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# PORT OF HASTINGS DEVELOPMENT PROJECT

DESIGN AND ENGINEERING

## Geotechnical Interpretive Report

Document Ref: AGH-CEP0-EG-REP-0009

In May 2016 the Special Minister of State asked Infrastructure Victoria to provide advice on the future capacity of Victoria's commercial ports. Specifically, the Minister has asked for advice on when the need for a second container port is likely to arise and which variables may alter this timeline. The Minister has also asked for advice on where a second container port would ideally be located and under what conditions, including the suitability of, and barriers to investing in, sites at the Port of Hastings and the Bay West location.

In undertaking this task, Infrastructure Victoria reviewed work that was completed as part of the Port of Hastings development project before it was cancelled in 2014. This document forms part of the initial work undertaken for the proposed port development at Hastings. Infrastructure Victoria considers that much of the previous Hastings work, although preliminary in nature, is relevant and suitable for informing a strategic assessment. Therefore, Infrastructure Victoria has made the reports previously commissioned for the development project part of the evidence base on which Infrastructure Victoria will use in providing the Minister with advice.

The opinions, conclusions and any recommendations in this document are based on conditions encountered and information reviewed at the date of preparation of the document and for the purposes of the Port of Hastings Development Project.

Infrastructure Victoria and its consultants have used the information contained in these reports as an input but have not wholly relied on all the information presented in these reports.

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# Port of Hastings Development Project – Design and Engineering

## Geotechnical Interpretive Report

Client: Port of Hastings Development Authority

ABN: 33 737 350 749


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# Executive Summary


In 2013 Port of Hastings Development Authority (PoHDA) commissioned terrestrial and marine geotechnical investigations, and a marine geophysical survey, to inform planning and business case studies for the port expansion.  These investigations were used to infill data gaps in information collected as part of previous investigations, and to provide sufficient information to support business case studies.

This Geotechnical Interpretive Report provides an overview of the investigations undertaken to date, explains the geological and geomorphological setting of Western Port Bay, presents the findings of the subsurface investigations and associated testing, provides discussion on the geotechnical considerations for Port Expansion, together with characteristic engineering parameters for use in concept design.

The key findings of the geotechnical and geophysical investigations as described in the Geotechnical Interpretative Report are given below:

- Bedrock is not expected to be encountered during dredging in the Port Area or the North Arm, however dredging of cemented materials will be required. Dredge materials in the Port Area and the North Arm are not expected to pose significant challenges for conventional dredging plant and equipment. Localised dredging and removal of high spots in the Western Channel and Anchorage would likely encounter weathered basalt rock.
- Within the BlueScope Steel terrestrial SUZ1 area and across most of the offshore Port Area relatively competent Baxter Formation soils are present at shallow depth which are deemed to provide suitable founding strata on which to place reclamation fill. Relatively thin low strength recent deposits are present below sea bed within the Port Area. It may be beneficial to remove these materials in areas of reclamation prior to filling.
- The dredge materials, predominantly comprising Baxter and Sherwood Formation soils, exhibit vertical and lateral variability. As a result of this variability it may not be possible to selectively dredge these sand and clay deposits. Reclamation using dredged Baxter and Sherwood Formation soils would require careful management to prevent unconsolidated slurry from becoming entrapped within the reclamation fill. Ground improvement would be required for reclamation fills derived from these materials to densify soils and reduce post reclamation settlement. Recent soft marine clays would be unsuitable as fill for reclamation.
- Foundation materials are expected to be suitably competent to support quay structures and associated facilities using conventional forms of construction including piled bulkhead walls or piled quay decks. Piles for wharf structures are expected to be driven to refusal on bedrock or a specified driving resistance in Baxter or Sherwood Formation soils. Foundations for gravity structures may require construction of a rockfill base along the quay line within the Baxter or Sherwood Formations due to the high bearing pressures associated with these structure types.

In summary, the 2013/ 2014 investigations provide a better understanding of geotechnical conditions and risks across the area of potential port expansion, and is considered to provide suitable definition to support business case studies.

# 1.0 Introduction

## 1.1 Background Information

The Victorian Government has identified the Port of Hastings as the preferred site for the state's next major container port. This port is considered to be essential for the long-term economic growth of Victoria as container trades are increasing and the Port of Melbourne is expected to reach capacity.

The Port of Hastings Development Authority (the Authority) is progressing staged planning of the Port of Hastings Development Project from 2014 to 2018, culminating in the development of a rigorous business case and a full environmental and social impact assessment.

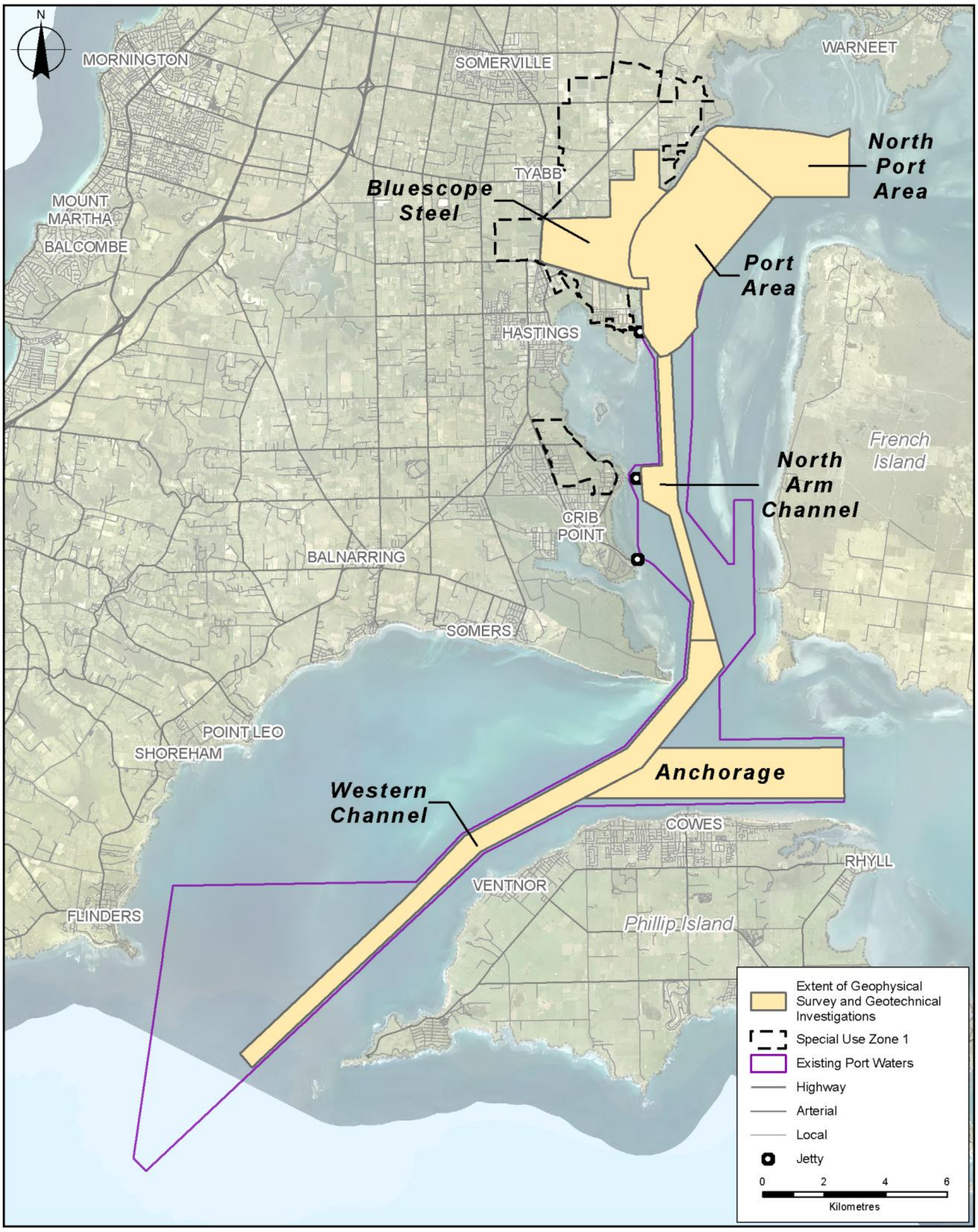
The Authority has selected a team of specialists to undertake detailed environmental, social and economic studies that will form part of a strict approval process. Specialists will also plan the conceptual design of new port infrastructure including wharf facilities and a logistics precinct, with road and rail access to the Port. Involvement of community and industry will be a critical part of the success.

By the mid-2020's it is envisaged that a world-class sustainable container port facility will begin operations at Hastings, handling up to 3 million twenty foot equivalent units (TEUs) each year, increasing to a minimum of 9 million TEU by 2060.

Prior to 2013, only limited existing site investigation information was available in the Port of Hastings area, comprising previous onshore and offshore investigations in the vicinity of wharf and jetty facilities at Long Island Point (ESSO), BlueScope Steel, Crib Point, and Stony Point, and limited investigations undertaken as part of studies in 2009 for a possible Stage 1 expansion of the Port of Hastings facilities between the Long Island Point and BlueScope Steel wharfs.

As part of the current study, in 2013 PoHDA commissioned terrestrial and marine geotechnical investigations, and a marine geophysical survey, to obtain subsurface information to inform planning and business case studies for the Port of Hastings expansion. This Geotechnical Interpretive Report has been prepared following completion of these studies. The extent of these studies is given in Figure 1-1.

**Figure 1-1 Extent of 2013-2014 geophysical survey and geotechnical investigation**




G:\31\31439\GIS\Maps\Deliverables\DE\20141021_Geotech Report\AGH-CEP0-DE-FIG-0085-ExtentOfSurveys_Em_A.mxd Data source: VicMap (DEPI, 2014), Port of Hastings Development Authority

## 1.2 Purpose of this Report

The objective of this Geotechnical Interpretative Report is to assimilate and summarise previous and recently completed geotechnical and geophysical investigation works, and to provide a preliminary assessment of ground conditions within the development area/project site. The information has been provided in a form that is suitable for subsequent use in the concept design and referral design studies for the proposed Port of Hastings expansion.

The structure of the Geotechnical Interpretive Report is as follows:

- Section 2.0 provides a summary of current thinking in respect to proposed port expansion options
- Section 3.0describes the regional geological and geomorphological setting of Western Port and surrounds.
- Section 4.0 describes site history and site conditions, including a summary of port development over time.
- Section 5.0 summarises historical site investigations that have been undertaken including drilling and geophysical surveys undertaken in the 1970s and 1980s.
- Section 6.0 describes the recently completed site investigation and geophysical survey works commissioned by PoHDA.
- Section 7.0 describes the site specific findings of the investigations and includes a delineation of ground conditions across different areas of the project site including the SUZ1 area, the port and reclamation area, turning basin and dredge areas, the North Arm, the Western Channel, and the Anchorage. Annotated geological cross sections and long sections and summaries of insitu and laboratory test data for different geological materials are presented.
- Section 8.0 provides an overview of the strata present across the development area and summarises the range of geotechnical properties from the field and laboratory testing for each stratigraphic unit.
- Section 9.0 provides a summary of geotechnical considerations for the proposed Port Expansion in respect to dredging, reclamation, ground improvement, and design of wharf and ancillary structures/ facilities.
- Section 10.0 briefly describes the development of a 3D geological model (to be reported separately) which will be used as input into various upcoming engineering studies including dredge and reclamation, dredge materials management, and wharf and terminal design packages.
- Section 11.0 provides recommendations for future studies and geotechnical investigations to be undertaken as part of future study phases.

## 1.3 Scope and Limitations

At the time of preparing the Geotechnical Interpretive Report the exact footprint, nature, and layout of the proposed port and associated facilities are still being developed. It is also noted that the scope of site investigations undertaken to date, although covering an extensive area, have been undertaken on a relatively wide grid spacing of approximately 500 x 500 m. Consequently the discussions and recommendations provided in this report, interpretation of ground conditions and geotechnical considerations, are by necessity general in nature, and are subject to further review and specific interpretation as studies progress.

It is also noted that the following areas of study, although briefly discussed in this report, are not covered in any detail. Specific detailed studies for these are being undertaken as part of other work packages or work streams:

- Regional groundwater/ hydrogeological assessment
- Environmental assessments
- Construction staging and dredge and reclamation sequencing
- Dredge methodology, dredge volume assessments and dredge materials management
- Reference design (dredging/ reclamation, foundations, seawalls, ground improvement etc.)

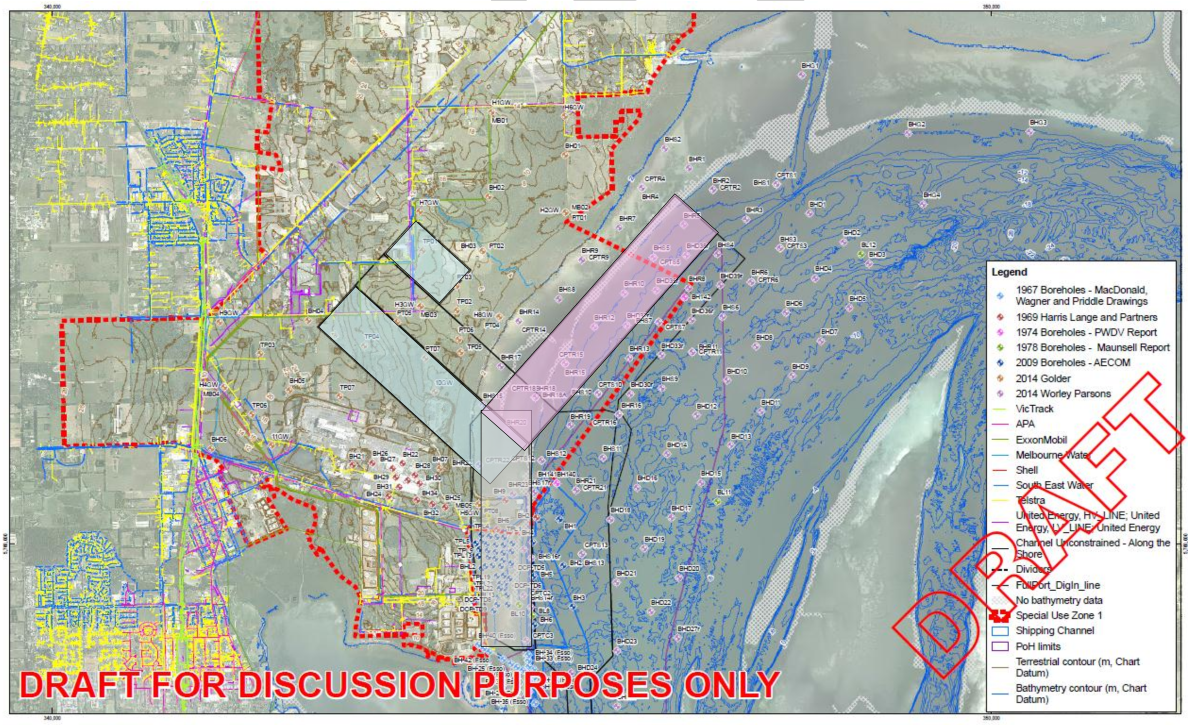
## 2.0 Development Outline

Currently the Port of Hastings Development Authority is exploring a range of potential business opportunities and development options. While project definition is currently being resolved, it is anticipated the Port of Hastings expansion and infrastructure development will include the following activities:

- Dredging of new or deepened shipping channels, berth pockets, swing basins and anchorages
- Beneficial re-use of dredged materials, or disposal to an existing or new dredged material ground inside or outside Western Port
- Construction of wharves and shipping berths, including the potential for land reclamation and provision for significant areas of hard stand
- New terminal facilities, including container stacking areas and equipment
- Port and logistics related developments in the port environs, including road and rail circulation within the port environs
- New land use and development associated with the ongoing operation of the port
- Upgrade of the arterial road network outside the port environs to increase capacity consistent with the increase in trade
- Construction of a new rail line between the port and the existing rail network.

A plan of the Port Area showing the envelope of concept options for terminal footprint and shipping channels is provided in Figure 2.1 for illustrative purposes.

**Figure 2-1 Envelope of concept options for terminal footprint and shipping channels (terminal areas shown shaded)**



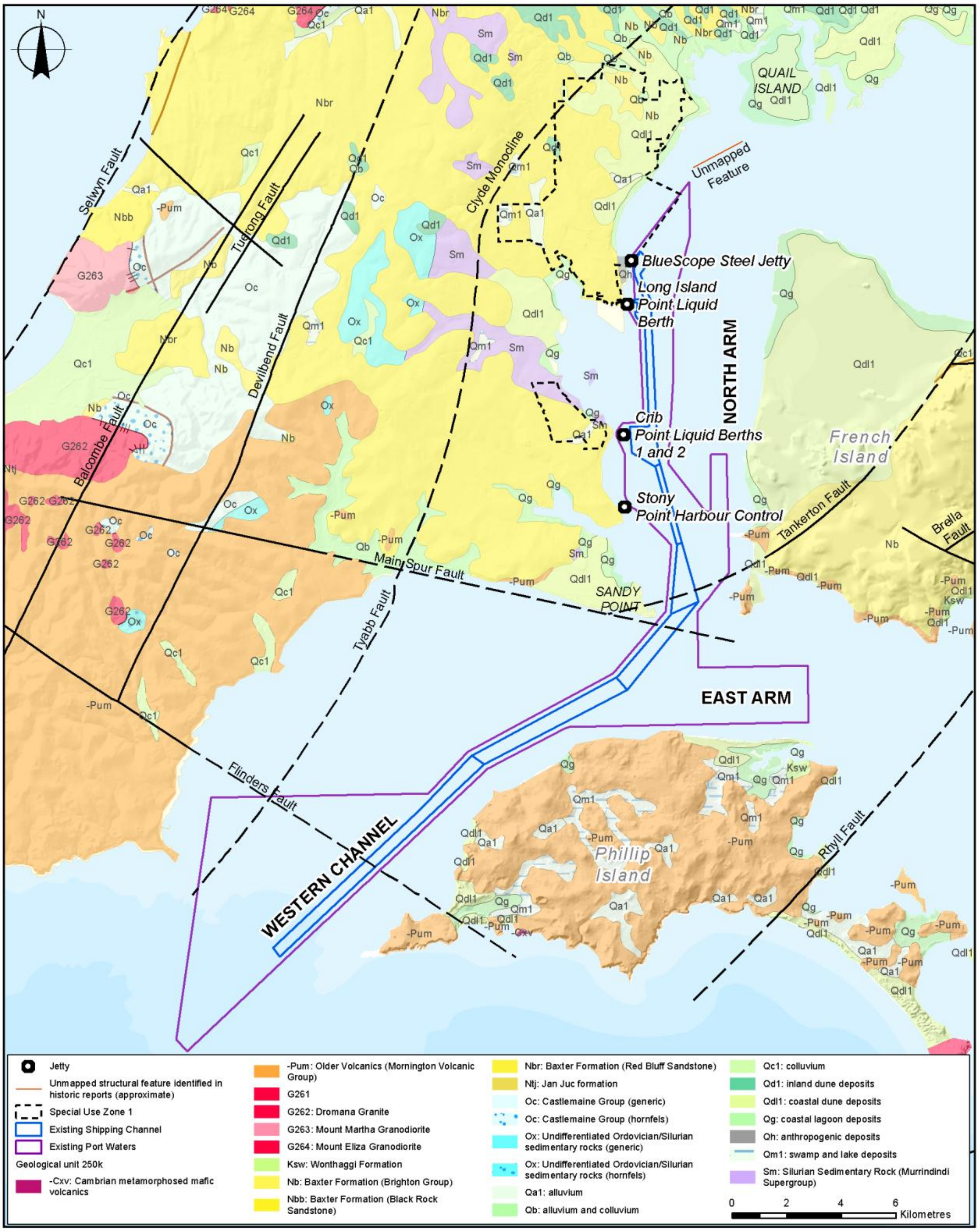
# 3.0 Regional Setting

## 3.1 Regional Geology

Western Port is a marine embayment with a water surface area of 680 $km^2$, of which 270 $km^2$ (approximately 40%) is intertidal. Apart from limited areas of Silurian marine and Lower Cretaceous non-marine sedimentary rock exposure, the geology of the coast is of Cainozoic age. Western Port lies within the Westernport Basin geological feature, which is a complexly faulted and eroded basin.

The surface geology and the location of inferred geological faults in Western Port are shown in Figure 3-1.

**Figure 3-1Surface Geology and Faults of Western Port**

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Data source: VicMap (DEPI, 2014), Port of Hastings Development Authority

### 3.1.1 Stratigraphy

The stratigraphy of the western side of Western Port (which encompasses the entire project area) consists of the following geological units (in order from oldest to youngest):

**Palaeozoic**

Silurian marine sedimentary rock (Murrindindi Supergroup) underlies the Cainozoic deposits and outcrops along the coast at Sandstone Island and in an area north of Crib Point. According to Lakey & Tickell (1978) this strata consists of well bedded and moderately steeply folded pyritic mudstone, siltstone and sandstone.

**Cainozoic**

*Childers Formation*

Fluviatile sediments underlying the Older Volcanics, consisting of sand and gravel interbedded with lignite and clay beds. The Childers Formation is generally confined to the deeper parts of the Westernport Basin, occupying and partly filling the early Cainozoic drainage system. The Childers Formation was not identified in boreholes drilled within the project area.

*Older Volcanics (also known as Mornington Volcanic Group)*

Extensive areas of the bay are underlain by Eocene and Oligocene basalt lava flows. The Older Volcanics outcrop extensively along the southern coast of French Island, the coast of Phillip Island, and the western area of the Western Port coastline south of Merricks Beach. The Older Volcanics strata consist of variably weathered materials ranging from residual soils to fresh basalt with interbedded sedimentary deposits. Drilling by the Ports and Harbours Division of the Victorian Department of Public Works in the main shipping channel showed that the eroded basalt surface falls away southwards from Stony Point (Spencer-Jones 1975). Outcrops of Lower Cretaceous rocks and Older Volcanics on the south coast of French Island indicate that the post-volcanic cover of Tertiary and recent sediment within the main channel are relatively thin. Off Sandy Point the maximum proven thickness of cover consisted of 6m of Quaternary sediments (Barton 1974).

*Yallock Formation - Western Port Group*

Thought to be Oligocene in age, the Yallock Formation overlies unconformably or disconformably the Older Volcanics and the Silurian rock, and does not outcrop at surface within the project area. The fluviatile and paludal deposits consist of sands and gravels interbedded with lignite and carbonaceous clays. The Yallock Formation has in the past been considered part of the Baxter Formation, and the extent is not well documented in the literature.

*Sherwood Formation (also known as Sherwood Marl) - Western Port Group*

The Early to Middle Miocene Sherwood Formation lies disconformably on the Older Volcanics or the Silurian rock, or overlies the Yallock Formation. The Sherwood Formation does not outcrop at surface within the project area, and is generally concentrated in the western half of the bay. It is a marine deposit of calcarenite, marl and shelly sandstones which grades towards the north into near-shore and onshore sand and shelly beach deposits under Koo-wee-rup swamp where the deposit can be up to 65m thick. This unit is described in Lakey and Tickell (1978) as a variable greenish grey marine sedimentary unit of fine grained calcareous clayey sand and silt, which towards the Tyabb area a clayey matrix predominates.

Evidence from channel drilling in the 1970s was interpreted to indicate that the Sherwood Formation pinches out on a basalt palaeo-slope south of Long Island Point and is overlapped to the south by the younger Baxter Formation. The apex of the ridge appears to occur in the vicinity of Stony Point (Barton 1974).

