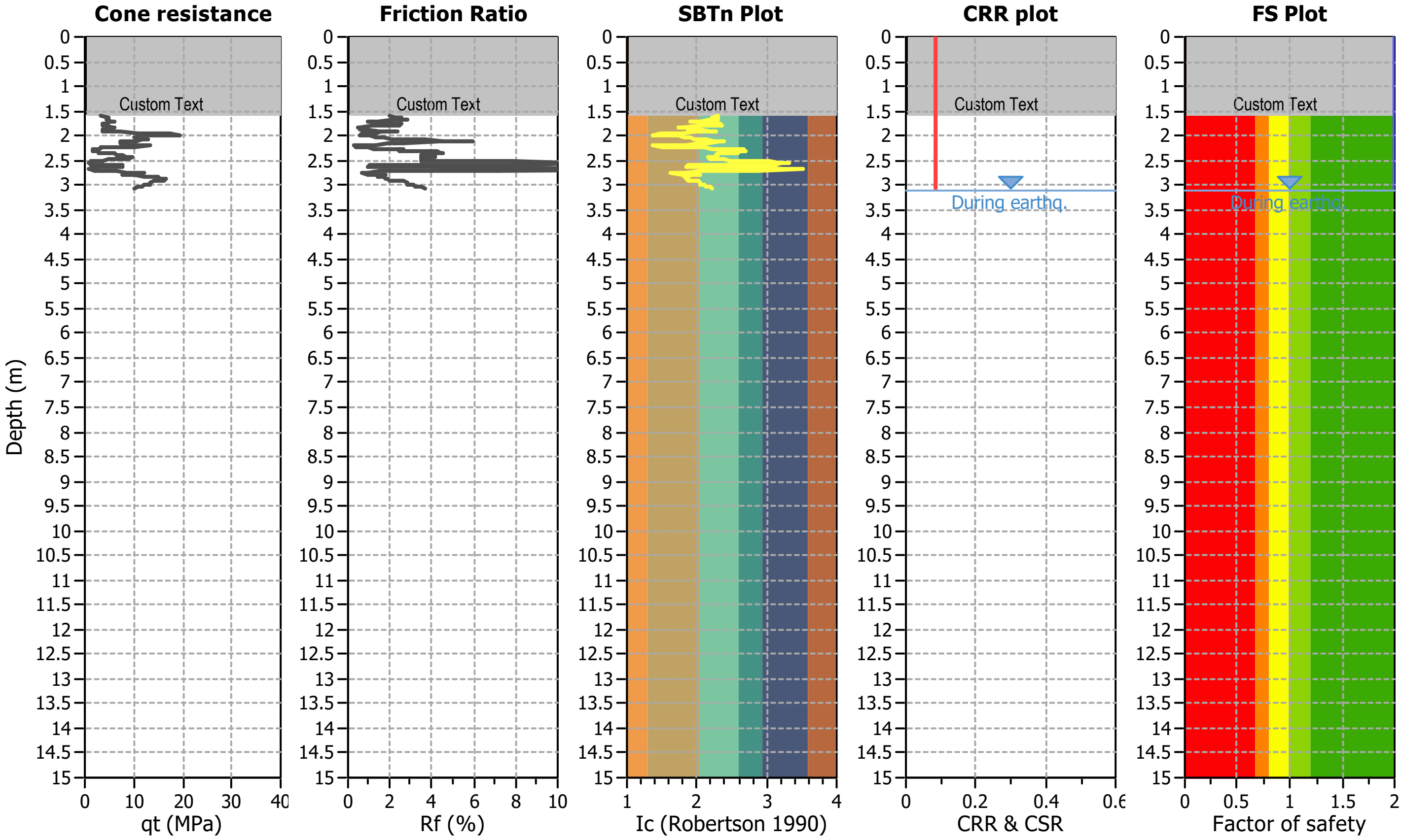
Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

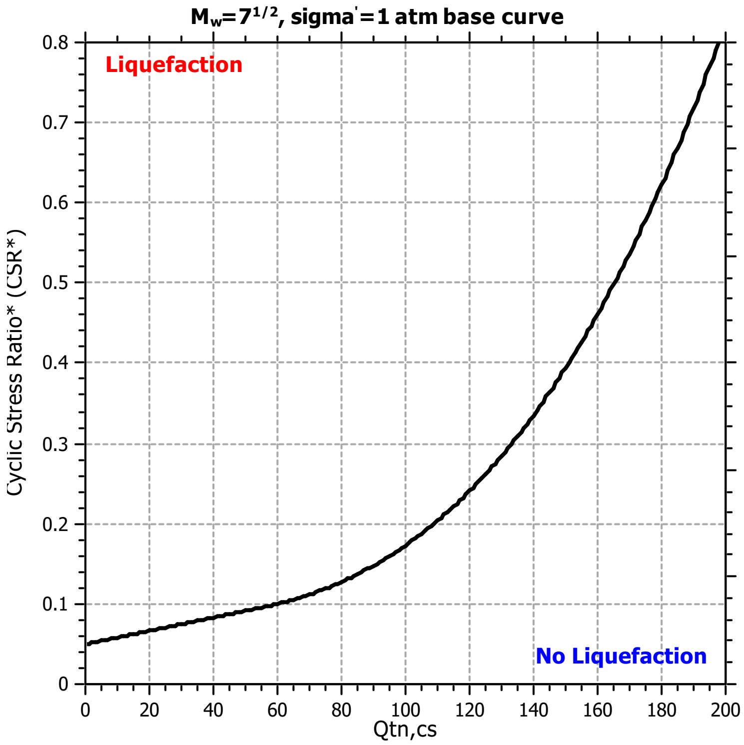
Project title : PS212776 Macquarie Point **Location : Hobart, Tasmania**

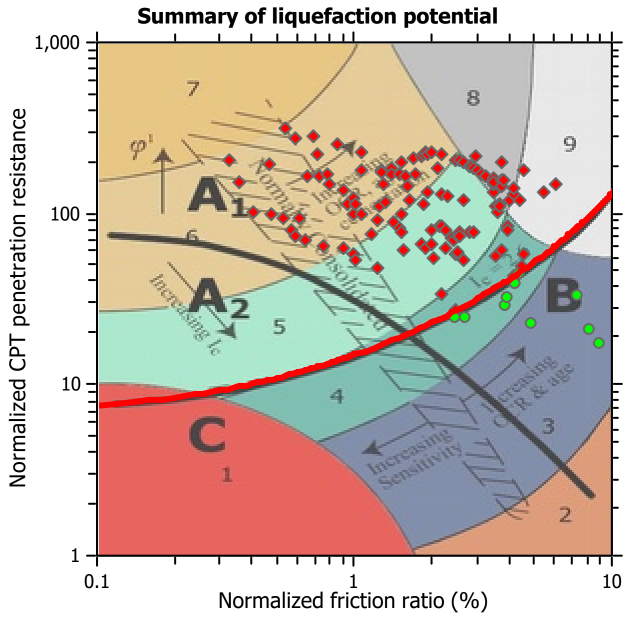
CPT file : CPT-04

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	3.10 m	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	3.10 m	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.50	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.19	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		







Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

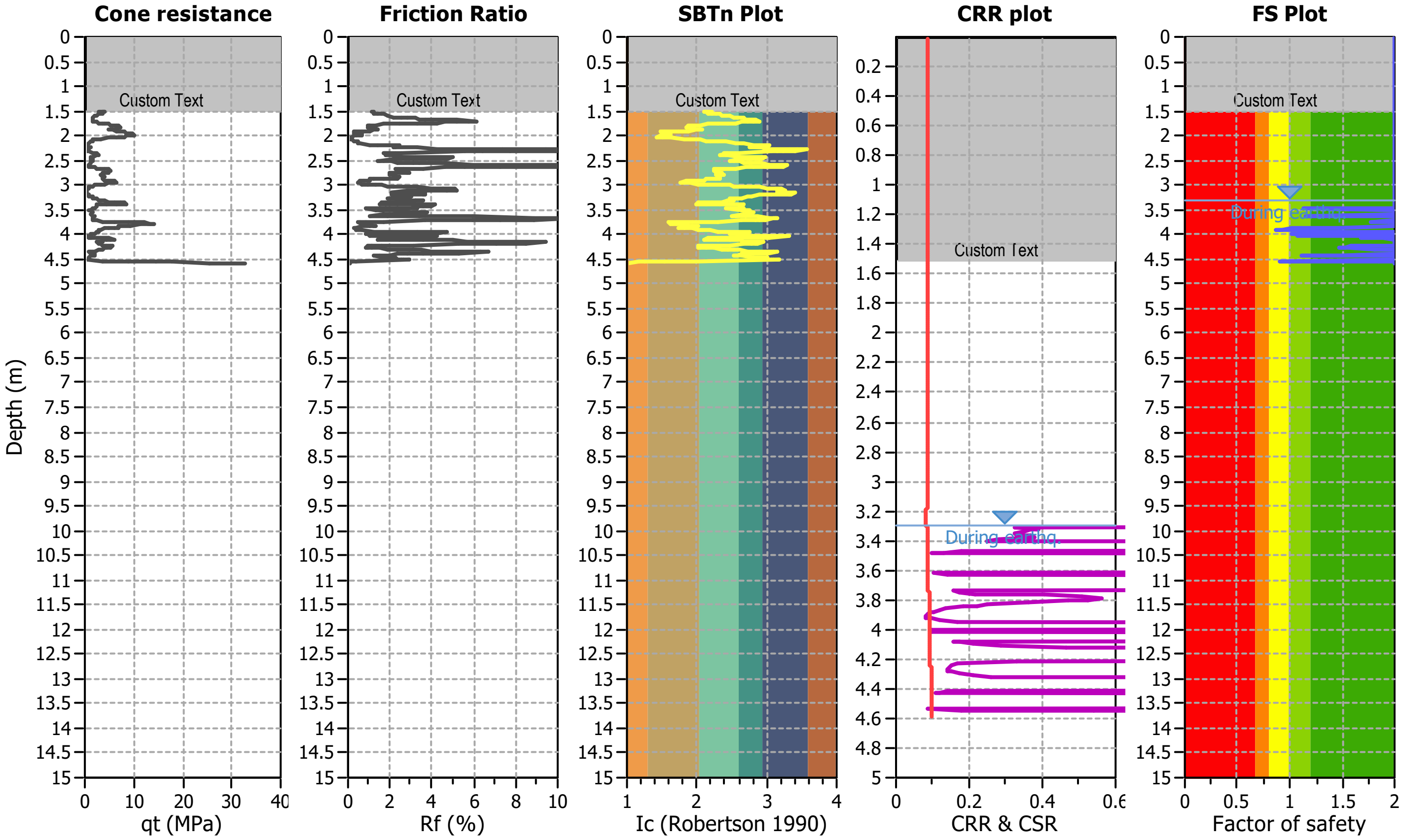
LIQUEFACTION ANALYSIS REPORT

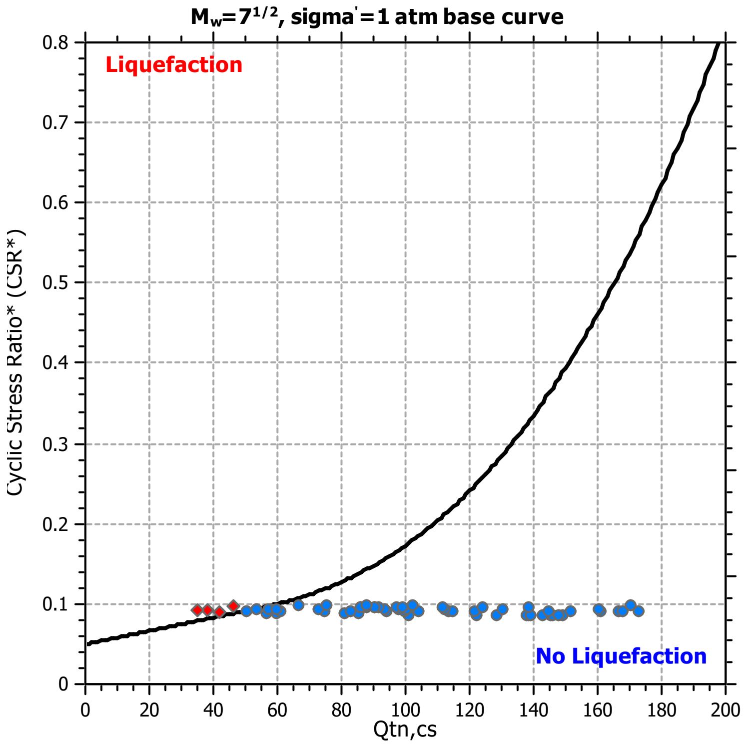
Project title : PS212776 Macquarie Point **Location : Hobart, Tasmania**

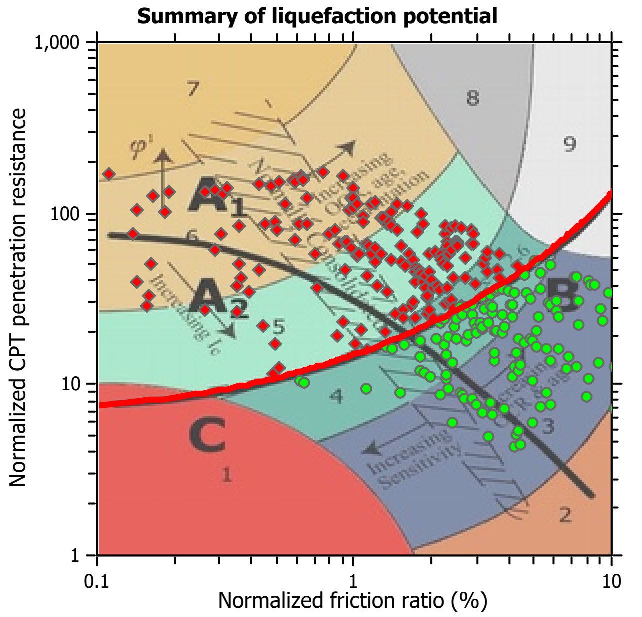
CPT file : CPT-04a

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	3.30 m	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	3.30 m	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.50	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.19	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		







Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

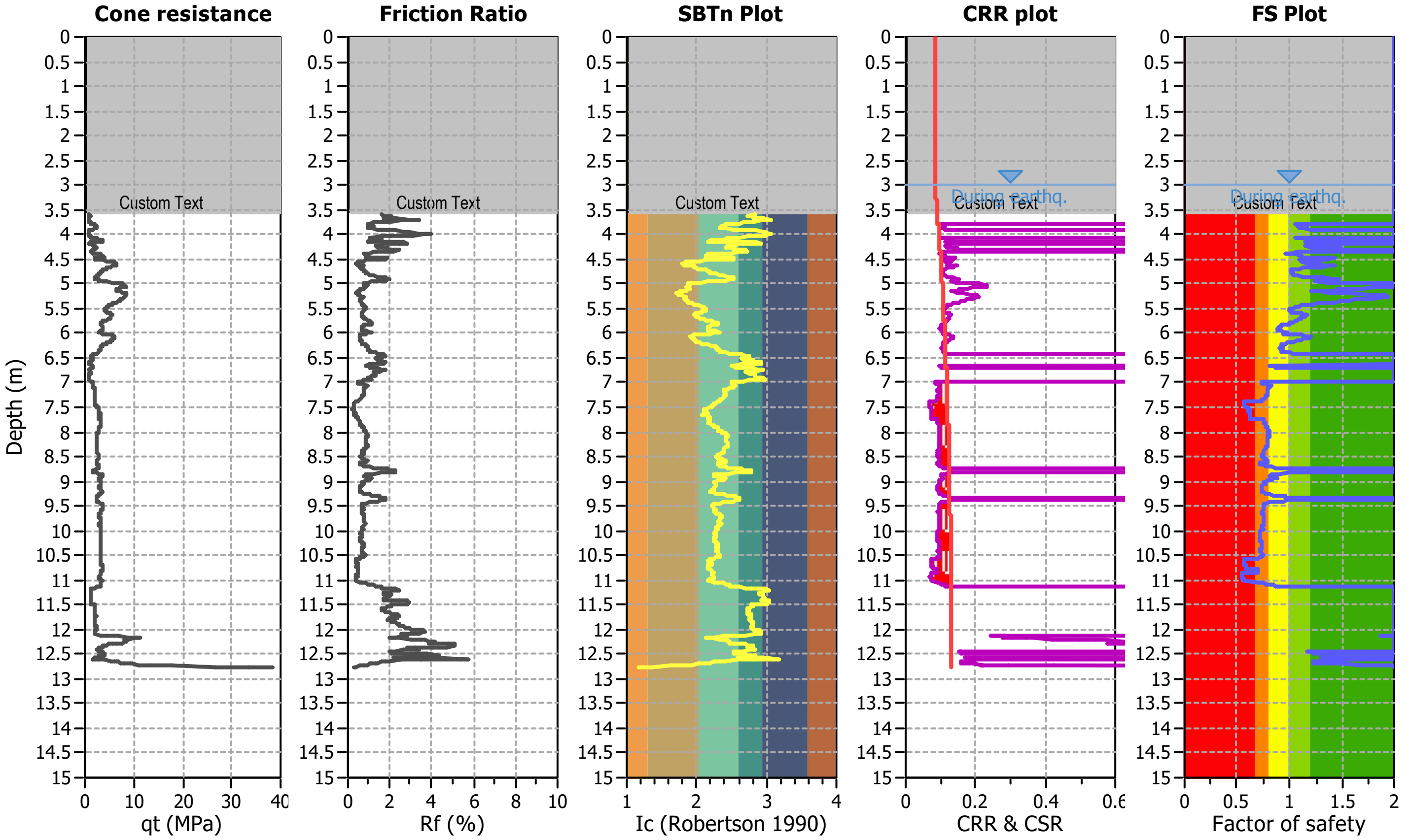
LIQUEFACTION ANALYSIS REPORT

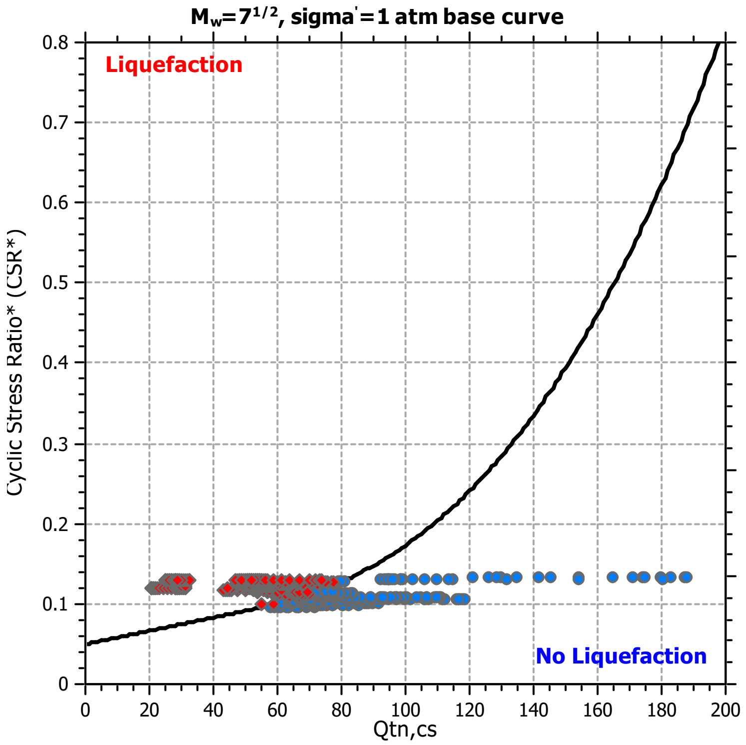
Project title : PS212776 Macquarie Point **Location : Hobart, Tasmania**

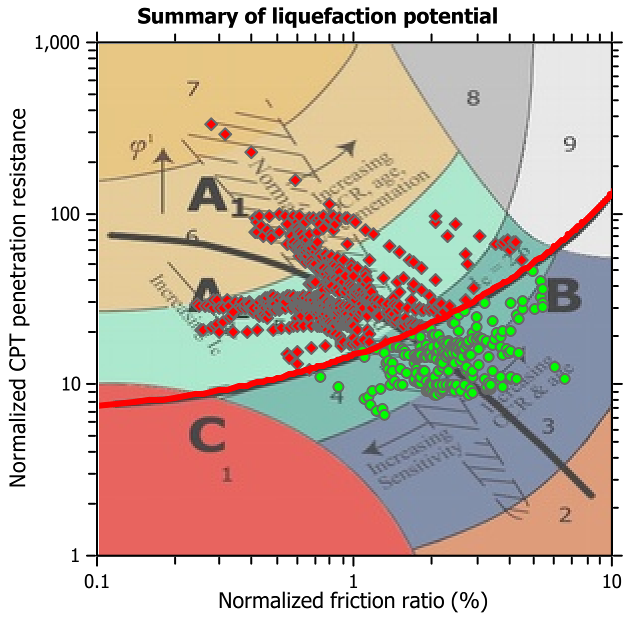
CPT file : CPT-05

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	3.00 m	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	3.00 m	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.50	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.19	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		







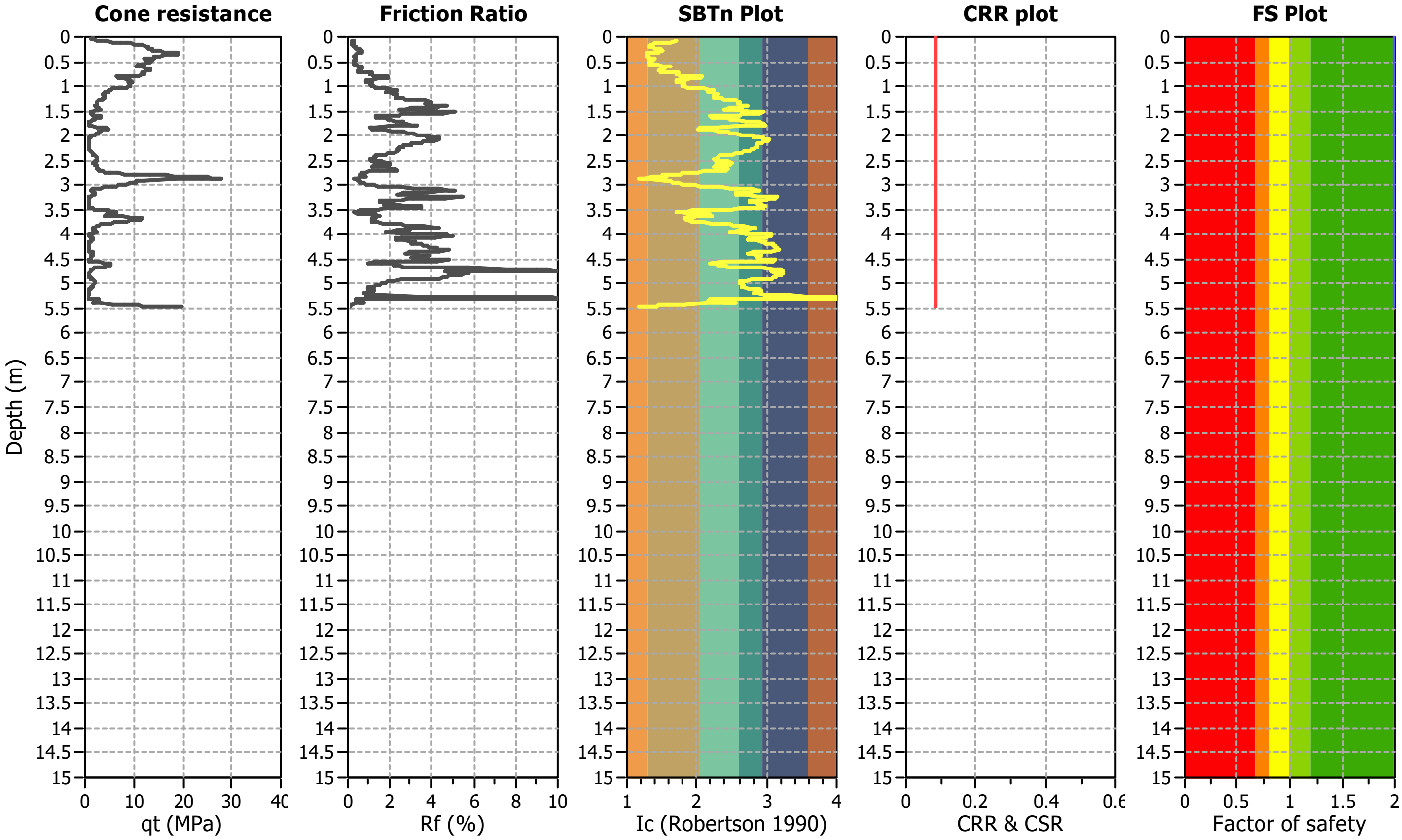
Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

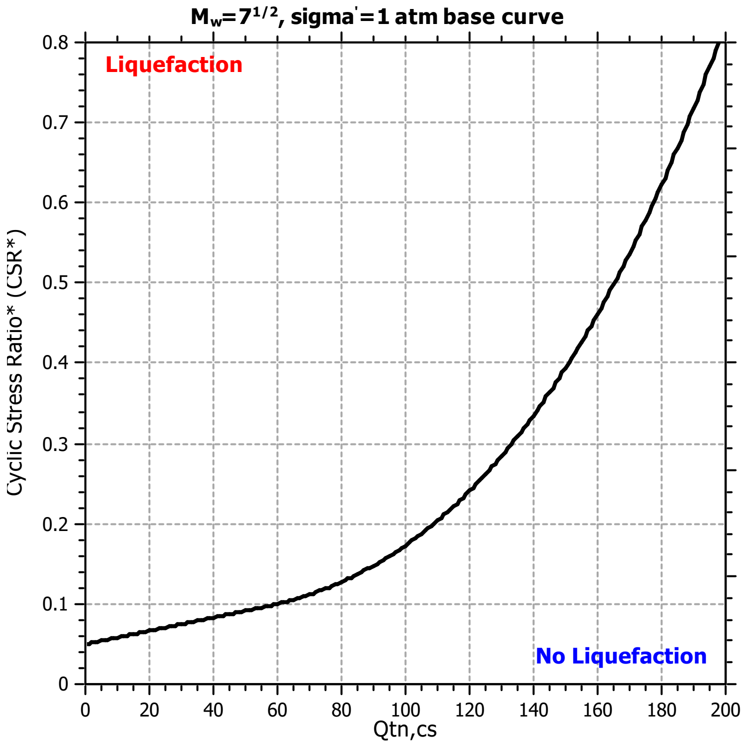
Project title : PS212776 Macquarie Point **Location : Hobart, Tasmania**

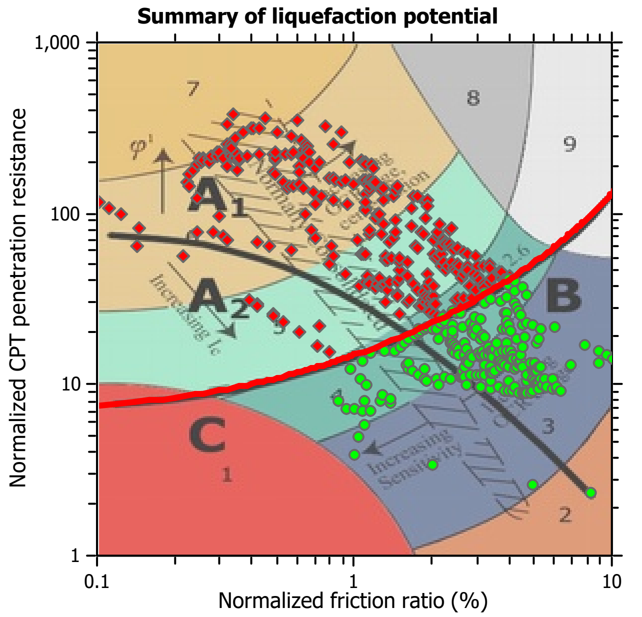
CPT file : CPT-06

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	25.00 m	Use fill:	No	Clay like behavior	
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	25.00 m	Fill height:	N/A	applied:	Sands only
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth applied:	No
Earthquake magnitude M_w:	6.50	Ic cut-off value:	2.60	Trans. detect. applied:	No	Limit depth:	N/A
Peak ground acceleration:	0.19	Unit weight calculation:	Based on SBT	K_σ applied:	Yes	MSF method:	Method based







Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

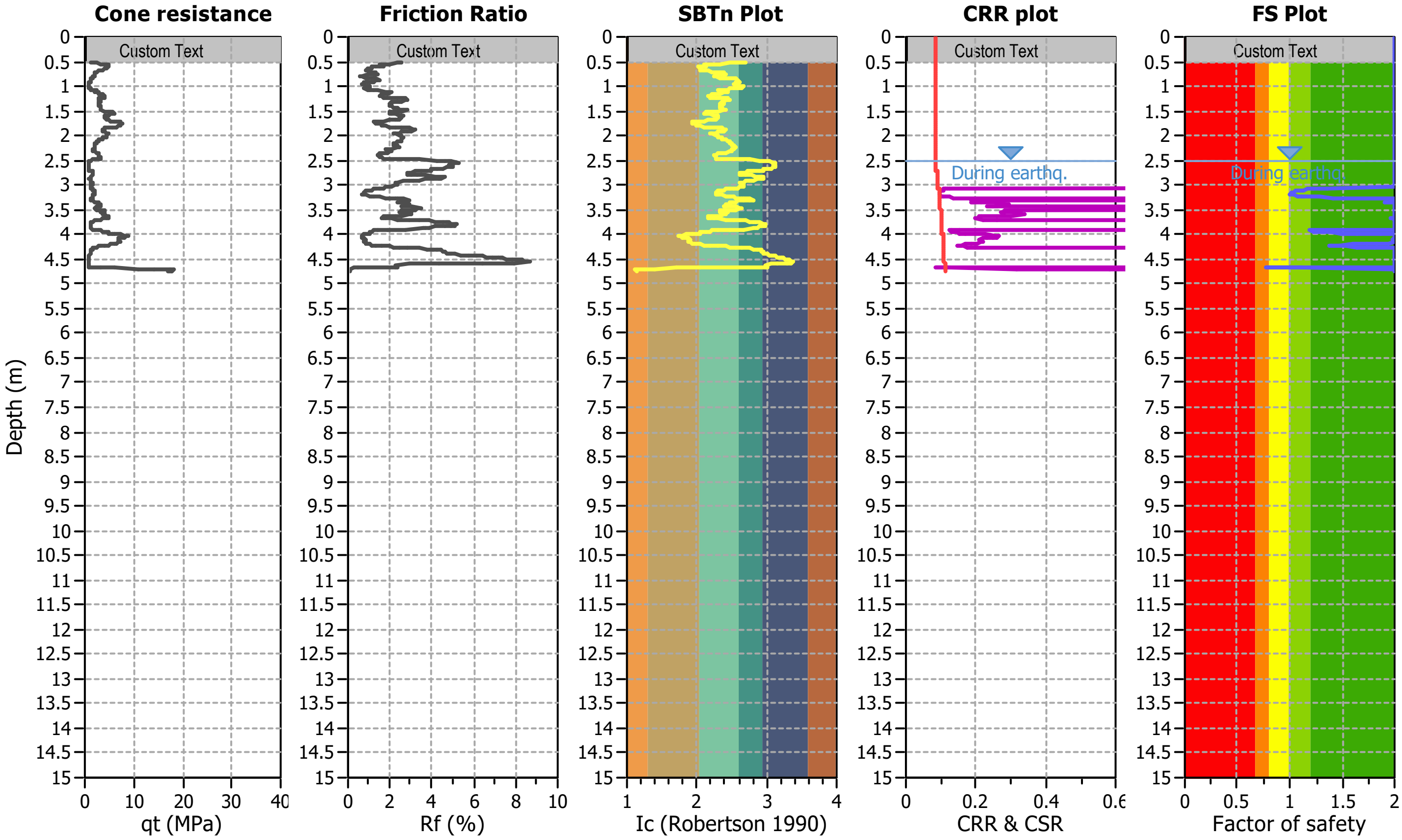
LIQUEFACTION ANALYSIS REPORT

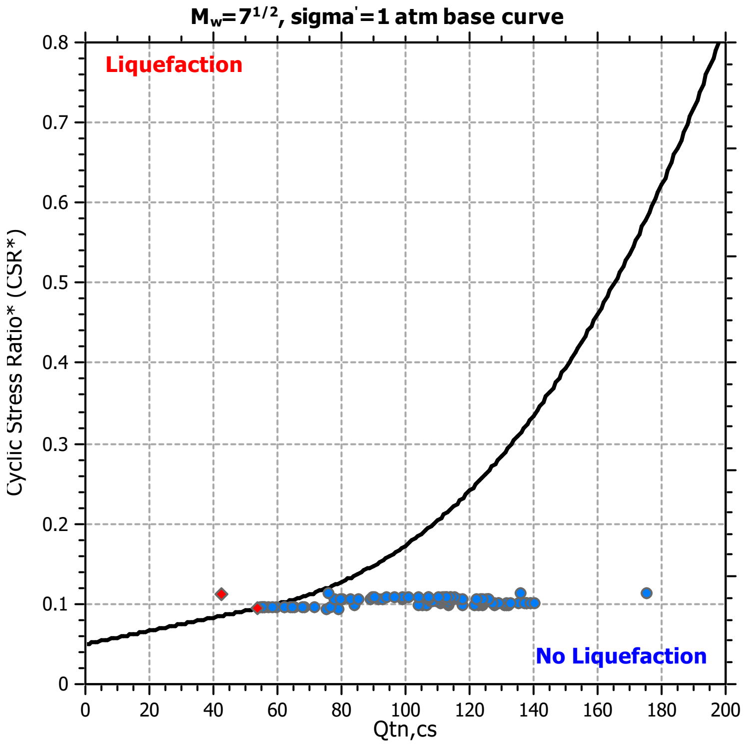
Project title : PS212776 Macquarie Point **Location : Hobart, Tasmania**

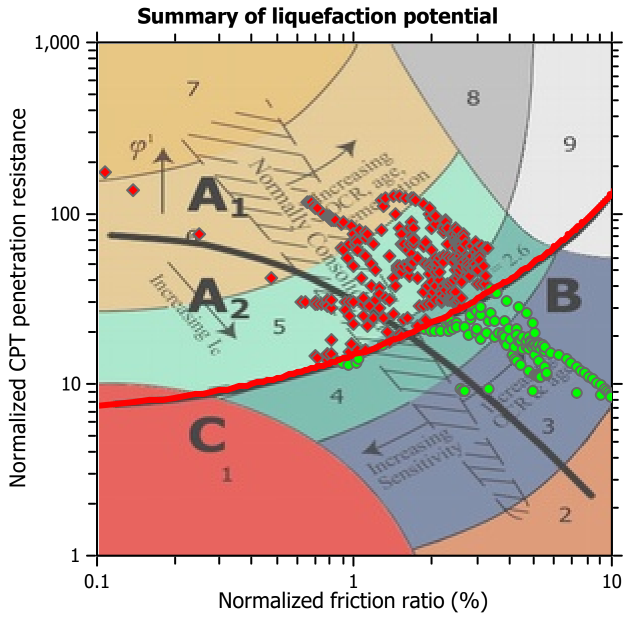
CPT file : CPT-07

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	2.50 m	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	2.50 m	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.50	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.19	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		







Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

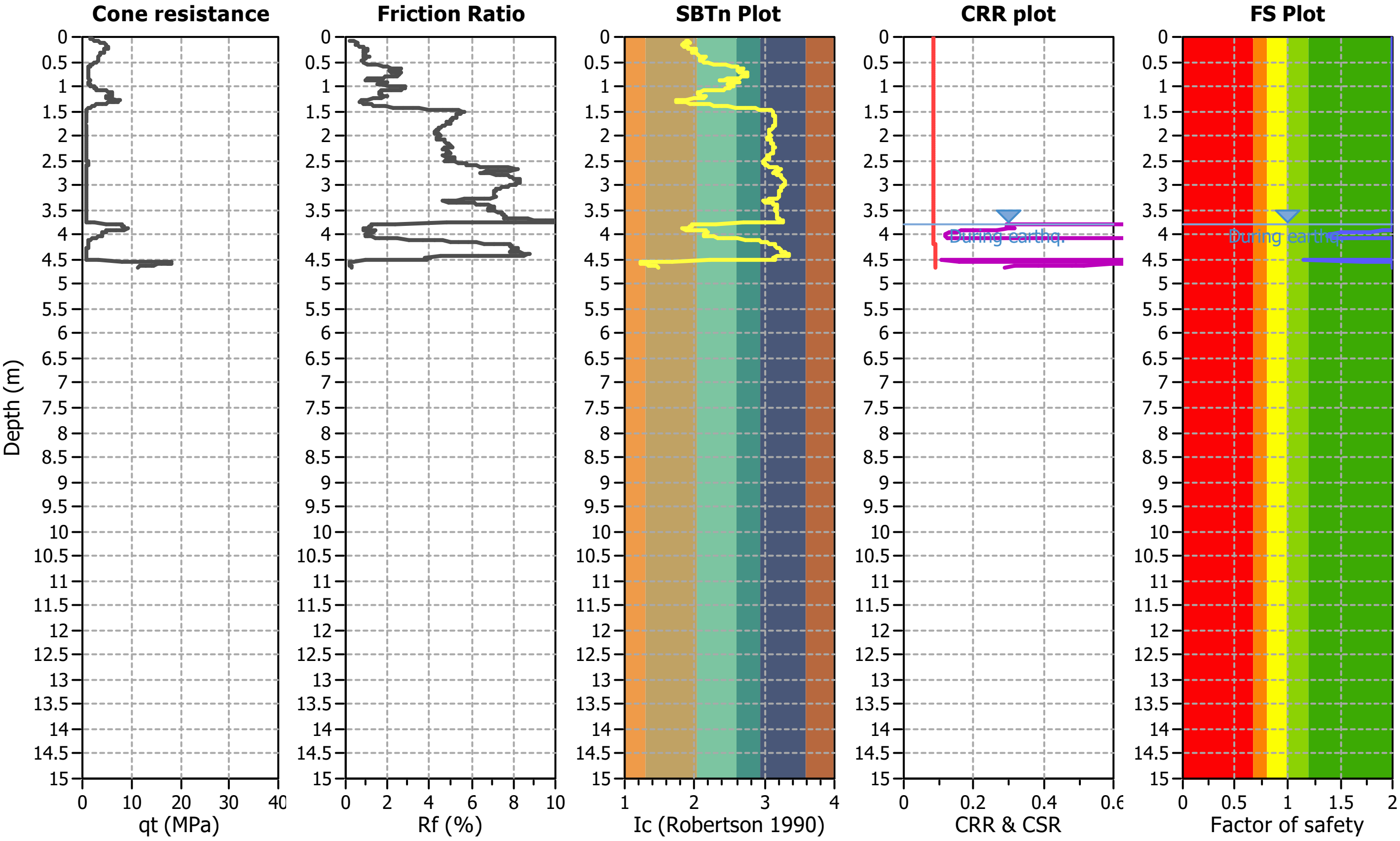
LIQUEFACTION ANALYSIS REPORT

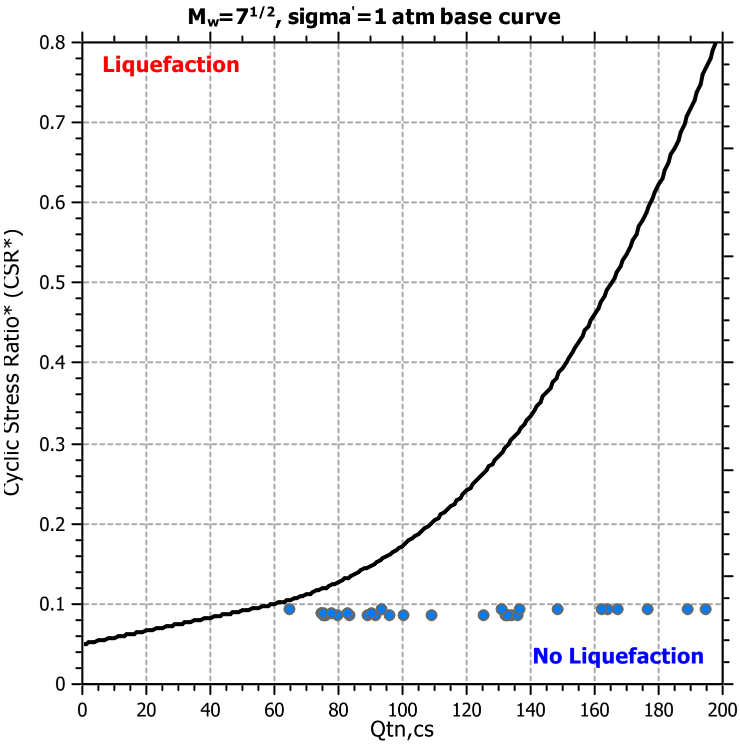
Project title : PS212776 Macquarie Point **Location : Hobart, Tasmania**

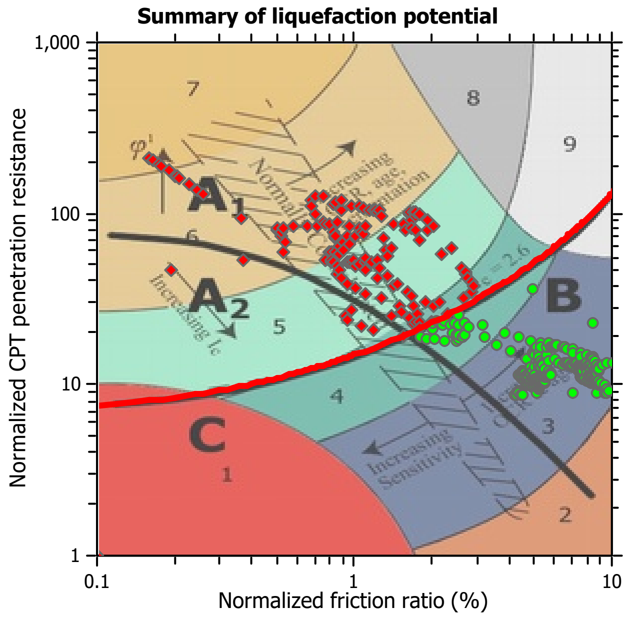
CPT file : CPT-07a

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	3.80 m	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	3.80 m	Fill height:	N/A		
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth applied:	No
Earthquake magnitude M_w:	6.50	Ic cut-off value:	2.60	Trans. detect. applied:	No	Limit depth:	N/A
Peak ground acceleration:	0.19	Unit weight calculation:	Based on SBT	K_σ applied:	Yes	MSF method:	Method based







Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

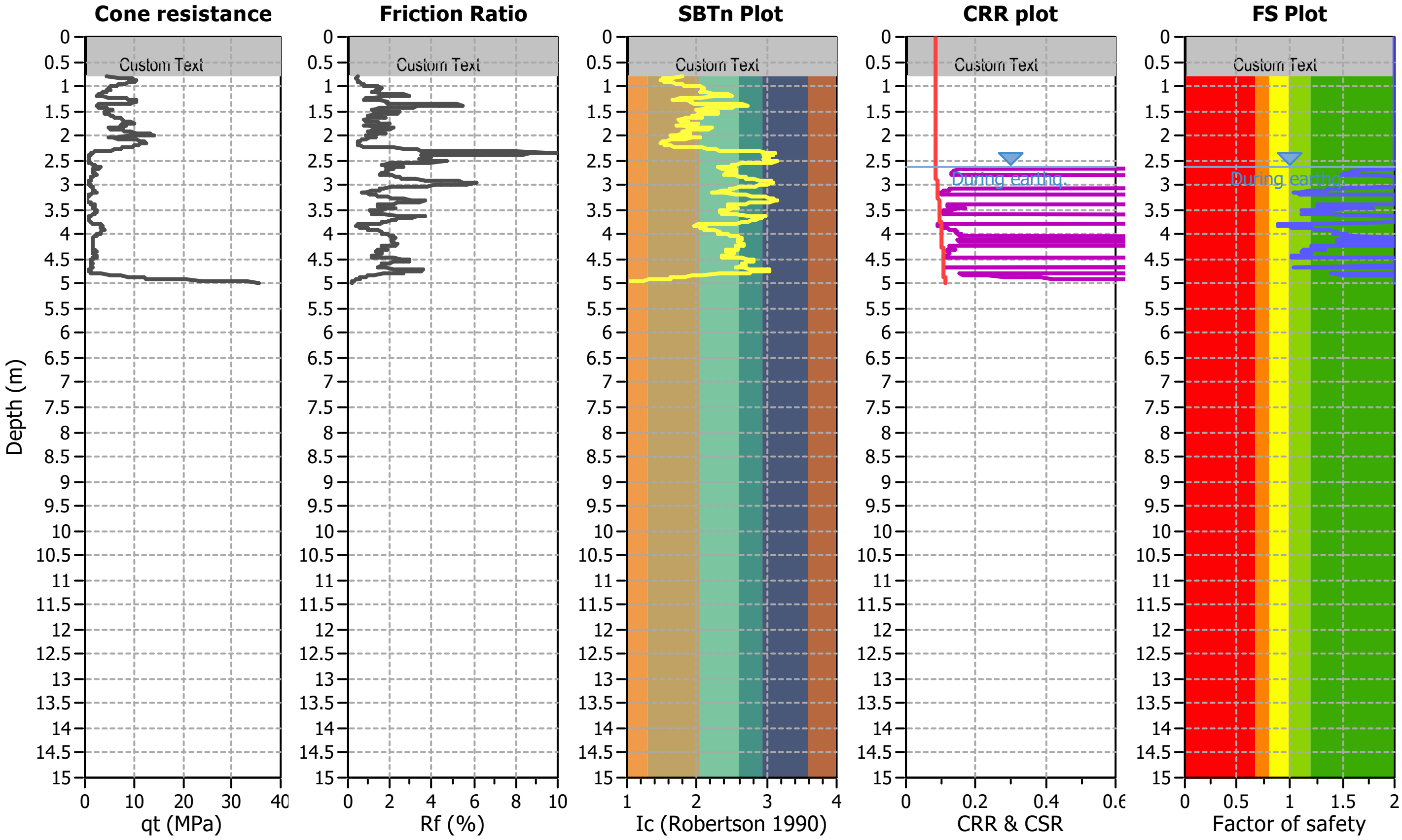
LIQUEFACTION ANALYSIS REPORT

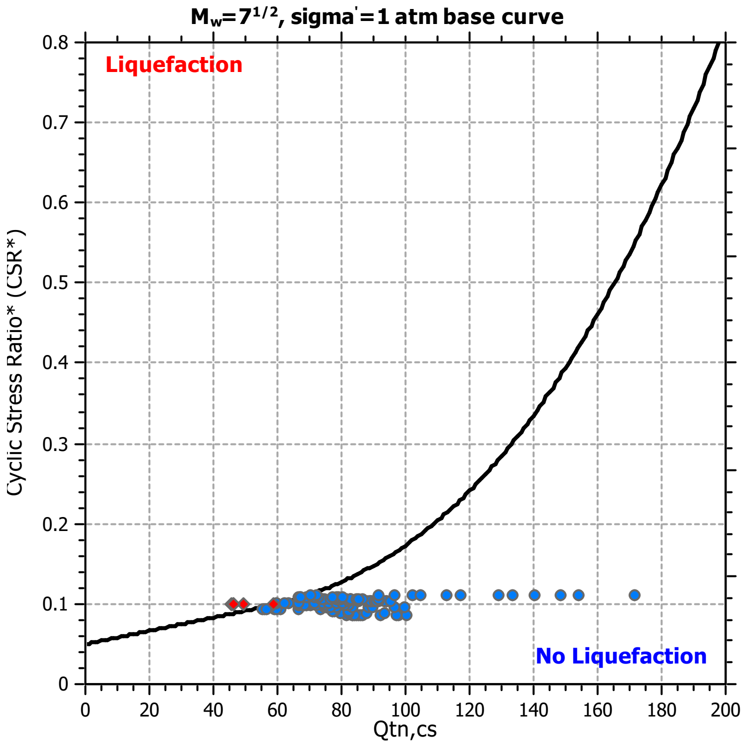
Project title : PS212776 Macquarie Point **Location : Hobart, Tasmania**

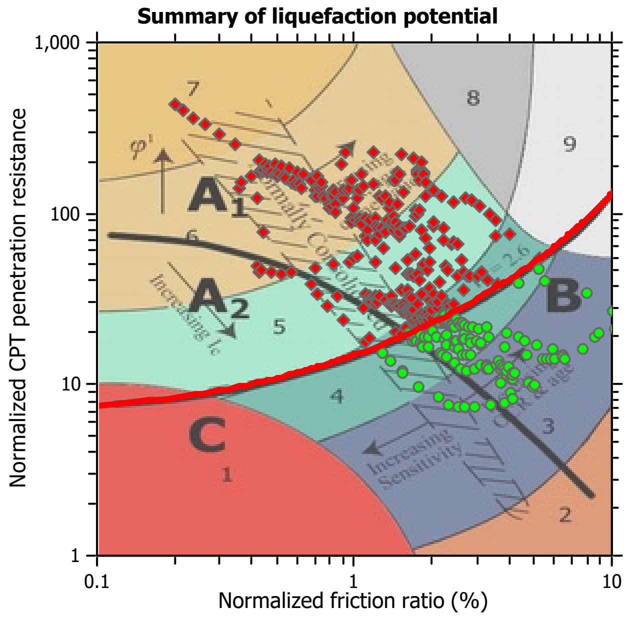
CPT file : CPT-08

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	2.65 m	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	2.65 m	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.50	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.19	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		







Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

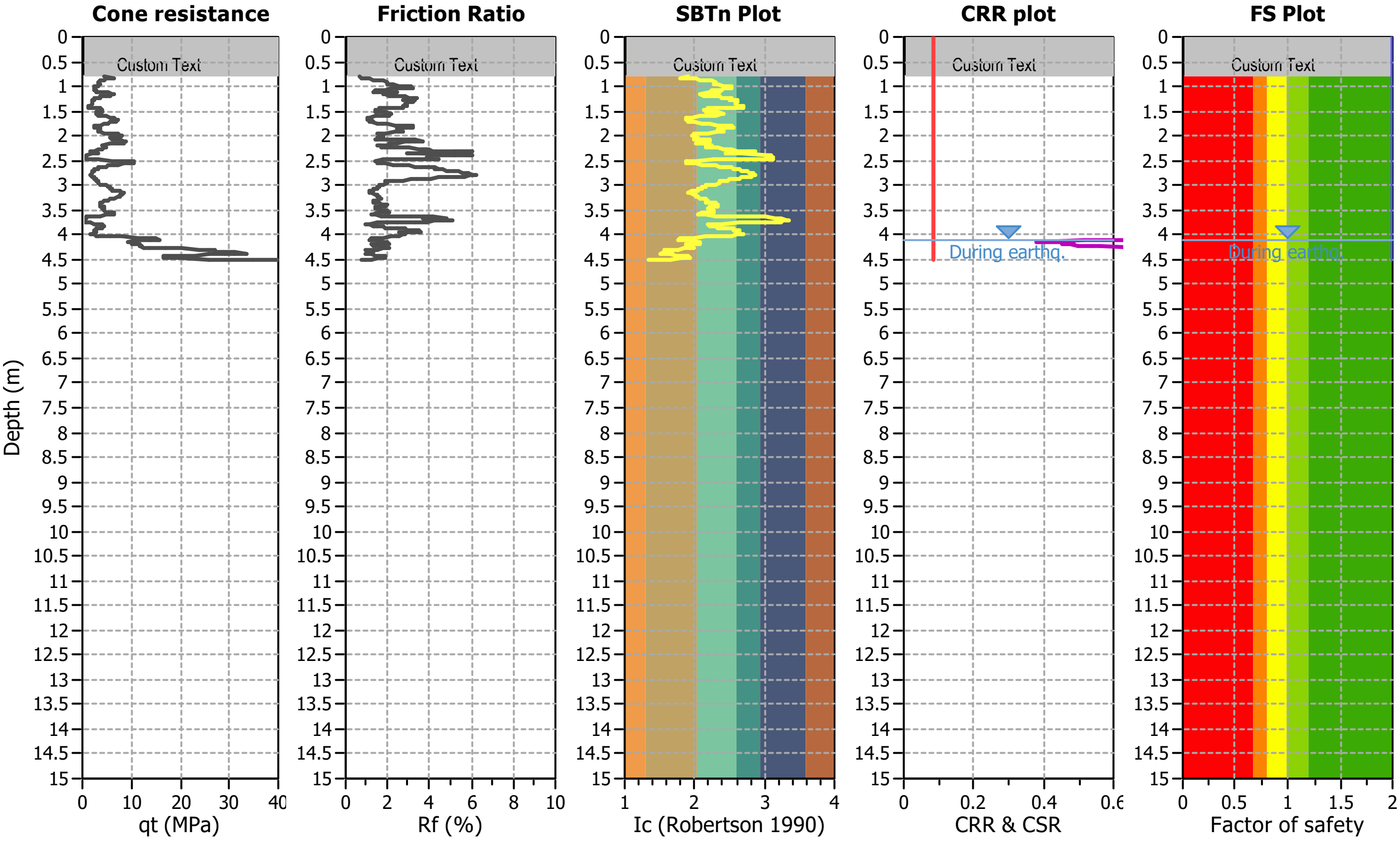
LIQUEFACTION ANALYSIS REPORT

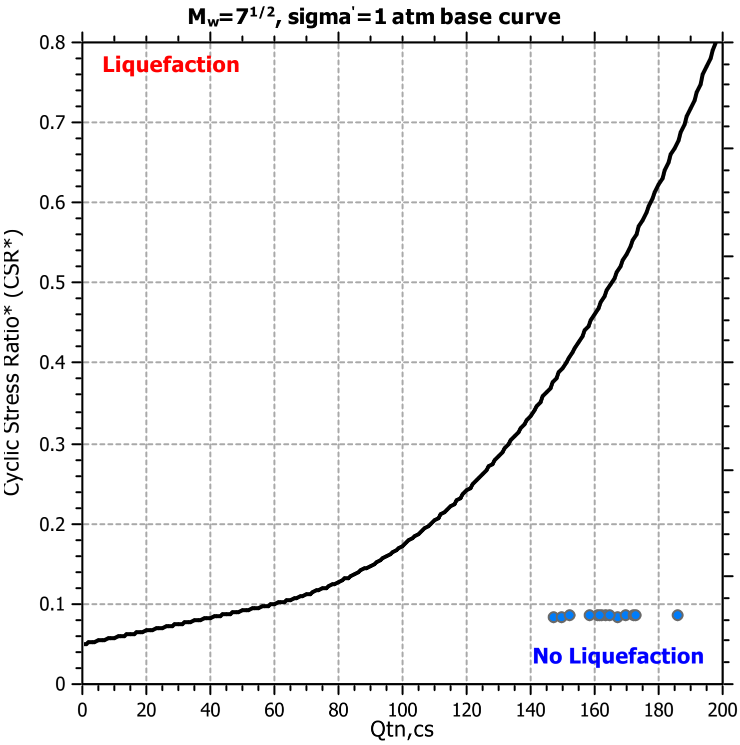
Project title : PS212776 Macquarie Point **Location : Hobart, Tasmania**

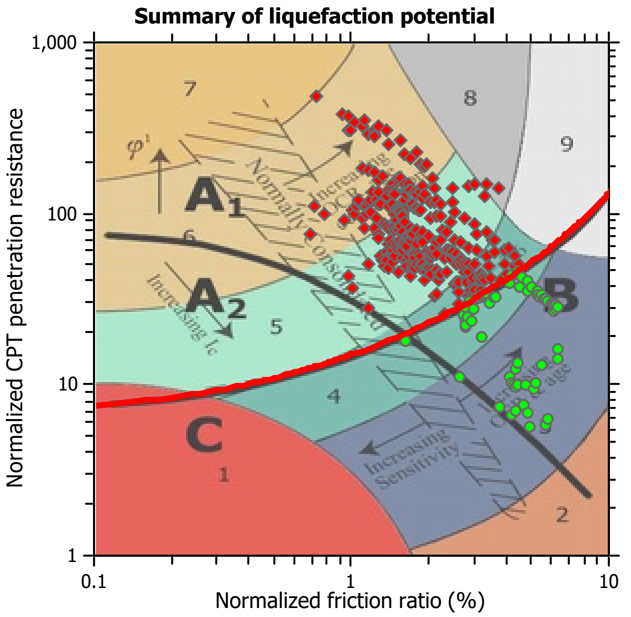
CPT file : CPT-11

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	4.12 m	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	4.12 m	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.50	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.19	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		







Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

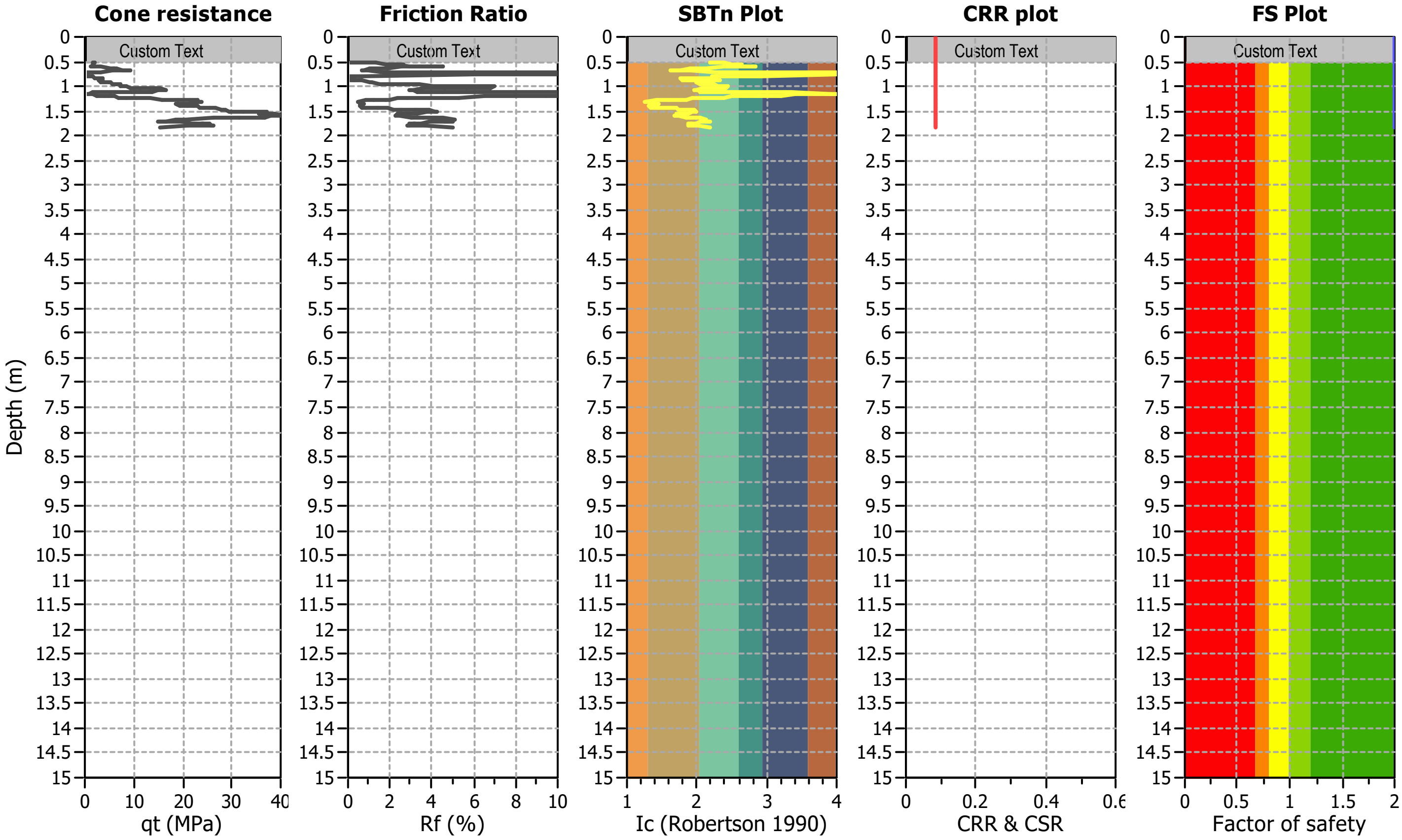
LIQUEFACTION ANALYSIS REPORT

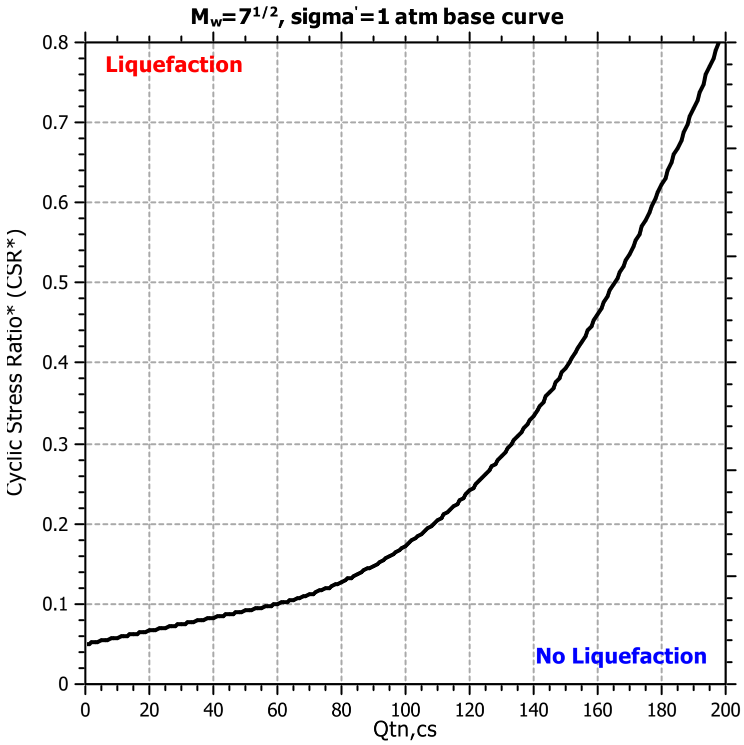
Project title : PS212776 Macquarie Point **Location : Hobart, Tasmania**

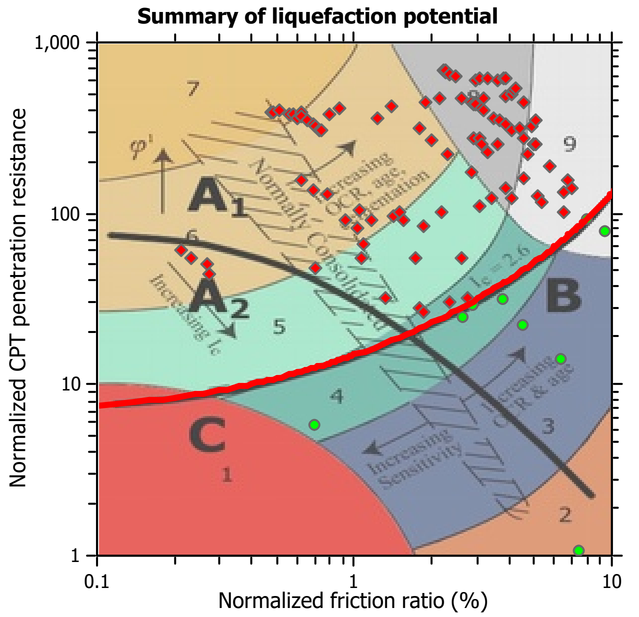
CPT file : CPT-12

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	25.00 m	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	25.00 m	Fill height:	N/A		
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth applied:	No
Earthquake magnitude M_w:	6.50	Ic cut-off value:	2.60	Trans. detect. applied:	No	Limit depth:	N/A
Peak ground acceleration:	0.19	Unit weight calculation:	Based on SBT	K_σ applied:	Yes	MSF method:	Method based







Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

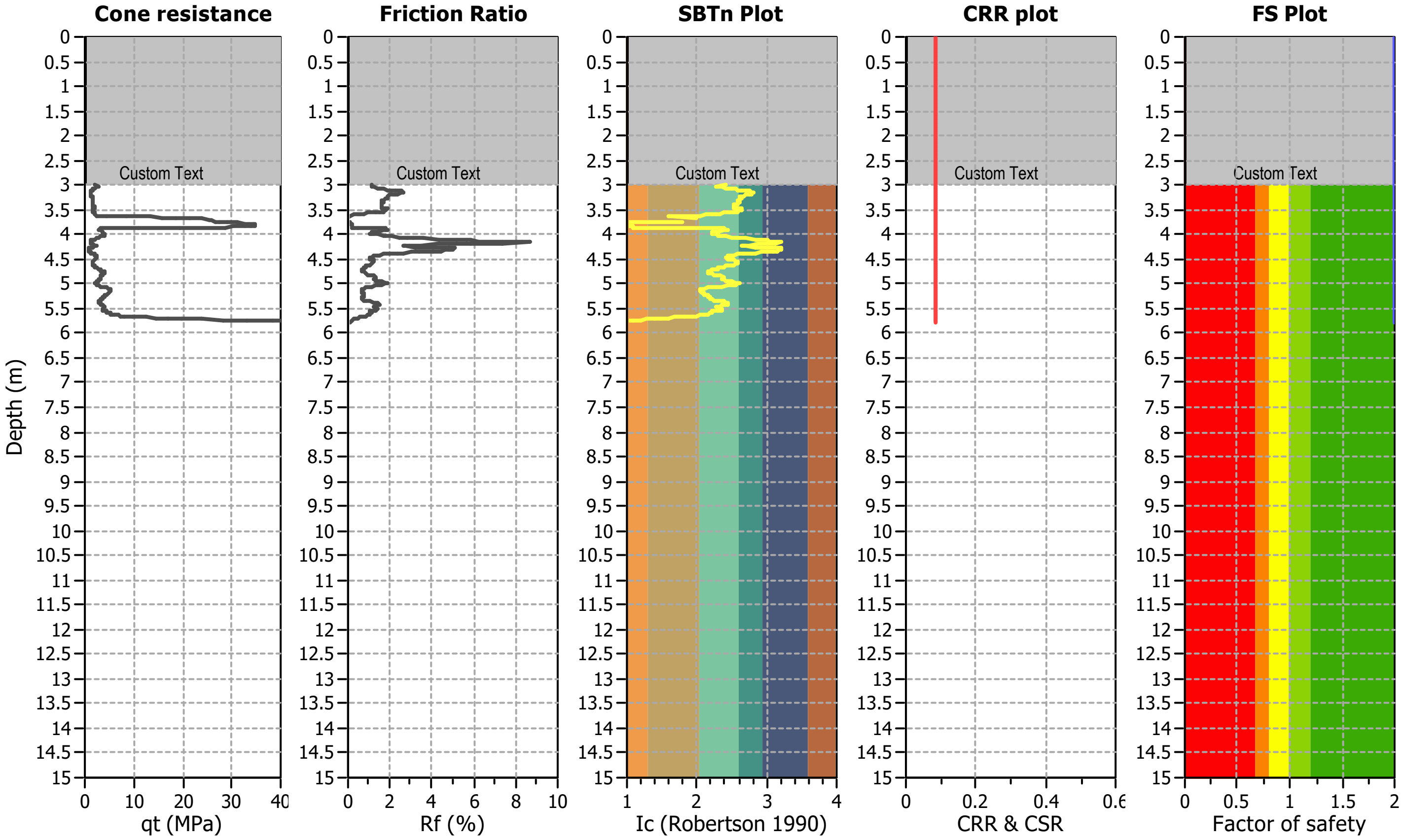
LIQUEFACTION ANALYSIS REPORT

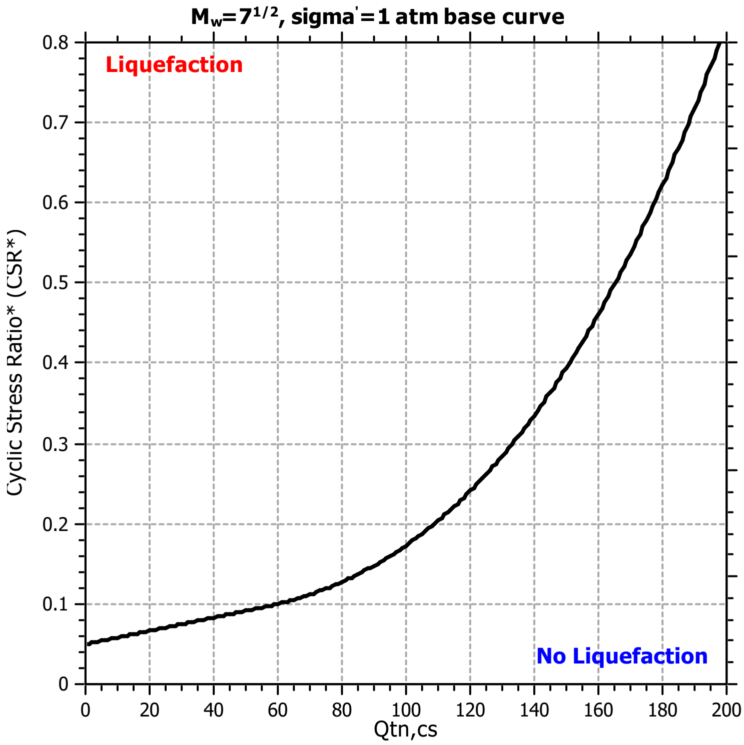
Project title : PS212776 Macquarie Point **Location : Hobart, Tasmania**

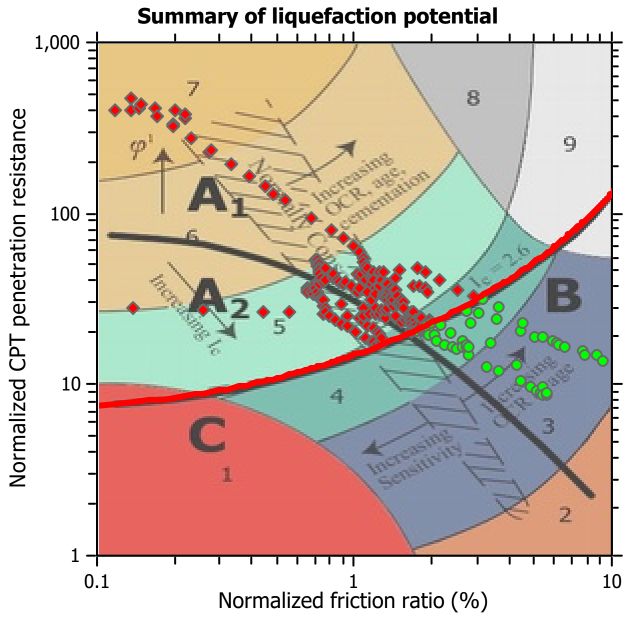
CPT file : CPT-12a

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	25.00 m	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	25.00 m	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.50	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.19	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		







Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

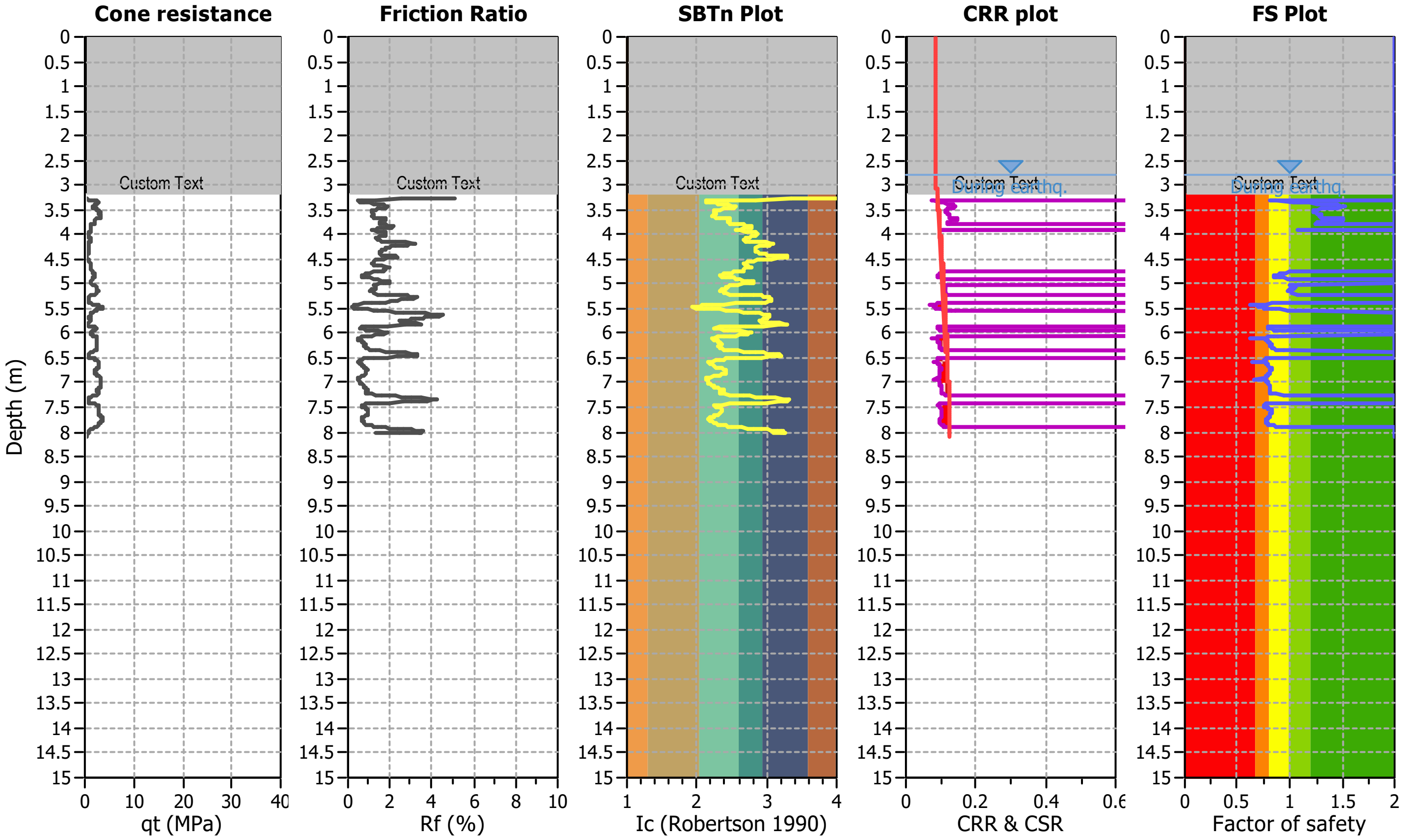
LIQUEFACTION ANALYSIS REPORT

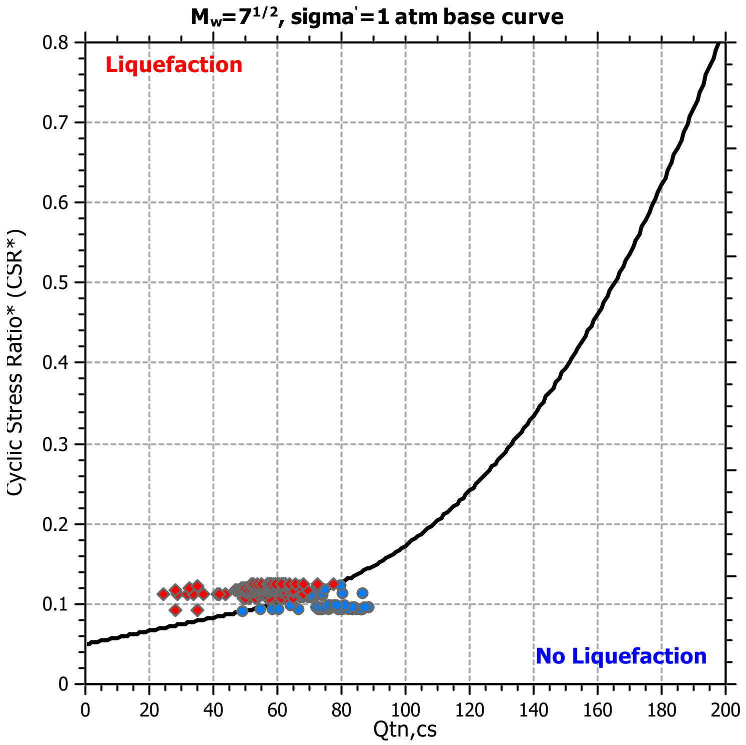
Project title : PS212776 Macquarie Point **Location : Hobart, Tasmania**