*Baxter Formation (also known as Baxter Sandstone and Red Bluff Sandstone) - Western Port Group*

The fluviatile and marginal marine Baxter Formation was deposited landward of the extent of the Sherwood Formation, and deposition continued extensively across the region during the Upper Miocene following uplift and consequential shoreline regression. The formation disconfomably overlies the Sherwood Formation in deposits up to 30m thick, and elsewhere disconformably overlies the Older Volcanics and unconformably overlies the Silurian rock. The formation consists of clays, sands and gravels that may include thin brown coal

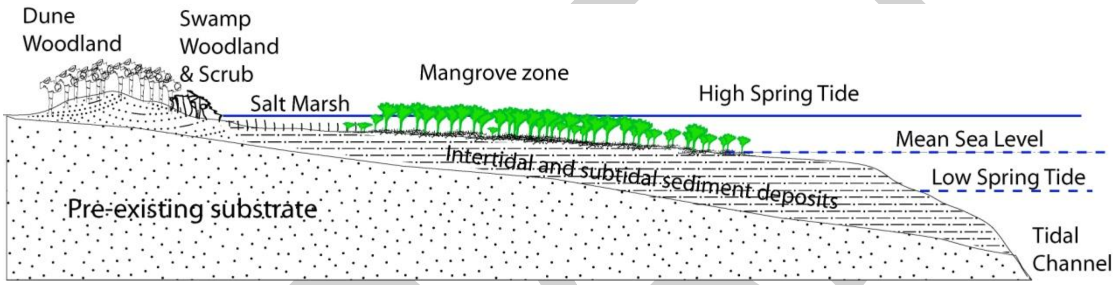
seams and other carbonaceous deposits. These deposits are frequently ferruginous near surface, however this characteristic does not persist at depth.

The depositional environment of the Baxter Formation is likely to result in frequent horizontal and vertical variation in lithology within the unit. Comparison can be made with modern day river systems to assist in the understanding of the environment in which the sediments were laid down, and the potential channel migration and swamp, backswamp, flood and delta deposits etc. that would likely have occurred within the larger scale Cainozoic river systems.

**Quaternary**

Onshore, particularly to the north and west, there are extensive deposits of dune sands that were active during Holocene times of low sea level. The most recent marine submergence occurred in the last 6,000 years during the Holocene post-glacial rise in sea level. The northern and north eastern margin of the bay was an extensive and low-lying freshwater wetland, now almost completely drained for agricultural use. Overlying the Baxter Formation along the coastal strip are Quaternary swamp and lagoon sediments. These sediments often comprise loose sands, and soft silts and clays with organic matter, deposited in the intertidal and sub-tidal zone. A schematic cross section of this depositional environment is shown in Figure 3-2. Quaternary alluvial deposits are present within watercourses onshore.

**Figure 3-2 Typical Western Port near shore depositional environment (after Bird and Barson, 1975).**




### 3.1.2 Geological Structure

The major regional geological structures are presented in Figure 3-1.

Geological structures define the morphology of the bay, with its configuration being broadly determined by northeast trending fault lines that have defined a broad central area of subsidence (known as the Western Port Sunkland, Keble 1950), bounded by the uplifted block of the Mornington Peninsula to the west, and the South Gippsland Hills or Strzelecki Ranges to the east. Tectonic activity has also led to the formation of French and Phillip Islands. Seismic activity has continued into the Quaternary and the area remains seismically active. However, on a global scale the seismic risk is considered to be low (Gibson and Brown, 2003). The Tertiary beds dip at a very low angle to the southeast as a result of movement along the Clyde Monocline and Tyabb Fault which pass across the western part of the peninsula. An additional 'unmapped' northeast trending structure has formerly been detected just off the Tyabb coast by a previous sparker seismic survey. The historical borehole information interpreted across this feature indicated that no major displacement (fault rupture) is involved and thus the structure was inferred to represent a small monocline.

## 3.2 Geomorphology

### 3.2.1 General

The geomorphology of the Westernport area has been described in the 2009 AECOM report – *Port of Hastings Stage One Scoping- Geotechnical and Environmental Investigation Report – M8 – Geomorphology, Geology and Geotechnical,* based on the findings of coastal geomorphological studies by Neville Rosengren in his report entitled "*Geology and geomorphology, Port of Hastings Stage 1 development"* prepared for AECOM, dated 2009. The following summary of geomorphological conditions within Westernport is generally based on this earlier work.

It is noted that further geomorphological studies of the Westernport area are being undertaken as part of the hydrodynamics work stream studies.

### 3.2.2 Western Port: Bathymetry, Tidal and Marine Processes, Sedimentology

The bathymetry of Western Port reflects the dominance of tidal movement and wave and current dispersion around Phillip Island and French Island. The deepest area of the bay is the western entrance channel which reaches 20 metres west of Point Grant. A dredged channel to 14.3 metres at low tide links this entrance to the Long Island Point terminal. Inside the bay, tidal ebb and flood movement has developed long narrow channels from 5 to 10 metres deep alternating with shallow sandy and muddy shoals and drying banks. A distinctive and unusual feature of the bay is the broad area of intertidal mudflat west of Lang Lang that at low tide forms a tidal divide between two sets of dendritic tidal and sub tidal channels. Spring tide range in the northern parts of the bay is in excess of 3 metres.

Sediment in the bay is dominated by sand in the entrances and broader channels with high concentrations of mud in the smaller embayments and at the head of the tidal channels. There is often considerable local variation in sediment type and stratigraphic (vertical) variation is common. Locally abundant accumulations of shell beds occur. Sediment movement is complex with ebb-flow and flood-flow differentiation.

### 3.2.3 Western Port: Coastal Geomorphology and Processes

A variety of coastlines occur around the bay including active rock and earth cliffs, shore platforms, inactive cliffs and bluffs, sandy beaches and mangrove and salt marsh fringed shores. Active cliffs contribute sediment to shorelines as does discharge from the drains across the former Dalmore, Koo Wee Rup and Tobin Yallock swamps. Removal of mangroves has accelerated the rate of shoreline erosion in some areas, although in some parts of the bay there is vigorous mangrove regeneration. Shallow water caused by ebbing tides allows mobilization of sandy and shelly sediments.

### 3.2.4 Long Island Point and Hastings Bight: Geology and Geomorphology

Long Island Point forms the northern edge of the shallow embayment of Hastings Bight. The southern edge is the headland at Crib Point. Sandstone Island is a low ridge of Silurian sedimentary rock surrounded by tidal channels and sand and mud flats. This is the only area of Western Port where there is Palaeozoic rock outcrop. Late Quaternary sediments of tidal and nearshore origin form the surface and shallow sub-surface cover.

### 3.2.5 Long Island Point and Hastings Bight: Coastal and Nearshore Processes

Hastings Bight is a sheltered embayment with an extensive (although remnant) mangrove and salt marsh. The tide range of almost 3.0 metres exposes broad areas of intertidal flat at low tide, crossed by a network of tributary and main tidal channels. The western edge of North Arm tidal channel and the dredged channel have a steep slope falling from the intertidal flat north of Long Island Point. Strong ebb and flood tide currents and local wave action spilling from the deeper water channel have the capacity to move coarse-grained sediment across the tidal flats forming elongate banks of sand and shell. Mangroves were once more extensive along this coastline but have been reduced in area due to cutting for boat access, reclamation of the intertidal zone for port and industrial expansion, and movement of sand across formerly muddy substrate.

### 3.2.6 Long Island Point and Hastings Bight: Historical Changes

European settlement has had substantial impacts on much of the shoreline of Western Port, including the Long Island Point and BlueScope Steel area under consideration in this report. Three broad groups of direct change can be identified:

a) Removal of mangrove and salt marsh, in places as a resource for soda ash but more often to allow shoreline access;

b) Alteration of foreshore and nearshore topography by construction of jetties and mooring sites, by covering tidal flats with fill to use as building or hardstand sites, and by dredging areas to allow shipping access;

c) By altering hinterland vegetation and hydrology by clearing and especially by draining former swampland and replacing natural channels with excavated drains.

These direct impacts have often triggered secondary impacts that change the source, movement and deposition of sediments on the shoreline as well as in the intertidal and subtidal zones.

### 3.2.7 Tyabb Tidal Flats and Banks

This is the marginal but narrow unit running northwards from Long Island to Quail Island, and enlarging where it extends into the entrance of Watsons Inlet. The coastline is low and inclines a broad salt marsh zone with mangrove fringe of variable width and continuity. In front of the mangroves there is a wide and predominantly muddy intertidal flat with poorly developed tidal channels (DEPI, 2014). The intertidal surface is locally sand covered with ridges and sheets of sand and shell. The orientation of the intertidal ridges reflects their formation by wave and tidal current action during ebb and flood tides.

Gaps have been cut in the mangrove fringe in a number of locations to permit boat access including the boat landing at Denhams Road (Bird, 1975). At Yaringa a canal has been cut across the salt marsh, through the mangrove fringe, and out across the tidal mudflats to access the deeper channel which opens southwards from Watson Inlet, with dredged material having been dumped on the adjacent salt marsh. A first attempt to cut such a channel approximately 400 m to the south as shown in Figure 3-3 was abandoned when it was realized that the mudflats off this sector consisted of deep, soft mud, whereas further north an extensive underlying shell bed offered a firmer substrate for canal extension offshore. Mangroves have since recolonized the mouth of the first canal (Bird, 1975). Mangroves and saltmarsh in the Port Area are shown on Figure 3-4 and were mapped between 2008 and 2011 (Geoscience Australia, 2014). Figure 3-4 also shows three rivers or streams entering Western Port north of the BlueScope Steel reclamation.

**Figure 3-3 Existing channel to Yaringa Marina and historical abandoned channel**

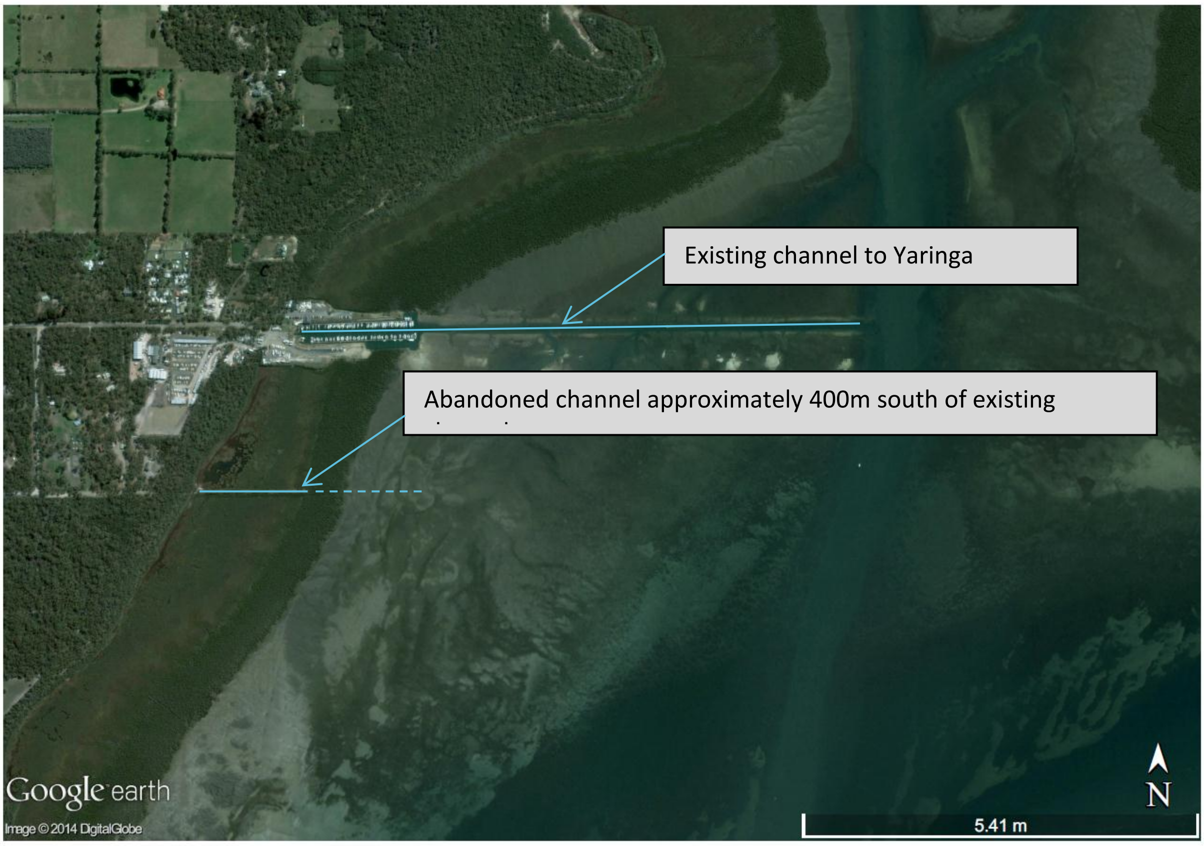
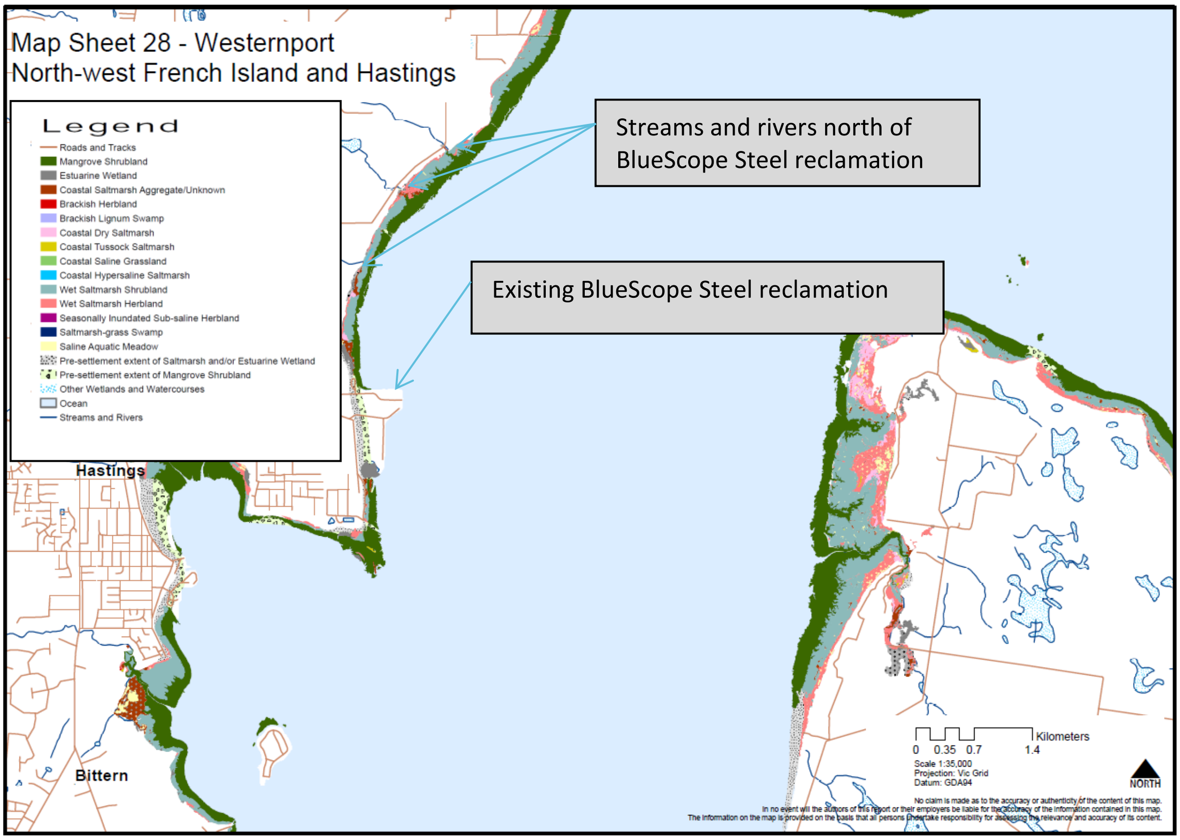


**Figure 3-4 Mangrove and salt marsh mapping**




## 3.3 Seismicity

### 3.3.1 Hastings Structural Settings

Hastings is within the southwest of the Melbourne Zone of the global Victorian structural zones. The Melbourne Zone is comprised of a thick, unbroken sequence (9 – 10 km) of early Silurian to Middle Devonian sedimentary rocks (Murrindindi Supergroup) that overly Ordovician shales and turbidites. The basement rocks are Cambrian volcanic and volcanoclastic rocks of the Selwyn Block.

The predominant structural features in the Hastings area are northeast trending folds and faults that are a result of east-west shortening that occurred in the Middle Devonian, subsequent to the deposition of the Murrindindi Supergroup sequence. Faulting is mainly bedding-parallel. A lesser period of north-south shortening that occurred around the same time led to a number of east-west trending faults and interference folds in the region.

The regional geological map (Figure 3-1) shows the following faults in the Hastings area:

- Tyabb Fault (northeast)
- Devil Bend Fault (northeast)
- Clyde Monocline (northeast)
- Tuerong Fault (northeast)
- Balcombe Fault (northeast)
- Tankerton Fault (northeast)
- Selwyn Fault (northeast)
- Main Spur Fault (eastwest)

The most significant of the these faults with proven recorded recent activity is the Selwyn Fault that runs along the eastern edge of Port Philip and is estimated to be approximately 50 km long. The Selwyn Fault has been active since the Late Cambrian and is still showing recent displacement. It has been estimated (Janssen 2001) that the current displacement along the Selwyn Fault is around 50 m / Ma. The Port of Hastings area is located at approximately 16 km South East from the Selwyn Fault.

The Selwyn Fault trace is associated with the prominent linear border forming the eastern side of Port Phillip, and together with the Rowsley and Bellarine Faults, has accommodated more than 1000 m of Late Tertiary and Quaternary sedimentary deposition in the Port Phillip Basin (Holdgate et al. 2001, 2002). The Victorian Geological Survey maps the fault as 97 km in length (Vandenberg, 1997), although onshore geomorphic expression is limited to about 55 km. A marine seismic reflection survey across the fault south of Nepean Peninsula (Dickinson et al. 2002) indicates a moderate easterly dip (~50 degrees). Closely spaced boreholes on the Nepean Peninsula near to Rosebud (Wannaeue 36 and Wannaeue 38) indicate ~16.5 m of east side up displacement across the base of the Pleistocene Bridgewater formation, which may be as old as 800,000 years (Guy Holdgate, Melbourne University, pers. comm. 2012). This estimate is commensurate with the estimate of 20 m across the base of the Pleistocene by Mallet & Holdgate (1985). In contrast to the Bridgewater Formation, which thickens by <3 m across the fault at this location, the Pliocene Wannaeue Formation thickens by ~30 m (97 ft) across the fault (Guy Holdgate, Melbourne University, pers. comm. 2012). Purportedly last interglacial beach deposits are reported at an elevation of 7 m above present sea level at Anthony's Nose, Dromana (Hills 1975; Janssen 2001). If proven, this suggests that 5 m of the 16.5 m of Pleistocene uplift has occurred in the last 120 ka. Janssen (2001) also report apparent reverse reactivation and folding associated with a westerly dipping normal structure called the Fossil Cove Fault (see also Gostin 1966), which might be expected to be part of the Selwyn Fault system.

### 3.3.2 Regional Seismicity

The Indo-Australian tectonic plate is moving north - north-eastwards at approximately 10 cm/year and is presently colliding with the Euro Asian and Pacific plate in the areas of the Indonesian arc, New Guinea and New Zealand. This process generates enormous compression forces inside the crustal rock masses of the collision zones. These pressures are transferring laterally and affect the sensitive stress balance of currently seismically inactive zones. The relative shallow depths characteristics of some Australian earthquakes might be associated with these compression effects.

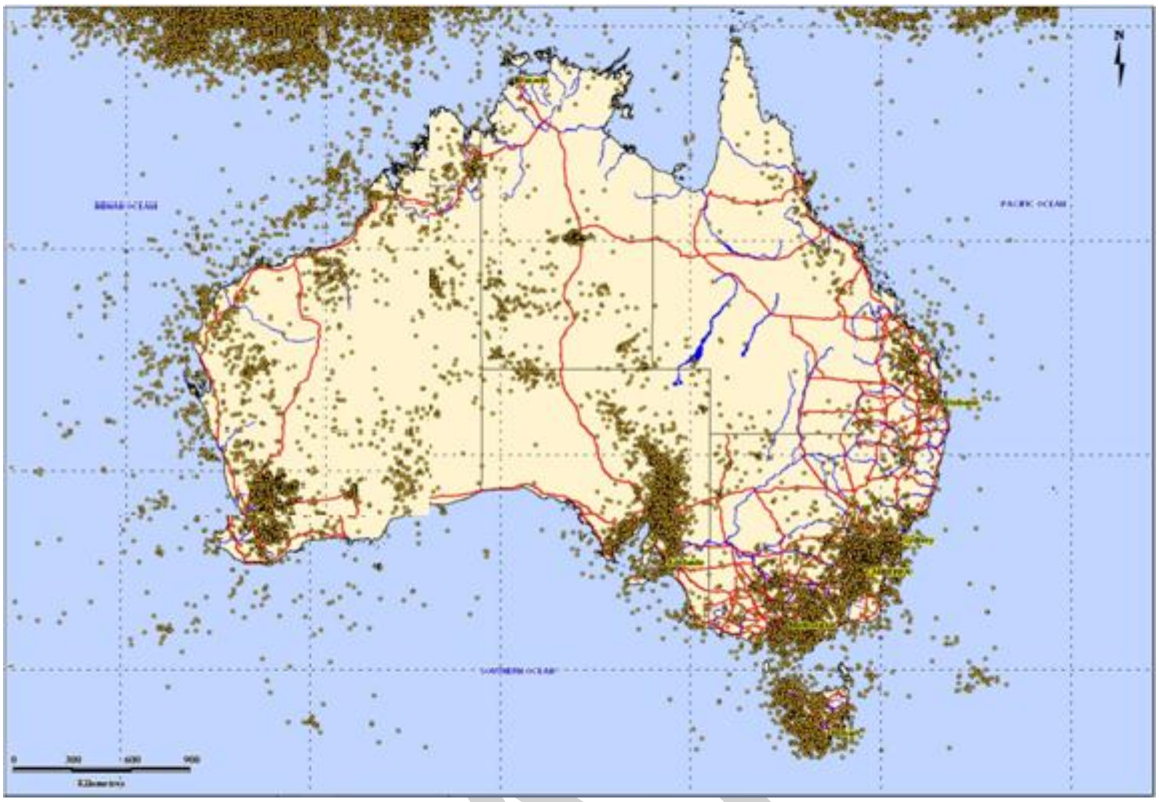


**Figure 3-5 Regional Seismicity of Australia (Source: Geoscience Australia earthquake record - 2014)**

The regional historic seismicity that includes all events recorded by the Australian seismological network (mostly comprised of earthquake records obtained from Geoscience Australia Earthquake Database) is presented in Figure 3-5. The figure includes all recorded earthquakes with a moment magnitude (Mw) range from Mw 0 – 7.0.

For the region of Victoria, however, very large seismic events with creation of large slip planes, is highly unlikely.

### 3.3.3 Local Seismicity

The Port of Hastings area is relatively remote from the above mentioned impact regions, and there has been limited recorded seismic activity of relatively moderate seismicity in the area since seismic recording started (Table 3-1).

Recorded seismicity data is provided from publicly available information sets recorded by regional seismic networks and from relatively recently installed local seismic networks capable of recording weak earthquakes.

The complex geotectonic setting (Section 3.3) of the area does indicate a possibility of relatively recent (geologically) activity in the region.

The current seismological network consisting of Geoscience Australia (GA) seismic stations in the area close to the Hastings site is capable of accurately locating weak earthquakes. This level of detection has been achieved relatively recently.

The contemporary methods of seismic hazard calculation can accommodate the uncertainties caused by gaps in data (less certain data records) with an appropriate activity determination that is adjusted for the cases of unequal period of completeness (Wiechert, 1980).

The strongest earthquake within the potentially "effective" distance from the Hastings site was the earthquake that occurred on 06/07/1971. The event was characterized with a Richter local magnitude (ML) of ML 5. The earthquake was located ~14.3 km to the south west of Hastings. (Source: Geoscience Australia earthquake

record– 2014). The "effective" distance is related to the relative epicentral distance of a certain earthquake that can cause damage to a structure on site. This is also related to the earthquake depth and magnitude.

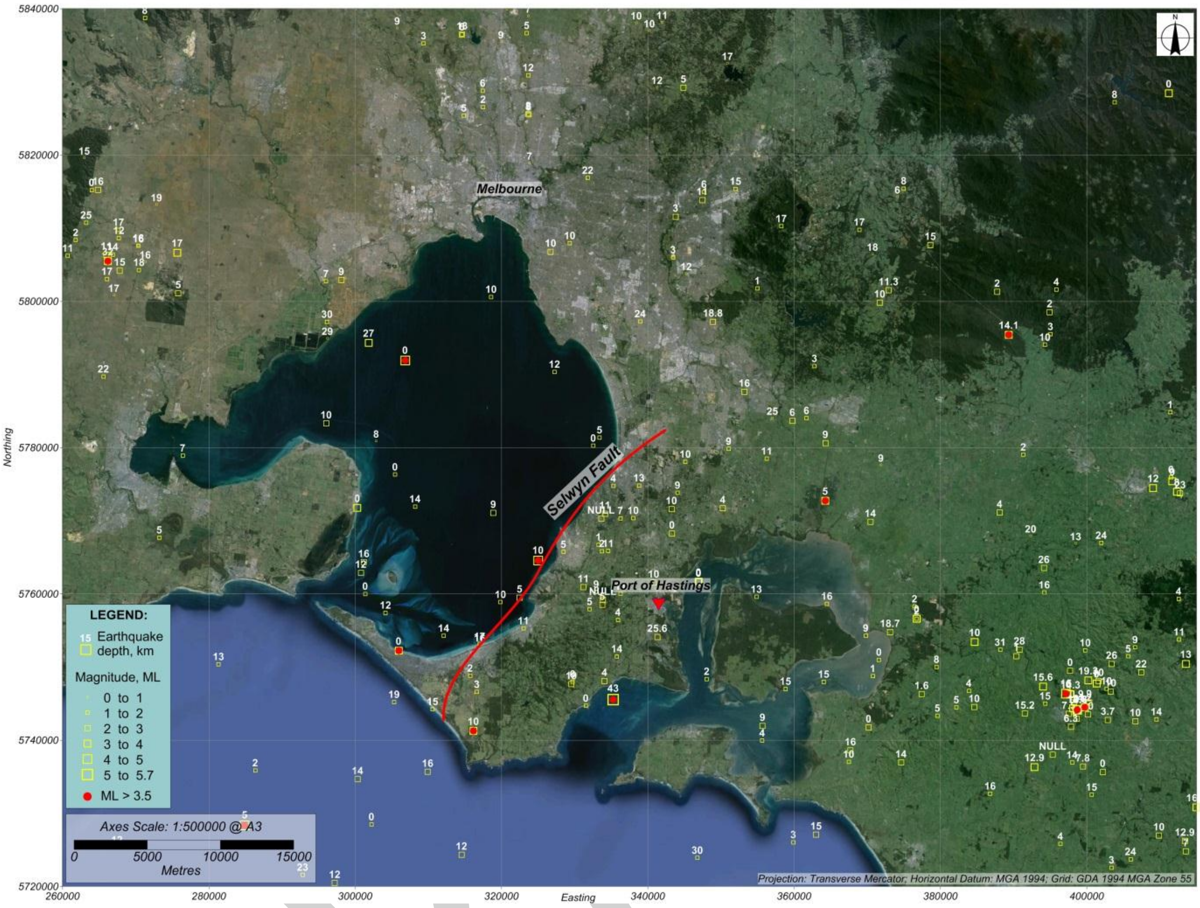



**Figure 3-6** **Local Seismicity of Hastings Area (Source: Geoscience Australia earthquake record - 2014) with indicated position of the Selwyn Fault (the number above earthquake symbols indicates depth of earthquake in km; the red earthquake symbols indicate earthquakes with ML>3.5).**

Earthquakes with ML>3.5 recorded in the area inside the 100 km radius from the Hastings are presented in Table 3-1 below:

**Table 3-1** **Earthquakes with ML>3.5 in 100 km Radius from Hastings (sourced from Geoscience Australia)**

| Date | Time (UTC) | E GDA 94; Zone 55 | N GDA 94; Zone 55 | Distance from Hastings, km | Depth, km | Magnitude, ML |
|---|---|---|---|---|---|---|
| 6/07/1971 | 21:55:00 | 342034.7 | 5758161 | 14.30 | 43 | 5 |
| 2/09/1932 | 18:22:30 | 342034.7 | 5758161 | 18.20 | 10 | 4.5 |
| 21/08/2001 | 23:13:03 | 342034.7 | 5758161 | 26.57 | 5 | 3.6 |
| 20/01/1997 | 11:03:21 | 342034.7 | 5758161 | 30.91 | 10 | 3.5 |
| 27/09/1998 | 22:32:08 | 342034.7 | 5758161 | 36.58 | 0 | 3.8 |
| 22/12/1902 | 12:45:00 | 342034.7 | 5758161 | 48.76 | 0 | 4 |
| 17/02/1973 | 8:56:24 | 342034.7 | 5758161 | 55.97 | 20 | 5 |
| 18/03/2009 | 5:28:17 | 342034.7 | 5758161 | 56.38 | 15 | 4.6 |
| 6/03/2009 | 9:55:36 | 342034.7 | 5758161 | 58.35 | 14.8 | 4.6 |
| 12/01/2009 | 8:48:42 | 342034.7 | 5758161 | 59.28 | 4.2 | 3.5 |
| 8/03/2007 | 2:34:11 | 342034.7 | 5758161 | 60.23 | 14.1 | 3.5 |
| 1/06/1961 | 9:53:08 | 342034.7 | 5758161 | 64.44 | 5 | 4 |

| Date | Time (UTC) | E GDA 94; Zone 55 | N GDA 94; Zone 55 | Distance from Hastings, km | Depth, km | Magnitude, ML |
|---|---|---|---|---|---|---|
| 5/02/2009 | 4:04:58 | 342034.7 | 5758161 | 81.60 | 0 | 3.5 |
| 25/08/1978 | 20:23:15 | 342034.7 | 5758161 | 87.11 | 2 | 3.7 |
| 29/12/2000 | 23:39:45 | 342034.7 | 5758161 | 87.12 | 15 | 3.6 |
| 24/09/2002 | 7:42:57 | 342034.7 | 5758161 | 87.25 | 3.4 | 4 |
| 24/09/2002 | 13:39:27 | 342034.7 | 5758161 | 88.51 | 5 | 3.5 |
| 14/09/1965 | 1:25:31 | 342034.7 | 5758161 | 89.10 | 10 | 5.7 |
| 22/07/1978 | 0:37:49 | 342034.7 | 5758161 | 89.40 | 32 | 3.5 |
| 6/01/1965 | 19:44:27 | 342034.7 | 5758161 | 89.61 | 10 | 3.5 |
| 31/05/1970 | 1:50:01 | 342034.7 | 5758161 | 92.61 | 0 | 4.1 |
| 1/06/1961 | 9:35:36 | 342034.7 | 5758161 | 94.61 | 0 | 4.1 |
| 22/10/1984 | 0:04:30 | 342034.7 | 5758161 | 94.72 | 15 | 3.5 |
| 22/06/1969 | 16:37:43 | 342034.7 | 5758161 | 94.73 | 0 | 4 |
| 20/10/1984 | 5:16:21 | 342034.7 | 5758161 | 95.71 | 14 | 4.3 |
| 2/12/1977 | 13:32:33 | 342034.7 | 5758161 | 95.96 | 16 | 4.7 |
| 29/08/2000 | 12:05:52 | 342034.7 | 5758161 | 96.53 | 18 | 5 |
| 14/09/1965 | 12:34:36 | 342034.7 | 5758161 | 96.82 | 10 | 5 |
| 14/09/1965 | 1:24:45 | 342034.7 | 5758161 | 96.82 | 10 | 3.5 |
| 20/06/1969 | 11:15:28 | 342034.7 | 5758161 | 98.27 | 19 | 5.3 |

*Universal Coordinated Time - Temps Universel Coordonné (UTC).

From Table 3-1 it is obvious that there is a data gap between the strong historic earthquakes, which have been recorded and characterized, based on published isoseismic maps (maps of the earthquakes intensity) and relatively smaller earthquakes coinciding with the increased level of the instrument coverage and the quality of the records collected in the area.

A seismic hazard map representing the seismic hazard factor (Z) for NSW, Victoria and Tasmania which is equivalent to the earthquake with an annual exceedance probability of 1 in 500 is presented in Figure 3-7. The seismic hazard map, sourced from AS 1170.4 -2007 predicts a hazard factor (Z) of between 0.09 and 0.1 for the Hastings area, which is equivalent to a peak ground acceleration of 0.09 to 0.1 g.

The National Seismic Hazard Map of Australia, published in 2012 by Geoscience Australia, estimates the peak ground acceleration from an earthquake with an annual exceedance probability of 1 in 500 in the Hastings area is approximately 0.07. These revised values have not been incorporated into design standards to date.

Design of new structures will need to consider seismic loadings in additional to other normal in service loadings. For design, seismic loadings are typically applied as a peak horizontal ground acceleration force, the magnitude of which is heavily reliant on earthquake magnitude (as a function of return period) and local geology influences. The site specific conditions of the soil and rock materials which underlie the proposed Port facility will affect the duration and amplitude of the resultant earthquake forces and hence the final horizontal ground acceleration force felt at surface. Properties such as degree of saturation and density, as well as the thickness and layering within the subsurface can significantly influence the final earthquake forces felt at ground surface.

At the Port of Hastings detailed seismic studies and testing are yet to be completed and therefore in this report the above local seismicity information has been presented for the preliminary purposes of informing any future studies. For later design stages (e.g. detailed design) it will be important to understand how an earthquake is affected by local site conditions, especially in terms of resonance or damping of the seismic wave front as it travels from the bedrock layer to the surface through any intervening soil layers. Site specific seismic investigations (S-wave& P-wave surveys) used to gather soil parameters such as shear modulus, density, etc. should be considered to inform future design stages of the project.

In the absence of site specific seismic design parameters it is proposed that preliminary design for most port structures can be addressed through application the Australian Standard (AS 1170.4 -2007) and refined at a later date.

For the design of structures covered by AS 1170.4 -2007 a Z value of 0.1 may be adopted. In most cases for onshore development a site sub-soil class of $C_e$ may be adopted, based on borehole findings. In some areas, such as sites where significant depths of fill are located (e.g. Old Tyabb reclamation area) the site sub-soil class may fall within Class $D_{e.}$

For structures that fall outside the scope of AS 1170.4 -2007 (for example offshore structures, bunds) a site specific seismic hazard assessment should be carried out to determine design parameters, taking into account the importance level of the structure.

Another important aspect with regard to seismic activity in the vicinity of the project is the potential for liquefaction of soils during a seismic event. Materials such as very loose and loose fine sands and silts are typically more prone to liquefaction. Detailed assessment of liquefaction would therefore need to be undertaken as part of design studies based on port layout, materials and method of placement reclamation fills, and level of risk to be accepted for various structures and facilities. However, it is anticipated that ground improvement and densification of reclamation sand fill and near surface in situ loose granular soils would need to be considered to reduce risks of damage to important infrastructure such as wharf structures and associated critical facilities.

AS 1170.4—2007 20

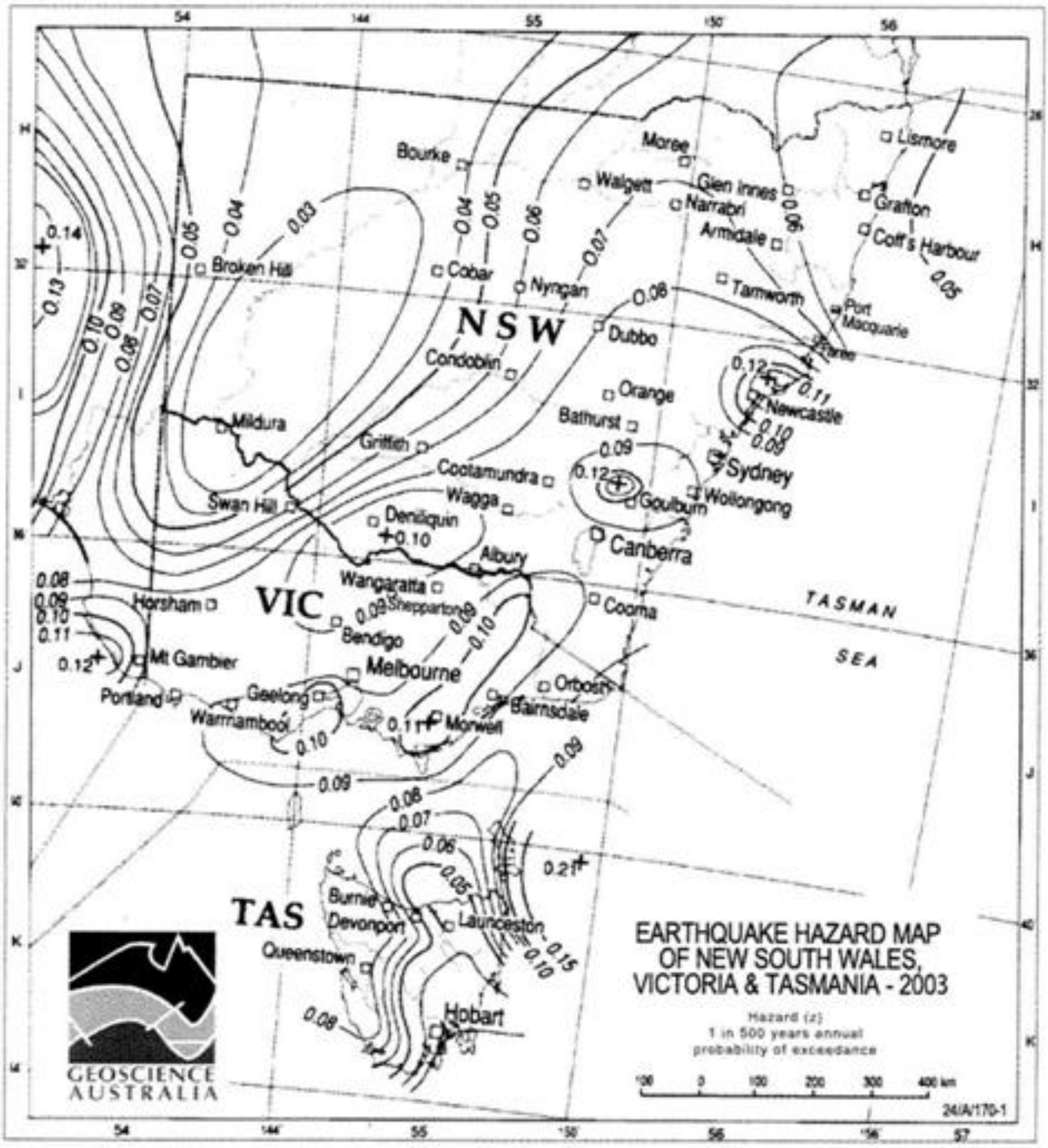



FIGURE 3.2(A) HAZARD FACTOR (Z) FOR NEW SOUTH WALES, VICTORIA AND TASMANIA

**Figure 3-7** **Seismic Hazard Map for NSW, Victoria and Tasmania (AS 1170.4 – 2007)**

# 4.0 Previous Site Development

## 4.1 General

This section summarises the existing conditions within the port development area.

It is noted that a detailed study of historical land use development has not been undertaken as part of the geotechnical investigation works. It is understood that such investigations are to be undertaken as part of separate environmental works package studies. The purpose of the following discussions is to simply provide a general description of land use infrastructure development, and change in land form, with the expectation that during development some level of ground disturbance would have occurred, ranging in scale from larger dredging and reclamation operations, to local excavation and filling to construct buildings, road and rail infrastructure, and buried services.

## 4.2 The Port

The Port of Hastings is an operating commercial port serving international and domestic shipping. The port handles import and export of crude oil, LPG, ULP, general cargo, project cargo, ship to ship transfer, pipe laying operations and the lay-up/repair of oil rigs/floating platforms. However the current Port facilities do not handle containerised freight. The current port has facilities to handle steel.

The existing Port consists of three precincts – Long Island Point, Crib Point, and Stony Point Jetties. Studies completed to date have focused on port expansion activities in the area to the north of Long Island Point and in and around the 3000 hectares of land north of Hastings surrounding the Port zoned for Port related uses – the Special Use Zone (SUZ1).

Within the Port environs, land is utilised by BlueScope Steel, the Esso-BHP Billiton facilities, and other land owners and tenants. The existing shipping channels are owned and managed by the Victorian Regional Channels Authority.

Further details are contained in the Port of Hastings 2013 Description Report.

## 4.3 Shipping Channels and Previous Dredging

The shipping channel to the Port of Hastings consists of a two-way channel from the west of Phillip Island to Sandy Point and a one way channel from north of Sandy Point to the existing berths. Channel sections, approaches, swing basin, anchorage and berth pocket outlines are shown in Figure 4-1.

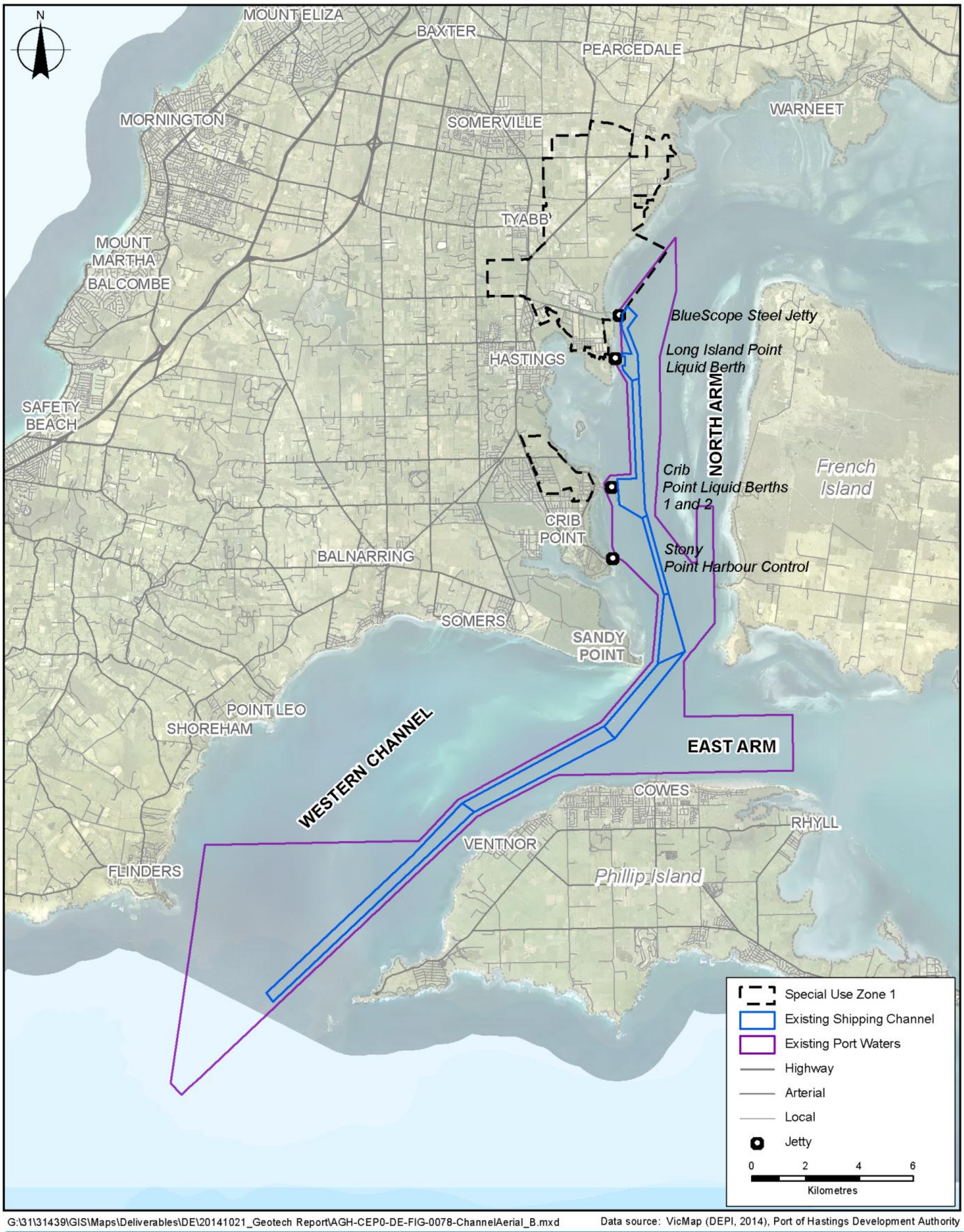



**Figure 4-1. Port of Hastings Approach Channel**

Initial Channel dredging works were completed in the 1960s and 1970s in the general Western Port area including Stony Point, Crib Point and at Long Island Point. Dredging was carried out in a number of stages as follows (Atkinson, 1981):

- Stage 1 – up to and including Crib Point (1964-65)
- Stage 2 - from Crib Point to Long Island (1968-70)
- Stage 3 – at Long Island Point for the Steelworks Berth (1971-72)

The material dredged is reported to be widely varied and to have consisted of silt, sandy clay, and clay sometimes stiff and, in isolated spots, boulders of varying sizes. Dredging to the south of Sandy Point was confined to some isolated high spots while dredging to the north of Sandy Point was more extensive, particularly in the swing basins (Atkinson, 1981). Dredging equipment included cutter suction dredgers, bucket dredgers and grab-dredger. Drilling and blasting of rock is also reported to have been undertaken at Stony Point (Evers, 2009).

## 4.4 Port Terminal Infrastructure

This section summarises the terminal infrastructure at the existing Long Island Precinct. Further information is included in the Port of Hastings 2013 Descriptive Report.

The Long Island Precinct is an extensive area of largely rural coastal hinterland situated between the towns of Hastings, Tyabb and Somerville. The precinct includes the existing Steel Works Wharf, owned by BlueScope Steel, and the Long Island Point Jetty, used by Esso for the export of LPG and until recently crude oil. The precinct also includes a heavy industrial estate and the Old Tyabb Reclamation.

Within the BlueScope Steel site there are two berths owned by BlueScope Steel and operated by Patrick Western Port Stevedores. Berth 1 is used for quarter stern ramp Ro-Ro vessels up to 16,000t. Berth 2 is a conventional general cargo wharf, 152m long.

Based predominately on an overview of the aerial photographs, there appears to be a progressive change of land use in the area around Hastings Bight. In particular, there has been intense development commencing from the late 1960s with changes in land use from orchards, horticulture and grazing to residential and industrial. In addition to changes in land use, the coastal morphology of the area has also changed and includes modification to the shore profile, from construction of sea defences, construction of port facilities for ESSO and BlueScope Steel, and land reclamation.