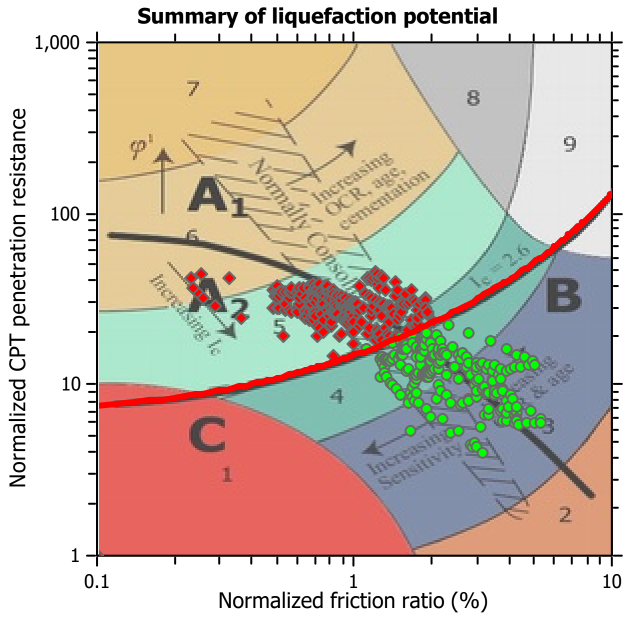
CPT file : CPT-13

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	2.80 m	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	2.80 m	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.50	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.19	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		







Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

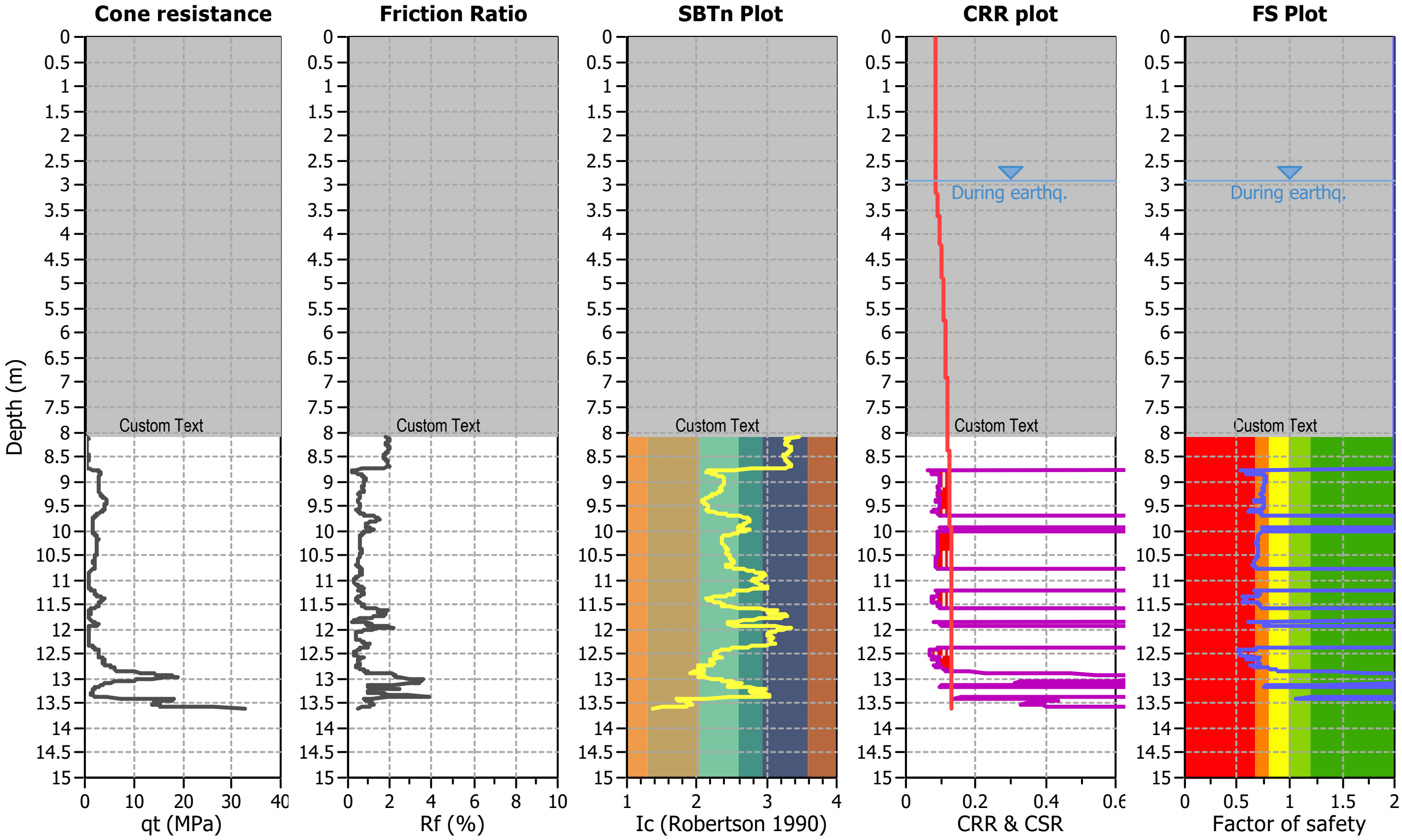
LIQUEFACTION ANALYSIS REPORT

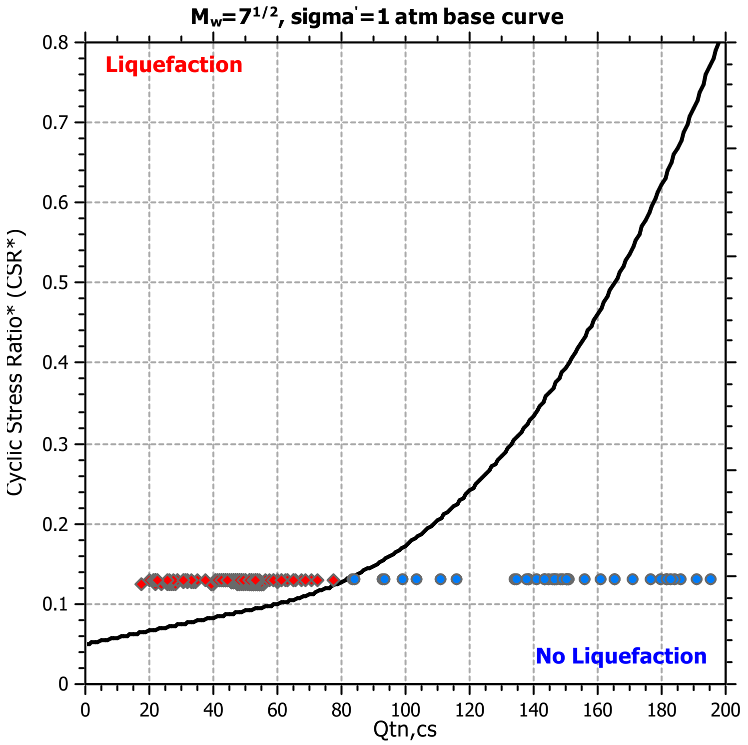
Project title : PS212776 Macquarie Point **Location : Hobart, Tasmania**

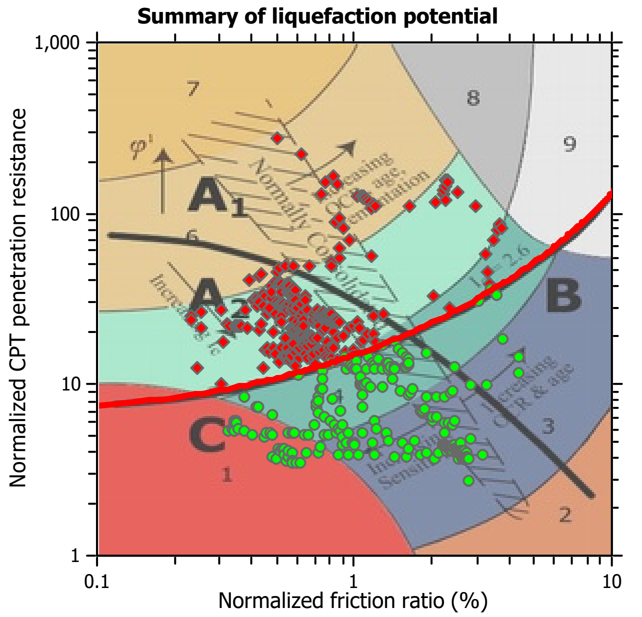
CPT file : CPT-13a

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	2.90 m	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	2.90 m	Fill height:	N/A		
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth applied:	No
Earthquake magnitude M_w:	6.50	Ic cut-off value:	2.60	Trans. detect. applied:	No	Limit depth:	N/A
Peak ground acceleration:	0.19	Unit weight calculation:	Based on SBT	K_σ applied:	Yes	MSF method:	Method based







Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

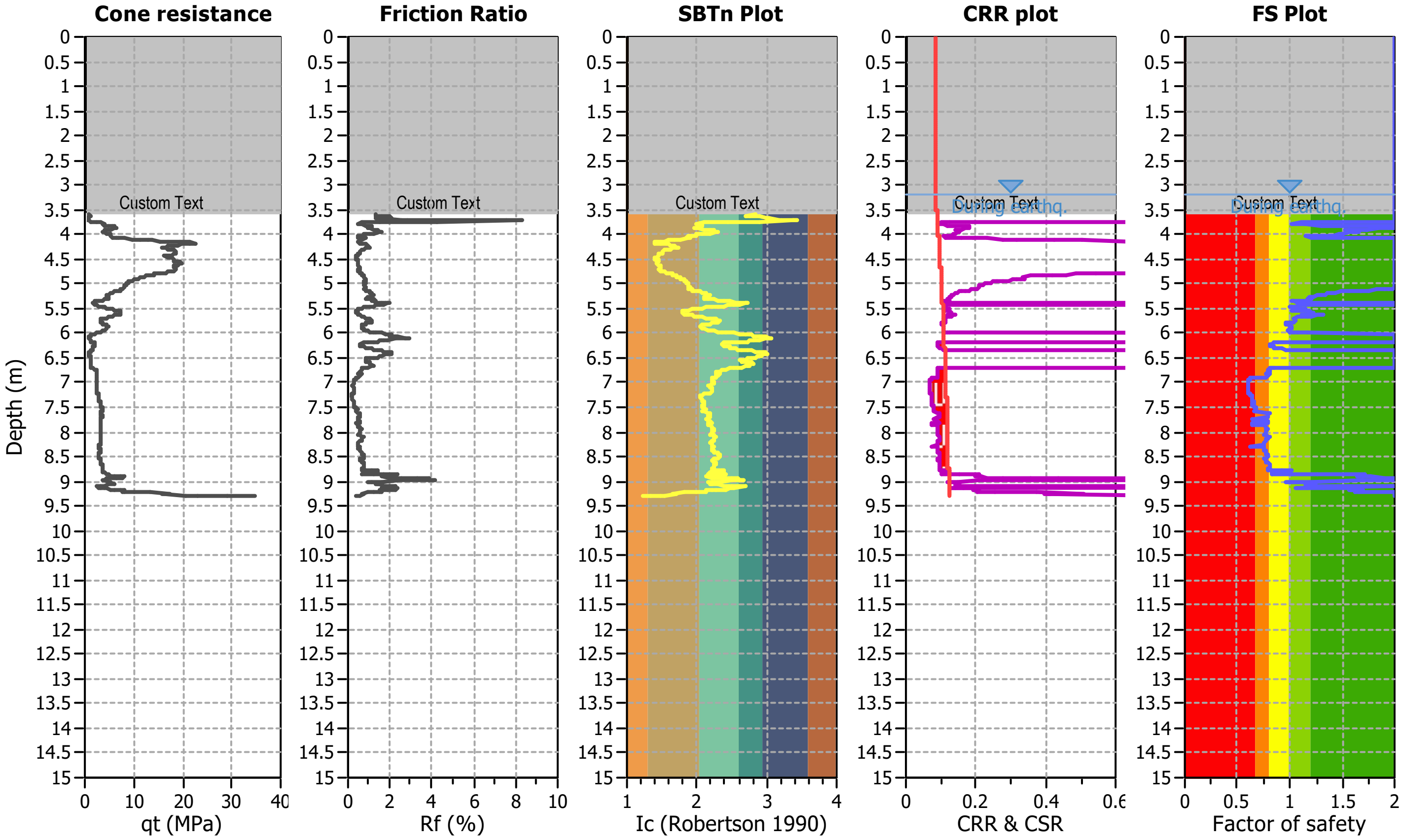
LIQUEFACTION ANALYSIS REPORT

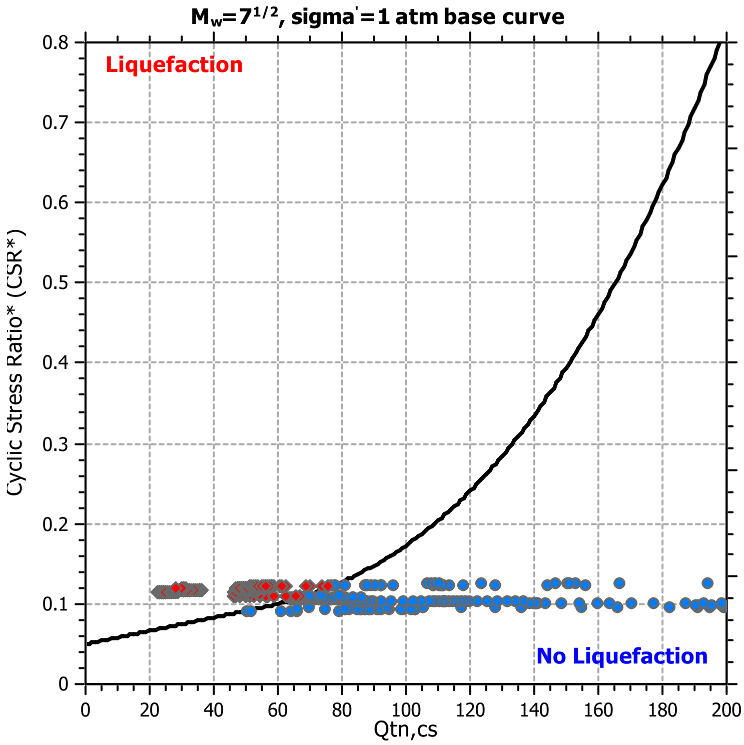
Project title : PS212776 Macquarie Point **Location : Hobart, Tasmania**