Figure 4-2, Figure 4-3, Figure 4-4 and Figure 4-5 provide visual representations of the following site developments:

1) A 1971 oblique aerial photograph showing apparent land based reclamation of the BlueScope Steel wharf site. Note from Figure 4.3 it is evident that the Old Tyabb reclamation was not present at that time.
2) Copy of a 1962 aerial photography with superimposed area bounded by present day reclamation fill and existing port development facilities (shown in red)
3) A 2009 oblique aerial photograph showing the extent of existing reclamation filling, and
4) A 2007 aerial image showing the reclamation area and interpreted local geomorphological processes.

It can be seen from the sequence of photographs, existing reclamation fill covers the near linear salt marsh, mangrove, and beach area that pre-existed at the site up until the 1960s.


**Figure 4-2 –Aerial oblique photograph taken in 1971 showing land based reclamation of the BlueScope Steel site**

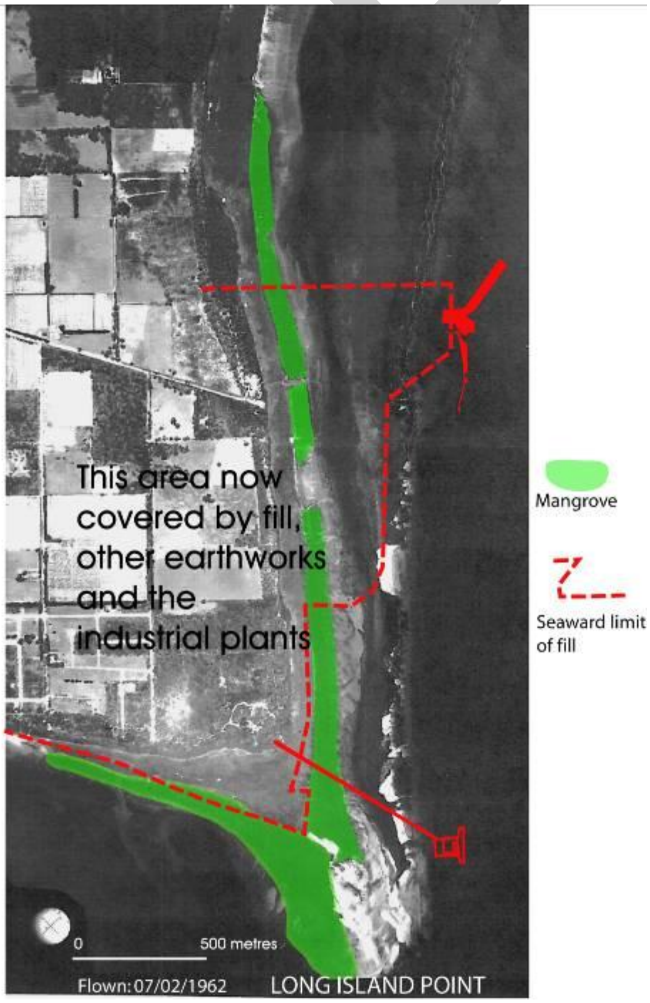



**Figure 4-3 – Vertical aerial photograph taken Feb. 1962 with overlay showing the extent of coastal reclamation and existing port facilities**

Figure 4-4 - Aerial oblique photograph of Long Island Point (Neville Rosengren 23/04/2009). The red stipple shows the extent of fill over mangrove and salt marsh

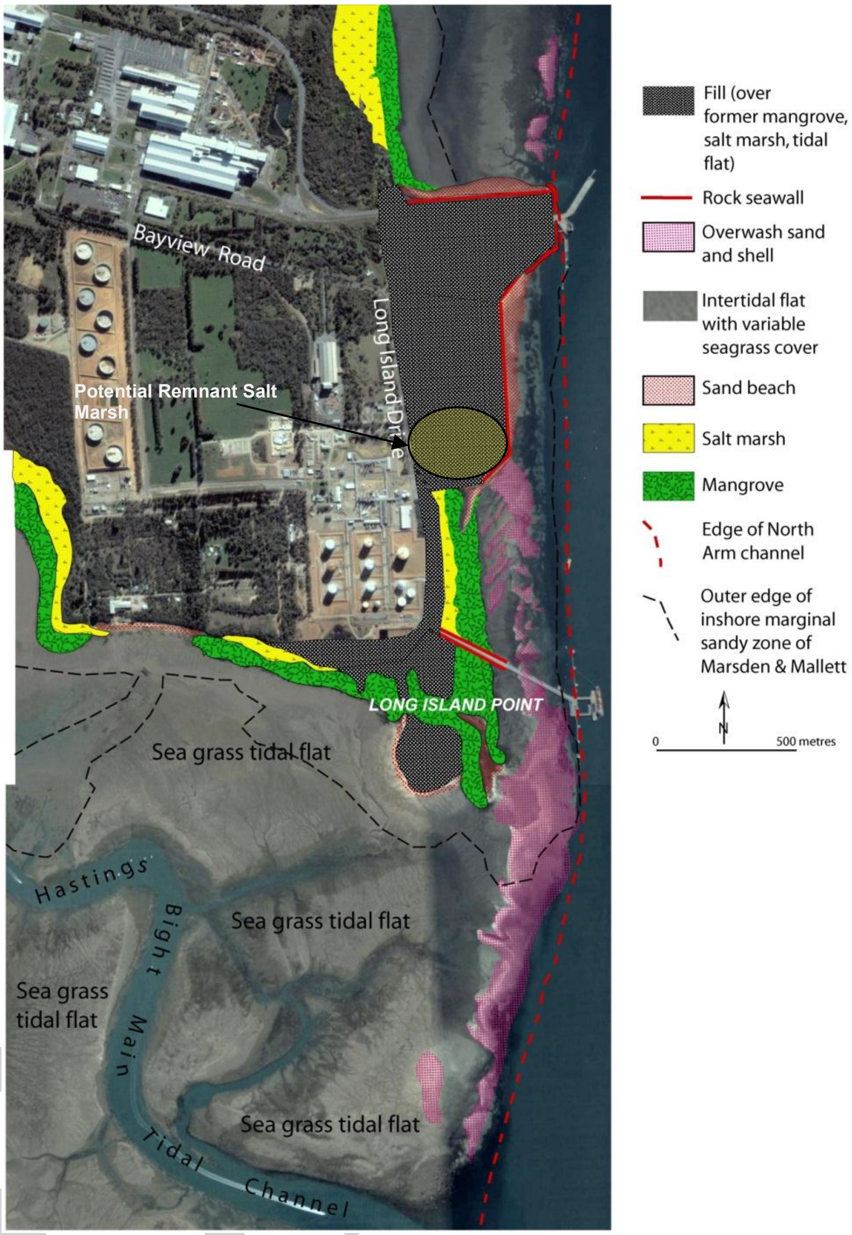



**Figure 4-5 – Coastal and nearshore geomorphology, Long Island Point to BlueScope Steel (2007 image)**

## 4.5 Special Use Zone (SUZ1)

The boundaries of the SUZ1 area and the BlueScope Steel site are shown in Figure 4-6.

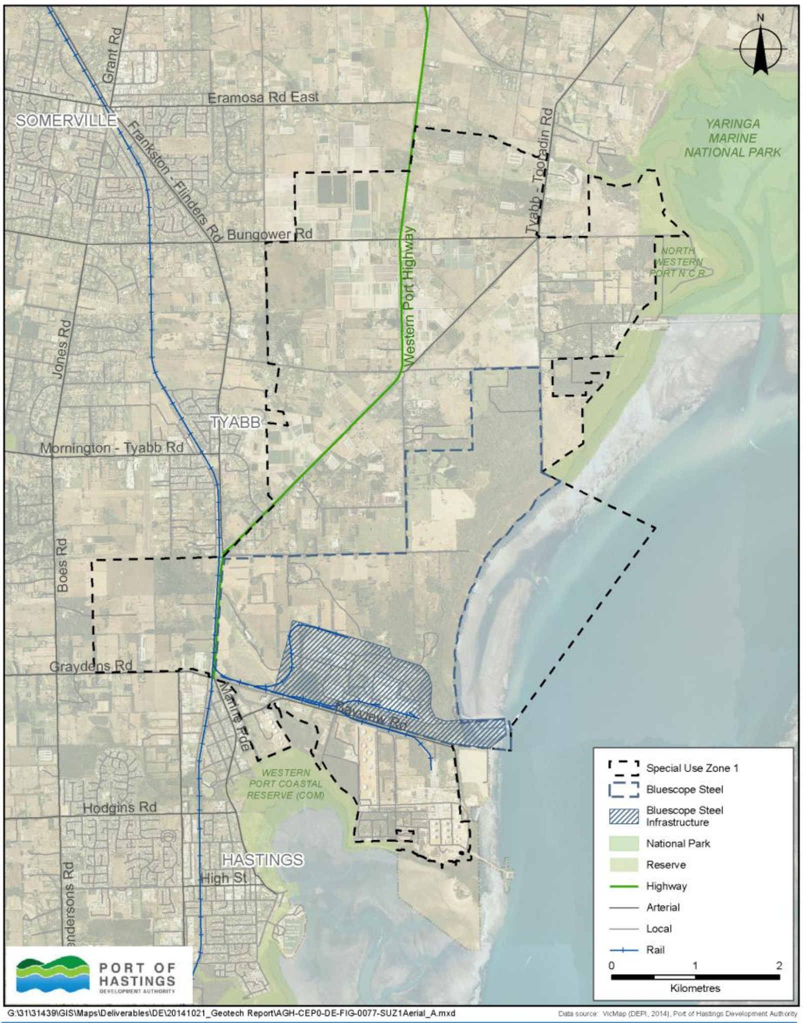



**Figure 4-6. SUZ1 and BlueScope Steel area**

Outside of the Long Island Point port environs and heavy industry areas, land use within the SUZ1 is predominantly pastoral, with local interspersed wooded areas. Coastal areas have not been developed and comprise coastal scrub and bush, salt marsh and mangrove areas.

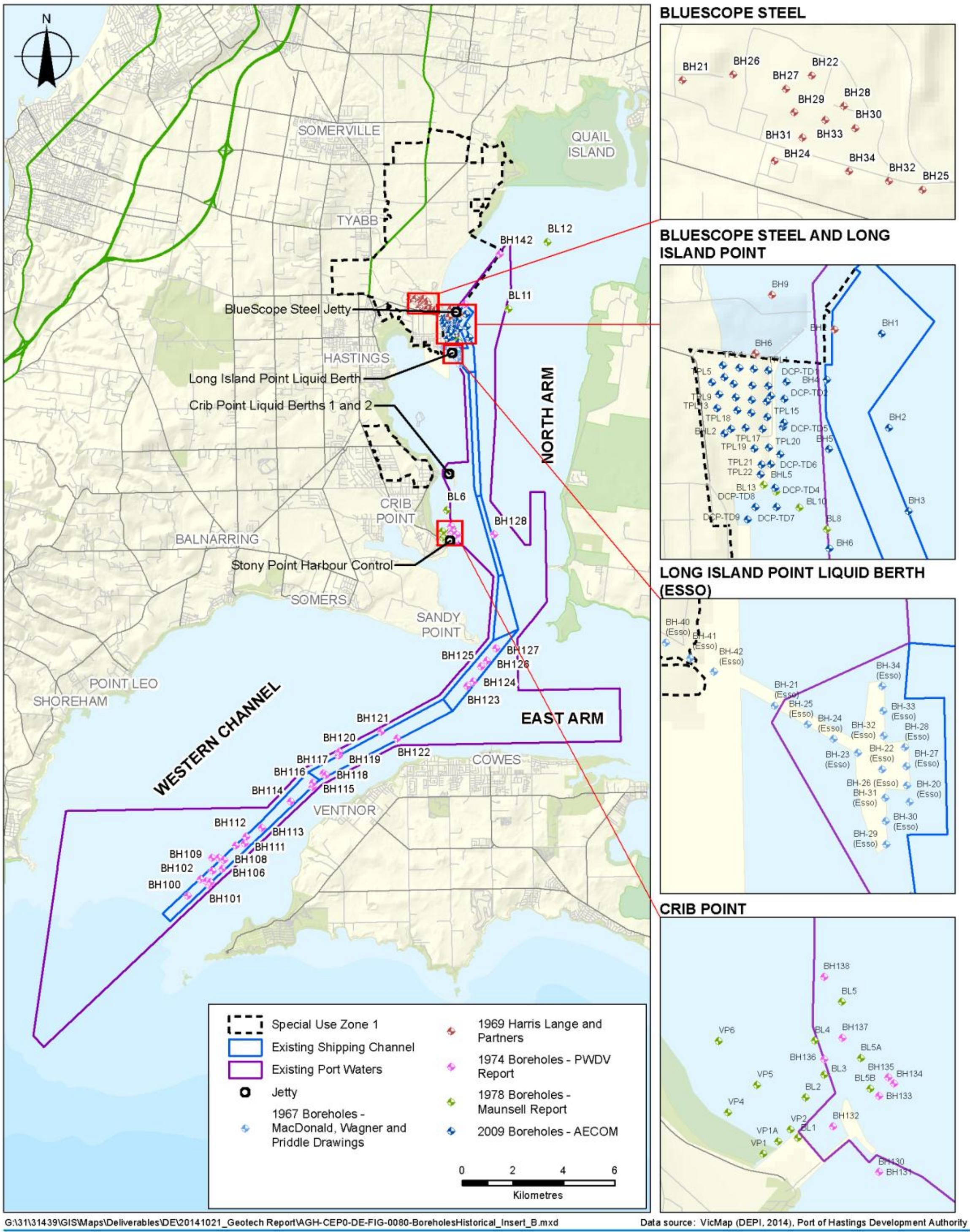
# 5.0 Pre 2013/2014 Site Investigations

Prior to 2013, previous site investigations that have been undertaken in the Hastings area of Western Port include drilling boreholes at:

- ESSO Long Island Point (LIP) wharf;
- BlueScope Steel wharf;
- Widely spaced boreholes between the ESSO and BlueScope Steel wharfs drilled as part of Stage 1 Hastings development investigations;
- Crib Point and Stony Point wharfs;
- Along the existing navigation channel within VRCA waters.

As part of the geotechnical interpretive report a review of the available reports and documentation of geotechnical investigations carried out within the study area has been undertaken. Documents relevant to the study area from which subsurface information has been obtained are given in Section 12.0. The locations of relevant site investigation data are shown in Figure 5-1.

**Figure 5-1 Known existing borehole locations**



## 5.1 Long Island Point

### 5.1.1 Onshore - Existing Reclamation Area / Old Tyabb Reclamation

The 2009 AECOM investigation reported that the soil sequence encountered in the existing reclamation area generally comprised a variable thickness of fill, overlying a limited thickness of very soft to firm clay and very loose to loose sand, overlying more competent stiff clay and medium dense to very dense sand deposits.

The fill thickness varied between 1.3m and 4.0m and generally comprised a mixture of sand and clay, in places observed to include cobble sized lumps of clay in a sand/sandy clay matrix. The basal fill materials more typically comprised very loose to medium dense sand and soft to firm sandy silt/ clay. The cobble sized lumps of clay are interpreted as being remnant “balling” of material from cutter suction dredge operations.

Natural soil materials underlying the fill were typically dark grey to black very loose and loose clayey/silty sand and very soft to firm silts and clays, between 0.9m to 2.5m thick. Soils were noted to contain organic matter, shells and have an organic odour. These materials were inferred to be Quaternary marine lagoon/ swamp deposits, which were left in place beneath the reclamation fill. There was no evidence that these deposits are inter-mixed with the fill materials as a result of the reclamation process.

The total combined thickness of the basal low strength fills and the underlying low strength marine deposits ranged between 2.7m to 4.5m at the borehole locations. At the southern extent of the existing reclamation area, within the ‘salt marsh’ where the ground surface is lower, the fill encountered was thinner (1.3m).

Borehole BL13 in the1978 Maunsell report is located on the south eastern corner of the existing reclamation. Fill material was encountered to a depth of 3.2m overlying silty sand, very stiff sandy clay, very dense clayey sand and stiff silty clay to a depth of 26m. *Mudstone refusal – Not Proven* is noted on the borehole log at 26m depth (RL -21.9mWCD).

The 1970 Harris Lange and Partners Drawing includes two boreholes located at the BlueScope Steel reclamation which were drilled to a depth of around 13m (pre-reclamation). These boreholes encountered clayey sand and silty clay overlying stiff sandy silty clays and medium dense to dense silty sand and sandy silt.

### 5.1.2 Onshore- BlueScope Steel Property

The 1970 Harris Lange and Partners Drawing includes 13 onshore boreholes at the BlueScope Steel site north of Bayview Road. The boreholes were drilled to depths of around 10m and typically encountered stiff to hard sandy silty clay overlying dense to very dense slightly clayey sand.

Several boreholes are reported in the groundwater online database that were drilled within the Terrestrial Project Area and have lithological logs available. These logs are from driller’s notes and tend to contain only basic information.

Four boreholes drilled to install groundwater monitoring bores in the southern part of the area in 1992 to depths of between 19 m and 24 m encountered various thickness beds of sandy clay, clayey sand, sandy silt, silty sand, silt and sand. These materials are interpreted as being Baxter Formation, and possible Sherwood Formation.

Two boreholes were drilled in 1978 at the western end of Denham Road to install state groundwater observation bores. The boreholes encountered a thin layer of sand overlying clay and sandy clay which is interpreted as being Baxter Formation. At depths varying from 10 m to 12 m clay, silty clay, silty marl, marl and clay was encountered which is interpreted as being Sherwood Formation. Brown clay was encountered towards the base of one borehole, terminated at 44 m. Brown coal/clay was encountered at 43 m in the other borehole, becoming grey clay as 49 m. This is interpreted as being Yallock Formation. At 55.5 m the borehole encountered hard sedimentary rock, interpreted as being Silurian rock, and the borehole was terminated at 56 m. A coastal bore drilled in 1919 immediately northeast of the Terrestrial Project Area encountered similar materials, with stratigraphic interpretation inferring likely Baxter Formation materials to 12.8 m overlying Sherwood Formation materials to 47.5 m. Inferred Yallock formation materials consisting of brown coal, sand and hard clay were then encountered with ‘slate’ (inferred Silurian rock) encountered from 53.3 m to end of hole at 56.4 m.

### 5.1.3 Intertidal Area

The 2009 AECOM investigation in the intertidal area was limited to hand held dynamic cone penetration (DCP) testing and indicated very soft/very loose materials to depths of between 0.7m and 2.2m. Hand samples of materials at surface were observed to comprise sands, silts and clays, in places with an overlying layer of shells (up to 0.1m thick), sand and organic matter.

Borehole BL10 in the 1978 Maunsell report is located towards the southern end of the intertidal zone. Very soft dark grey clayey silt materials were encountered from the seabed to a depth of 1.5m. These materials are inferred to be recent marine sediments. Underlying these materials were medium dense to very dense sand and clayey sand, interbedded with silt and clay layers. Depth of rock is not clear on the borehole log but top of rock is expected to be approximately 29.5m depth (RL -38.7mCD).

### 5.1.4 Offshore

The 2009 AECOM investigation materials encountered within the then proposed depth of dredging (RL -9 m to -15m CD) generally comprised a surficial layer of very loose to loose sand/silty sand/ clayey sand, overlying loose to dense silty sand/ clayey sand and firm to hard sandy clay. SPT 'N' values undertaken within these materials ranged from 5 to 42.

Materials encountered below around RL -15.0m CD where highly variable and comprised soft to stiff interbedded layers of sandy clay/ clay, over medium dense to very dense clayey sand /silty sand. Generally lower strength (soft, firm, and loose) materials were encountered to a greater depth towards the south of the site towards the existing Esso facility. The original location for BH6 was abandoned when the barge platform legs reached full extension without encountering strata of sufficient bearing capacity to lift the structure (at approximately RL -17.75m CD). This borehole is shown as BH6ABN on the borehole plans in Appendix A.

Rock was encountered at depths between RL -35.5m CD and RL -41.8m CD. The rock comprised very low to high strength fractured siltstone and sandstone, and was proven to a maximum depth of around RL -50 m CD.

Two boreholes are given on the GeoVic Database which are located approximately 2.0km east and 4.0km north east of the BlueScope Steel Wharf and were drilled to depths of 51 and 54m respectively. It is stated in the 1978 report that these boreholes were drilled for the Department of Minerals and Energy as part of an investigation into regional groundwater. Borehole logs are only available to 5.0m depth for the location closest to the BlueScope Steel Wharf and show sand with shell fragments overlying sand and clayey sand. For the location further offshore the materials encountered comprised grey clayey sand, sand and silty sand to a depth of approximately 10m, overlying yellow/brown/orange clayey sand/sandy silty clay to a depth of 18.5m, overlying khaki/olive slightly calcareous clayey sand, silty clay and silty sand to a depth of 54.5m. Chips of what may be Silurian quartzite hard rock were reported at 54.5m (-63mCD).

Three offshore boreholes were drilled as part of the 1974 investigation by Public Works Department Victoria. BH 140 and BH141 are located approximately 300m to the north east of the BlueScope Steel Wharf and BH142 is located approximately 3km to the north east. BH140 and 141 were only drilled to depths of 1.68m and 1.52m respectively and encountered soft sandstone overlying grey silty fine sand (BH 140) and light grey sandy clay (BH 141). BH 142 encountered sandy clay, sandy silt and clayey sand with bands of limestone to a depth of 12.9m overlying sandy calcareous clay and sandy calcareous silt with frequent rock layers, boulders and limestone layers to a depth of 28.96m (RL-36.68mCD).

The 1967 MacDonald, Wagner and Priddle Drawing includes seven boreholes along the alignment of the access jetty to the Esso wharf, 10 boreholes at the Esso wharf and two boreholes to the south of the main jetty head. The ground conditions encountered along the access jetty alignment typically comprised interbedded medium dense to very dense clayey and silty sand; stiff to firm silty and sandy clay; and very soft to very stiff sandy clayey silt. These layers were underlain by weathered siltstone, sandstone, mudstone and cemented calcareous sand. The ground conditions along the dolphin and jetty head alignments typically comprised interbedded silty and clayey sand ranging from very loose to very dense; soft to stiff clayey and sandy silt; soft to stiff silty sandy clay; and thin horizons of limestone and siltstone. These units were underlain by siltstone, sandstone and mudstone. Thin layers of brown coal were also encountered in a number of these boreholes.

One offshore borehole (BH2) is included on the 1970 Harris Lange and Partners Drawing which was drilled to a depth of 5.3m and encountered silty sand with numerous shells, overlying medium dense silty sand and medium dense clayey silty sand.

## 5.2 North Arm – Between Crib Point and Long Island point

### 5.2.1 Onshore

No previous site investigation data is available for this area.

### 5.2.2 Offshore

No previous site investigation data is available for this area.

## 5.3 Crib Point

### 5.3.1 Onshore

No previous site investigation data is available for this area.

### 5.3.2 Offshore

One borehole (BL6) was drilled as part of the 1978 investigation approximately 1500m south of Crib Point. This borehole encountered loose silty sand and sandy silt overlying firm to stiff sandy silty clay to a depth of 13.5m (RL -21.2mWCD). A layer of basalt gravel was encountered at 13.5m overlying sand and gravel to a depth of 18m (-25.7mWCD), overlying a very stiff silty clay to a depth of 27.5 (-35.2m WCD).

## 5.4 Stony Point

### 5.4.1 Onshore

One borehole (BL1) was drilled as part of the 1978 investigation at the existing reclamation to prove inshore stratigraphy and to investigate the existing reclamation. This borehole encountered fill comprising gravelly sandy clayey silt and sandy clayey silt to a depth of 3.5m (0.9mWCD). Below this level sand becoming very dense overlying hard silty clay was encountered to a depth of 27.5m (-23.1mWCD). Highly to moderately weathered basalt was encountered from 27.5m to 29m depth (-24.6mWCD).

A number of vane shear tests were attempted adjacent to the existing reclamation as it was thought that there may be significant depth of soft sediments adjacent to this reclamation, however the vanes could not penetrate more than about 1m and it was decided to replace these tests with DCP tests which confirmed that loose surface sediments were relatively shallow and were found to overlie stiff clays.

### 5.4.2 Offshore

Nine boreholes were drilled at Stony Point as part of the 1974 investigation. These boreholes were drilled in relatively shallow water with depths ranging from -1.22m to -5.79mCD. Two boreholes were drilled to around 37m, three to around 13-14m and the remaining boreholes were drilled to between two and six metres depth. The 1974 report states that the boreholes drilled at Stony Point indicate an appreciable variation in soil conditions particularly in the upper 9m. The predominant soil type was a residual clay but a number of significant sand zones were also encountered. The clay is reported as being generally stiff to very stiff but a significant stratum of soft to firm clay was encountered in BH 131. Weathered Basalt was encountered in BH 131 and BH 136 at depths of 35.97 and 32.46m respectively. In both of these boreholes the top of the bedrock was found to consist of weathered basalt boulders in a residual clay matrix changing to intact weathered basalt one to two metres below this level.

One borehole (BH 128) was drilled close to the channel off Stony Point as part of the 1974 investigation which encountered fine sand to a depth of approximately 2m overlying sandy clay and silty sand to a depth of 10.67m (-25.91m CD).

Three boreholes (BL2-BL4) were drilled at the proposed structure alignment at Stony Point as part of the 1978 investigation. Silt was encountered at bed level up to a maximum depth of 1.6m and was described as soft in two of the three boreholes. The silt is underlain by clay interbedded with sand and gravel in the upper portion

of the boreholes to a maximum depth of 16m. Clay was typically stiff to very stiff becoming hard at around -30 to -35mWCD. BL3 encountered a 0.5m deep zone of moderately weathered to fresh basalt t at 11m depth (-15.50mWCD) underlain by soft clay at a depth of 12.5m (-17mWCD). Highly weathered becoming slightly weathered and moderately weathered Basalt was encountered in two of the boreholes at depths of 39m (-43.5mWCD) and 28.5m (-36.2mWCD).

Three boreholes (BL5, 5A and 5B) were also drilled at the proposed dredging area as part of this investigation to a level of approximately -7.0mWCD. They encountered clay interbedded with sand and some silt and gravel. Clays were typically stiff to very stiff however soft clay was encountered in one borehole at a depth of 3-3.5m. Gravel logged as basalt overlying clay and gravel conglomerate was encountered in one borehole at a depth of 0.78-1.10m.

## 5.5 Western Channel

27 of the 41 boreholes drilled as part of the 1974 investigation are located between the entrance to Western Port at The Nobbies and Sandy Point. These boreholes ranged in depth from 1.5 to 10.7m with an average depth of around 6m corresponding to an average reduced level of around -24mCD. Materials encountered typically included sand and gravels at seabed, often calcareous and or cemented, overlying clayey gravel, gravelly clay and/or sandy and silty clays and in some areas rock. Rock including weathered basalt, sandstone and mudstone was encountered in nine of these boreholes. A summary of the depths and recorded description of rock materials in these boreholes is given in Table 5-1 below.

**Table 5-1 Rock encountered in Western Channel in 1974 Investigation**

| Borehole | Depth (m) | RL (mCD) | Description |
|---|---|---|---|
| BH101 | 1.90-2.44 | -18.36 to –18.90 | Boulders and cobbles of weathered basalt in a clay matrix |
| BH101 | 2.44–4.27 | -18.90 to –20.73 | Weathered basalt |
| BH105 | 3.51–6.10 | -21.80 to –24.39 | Tuffaveous mudstone with gravelly clay |
| BH106 | 0.00–8.23 | -16.46 to –24.69 | Clayey gravel with cobbles and boulders of weathered volcanic tuff and basalt |
| BH108 | 0.00–3.96 | -17.68 to –21.64 | Sand and gravel with cobbles of cemented sand and basalt |
| BH108 | 5.03 – 6.71 | -22.71 to –24.39 | Sand and gravel with cobbles of basalt and cemented sand |
| BH109 | 0.00 – 6.10 | -18.29 to –24.39 | Sand and gravel with occasional fragments of cemented sand and basalt |
| BH111 | 0.00 – 2.13 | -17.98 to –20.11 | Sandstone boulders and coarse sand changing to grey clay at 2.13m |
| BH117 | 3.98 – 6.10 | -21.87 to –24.08 | Weathered basalt and clay |
| BH120 | 2.44 – 5.18 | -19.81 to –22.55 | Clay with numerous highly weathered basalt remnants |
| BH120 | 5.18 – 7.32 | -22.55 to –24.69 | Weathered basalt |
| BH121 | 0.46 – 6.71 | -18.75 to –25.00 | Clay with fragments of highly weathered basalt |

The 1974 Public Works Department Victoria report states that boulders were encountered in a number of the boreholes located in the south-eastern half of the approach channel which is described as being an area of shallow bedrock and that the boulders may be associated with the weathered upper bedrock strata. This report also states that relatively hard cemented sand layers were encountered in a number of boreholes but that the distribution of this stratum appears to be random.

In the 1974 investigation rotatory drilling techniques were only used for five boreholes with the remainder of the boreholes being drilled using percussion drilling techniques with rotary coring in rock in three boreholes.

## 5.6 East Arm (Anchorage)

No previous site investigation data is available for the East Arm anchorage area.

## 5.7 1974 Geophysical Survey

The 1974 Public Works Department Victoria report on seabed investigation included a geophysical survey using continuous profile sparker seismic reflection equipment. Interpretation of this survey was only carried out for the approach channel seaward of Sandy Point.

The report states that the seabed strata over the area interpreted is complex with generally layered sediments, frequently coarse, overlying weathered and unweathered bedrock. The strata included numerous cemented layers and layers of cobbles and boulders. Bedrock is reported as being shallower on the south east side of the present approach channel and deeper to the north-west of the channel.

Interpretation of the seismic data included correlation with drilling records however the seismic interpretation included in the report states that the seismic data is considered to be of poor to medium quality and that a number of non-geological and geological influences had an adverse effect on the data quality which reduced the accuracy and reliability of the seismic interpretation.

The report states that there would be some advantage in planning future dredging and channel widening on the north west side of the present approach channel rather than on the south east side, both to avoid shallow bedrock and the probable boulder areas evident on the south-east side of the present approach channel.

# 6.0 2013/2014 Site Investigations

## 6.1 Overview

Geophysical survey and geotechnical investigations have been undertaken to supplement existing information for the current phase of design development. Boreholes have been located on a widely spaced grid of approximately 500m in the Port Area and have been aimed at providing general site coverage with the intention of providing information to allow assessment of development options and to address risk and uncertainty.

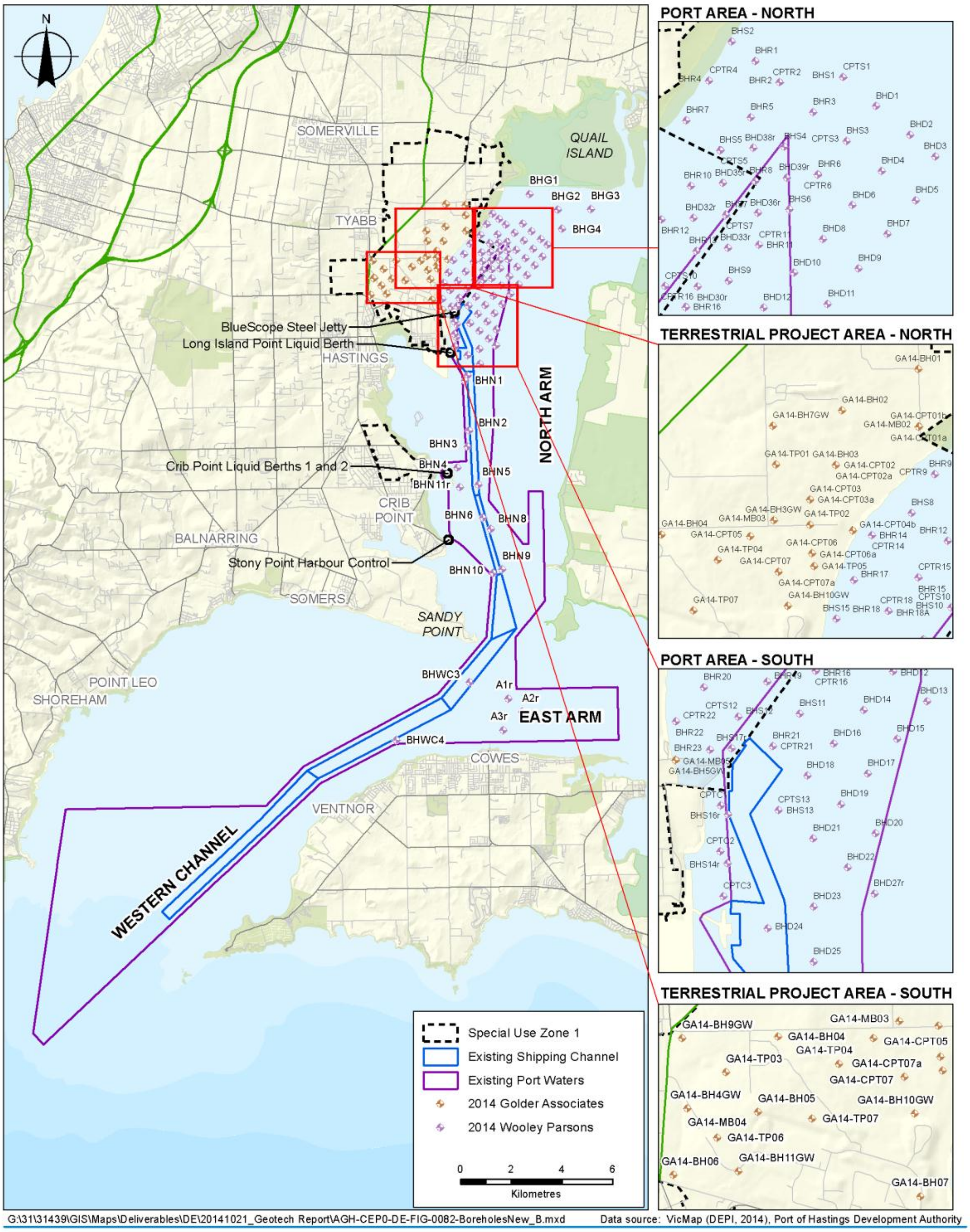
The current investigations have included the following:

- Marine geophysical survey in the Western Channel, Anchorage, North Arm Channel, Port Area and North Port Area by Aurecon
- Marine geotechnical investigation in the Western Channel, Anchorage, North Arm Channel, Port Area and North Port Area by WorleyParsons
- Terrestrial geotechnical investigation within the BlueScope Steel property area (the Terrestrial Project Area) by Golder Associates

The marine geophysical survey was undertaken concurrently with the marine geotechnical investigation. As such there was limited opportunity to revise borehole locations based on the findings of the geophysical survey however borehole locations in the North Arm and Western Channel were revised from their original location with the revised locations selected to target areas where rock was expected within the dredge depth or where other complex reflectors were identified by the geophysical survey.

Details of the geophysical survey and geotechnical investigations are given in this section. Marine and terrestrial boreholes completed as part of the 2013/2014 investigations are shown in Figure 6-1. Additional plans showing all borehole locations within the study area are given in Appendix A. The findings of these investigations are discussed in Sections 7.0 and 8.0.

**Figure 6-1 2013/2014 Marine and Terrestrial boreholes**



## 6.2 Marine Geophysical Survey

The aim of the marine geophysical survey was to supplement the geotechnical investigation and to assist in understanding and interpretation of geological materials across the site.

The geophysical survey included the following types of survey:

**Bathymetric survey**

Single beam bathymetry was recorded across the site and provided as contour plans with levels reduced to Lowest Astronomical Tide (LAT).

**Side scan sonar (SSS)**

Side scan sonar was used to classify seabed materials (fine-grained, coarse-grained, cobbles, bedrock etc.) and detect other objects on the sea floor that might have an adverse effect on construction. The data provided full coverage of the survey area with the exception of the North Port Area where lines were widely spaced, and at the northern most line at the Anchorage. The side scan sonar data was provided as a mosaic along with preliminary interpretation of seabed features and character.

**Sub-bottom profiling (SBP)**

Two types of sub-bottom profiling systems were used in the works as follows:

- A higher frequency chirp sub-bottom profiler system which was used in deep water areas where only shallow penetration was required due to the limited depth of dredging. This system had a very limited penetration across the survey areas due to the seafloor material being predominantly sandy in nature.
- A lower frequency boomer sub-bottom profiler which was used in more shallow water areas where deeper levels of penetration were required.

**Multi-channel seismic reflection**

A multi-channel seismic reflection system was also used in the shallow water areas of the Port Area and North Port Area.

### 6.2.1 Interpretation of the Geophysical Data

Interpretation of the survey records has been completed by Aurecon and their sub-consultant Marine and Earth Sciences and is reported in Aurecon's Marine Geophysics Survey – "Final Interpretative Report" dated 10 October 2014 (ref No.: 239769-02).

This has involved identifying and mapping laterally continuous sub-surface reflectors and calibration with borehole logs. The accuracy of the interpreted seismic reflectors levels is reported to be in the order of +/- 5%. It is noted by Aurecon that it is difficult to accurately detect layers thinner than 1.0 to 1.5 m.

The report include figures showing vessel trackplots, contoured bathymetric data, interpreted seismic sections, contour plots for major reflectors, side scan sonar mosaics and seafloor mapping plans for each survey area. Selected outputs from the geophysics reports have been reproduced in Appendix I.

A summary of the interpretation process is described below. Reference should be made to the Aurecon report for details of the survey findings and interpretation.

The simplified borehole sticks and the CPT graphic symbols were superimposed over the SBP profiles. Only boreholes and CPT results at up to 10 m distance from the seismic lines were used in the correlation and those were displayed over the SBP profiles in figures. This allowed for the correlation of the seismic results.

Figure 6-2 represents a magnified section of the interpreted SBP line PA01 with the zone of interpretation close to BH R12 for demonstration purposes.

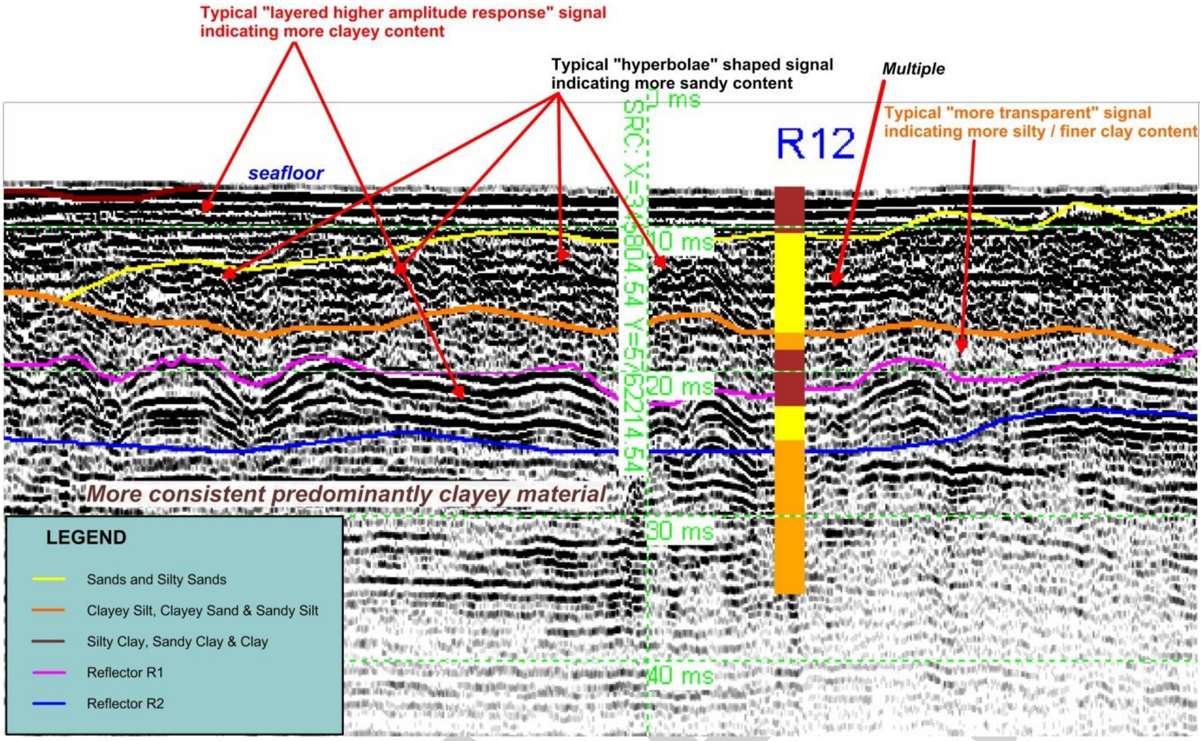



**Figure 6-2 Typical interpretation of SBP based on BH correlation with the seismic signal "signature".**

As part of the interpretation process layers were identified based on the specific seismic “signature” and amplitude character they exhibit taking account of general correlations with the boreholes. The signal amplitude and overall signal image trends were followed along the SBP lines and interpreted using the borehole logs and CPT graphs where available. Typically the more clayey material generated a higher amplitude response, and material with more silty content generated more “transparent” seismic “image”. The more sandy material generates more diffraction; hence hyperbolae shaped forms on the SBP amplitude images. A more defined layering was not possible based on the signal signature and characteristics alone.

As part of the geotechnical interpretive reporting process, the geophysical survey reflectors and borehole/ CPT data have been imported into the 3D geological model. This model is to be used as input into future workstream activities in respect to, for example, dredging and reclamation, dredge material management, and wharf and terminal design. It is noted that the 3D geological model is to be reported under separate cover. The screen captures of this model presented in the following sections of this report are for illustrative purposes only, and should not be used for interpretation.

### 6.2.2 Geophysical Survey Limitations

An increase in output power gives better penetration into the sub-bottom layers. Sometimes, however, if the bottom is very hard or not very deep, the increase in power will cause more signal to be reflected back off the water / sediments interface. The signal might then be reflected off the water surface, leading to multiple reflections and "noise" in the data.

Signal frequency also has an effect on system performance. Higher frequency systems (2 to 20 kHz) will produce high definition data of the upper sediment layers. These higher frequency signals have shorter wavelengths, and they are able to discriminate between layers that are close together. Lower frequency systems will give greater penetration but at a lower resolution.

Longer sound pulse length transmits more energy and yields deeper penetration. However, a long pulse length may decrease the ability to discriminate between adjacent reflectors, thus decreasing the system resolution. The SBP systems used in the survey have resolution from ~ 0.1 m (CHIRP) to ~0.5 m (MCSR). The boomer system has expected resolution of ~ 0.2 m.

Similarly the penetration depth depends on the hardness of the overlying layers and the presence of gas deposits, such as methane within sediments. The interpretation of the SBP profiles depend on the signal quality

(subject to geology) and interpreter's experience. The interpretation of the seismic reflectors is a partly subjective process.

Figure 6-3 below represents an example where only partial correlation of the SBP interpretation with boreholes is achieved (the interpreted SBP line profile PA07). Some limitations of the SBP technique are demonstrated first in capability to detect the relatively thin sandy layers (yellow interbedded layer in borehole R6 at ~ 20 ms time gridline). Also, the deeper layer in borehole S3 (the silty layer at ~ 53 ms in borehole S3) does not have a recognisable signature possibly due to decreasing of the seismic signal amplitudes with depth. However, the prominent reflector R1 which can be followed along the SBP profile is not detected in borehole R12, which may be a function of drilling methodology, and gradual material changes that can be difficult to differentiate during the borehole logging process.

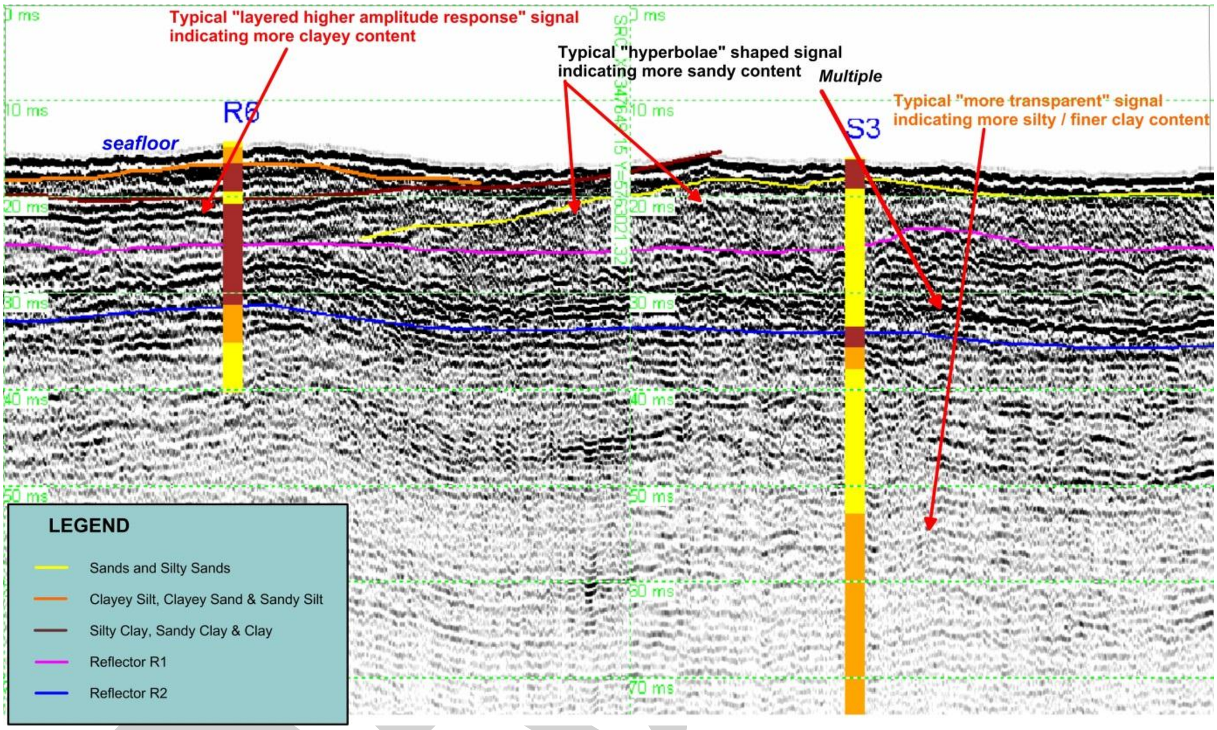



**Figure 6-3 Interpreted SBP profile Line PA07 including some limitations with regards to the interpretation and correlation with boreholes**

### 6.2.3 North Port Area

The SBP interpretation in the North Port Area has identified two major continuous reflectors, R1 and R2. The material between reflector R1 and seabed has been interpreted as deposits comprising clays, silts, and sands. Localized discontinuous reflectors are reported within this layer and possibly represent minor density boundaries.

The material between reflectors R1 and R2 is reported to comprise interbedded clays, silts and sands, and the material immediately below R2 is reported to be predominantly represented by interbedded finer materials such as clays and silts.

R1 is an irregular reflector and has been interpreted as an erosional surface. R2 is reported to display more simple parallel reflections and it is noted that the base of R2 was difficult to detect in all places due to the presence of the sea floor multiple.