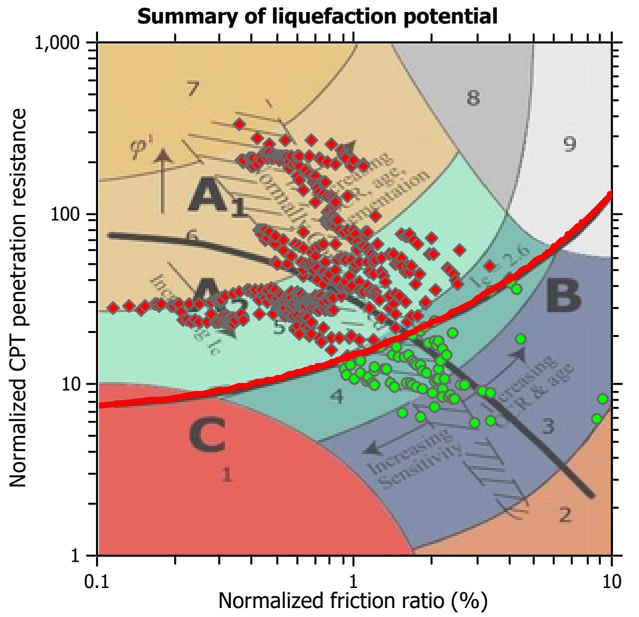
CPT file : CPT-14

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	3.20 m	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	3.20 m	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.50	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.19	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		







Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

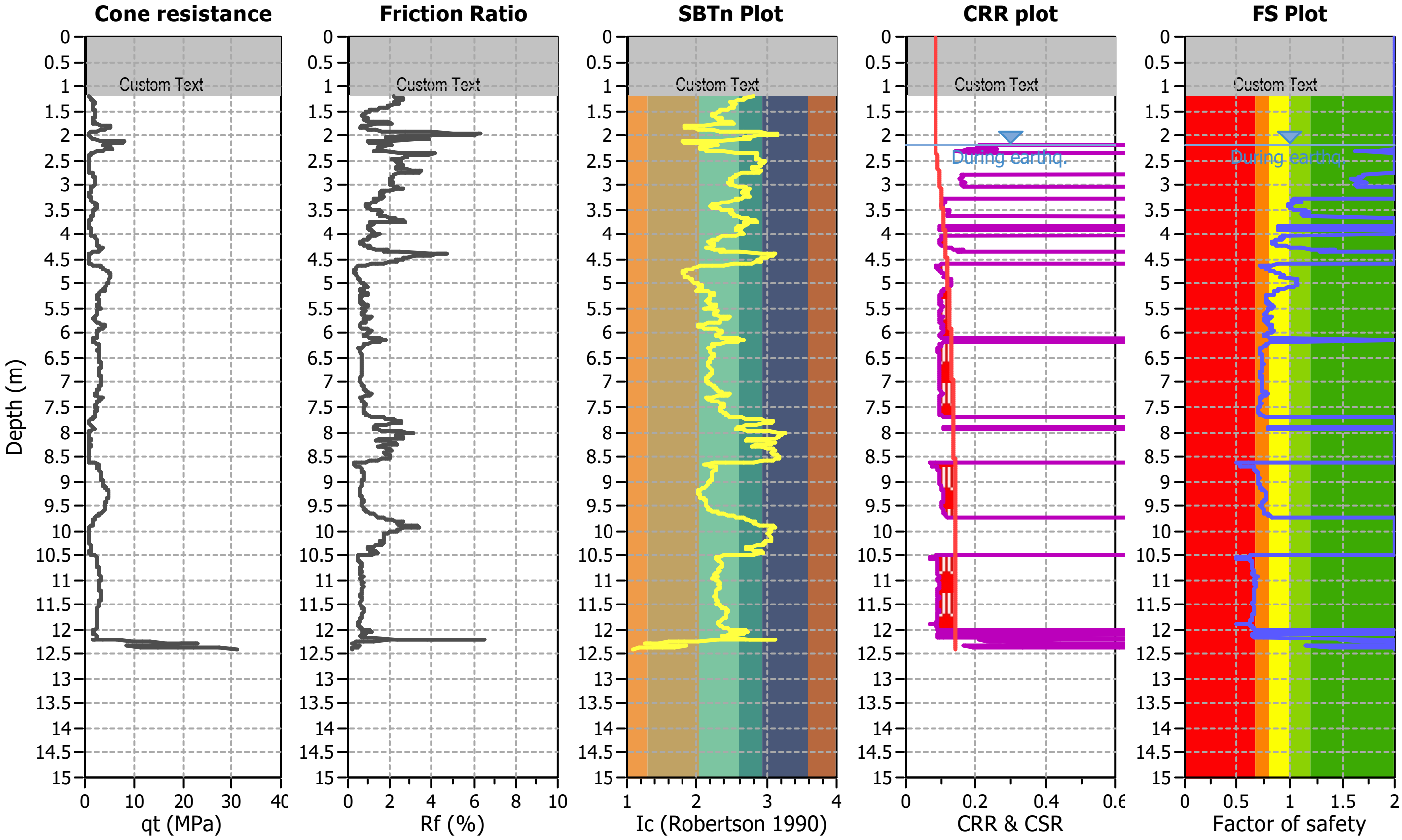
LIQUEFACTION ANALYSIS REPORT

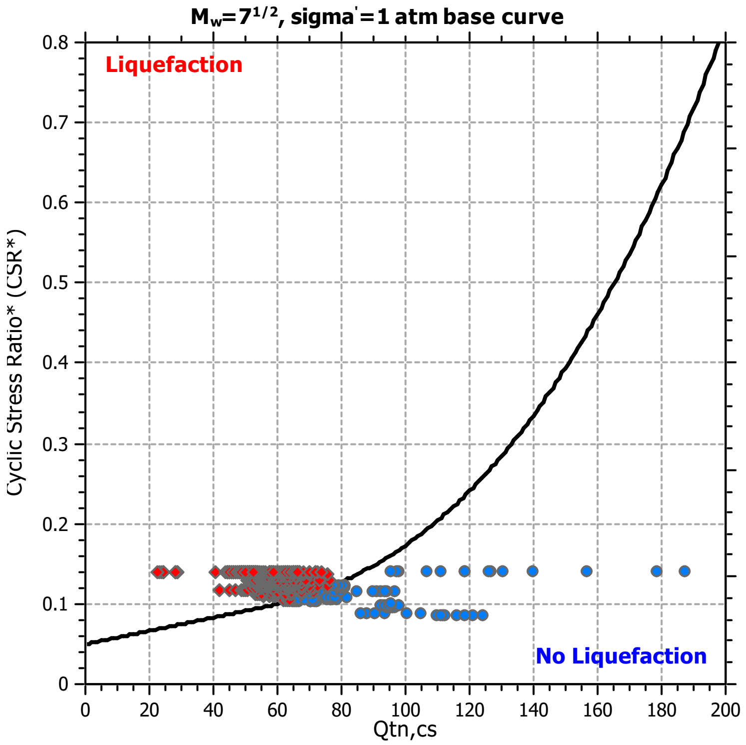
Project title : PS212776 Macquarie Point **Location : Hobart, Tasmania**

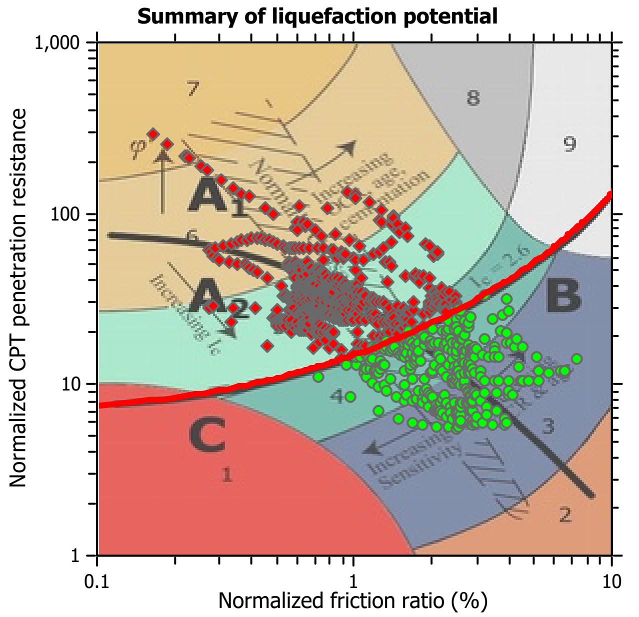
CPT file : CPT-16

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	2.20 m	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	2.20 m	Fill height:	N/A		
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth applied:	No
Earthquake magnitude M_w:	6.50	Ic cut-off value:	2.60	Trans. detect. applied:	No	Limit depth:	N/A
Peak ground acceleration:	0.19	Unit weight calculation:	Based on SBT	K_σ applied:	Yes	MSF method:	Method based







Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

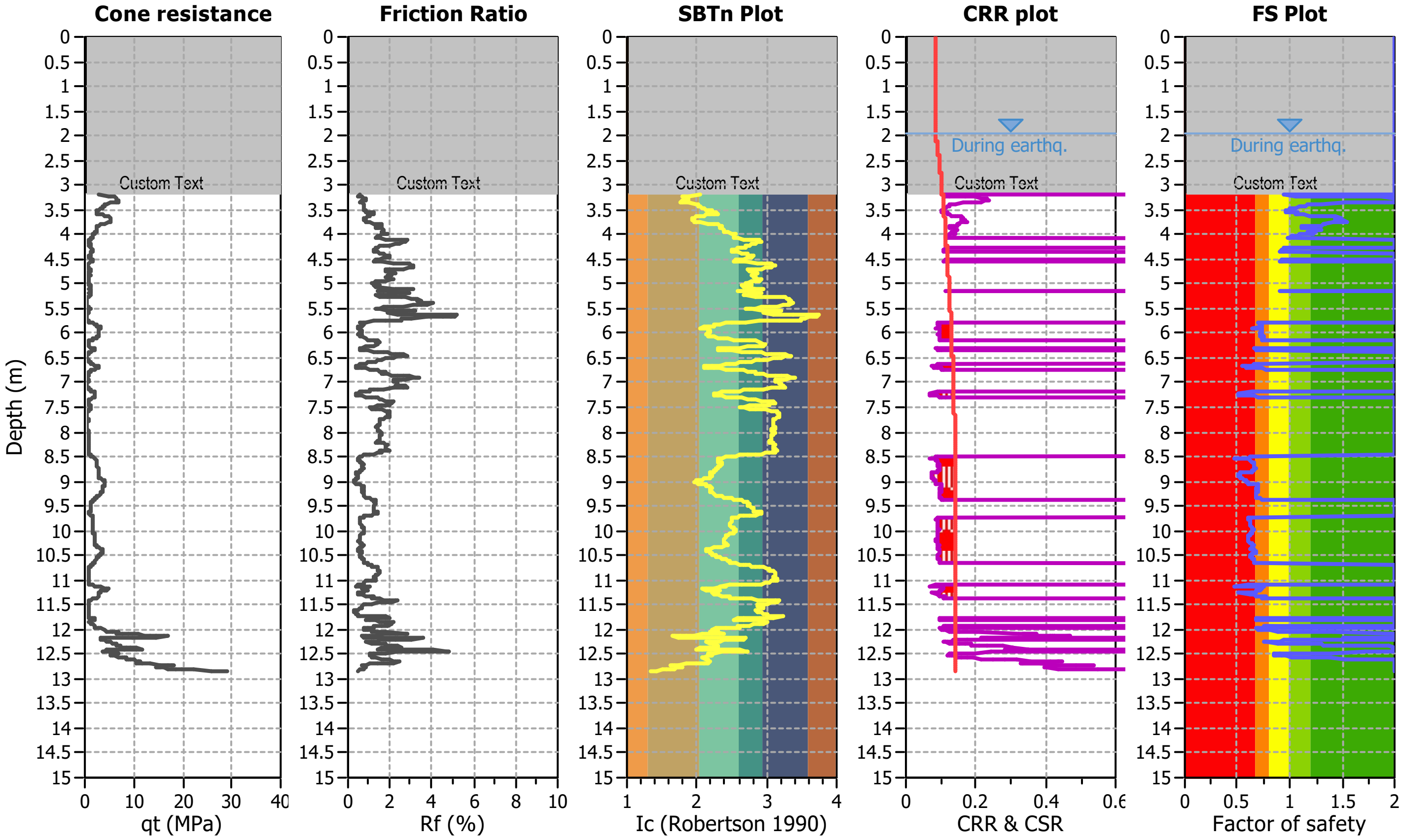
LIQUEFACTION ANALYSIS REPORT

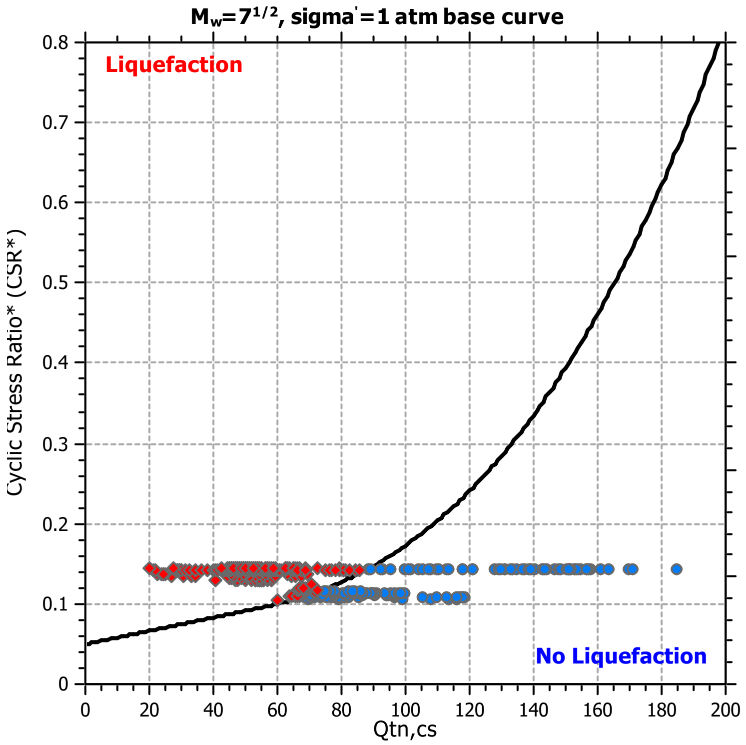
Project title : PS212776 Macquarie Point **Location : Hobart, Tasmania**