Based on the interpretation the rock levels are highest at the western part of the North Port Area. Slight uplift in rock levels can be observed towards the North North West with the shallowest levels estimated at RL -32 m. The groundwater boreholes G1 to G4 drilled in the North Port area terminated above rock level.

### 6.2.4 Port Area

The SBP interpretation in the Port Area has identified two major continuous reflectors, R1 and R2. The material between reflector R1 and seabed has been interpreted as recent deposits comprising clays, silts and fine sands.

Following the above mentioned approach (Section 6.2.1 and Figure 6-2), three units (represented by three different reflectors) have been identified within this layer which are typically non-continuous, and are shown on the interpreted cross-sections as follows:

1) Clays, Silty Clays and Sandy Clays
2) Clayey Silt, Clayey Sand and Sandy Silt
3) Sands and Silty Sands

The material between reflectors R1 and R2 is reported to comprise interbedded sands, silts and clays and the material below reflector R2 is reported to be interbedded fine grained materials such as clay and silts. The interpreted material types between R1 and R2 are reported to agree well with the CPT data in this interval. However, the geotechnical borehole logs are reported not to detect this interbedded material well. The top of the R1 reflector is irregular and is reported to include minor erosional features which have been infilled, whereas reflector R2 displays more parallel reflections.

A rock reflector has been identified along the western side of the site and is reported to be highly variable in elevation but does correlate well with rock levels encountered in the boreholes between Long Island Point and the BlueScope Steel wharf.

The shallowest rock was interpreted at Long Island Point towards the south west of the Port Area. The rock level interpreted in this zone is ~ -29 m CD. The Silurian rock levels dip from the land eastwards (Figure 6-4). The rock reflector in the Port Area north of the BlueScope wharf is interpreted to be below borehole termination depths.

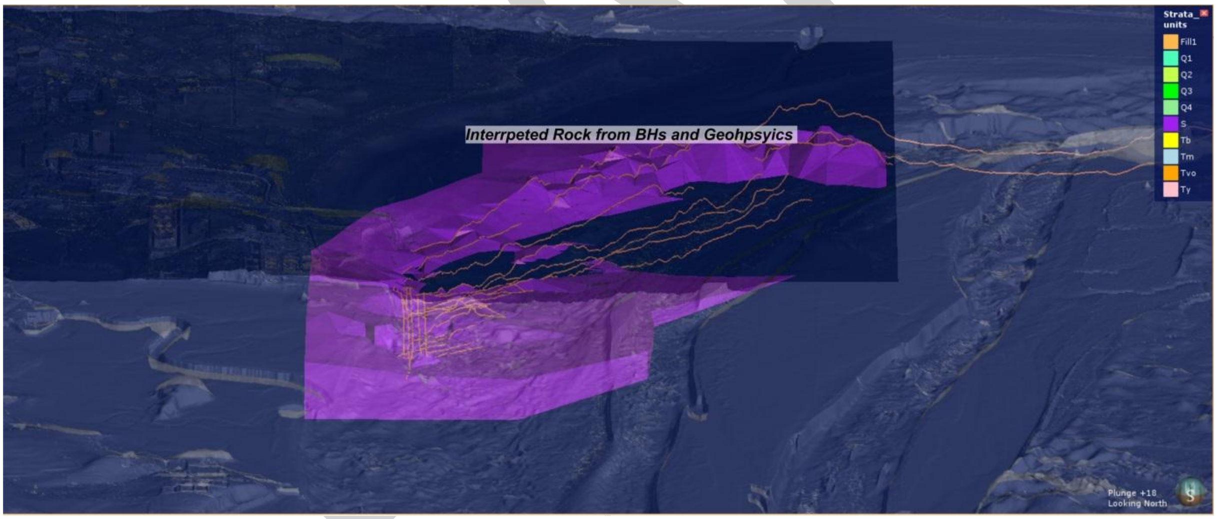



**Figure 6-4 Interpreted 3D Silurian rock based on boreholes and Seismic Interpretation - Port and North Port Area - view from South. The purple surface is the modelled Silurian Rock, with the pink lines the interpreted geophysics lines for rock surface. Note that the gap in the purple surface in the centre of the figure is due to the geophysics lines extending deeper than the extent of the 3D model.**

Based on the correlation with the boreholes in the Port Area the interpreted reflector R2 appears to respond to the approximate interface between the Baxter and Sherwood Formations. The R2 reflector has a slight dipping trend towards the East with a minimum elevation of around -14 m CD near shore deepening away from shore to around ~ -20 m CD.

Similarly to the R2 reflector, the reflector R1 shows a dipping trend from nearshore towards the east.

Above the R1 reflector a number of additional reflectors have been identified from the geophysical survey. These reflectors appear to be variable in respect to depth and orientation, and are discontinuous.

Figure 6-5 below shows a 3D view of the complex array of interpreted geophysics reflectors above R1. The reflectors seem to generally correlate with the type of material encountered in the boreholes, although there are some discrepancies due to intrinsic geophysical and drilling methodology limitations.

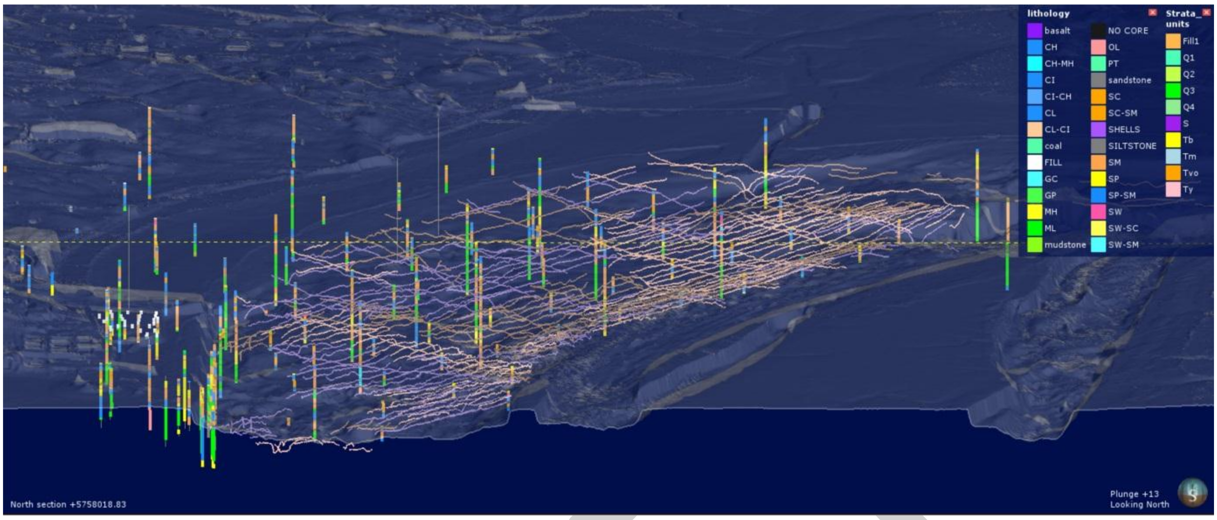



**Figure 6-5 3D model with borehole s and complexity of interpreted geophysics lines above the R1 reflector – view from South**

The general classification of seabed materials based on the geophysics results (combining BH correlated results and the Side Scan Sonar interpretation for the Port Area is presented in Figure A13 of the Aurecon report.

### 6.2.5 North Arm

Four major reflectors have been identified in the North Arm including a rock reflector; the other three reflectors have been named R1 to R3.

**North Arm – North Channel**

The parallel dipping reflectors interpreted in the Port Area are reported to continue into the northern end of the North Arm for approximately 1000m before abruptly discontinuing. It is reported that this sudden termination may represent a geological boundary.

The material between seabed and reflector R1 has been interpreted as deposits dominated by fine material such as clays and silts with a thin veneer of overlying sandy material. Typical thicknesses of this material are 3-5 m. The material between reflectors R1 and R2 has been interpreted as interbedded sands, silts and clays. The material immediately below R2 is interpreted to be interbedded finer materials such as clays and silts.

The position of the R2 reflector indicates that this material is deposited in the deeper channels and this corresponds with the position of the deeper R3 reflector in the Crib Point area and North Channel.

The top of R1, which is a discontinuous reflector, is reported to be irregular with minor erosional features which have been infilled with recent deposits. Reflector R2 underlies R1 and laps onto R3 in the southern end of the channel. R3 is reported to be a very irregular reflector which defines the surface of a predominantly silt and clay layer. The material below the reflector is also reported to display numerous diffractions indicative of gravels, cobbles and boulders.

The rock reflector (which is below likely dredging depth) has been interpreted as underlying reflector R3 and has been detected in the southern end of the North Arm North Channel. The interpretation of the rock levels was based on correlation with the recently drilled boreholes. The shallowest interpreted rock level based on the geophysics interpretation is observed in the northwest area of the North Arm North Channel area with the RL at -19 m CD.

**North Arm – Crib Point**

In the area around Crib Point wharf reflector R1 (interbedded sands, silts and clays) is again discontinuous (localized discontinuous reflectors possibly represent minor density boundaries) and overlies reflector R3 which is interpreted as the top of a silt and clay layer. Limited correlation with boreholes was available due to the

paucity of borehole data. Similar to the North Arm – North Channel area the material below this reflector is also reported to display numerous diffractions indicative of gravels, cobbles and boulders. The shape of this reflector corresponds well with the character of the bedrock as interpreted in this zone. It exhibits a sudden dip in the zone of Crib Point potentially associated with a geological feature.

A rock reflector (which is below likely dredge depth) has also been interpreted in this area and is found to be shallowing to the north (Figure 6-6)

**North Arm – South Channel**

The seabed in this area is dominated by sand waves with wave heights ranging from 1 m to 5 m. A shallow discontinuous horizontal reflector is reported to exist immediately beneath the sand waves and is interpreted to represent the boundary between the recently deposited sand deposits and the underlying finer clay and silt layers.

The seismic data in this area is reported to be very complex and has identified numerous reflectors including discontinuous R1 and R2 reflectors, R3 and a rock reflector (below dredge depth). Reflector R1 is limited to the northern extent of this section of channel with the material between seabed and reflector R1 being interpreted to comprise clays and silts in the northern extent and grading to sandy material at the southern end.

The level of Reflector R3 is highly variable and intersects the sea floor at 1600 m and 3400 m from the southern end of the North Arm. The material between seabed and reflector R3 has been interpreted as a predominantly silt and clay layer with some localised diffraction zones of gravels, cobbles and or bounders. This unit includes two major channel features which have been infilled with interbedded fine and coarse deposits and exhibit a series of dipping and onlapping reflectors which terminate on the unit between R3 and the rock reflector.

Based on the historical boreholes drilled in the area the rock undulates with some deeper zones in the central North Arm North Channel area and at the North Arm South Channel area possibly indicated by the discontinuous characteristics of the reflector (Figure 6-6). The gaps in the reflector might be caused by deeper bedrock in these zones or the potential presence of structural features.

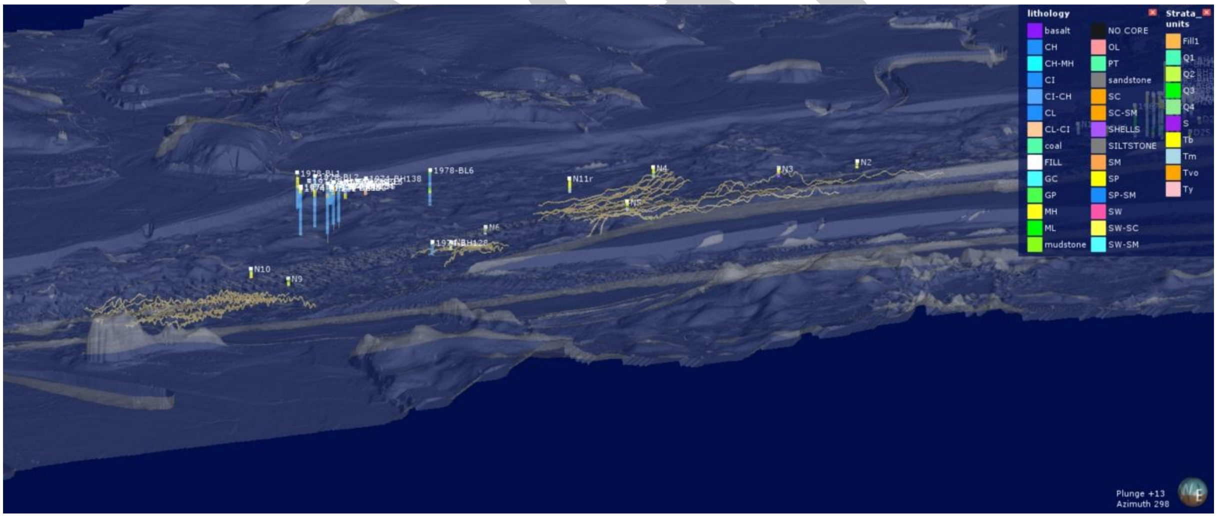



**Figure 6-6 3D view over the North Arm area with the observed "discontinuous" rock reflector (grey lines) appearance.**

The rock reflector continues from the North Arm North Channel and Crib Point area but is not detectable continuously and is reported to be highly variable in level with a maximum elevation of approximately -21 m CD.

### 6.2.6 Western Channel

The CHIRP system is reported to have had limited penetration over much of this area and while the boomer system generally provided good penetration the rock boundary is reported to be indistinct in places.

**Western Channel - Northern Section**

Dense seafloor materials are reported in this section of the Western Channel and have been interpreted as predominantly coarse material such as sands and gravel. Rock levels are reported as being variable with the shallowest rock being encountered on the eastern side of the channel at the northern end of this section at -14m CD with the surface dip trending towards South.

No reflectors are observed that can be correlated with gravelly material in this zone. A clay layer is reported to overly the rock reflector in places which may represent extremely weathered rock or residual soils. Seismic penetration below the rock reflector is reported to have been limited.

A shallow bank is reported to extend into the channel from the eastern side. The survey results suggest that dense gravels or a cemented material is present from seabed over this area. In a few locations on the seismic records a very high amplitude reflector is observed and is interpreted as the surface of an indurated or cemented layer (Figure 6-7).

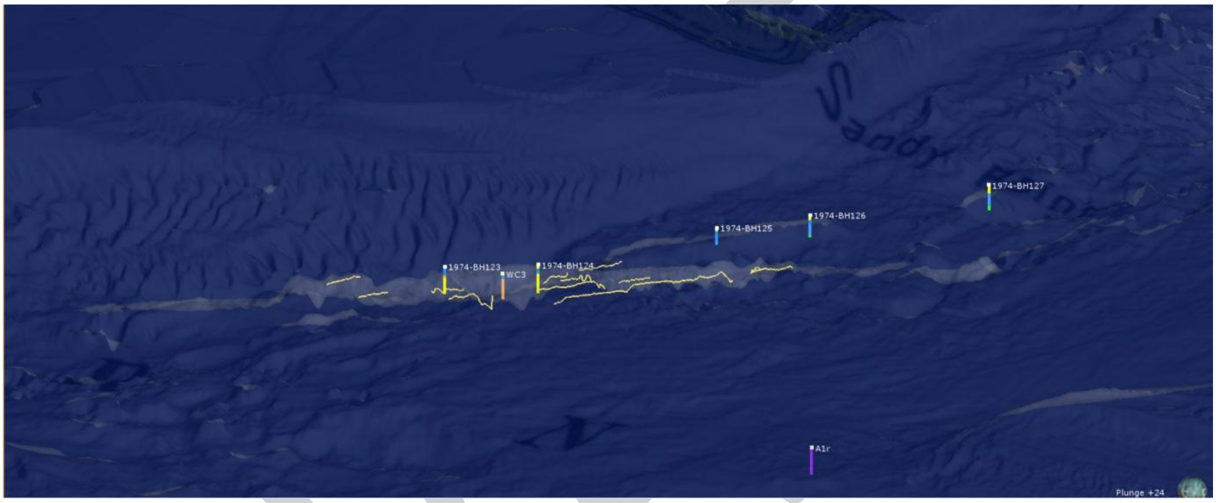



**Figure 6-7 3D view of the interpreted Cemented layer in the WC North Area - view from East.**

**Western Channel - Mid Section**

This section of channel has a significantly variable seabed which includes numerous high spots and rock has been identified at six locations above RL -22.5 m LAT. The interpretation based on the limited borehole information indicate high level of correlation of the geophysics with the available data. The minimum rock level at the north central area of the Western Channel is ~ -19 m LAT. The rock level has an undulating character in this zone with a drop in the rock levels in the mid area, and with an increase in level towards South with the minimum level of ~ -18 m LAT.

The material between the seafloor and the rock reflector is reported to be very dense and include clays, sands, gravel and or cobbles. The seismic character of this material is deemed to be very variable and lacking internal reflections. Limited soil material reflectors were interpreted above the rock layer in this section.

**Western Channel - Southern Section**

This section of channel includes numerous high points and a localised band of higher seafloor that crosses the channel. Rock has been identified at ten locations above RL -24.0 m CD. At the northern end of the Western Channel Southern Section the maximum elevation of the rock surface was recorded at ~ 17.2 m CD. In the central zone of the Western Channel Southern Section the maximum elevation of the rock surface was recorded to be ~ -21 m CD and at ~ -23 m LAT in the southern end of the Western Channel Southern Section the maximum elevation of the rock surface was recorded at ~ -21 m CD.

The material between the seafloor and the rock reflector is reported to be very dense and include clays, sands, gravel and or cobbles. The cemented and gravel layers were indicated in the SBP profiles by presence of numerous hyperbolae shaped features. The layers above the R1 reflector were sparse and some more interbedded layers were detected above the rock layer, however no boreholes were available for correlation.

The snapshot of the layered cross section shown in Figure 6-8 below indicates the depositional layering of sediments within the rock depressions (note the light blue and navy blue reflectors infilling the depressed rock zones).

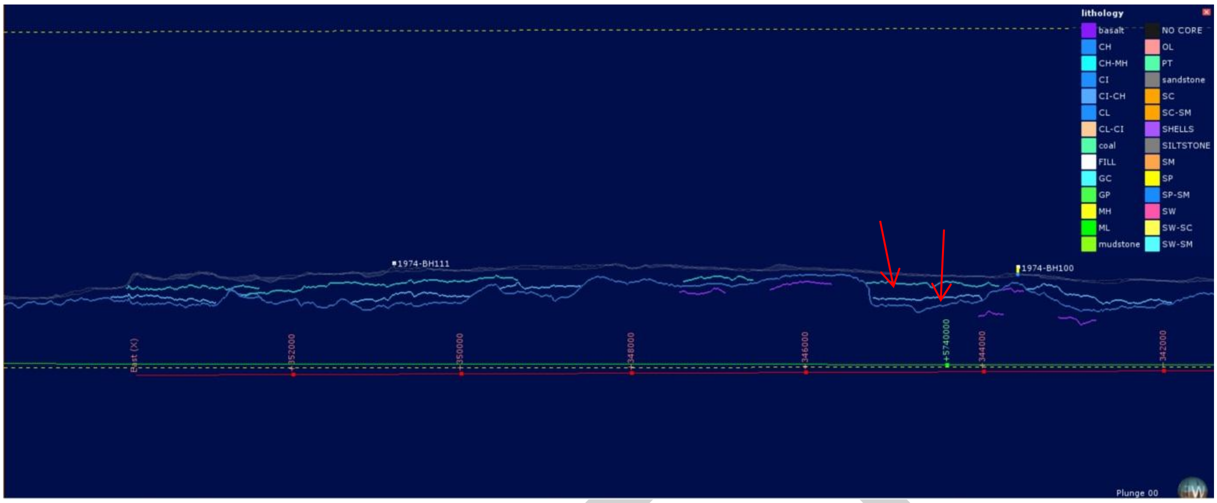



**Figure 6-8** **Cross section showing the bedrock layer - dark blue and magenta lines, and infilled soil - light blue and navy blue lines– layer surfaces (examples marked with red arrow) in the Western Channel Southern Section(looking from West).**

### 6.2.7 Anchorage

The SBP interpretation in the Anchorage Area has identified three major reflectors, R1, R2 and an interpreted rock reflector. The material between reflector R1 and the seabed has been interpreted as recent deposits dominated by silts and sands and includes localised discontinuous reflectors which are reported to represent the surface of deposits of coarser materials or clay layers. The dominant material between reflector R1 and R2 is reported to be interbedded sands and silts and the material below reflector R2 is reported to comprise interbedded fine materials such as clays and silts.

R1 is an irregular reflector limited to the eastern extent of the anchorage and displays ‘chaotic’ patterns with erosional features while reflector R2 displays more parallel reflections.

A relatively good correlation of the rock levels with the available boreholes has been achieved in the geophysical interpretation (both with the extremely weathered rock and the more competent deeper rock as interpreted in SBP profiles). The rock reflector has been continuously mapped across the anchorage with elevations ranging from -12m CD in the central northern Anchorage area to below -20 m CD towards the south west and south east ends. Figure 6-9 below represents the interpreted rock reflector layer which shows an elevated rock surface (arrowed in red) towards the central part of the area.

The interpreted rock depth contour map is also displayed in Figure C17 in the Aurecon Final Interpretative Report.

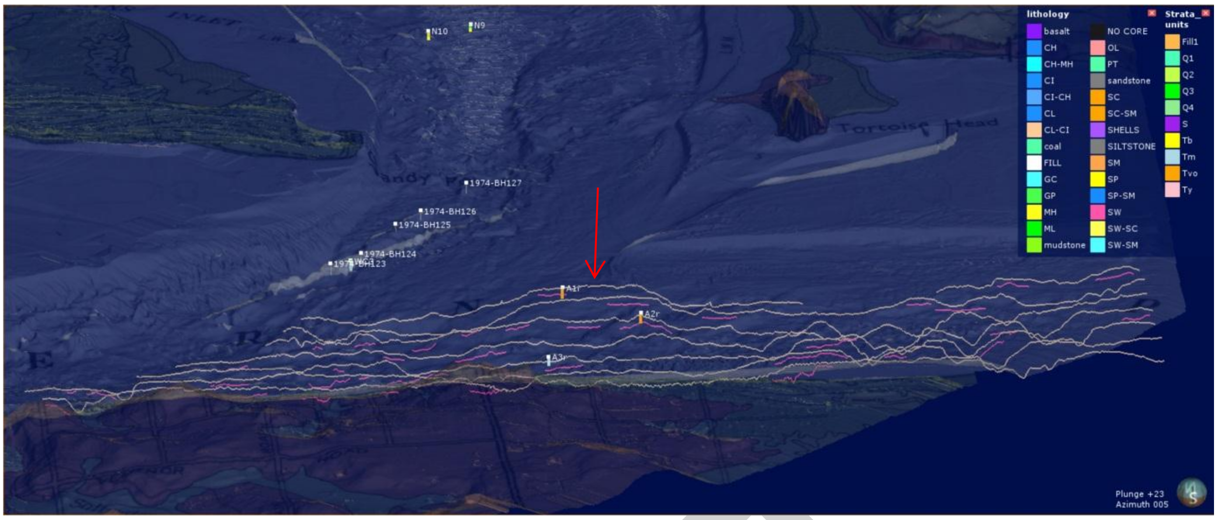



**Figure 6-9 3D model of the Anchorage Area - rock reflectors (looking from South). Elevated rock surface marked with red arrow.**

The penetration below the rock reflector is laterally variable and is likely related to degree of weathering, with the SBP images indicating some areas of intensely weathered rock. However the rock strength appears to increase rapidly with depth.

### 6.2.8 Summary

The interpretation of the geophysics survey results can be summarised as follows:

- Generally the interfaces between major geological units (e.g. between soil and rock) have been identified with reasonable clarity due to sufficient density contrast. However, the subtle changes in stratigraphy within recent depositional units were not clearly identifiable based on the correlation of geophysics results and the boreholes and CPTs. This is potentially either due to the gradual changes in density and grain size between the deposited materials, and/ or the intrinsic limitation connected with the drilling process and geophysical methods. These limitations were especially evident in the case of the Port Area. Examination of the SBP cross sections and borehole results from the Port Area indicate that the shallow material beneath the seafloor is dominated by predominantly fine interbedded sediments (sand, clay, and silt).
- The rock levels in the Port Area are interpreted significantly deeper than the proposed dredge level (-20 m CD).
- The interpretation in the North Arm Channel, Western Channel and Anchorage Areas is more straightforward since a sufficient density contrast between the soil and rock layers exists. Some variable levels of rock were interpreted in these areas with the highest rock level interpreted at ~ -12 m CD towards the centre of the Anchorage Area (Rock contour Map in Figure C17 – Aurecon "Final Interpretative" Report).
- The depth to rock in the North Arm is interpreted to be generally deeper than the proposed dredge level (-20 m CD). The relatively higher elevation rock levels (~ -22 m CD) were interpreted in the south west part of the North Arm North Channel area (see Figure B10 in the Aurecon report).
- In the Western Channel, the shallowest rock can be expected in the north west corner of the Western Channel North Section with the rock levels shallower than -18 m CD (Rock contour Map in Figure C17 in the Aurecon Report).
- The rock levels in the Western Channel Mid Section are shallowest towards the central western zone (rock levels shallower than -20 m CD) of the site with limited shallow rock in the central channel area. A localised rock high point was interpreted in the south western part of the section, with the rock surface at ~ -20 m CD (Rock contour Map in Figure C18 of the Aurecon Report).
- The rock levels in the Western Channel Southern Section are more variable with several high points (in the north - ~-22 m CD, centre <20 m CD and at the South West at ~ -24 m CD (Rock contour Map in Figure C19 – Aurecon "Final Interpretative" Report).

- Some non-continuous cemented and gravelly layers were interpreted in the Western Channel Southern Section (refer Section 6.2.6 and Figure 6-7). These layers can impact on the dredging operation. Some very undulating seafloor features in parts of the Western Channel and the Port Area were interpreted to be sand waves based on the cross examination of the more detailed bathymetry data from Lidar and the seafloor as recorded in the SBP profiles.
- Due to the insufficient penetration of the CHIRP system in the Western Channel area, it is strongly recommended that this area be surveyed with an infill SBP boomer, seismic refraction lines and possibly additional multichannel reflection lines in order to better delineate the rock surface, and to characterise the zone in more detail.

## 6.3 Terrestrial Geotechnical Investigation

### 6.3.1 Scope of Investigation

The terrestrial investigation was scoped in 2013 with the following objectives in mind:

- To define the geological stratigraphy and associated geotechnical design parameters of the terrestrial site;
- To characterise the physical, chemical and engineering properties of the underlying soil and rock;
- To provide interpretation of the subsurface ground conditions to inform decisions on the suitability of founding future structures;
- To undertake ground water level monitoring to inform future studies.

The investigation was constrained to within the BlueScope Steel property, with test locations limited to those areas where native vegetation clearance would not be required. Operational areas of the BlueScope Steel facility were excluded, together with adjacent areas that appeared to consist of fill.

The terrestrial geotechnical investigation was carried out between 19 February and 24 April 2014 by Golder Associates Pty Ltd (Golder) under Authority Contract no. 2013-002. The completed scope of investigation included:

- Drilling of 18 no. geotechnical boreholes to nominal depths of between 15 and 50 m, including 11 boreholes with groundwater monitoring wells
- Drilling and installation of 5 no. groundwater monitoring bores to approximate depths of up to 10 m.
- Excavation of 7 no. test pits to nominal depths of 3 m
- Undertaking Cone Penetration Tests (CPTs) with pore pressure measurement to refusal at 8 no. locations
- Undertaking wireline geophysical testing in select boreholes
- Undertaking in-situ testing including Dynamic Cone Penetrometer (DCP) testing, Standard Penetration Tests (SPT),
- Logging and photographing of samples
- Recovery of disturbed and undisturbed samples including pushed thin wall tube samples
- Development and monitoring of installed groundwater monitoring bores
- Geotechnical Laboratory testing including:
  - soil moisture content tests
  - soil classification tests
  - one dimensional consolidation tests
  - unconsolidated undrained triaxial tests
  - chemical testing

### 6.3.2 Summary of Boreholes Test pits and CPTs

#### *6.3.2.1 Boreholes*

Twenty three boreholes were drilled during the terrestrial investigation, in locations as shown on Figure 6-1 and in the figures included in Appendix A. The boreholes consisted of 11 no. geotechnical boreholes installed with groundwater monitoring wells including 5 no. holes drilled to a nominal depth of 50 m, 5 no. boreholes drilled to a nominal depth of 15 m and 1 no. borehole drilled to a depth of 29.95 m. These boreholes were designated with the suffix 'GW' to identify them as groundwater monitoring bores. Seven additional geotechnical boreholes were drilled to a nominal depth of 15 m and backfilled with grout upon completion.

Five boreholes were drilled adjacent to the nominal 50 m deep boreholes to approximate depths of between 7 m and 10 m and were completed as groundwater monitoring wells. These boreholes were designated with the prefix MB to identify them as dedicated groundwater monitoring bores with no geotechnical testing.

The boreholes were drilled using a truck mounted rotary rig, with solid auger techniques employed near surface followed by rotary wash boring techniques generally until termination depth. In one borehole NMLC rotary coring was undertaken towards the base of the hole where siltstone was encountered. Within geotechnical boreholes SPTs were performed or thin walled tube samples (U63) obtained at regular depths. Pocket penetrometer testing was undertaken in the material recovered in the U63 tubes.

#### *6.3.2.2 Test pits*

Seven test pits were excavated during the terrestrial investigation to depths ranging between 3.0 m and 3.15 m, in locations as shown on Figure 6-1 and in the figures included in Appendix A. The test pits were excavated using a backhoe, with disturbed soil samples collected at select depths. A DCP test was performed adjacent each test pit to a maximum depth of 1.5 m or refusal. Pocket penetrometer testing was undertaken at select locations within the pit walls.

#### *6.3.2.3 CPTs*

CPTs were carried out at 8 no. locations as shown on Figure 6-1 and in the figures included in Appendix A. In seven of the eight locations multiple tests were performed, either due to refusal of the CPT cone, or in order to perform a dissipation test. CPTs were pushed to refusal to depths ranging between approximately 2.5 m and 32.1 m. The CPTs were carried out using a purpose built truck mounted rig with a stated maximum pushing force of 20 tonnes fitted with a piezocone. A total of 5 no. dissipation tests were carried out during the fieldwork program at selected sites.

### 6.3.3 In situ testing

In situ testing carried out during the terrestrial investigation included SPTs, DCPs, pocket penetrometer testing, and wireline geophysical logging.

SPTs were carried out in geotechnical boreholes generally at 1.5 m intervals, except when alternated with thin wall tube sampling in fine grained soils. DCP tests and in situ pocket penetrometer testing was carried out in conjunction with test pits as described in section 6.3.2.2 above.

In order to assist screen placement for groundwater monitoring well installations wireline geophysical logging was carried out in 5 no. PVC cased boreholes drilled adjacent the 5 no. nominal 50 m deep boreholes specifically for this purpose. Natural gamma and bulk density logging was carried out down the full depth of each borehole. The PVC casing was backfilled with grout following completion of testing. In four boreholes the PVC casing was slotted prior to installation to allow grout penetration to mitigate the potential of hydraulically connecting possible upper and lower aquifers. This was not carried out in the borehole drilled for wireline geophysical logging adjacent GA14-BH5GW.

### 6.3.4 Groundwater well development and monitoring

Groundwater wells were installed in a total of 16 terrestrial boreholes. The wells were constructed of nominal 50 mm ID PVC casing, with machine slotted screens. As it was considered likely that multiple aquifers underlie the site, the well screens were installed at levels chosen with the intention of monitoring either an upper or lower aquifer.

Following installation each of the groundwater wells were developed using airlifting techniques. A number of the wells were installed with water level data loggers to facilitate continuous monitoring of groundwater levels. The purpose of undertaking groundwater monitoring is to inform hydrogeological studies to be undertaken as part of environmental assessment work stream studies.

#### 6.3.5 Laboratory Testing

Geotechnical laboratory testing was carried out on select soil samples obtained from boreholes and test pits during the terrestrial investigation. Laboratory testing was scheduled by Golder Associates, with input from the AECOM+GHD JV. Completed tests included:

- Soil moisture content (120 no.)
- Atterberg Limits (58 no.)
- Linear shrinkage (44 no.)
- Particle size distribution –sieving (52 no.)
- Particle size distribution –hydrometer (50 no.)
- Unconsolidated undrained triaxial -single stage (16 no.)
- One dimensional consolidation (3 no.)
- Chemical aggressivity suite - pH, Sulphate, Chloride, Electrical Conductivity (22 no.)
- Total organic carbon (5 no.)

#### 6.3.6 Factual Report

Results of the terrestrial geotechnical investigation are given in the following Golder Associates factual report - Factual Report on Terrestrial Geotechnical Investigation (137612127-010-R-Rev0).

### 6.4 Marine Geotechnical Investigations

#### 6.4.1 Scope of Investigation

The objectives of the marine geotechnical investigation were as follows:

- To define the geological stratigraphy and associated geotechnical properties of the seabed strata
- To characterise the physical, chemical and engineering properties of the underlying sediment, soil and rock
- To provide interpretation of the subsurface seabed conditions to inform decisions on the suitability of material for reuse in reclamation works and for preliminary assessment of dredging techniques and production rates
- To inform the concept design of seawalls, reclamation works and wharf structures
- To undertake complementary environmental sediment sampling to inform future studies

The marine geotechnical investigation was carried out between 14 December 2013 and 24 July 2014 by WorleyParsons. The completed scope of the marine geotechnical investigation included:

- Port Area - drilling 75 No. boreholes to nominal depths of between 5 and 50 m.
- Port Area – undertaking 21 No. cone penetrometer tests (CPTs). with pore water pressure measurement to refusal
- North Arm – drilling 10 No. boreholes to around -22.5 mCD
- Western Channel – drilling 2 No. boreholes to around -23.5 mCD
- Anchorage – drilling 3 No. boreholes to around -23.5 mCD

- North Port Area – drilling 4 No. groundwater boreholes to up to 50 m below the seabed.
- Undertaking in-situ testing including Standard Penetration Tests (SPT) and in situ vane testing
- Logging and photographing of samples
- Recovery of disturbed and undisturbed samples including thin wall tube samples and piston samples
- Geotechnical and environmental laboratory testing

### 6.4.2 Summary of boreholes and CPTs

#### *6.4.2.1 Boreholes*

Boreholes in the Port Area include the following:

- shallow boreholes to approximately 3 m below the anticipated dredge depth to enable material to be characterised and tested ('D' boreholes)
- deep boreholes to approximately 50 m depth or to prove adequate founding materials and for determination of key parameters for subsequent engineering design ('S' boreholes)
- boreholes to approximately 15 m depth or sufficient depth to identify and characterise sediments and areas of soft and compressible strata in areas of potential land reclamation, land side support infrastructure and landside structures ('R' boreholes)

The borehole naming system was used to assign termination criteria for each of the boreholes. As the final footprint and form of any future development is still under consideration, boreholes could not be located to target specific dredge, structure or reclamation areas. Boreholes assigned as 'D' represent areas where reclamation and/or wharf structures are unlikely to be constructed (with the exception of the D30 series boreholes) and therefore boreholes in this area were drilled to sufficient depth to investigate potential dredge materials. The seaward line of 'R' and 'S' boreholes represent the assumed maximum extent of reclamation or wharf structures, and boreholes between shoreline and this outer row of 'R' and 'S' boreholes provide a mix of deep (S) and shallow (R) boreholes. Boreholes in this area have not been positioned to target specific structure locations and, may also represent potential dredge areas.

Boreholes in the Port Area were typically drilled on a 500 m x 500 m grid and included 17 'S' boreholes, 34 'D' boreholes and 24 'R' boreholes. As shown on Figure 6-1 and in the figures included in Appendix A seven of the 'D' boreholes were drilled between the main 500 m gridlines to provide improved coverage in this part of the Port Area. These boreholes were selected to be drilled after sand was identified in a number of the surrounding boreholes on the main 500 m grid.

In the North Port Area four boreholes were drilled to approximately 50m depth to provide information on the lithological profile and to inform future hydrogeological studies.

In the North Arm 10 boreholes were drilled to approximately -22.5 mCD to target dredge areas along the anticipated channel alignment. Two boreholes were located to the west side of the Crib Point berth pocket where rock was expected to be present at seabed or where complex reflectors were identified by the geophysical survey.

In the Western Channel two boreholes were drilled to approximately -23.5 mCD to target dredge areas along the anticipated channel alignment and were positioned in areas where rock was expected to be present at seabed based on a preliminary interpretation of the geophysical survey results. Ten boreholes were originally planned to be drilled in the Western Channel however only two boreholes were completed as part of this phase of investigation.

In the Anchorage area three boreholes were drilled to approximately -23.5 mCD to target dredge areas and were located where rock or complex reflectors were identified by the geophysics survey.

All boreholes were drilled from a jack up barge using rotary mud techniques utilising a wireline cutting tool in conjunction with an HWT drill string. Where drilling in rock was completed the borehole was generally advanced using HQ size drill string in conjunction with a wire line HQ core barrel.

#### 6.4.2.2 *CPTs*

CPTs were undertaken at 18 borehole locations and at three standalone locations in the Port Area. CPTs were typically pushed from seabed to refusal however a drill-out procedure was implemented following initial refusal of the CPT at three locations, whereby potential and identified hard ground layers were drilled out and the CPT completed in the potentially weaker layers at depth. CPTs were pushed to refusal to depths ranging between approximately 0.5 m and 18 m. Six dissipation tests were also undertaken where very soft or soft soil was encountered.

The CPTs were carried out using a 200kN (maximum thrust) Hyson cone penetration testing unit mounted on the jack up barge and used Class 1 and Class 2 piezocones. Where soft soil was expected from seabed a Class 1 piezocone was used to penetrate the soft soil followed by testing with a Class 2 piezocone to refusal.

### 6.4.3 In situ sampling and testing

In situ sampling and testing at each borehole location typically included the following:

- Collection of seabed grabs samples using a 5 litre Van Veen sampler
- Split barrel sampling with Standard Penetration Test (SPT) at 1.5m intervals in cohesionless soils and in very stiff or hard cohesive soils
- Thin walled tube sampling (U63) at 1.5 m centres in firm and stiff cohesive soil
- Piston sampling in very soft or soft soil
- Windowless sampling to assist in the collection of environmental samples and disturbed samples for geotechnical testing
- In situ vane testing in very soft and soft soil including measurement of the residual value of undrained shear strength. Shear vane readings are reported on the borehole logs in terms of torque and estimates of the peak and residual shear strengths are given in the Factual Report.
- Hand shear vane testing was undertaken on undisturbed samples and is reported on the borehole logs along with the residual shear strength where recorded.

Materials recovered from the boreholes were logged in general accordance with AS 1726 – *Geotechnical Site Investigations*, by a geotechnical engineer or engineering geologist from WorleyParsons who supervised and managed the sampling and testing program.

### 6.4.4 Overview of Laboratory Testing

Laboratory testing has been undertaken to characterise the physical, chemical and engineering properties of the underlying sediment, soil and rock. A complementary environmental sampling and testing program has also been undertaken but is not discussed further in this report.

#### 6.4.4.1 *Potential reclamation and port structure areas*

*Laboratory testing in potential reclamation and port structure areas generally comprised the following:*

Where low strength and compressible soil was encountered laboratory testing was aimed at providing key parameters for compressibility and shear strength. Laboratory testing typically included the following:

- Systematic classification testing of soft soils for moisture content, Atterberg limits, and particle size distribution with hydrometer. Testing frequency was typically at 1.5 m depth intervals.
- Strength and compressibility testing of selected undisturbed samples of soft soil for undrained shear strength (unconsolidated undrained triaxial tests), effective stress triaxial testing (3 x single stage) and oedometer tests. Test frequency was approximately 1 to 2 tests on undisturbed samples per borehole. Classification testing was also undertaken on all triaxial and oedometer test samples.
- Organic content testing of selected samples where organic materials where identified

In materials other than soft soils laboratory testing was undertaken to confirm material classification and strength descriptions of materials. Laboratory testing typically included the following:

- Classification testing on selected samples to include moisture content, Atterberg limits and PSD with hydrometer for cohesive soils, and PSD with hydrometer for non-cohesive soils. Testing frequency was at approximately 3 to 5 m depth intervals.
- Undrained shear strength (unconsolidated undrained triaxial tests) on selected undisturbed cohesive samples. Testing frequency was at approximately 3 to 5 m depth intervals where undisturbed samples were taken in firm or stronger cohesive soils.
- Effective stress triaxial testing on selected undisturbed cohesive samples (3 x single stage). Classification testing was also undertaken on all triaxial test samples.
- Organic content testing of selected samples where organic materials were identified.

Where rock was encountered laboratory testing was undertaken to provide information on material strength and typically included the following:

- Point load testing
- UCS testing

#### *6.4.4.2 Potential dredge areas*

Laboratory testing in potential dredge areas generally comprised the following:

Samples in this area between existing seabed and RL -20 mCD were tested to categorise materials for dredging and potential re-use. Laboratory testing typically included the following:

- Classification testing for moisture content, Atterberg limits, and particle size distribution with hydrometer. Where clayey non cohesive soils were present Atterberg limits were also undertaken. Testing frequency typically included 1 to 2 tests in the upper near surface sediments and at 1.5 m depth intervals to RL -20 mCD.
- Undrained shear strength (unconsolidated undrained triaxial tests) of undisturbed cohesive samples. Testing frequency at approximately 3 m depth intervals where undisturbed samples were taken in cohesive soil in the potential dredge zone.
- Selected testing of cohesive soils and non-cohesive soils for particle specific gravity.
- Selected testing of abrasivity, particle angularity, and mineralogy of non-cohesive soils.
- Organic content testing of selected samples where organic materials were encountered

For soils below -20 mCD in this area laboratory testing was undertaken to confirm material classification and strength descriptions of materials. Laboratory testing typically included the following:

- Classification testing for moisture content, Atterberg limits, and particle size distribution with hydrometer for cohesive soils, and particle size distribution with hydrometer for non-cohesive soils. Testing frequency was typically at 3 to 5 m depth intervals.
- Undrained shear strength (unconsolidated undrained triaxial tests) testing of selected undisturbed samples of cohesive soils. Testing frequency was typically at 3 to 5 m depth intervals where undisturbed samples were taken in cohesive soil.

### 6.4.5 Laboratory Tests

The following laboratory tests have been carried out on selected soil and rock samples recovered from the ground investigation. Laboratory testing has been completed by NATA certified laboratories and in accordance with the relevant Australian or international standards. A summary of the number of each type of test completed is given in Table 6-1. A number of laboratory tests were scheduled to be completed on soft soils recovered in piston samples but were not completed due to sample disturbance which occurred during recovery, transport and/or storage, which is a function of their low strength.

**Table 6-1 Marine Geotechnical Investigation - Laboratory Test Summary**

| Test | Number | Standard | Comment |
|---|---|---|---|
| Soil moisture content | 861 | AS 1289.2.1.1 | |
| Atterberg Limits | 396 | AS 1289.3.1.1, AS 1289.3.2.1, AS 1289.3.3.1 | |
| Particle size distribution – analysis by sieving | 97 | AS 1289.3.6.1 | |
| Particle size distribution – Standard method of fine analysis using a hydrometer | 620 | AS 1289.3.6.3 | |
| Particle size distribution – Standard method of fine analysis using a hydrometer (using seawater in-lieu of distilled water). | 170 | AS 1289.3.6.3 | It is understood that the standard test method was modified to remove the addition of dispersant to the soil sample |
| Undrained triaxial compression without measurement of pore water pressure (Single Stage) | 137 | AS 1289.6.4.1 | |
| Saturated consolidated triaxial compression with pore water pressure measurement (Single Stage) | 7 | AS 1289.6.4.2 | |
| Saturated consolidated triaxial compression with pore water pressure measurement (3x Single Stage or multistage) | 18 | AS 1289.6.4.2 | Sets of 3 x Single Stage tests have been completed where sufficient sample was available, alternatively multi-stage testing was completed |
| One-dimensional consolidation properties of a soil (8 Stage) | 36 | AS 1289.6.6.1 | A number of tests report less than 8 load stages due to errors with the recording measurements |
| Particle Specific Gravity | 178 | AS 1289.3.5.1 | |
| Organic Content | 91 | APHA2540E (NATA) APHA5310B (NATA) | |
| Permeability Testing (constant head) | 22 | AS 1289.6.7.3 | |
| Abrasivity testing | 19 | ASTM GA075 | Standard test method for determination of slurry abrasivity (Miller Number) and slurry abrasion response of materials (SAR Number) |
| Angularity and Roundness – | 77 | - | Scanning electron microscope imagery |
| Petrographic Analysis of soil | 136 | - | Silica, quartz and carbonate content by X-ray diffraction |
| Rock point load test | 19 | AS 4133.4.1 | |
| Rock Uniaxial Compressive Strength (UCS) test | 11 | AS 4133.4.2 | |
| Rock - Petrographic Analysis | 5 | - | |
| Rock – X-ray diffraction and visual observation | 1 | - | |

| Test | Number | Standard | Comment |
|---|---|---|---|
| Rock - Cerchar Abrasivity | 2 | ASTM D7625 – 10 | |
| Durability Testing | 89 | Resistivity: inverse of conductivity by 1:5 aqueous extract | |
| | | pH – NEPM Schedule B(3) Method 103 | |
| | | Chloride: MGT Method 1100A | |
| | | Sulphate: MGT Method 1100A | |

#### 6.4.6 Factual Reports

Results of the marine geotechnical investigation are reported in WorleyParsons' Factual Report on Marine Geotechnical Investigation (301010-01290-SS-REP-0001). The environmental aspects of the marine site investigations including the results of the environmental testing are reported in WorleyParsons' Factual Report on Environmental Sampling (301010-01290-SS-REP-0002).

# 7.0 Site Geology

## 7.1 General

A general description of the main geological units encountered in the study area is given below. The site geology was found to be generally consistent with the published data discussed in Section 4. The Section 7 discussion below is intended to summarise the nature and presence of the geological units identified. The spatial distribution and material characteristics of the units are discussed further in Sections 8 and 9.

## 7.2 Anthropogenic Deposits

### 7.2.1 Old Tyabb Reclamation (Fill 1)

Old Tyabb Reclamation Fill comprises variable deposits of fill of sand and clay, and in places was observed to include cobble sized lumps of clay in a sand/sandy clay matrix. The basal fill materials more typically comprised very loose to medium dense sand and soft to firm sandy silt/ clay. The cobble sized lumps of clay are interpreted as being remnant "balling" of material from cutter suction dredge operations.

No reclamation records have been sourced but it is understood that the fill was likely placed in an uncontrolled manner (i.e. not systematically placed and compacted in a controlled manner). Lower levels of fill are soft/ loose and compressible.

### 7.2.2 Other Fill (Fill 2)

Reclamation for the approach to the BlueScope Steel Wharf consists of fill. Limited investigation suggests this fill varies from firm to stiff sandy clays to medium dense clayey sands.

Outside of the known reclamation areas fill encountered consists of dense to very dense sandy gravels, generally associated with road or track formation.