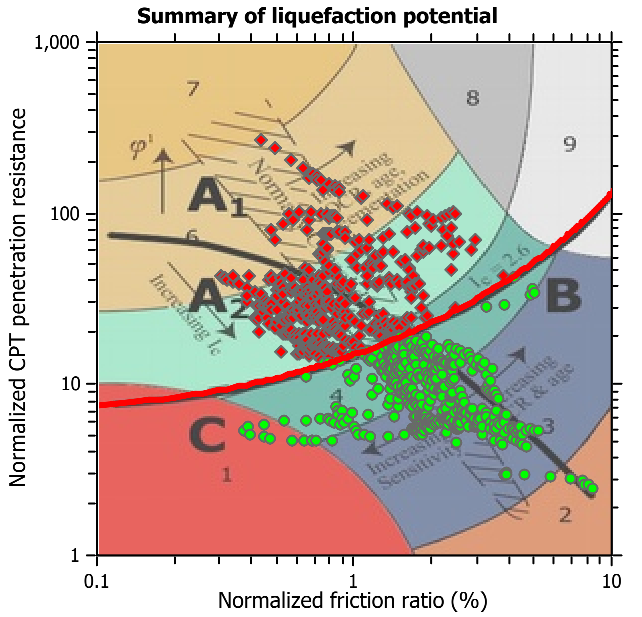
CPT file : CPT-17

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	1.95 m	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	1.95 m	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.50	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.19	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		







Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

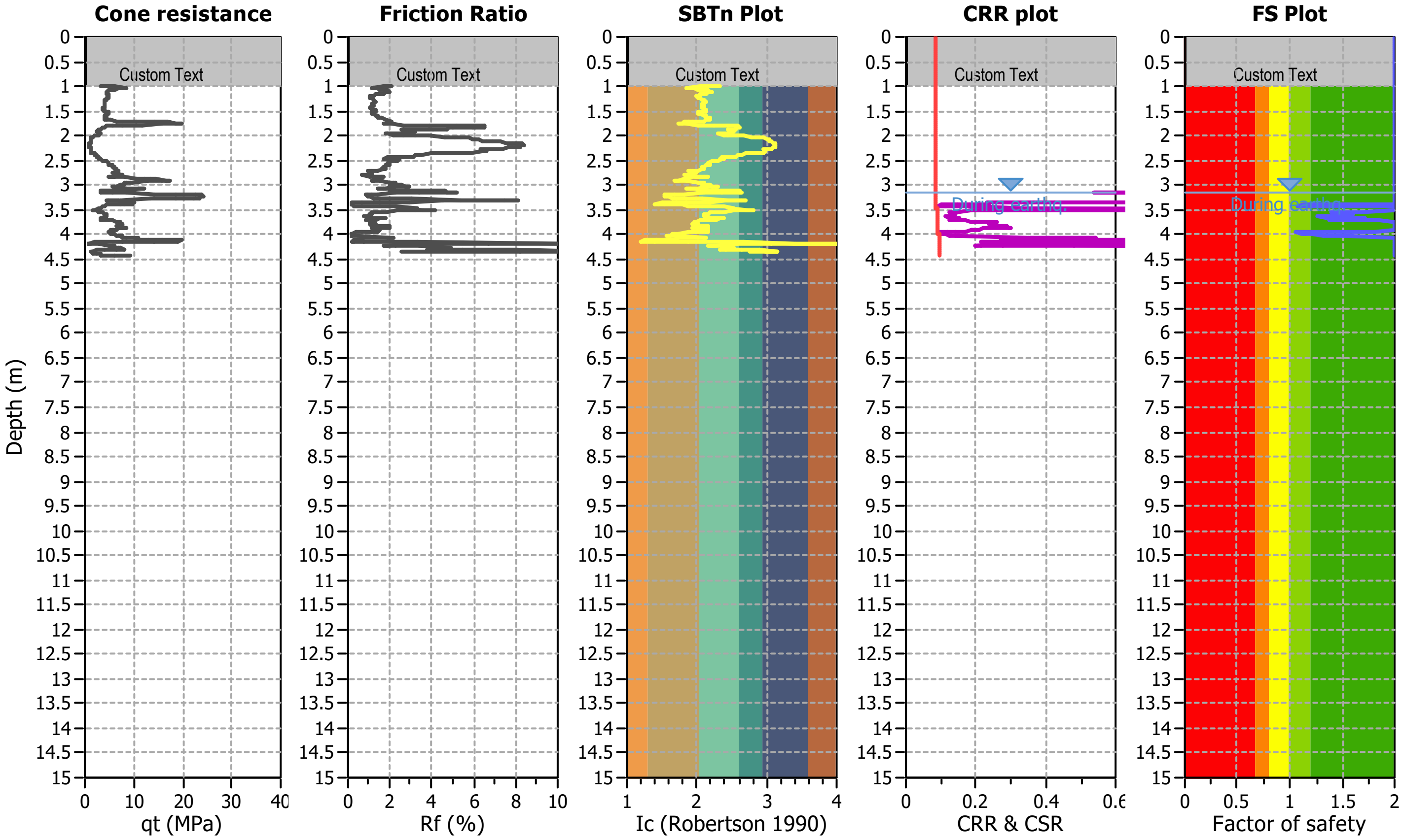
LIQUEFACTION ANALYSIS REPORT

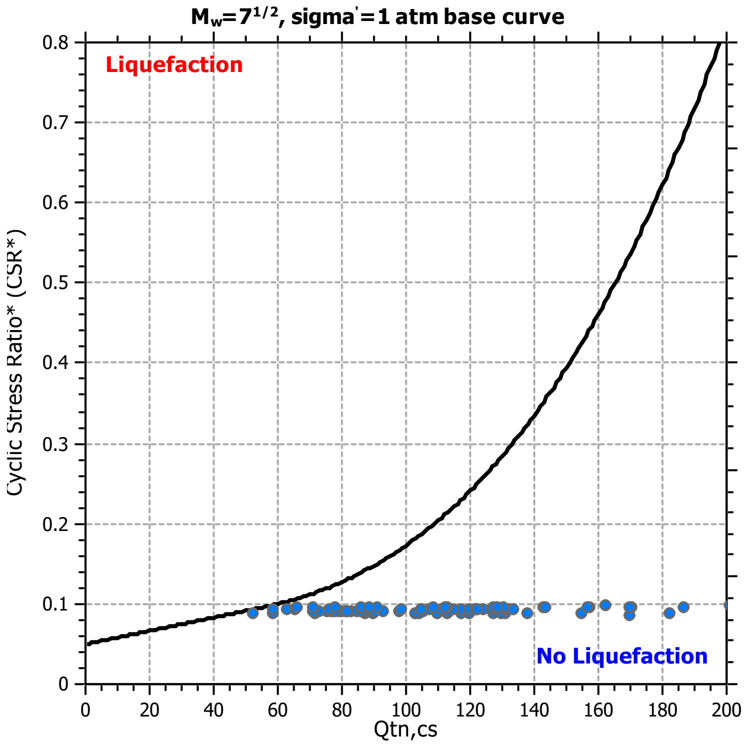
Project title : PS212776 Macquarie Point **Location : Hobart, Tasmania**

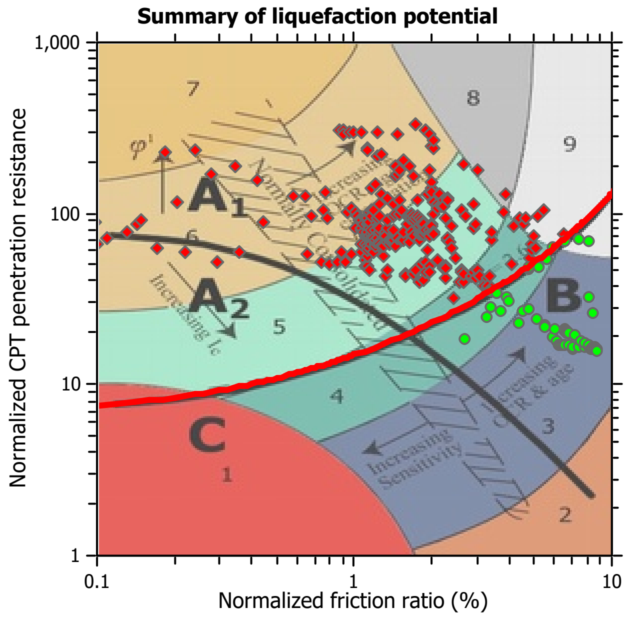
CPT file : CPT-19

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	3.16 m	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	3.16 m	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.50	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.19	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		







Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

Appendix H

Limitations

Limitation Statement: Geotechnical Site Investigation

SCOPE OF SERVICES

This geotechnical site assessment report (the report) has been prepared in accordance with the scope of services set out in the contract, or as otherwise agreed, between the client and WSP (scope of services). In some circumstances the scope of services may have been limited by a range of factors such as time, budget, access and/or site disturbance constraints.

RELIANCE ON DATA

In preparing the report, WSP has relied upon data, surveys, analyses, designs, plans and other information provided by the client and other individuals and organisations, most of which are referred to in the report (the data). Except as otherwise stated in the report, WSP has not verified the accuracy or completeness of the data. To the extent that the statements, opinions, facts, information, conclusions and/or recommendations in the report (conclusions) are based in whole or part on the data, those conclusions are contingent upon the accuracy and completeness of the data. WSP will not be liable in relation to incorrect conclusions should any data, information or condition be incorrect or have been concealed, withheld, misrepresented or otherwise not fully disclosed to WSP.

GEOTECHNICAL INVESTIGATION

Geotechnical engineering is based extensively on judgment and opinion. It is far less exact than other engineering disciplines. Geotechnical engineering reports are prepared to meet the specific needs of individuals. A report prepared for a consulting civil engineer may not be adequate for a construction contractor or even some other consulting civil engineer. This report was prepared expressly for the client and expressly for purposes indicated by the client or his representative. Use by any other persons for any purpose, or by the client for a different purpose, might result in problems. The client should not use this report for other than its intended purpose without seeking additional geotechnical advice.

THIS GEOTECHNICAL REPORT IS BASED ON PROJECT-SPECIFIC FACTORS

This geotechnical engineering report is based on a subsurface investigation which was designed for project-specification factors, including the nature of any development, its size and configuration, the location of any development on the site and its orientation, and the location of access roads and parking areas. Unless further geotechnical advice is obtained this geotechnical engineering report cannot be used:

- when the nature of any proposed development is changed
- when the size, configuration location or orientation of any proposed development is modified.

This geotechnical engineering report cannot be applied to an adjacent site.

THE LIMITATIONS OF SITE INVESTIGATION

In making an assessment of a site from a limited number of boreholes or test pits there is the possibility that variations may occur between test locations. Site exploration identifies specific subsurface conditions only at those points from which samples have been taken. The risk that variations will not be detected can be reduced by increasing the frequency of test locations; however this often does not result in any overall cost savings for the project. The investigation program undertaken is a professional estimate of the scope of investigation required to provide a general profile of the subsurface conditions. The data derived from the site investigation program and subsequent laboratory testing are extrapolated across the site to form an inferred geological model and an engineering opinion is rendered about overall subsurface conditions and their likely behaviour with regard to the proposed development. Despite investigation the actual conditions at the site might differ from those inferred to exist, since no subsurface exploration program, no matter how comprehensive, can reveal all subsurface details and anomalies.

The borehole logs are the subjective interpretation of subsurface conditions at a particular location, made by trained personnel. The interpretation may be limited by the method of investigation, and can not always be definitive. For example, inspection of an excavation or test pit allows a greater area of the subsurface profile to be inspected than borehole investigation, however, such methods are limited by depth and site disturbance restrictions. In borehole investigation, the actual interface between materials may be more gradual or abrupt than a report indicates.

SUBSURFACE CONDITIONS ARE TIME DEPENDENT

Subsurface conditions may be modified by changing natural forces or man-made influences. A geotechnical engineering report is based on conditions which existed at the time of subsurface exploration.

Construction operations at or adjacent to the site, and natural events such as floods, or groundwater fluctuations, may also affect subsurface conditions, and thus the continuing adequacy of a geotechnical report. The geotechnical engineer should be kept appraised of any such events, and should be consulted to determine if additional tests are necessary.

AVOID MISINTERPRETATION

A geotechnical engineer should be retained to work with other appropriate design professionals explaining relevant geotechnical findings and in reviewing the adequacy of their plans and specifications relative to geotechnical issues.

BORE/PROFILE LOGS SHOULD NOT BE SEPARATED FROM THE ENGINEERING REPORT

Final bore/profile logs are developed by geotechnical engineers based upon their interpretation of field logs and laboratory evaluation of field samples. Customarily, only the final bore/profile logs are included in geotechnical engineering reports. These logs should not under any circumstances be redrawn for inclusion in architectural or other design drawings. To minimise the likelihood of bore/profile log misinterpretation, contractors should be given access to the complete geotechnical engineering report prepared or authorised for their use. Providing the best available information to contractors helps prevent costly construction problems. For further information on this matter reference should be made to 'Guidelines for the Provision of Geotechnical Information in Construction Contracts' published by the Institution of Engineers Australia, National Headquarters, Canberra 1987.

GEOTECHNICAL INVOLVEMENT DURING CONSTRUCTION

During construction, excavation is frequently undertaken which exposes the actual subsurface conditions. For this reason geotechnical consultants should be retained through the construction stage, to identify variations if they are exposed and to conduct additional tests which may be required and to deal quickly with geotechnical problems if they arise.

REPORT FOR BENEFIT OF CLIENT

The report has been prepared for the benefit of the client and no other party. WSP assumes no responsibility and will not be liable to any other person or organisation for or in relation to any matter dealt with or conclusions expressed in the report, or for any loss or damage suffered by any other person or organisation arising from matters dealt with or conclusions expressed in the report (including without limitation matters arising from any negligent act or omission of WSP or for any loss or damage suffered by any other party relying upon the matters dealt with or conclusions expressed in the report). Other parties should not rely upon the report or the accuracy or completeness of any conclusions and should make their own enquiries and obtain independent advice in relation to such matters.

OTHER LIMITATIONS

WSP will not be liable to update or revise the report to take into account any events or emergent circumstances or facts occurring or becoming apparent after the date of the report.