## 7.3 Quaternary Deposits

### 7.3.1 Dune Sands (Q1)

Where encountered in the terrestrial environment the Quaternary dune sands typically consist of loose to dense silty sands and sands. The materials are typically pale grey to grey or pale brown to brown, with fine to medium grain sand.

### 7.3.2 Alluvium (Q2)

Inferred Quaternary alluvium encountered within the terrestrial project area consists of fine to medium grained pale brown to brown silty sand. This material was disturbed during the cultural heritage investigation and the insitu density is not known.

### 7.3.3 Quaternary marine deposits (Q3)

Quaternary marine deposits were encountered from seabed and typically comprise very loose and loose carbonate and siliceous sand, soft and very soft silty clay and sandy silt, and loose and very loose silty sand and clayey sand. These materials are typically brown and dark grey/ black and contain a high percentage of shell fragment, traces of coral, plant material and organic matter, ferricrete gravel and gravel sized shell fragments.

### 7.3.4 Undifferentiated Quaternary (Q4)

Undifferentiated Quaternary deposits were encountered beneath the Quaternary marine deposits and typically comprise loose to dense sands, silty sands, clayey sands and soft sandy silt with shell fragments and organic matter. The materials range in colour from dark grey, brown, orange, mottled grey and brown to pale grey. These deposits may have been derived from reworking of Tertiary deposits and in some areas were found to comprise similar materials as the underlying Baxter Formation but also contained shell fragments and were of significantly lower strength than typical Baxter Formation soils.

## 7.4 Baxter Formation (Tb)

Baxter Formation includes variable silty sand, silty clay, clayey sand, sandy clay, sand, clayey silt and clay materials with the predominant material types being clayey sand, sandy clay, silty sand and silty clay. These materials are commonly grey mottled red and orange brown, pale grey, dark grey and brown and locally contain gravel, iron cementation, and brown coal or carbonaceous material. This unit is typically relatively competent comprising clay and silt materials of stiff to very stiff consistency and dense to medium dense sand materials. However zones of weaker material are also present and comprising very loose and loose sand. Clay materials are typically medium to high plasticity and silts are typically of low plasticity. Sands are typically siliceous, fine to medium grained and sub-rounded to sub-angular.

Clay materials at the top of this unit occasionally exhibit fissuring ranging from well-developed and closely spaced to poorly developed fissures (up to 50mm across and 1.3m deep) which are typically infilled with softer materials.

This unit contains numerous weakly to moderately cemented material horizons (described on logs as pockets, band, layers, silt and sand beds, cemented clay) typically 10-100 mm thick.

Gravels, where present, typically occur as a minor component and comprise angular and sub rounded silica gravel, in sand, clayey sand or sandy clay matrix. Angular and sub-rounded basalt/igneous gravel in silty clay and silty sand matrix also occur as a minor component.

By inspection of borehole logs at 500 m centres there does not appear to be any significant lateral continuity of material types between adjacent boreholes. Material type also repeatedly changes with depth with the typical thicknesses of material beds in individual boreholes ranging from one to five metres. In the offshore boreholes differentiation between the terrestrially deposited Baxter Formation and the underlying marine deposited Sherwood Formation appears to be indistinct. The Baxter Formation is encountered underlying the Quaternary deposits across the site.

## 7.5 Sherwood Formation (Tm)

The Sherwood Formation comprises orange brown, brown, grey brown, grey, dark grey, green-grey and olive sand, silty sand, silt, sandy silt, silty clay and clayey silt, with local fine to medium carbonate shells and shell fragments. Silty sands and sands are typically fine or fine-medium grained, sub-angular or sub-rounded and of medium dense to very dense consistency. Sands are typically siliceous but also include carbonate and calcareous sand. Sandy silt, silt, silty clay and clayey silt are typically of stiff to hard consistency but also include zones of softer material. Clay and silt materials were typically of low to medium plasticity with some high plasticity material encountered.

This unit includes frequent cemented horizons (beds and bands) and gravel sized pockets of cemented material, typically described on borehole logs as weakly to moderately cemented and 10-200 mm in thickness. Well cemented horizons up to 200 mm thick are also present. Cemented horizons often occur towards the top of the Sherwood Formation (below the Baxter Formation) and at the base of this unit. Cemented materials include silty sand, sand, silt, clays, shell fragments and gravel size cemented/mudstone fragments. High SPT N values or SPT refusal was frequently encountered on these cemented horizons. However high SPT N values or SPT refusal was also frequently encountered in very dense sands which were not cemented. This unit also includes shells and carbonate nodules up to 7 mm and weakly cemented sand nodules up to 20mm diameter.

Differentiation of the Sherwood Formation from the Baxter Formation would appear to be difficult in some cases. By comparing adjacent borehole descriptions (albeit some 500 m apart) there does not appear to be an obvious continuity or consistency of material type across the site. The reason for this is not known but may be due to a number of factors including i) the presence of recent sediments deposited in eroded channels derived from re-working of the Baxter and/ or Sherwood Formation, ii) varied lateral extent of the respective formations, iii) varied depositional environment and transitional boundary that may have included both marine and terrestrial episodes, and iv) sub-aerial weathering and oxidation of the green grey Sherwood Formation during periods of exposure and erosion when sea levels were lower.

The unit is overlain by Baxter Formation and overlies the Yallock Formation in the northern part of the Port Area and Silurian sandstone and siltstone in the southern part of the Port Area. Only six boreholes in the port area proved the base of this unit.

## 7.6 Yallock Formation (Ty)

Yallock Formation comprises sand, silty sand, gravelly sand, sandy silt, silt, clay, peat and coal. Sands are typically grey-brown, pale brown to yellow and fine to coarse grained and sub rounded to sub angular, are of medium dense to very dense consistency and include traces of organic matter. Silts and clays are of low to high plasticity and relatively competent being of stiff to hard consistency and also include organic materials. Hard peat and very low strength dark brown-dark grey coal are also present in this unit.

The unit is overlain by Sherwood Formation and rests on Older Volcanics and basement rocks.

## 7.7 Older Volcanics Group (Tvo)

Older Volcanics were encountered at depth in the historical boreholes at Crib Point and Stony Point and near surface in the boreholes between The Nobbies and Sandy Point, as a variably weathered material ranging from fresh rock to extremely weathered boulders and cobbles in a clay matrix. Older Volcanics were also encountered in the recent investigations as a highly to moderately weathered basalt rock mass in the Western Channel from seabed and are described as extremely low to very high strength. This unit was also encountered in the Anchorage area from a depth of 0.7 to 1.5m below seabed and described as a gravelly clay of high plasticity and stiff consistency, and extremely to moderately weathered basalt, typically of medium to high strength. Variably weathered (extremely weathered to slightly) basalt of medium strength was also encountered in the Port Area below the Yallock formation at a depth of 49 m.

## 7.8 Silurian Sandstone and Siltstones (S)

Silurian sandstone and siltstone is present as a variably weathered basement rock and was encountered in the Port Area south of the BlueScope Steel wharf at depths ranging from 22 m to 47 m.

# 8.0 Geotechnical Conditions

## 8.1 Strata Overview

Stratigraphic units have been assigned to the ground profile based on interpretation of the results of the geotechnical investigation and used to develop geological cross-sections for the site. The cross-sections through the Port and North Port area, North Arm, Western Channel and Anchorage, including combined cross-sections through the marine and terrestrial project area are presented in Appendix B.

The cross-sections highlight the interbedded nature and lateral variation of materials across the site, in particular the sand and clay materials within the Baxter Formation where there does not appear to be any apparent lateral continuity of sand or clay horizons based on comparison between adjacent boreholes (noting the wide borehole spacing of 500 m in the port area), and from geophysical interpretation.

### 8.1.1 Terrestrial Project Area

The terrestrial project area generally comprises Baxter Formation materials, with a thin layer of Quaternary deposits present at surface over much of the site. Below the Baxter Formation, Sherwood Formation, Yallock Formation and Silurian rock were encountered. Inferred stratigraphy has been presented on the factual report logs. However, this has been reinterpreted during preparation of this report and some alternative interpretations adopted as discussed below.

**Dune Deposits**

Within the project area the Western Port 1:63 360 scale geological map refers to Quaternary fluviatile deposits. However, this unit on the adjacent Cranbourne 1: 63 360 scale geological map is labelled as 'Pleistocene siliceous sand dunes and sheets (including Cranbourne Sand)'. The 1:250 000 scale seamless geology mapping on GeoVic (accessed October 2014) refers to this unit as Quaternary Holocene coastal dune deposits. Interpretation of the terrestrial borehole logs in conjunction with the published geological mapping infers that materials representative of these deposits may have been encountered from surface, or below the fill layer in nine borehole or test pit locations. The identified deposits range in thickness from 0.3 m to 1.1 m. and is generally described as loose to medium dense silty sand, occasionally sand, with fine to medium grain size. Where encountered within test pits the material is described as weakly cemented, and medium dense to dense. The dunes are present in the central and northern regions of the Terrestrial Project Area, with the thickest deposits towards the north.

**Recent marine deposits**

Due to access and environmental constraints only one borehole was located within the salt marsh/mangrove swamp area of the foreshore. This borehole, GA14-BH8GW encountered what has been inferred as recent marginal marine deposits (salt marsh deposits) to a depth of 1.5 m below a layer of surficial fill. These deposits are described as loose silty fine to coarse sand and highly organic with root, wood and plant matter. A sample tested for total organic carbon returned a result of only 0.9% and may not be representative of the overall mass.

**Fill**

Fill was reported in a number of terrestrial boreholes. In a number of instances the fill was the result of reworked natural materials where the borehole was drilled in a previously excavated and backfilled cultural heritage test pit. Where pre-existing fill was encountered, it generally coincided with roads or tracks within the project area. Reported thickness of this fill varied from 0.1 to 0.7 m, and it consisted of dense to very dense sandy gravels.

Located on the approach to the BlueScope Steel wharf, Borehole GA14-BH5GW was logged as encountering 0.2 m of fill, however based on the known site history and re-interpretation of the materials encountered in CPT (GA14-CPT08) and the findings in other nearby boreholes it is considered likely that approximately 5.5 m of fill is present at this location. These fill materials vary from firm to stiff sandy clays to medium dense clayey sands.

**Alluvium**

Based on surface geological mapping and inspection of the site it is likely that Quaternary alluvium will be encountered in isolated low lying parts of the project area. It has been inferred that the 0.7 to 0.8 m of silty sand encountered from surface in boreholes GA14-BH03 and GA14-BH7GW may represent these deposits. The majority of boreholes were relocated during the investigation to avoid watercourses and therefore are unlikely to describe the full extent of alluvial deposits.

**Baxter Formation**

This unit was encountered at all borehole and test pit locations either from surface or immediately below the surficial Quaternary deposits. The material is described as clays, silts and sands with variable minor components. While there is a general trend of granular soil types being more prevalent at depth, this is not always the case. Clays are generally medium to high plasticity, with silts generally low plasticity. The grain size of the sand components varies from fine to coarse. Traces of gravel were encountered. Fine grained soils encountered range from soft to firm through to very stiff to hard consistency. Coarse grained soils encountered range from loose to very dense. Cemented zones are present in places. Baxter Formation materials were encountered to depths ranging from 9.8 m to 26.9 m below ground surface, with total unit thickness varying from 9.4 m to 22.0 m. All test pits terminated within the Baxter Formation.

**Sherwood Formation**

The Sherwood Formation was encountered in ten of the boreholes drilled, underlying the Baxter Formation. The fine grained soils are generally described as medium to high plasticity, firm to hard sandy clay, silty clay and clay, and low plasticity firm to hard sandy silt and clayey silt. Coarse grained soils are described as medium dense to dense fine to coarse grained silty sand, clayey sand and sand, frequently with shells and shell fragments and occasionally with fine to medium gravel. The material colour is generally shades of grey and green grey, orange brown, brown and grey brown.

The formation was encountered at depths ranging from 9.8 m to 26.9 m below ground surface. Three boreholes penetrated the base of the Sherwood formation, with the thickness of the unit ranging from 11.7 m to 29.0 m.

**Yallock Formation**

This unit has been interpreted in three of the boreholes drilled. The material encountered includes fine to coarse very dense sand, low plasticity stiff to hard silt, high plasticity hard clay, and hard peat. Sand fractions, fine gravel and woody organic material are frequently present. The peat is typically dark grey to black, with other material shades of brown, grey, grey brown, green blue grey and green blue.

The unit was encountered at depths ranging from 33.7 m to 43 m below ground level. Borehole GA14-BH03GW penetrated the base of the Yallock Formation, with the total thickness being 11.4 m. Borehole GA14-BH04GW was terminated 17.0 m into the Yallock Formation.

**Silurian rock**

Silurian siltstone was encountered towards the base of borehole GA14-BH03GW at a depth of 46.6 m and proven to a depth of 51.25 m. The contact was initially inferred at 49.6 m, however interpretation against laboratory data indicates the contact is likely to be at 46.6 m, with the upper layer potentially being either residual soil or extremely weathered rock, and logged as very stiff to hard pale grey silt with trace fine to medium sand. The siltstone is pale grey to dark grey, extremely weathered and extremely low strength.

### 8.1.2 North Port Area

Four boreholes were drilled in the North Port Area to provide details of the lithological profile and to inform future hydrogeological studies. Boreholes G1 to G3 were drilled in relatively shallow water with seabed levels ranging from +0.47 mCD to -2.42 mCD while borehole G4 was drilled in deeper water where the seabed level was -12.16 mCD.

Quaternary marine deposits were encountered from seabed to depths of 1.7 m, typically of very soft consistency, overlying Baxter Formation in three of the four boreholes. Undifferentiated Quaternary deposits were encountered in borehole G3 to a depth of 13 m overlying Baxter Formation. Borehole G2 encountered

dense to very dense sand from 3.5 to 11m overlying very loose to medium dense silty sand from 11 to 20 m and G3 encountered medium dense to very dense sand from 1.2 m to 17.5 m overlying medium dense to dense silty sand from 17.5 to 23.5 m. Undifferentiated quaternary deposits in borehole G3 comprised medium dense to very dense dark grey and dark brown sand with shells up to 13 m depth.

Sherwood Formation was encountered from depths of 4.5m and mainly comprised silty sand, sand and silt of dense to very dense and very stiff to hard consistency. Pockets of soft peat were encountered within grey-green sand at the base of borehole G1 which may represent the top of the Yallock Formation. Borehole G2 encountered loose silty sand at 21.5m depth overlying silty clay, silty sand, and silt with multiple cemented beds up to 300mm thick at 0.5 m to 1.0 m intervals from 23.4 m to 32 m depth.

### 8.1.3 Port Area

Ground conditions in the Port Area typically comprise Quaternary marine and undifferentiated Quaternary deposits from seabed overlying Baxter Formation which overlies Sherwood Formation. The base of the Sherwood Formation was only proved at five locations in the 2014 investigation; at depths of 22 to 28 m, south of the BlueScope Steel wharf, in boreholes S14 and S16 where it overlies Silurian mudstone, siltstone and sandstone; and at depths of 45 to 50 m in the northern part of the Port Area in boreholes S8, S5 and R1 where it overlies Yallock Formation. The base of the Yallock Formation was only proven in one borehole, R1, at the northern end of the Port Area where it overlaid Older Volcanics. Quaternary marine deposits are also present beneath the Old Tyabb reclamation fill, and within the intertidal zone.

**Quaternary marine deposits**

Materials encountered from seabed comprised a surficial layer of low strength Quaternary marine deposits typically very loose and loose carbonate and siliceous sand. Soft and very soft clay, silty clay, sandy silt and very loose and loose sands are present up to 6 m depth with the greatest thickness of these weak materials occurring in the area north of the BlueScope Steel wharf where soft and very soft clays are present from surface or are overlain by a thin layer of sand materials. Very soft and soft clayey quaternary deposits are typically grey to dark grey, of high plasticity and containing shell fragments. Boreholes in the intertidal zone north of borehole S15 typically comprised very loose sand up to 0.5 m thickness. There is a zone of deeper Quaternary deposits in the northern part of the port area in water depths ranging from around -1 mCD to -8mCD. Moving further offshore the depths of these surficial deposits are typically less than 0.5 m.

The three standalone CPT's between Long Island Point and BlueScope Steel Jetty all encountered a weak surficial layer of low strength materials assumed to be Quaternary marine deposits to depths of 3.3 to 4.0 m. These CPT's are located approximately 100 m inshore from the alignment of the 2009 boreholes BH4 – BH6 and where seabed levels are around -1.0 mCD. Interpretation of soil behaviour type as given by Robertson (2010), whereby soils are categorized into one of nine soil types based on the cone penetration test results, shows these materials to include organic soils, sensitive fine grained soils and clay.

The original location for the 2009 borehole BH6 was abandoned when the legs of the jack-up barge reached full extension without encountering strata of sufficient bearing capacity to lift the barge (at approximately RL -17.75 mCD). As part of the 2013/2014 investigation a borehole was planned at the original BH6 location however the legs of the jack-up barge again reached full extension without encountering sufficient bearing capacity.

The original location of borehole R16 was also abandoned and moved approximately 150 m as a result of soft or loose material at seabed and the jack-up barge being unable to achieve sufficient bearing capacity within the available leg length. At the revised R16 location soft sandy silt and loose silty sand was encountered to a depth of 1.2 m overlying firm silt to a depth of 4.2 m which overlaid loose to medium dense clayey sand to a depth of 7.6 m. The seabed level at the revised R16 location was -9.47m CD.

Undifferentiated Quaternary deposits were encountered in approximately 20% of the boreholes in the Port Area and North Port Area. They underlie the Quaternary marine deposits and are typically present up to depths of 2m. Dense to very dense sand is present in Borehole G3 in the North Port Area underlying very soft Quaternary marine deposits from 1.2 m to 12.0 m depth. These sands are fine to medium grained and contain carbonate shells up to 10 mm diameter.

The Quaternary deposits described above overlie the Baxter Formation which includes clayey sand, sandy clay, silty sand and silty clay. This unit is typically relatively competent comprising clay and silt materials of stiff to very stiff consistency and medium dense to dense sand materials. The materials within this unit exhibit vertical and lateral variability across the Port Area.

**Baxter Formation**

The Baxter Formation was encountered to depths of around 10 to 15 m and up to 20 m in some locations. The elevation of the base of the Baxter Formation is around -20 to -25 mCD in the outer four rows of northeast – southwest boreholes (Appendix B Geological cross-sections *Port Area 3* to *Port Area 6*) which are located in relatively deep water and at around -10 mCD in the boreholes in the intertidal and shallow water rows (Appendix B Geological cross-sections *Port Area 1* and *Port Area 2*).

This unit contains several zones of brown and dark brown silty sand, silty clay, silt, and sandy silt which are described as having an organic appearance but which were relatively competent, typically medium dense sands and very stiff to hard clays and silt. The average thickness of these "organic appearance" horizons is around 5 m, occurring in a cluster of boreholes in the northern part of the Port Area (D32, D35, D36, D38, D39, S6) at an elevation ranging from -10 to -18 mCD. Results of organic testing on these materials is discussed in Section 8.2.

Organic or potentially organic materials were also identified within the Baxter Formation in several other boreholes in the Port Area including a 3 m zone of dark brown loose silty sand in borehole R22 at an elevation of -8.73 mCD. Dark brown/black peat seams were identified in a 1.5m thick horizon of brown loose silty sand in borehole R18 at an elevation of -7.8 mCD, and in a 5.5 m thick horizon of dark brown silty sand and clayey sand from 0.7 m depth in borehole D30 at an elevation of -12.12 mCD.

Dark brown to black and dark grey to black horizons and thin lenses of materials including silty sand, sandy silt, clayey sand, silty clay and clayey sand are present in the Baxter Formation in a number of adjacent boreholes in the Port Area (D12, D14, D16, D17, R19, S11) at around a similar elevation from -15 mCD to -20 mCD. The consistency of these materials ranges from firm to very stiff and loose to very dense.

Although the Baxter Formation generally comprises relatively competent materials, there are frequent horizons of typically dark grey or dark brown very loose and loose silty and clayey sands with SPT N values of less than 10. The average thickness of these layers is around 1.5 m (which may be a function of the spacing of SPT tests which was also at 1.5m depth intervals) but can be as great as 5 m as seen in borehole R3 which encountered very loose to loose brown silty sand from 6 to 11 m depth (approximately -11 mCD to -16 mCD). Infrequent soft to firm horizons are also present within the Baxter Formation as summarised in Table 8-1 below. Low SPT N values of less than 5 and soft horizons are highlighted on the cross sections included in Appendix C and indicate that there does not appear to be any lateral continuity between these horizons of low strength materials.

**Table 8-1 Summary of Occurrence Low Strength Soils in Baxter Formation**

| Borehole | Material | Colour | Consistency | Depth | | RL | |
|---|---|---|---|---|---|---|---|
| - | - | - | - | (m) | | (mCD) | |
| | | | | From | To | From | To |
| D15 | Clayey silt | Dark brown | Soft-Firm | 7.5 | 8.7 | -21.58 | -22.78 |
| D23 | Sandy Clay | Dark grey, dark brown | Soft-Firm | 6.0 | 9.5 | -20.71 | -24.26 |
| R3 | Sandy clay | Brown | Firm | 15 | 15.75 | -20.01 | -20.76 |
| R8 | Clayey sand and clayey silt | Dark brown | Firm | 9 | 12 | -16.68 | –19.68 |
| R10 | Sandy clay | Grey | Firm | 15 | 17.5 | -18.09 | -20.59 |
| R16 | Silt | Dark brown | Firm | 1.2 | 4.2 | -10.67 | -13.67 |
| S11 | Silty Clay | Pale brown & pale grey mottled | Firm | 0.3 | 1.0 | -8.32 | – 9.02 |
| S11 | Sandy silt | Dark grey | Firm to stiff | 7.85 | 8.5 | -15.87 | -16.52 |

| Borehole | Material | Colour | Consistency | Depth | | RL | |
|---|---|---|---|---|---|---|---|
| S15 | Silty clay & sandy clay | Dark grey to black | Firm/Firm to Stiff | 10 | 14.5 | -18.02 | -22.52 |
| S16 | Silty Clay | Dark grey | Very soft & Firm | 2.5 | 3.5 | -13.27 | -14.27 |
| 2009 BH5 | Silty Sand, Clayey Sand, Sandy Clay, Clay | Grey-Dark Grey | Loose, Soft - Firm | 0.8 | 8.0 | -13.25 | 20.45 |

The geological cross sections in Appendix C also highlights SPT N values of 50 or greater (and N=R). The Baxter Formation includes multiple cemented horizons which are typically weakly to moderately cemented but have also been described as well cemented. These cemented horizons are not always coincident with high SPT N values; cemented materials can be present where SPT N values are as low as 1 and SPT refusal has been encountered in very dense sands which are not cemented. The cementation classification system used in the 2013/2014 marine geotechnical investigation is given in Table 8-2 and examples of low SPT N values in cemented materials is given in Table 8-3.

Borehole D23 encountered a layer of dark grey, dark brown medium plasticity soft to firm sandy clay from 6 to 9.55 m depth (-20.71 mCD to -24.26 mCD) which included 3-10 mm thick layers of moderately weakly to moderately cemented material from 7.5 m, and 2 -15 mm thick layers of well cemented silcrete from 8.5 m depth.

Siltstone/ Cemented Clay (brown ferruginised) was encountered in borehole R20 from a depth of around 16 m (-16.82 mCD) for 3 m, overlying cemented clay of the Sherwood Formation which was cored to a termination depth of 23.5 m. As a result of significant core loss in the Siltstone/ Cemented Clay the estimated strength of this material is only reported for a 0.5 m depth interval and ranged from extremely low to high strength with a defect spacing ranging from 300 mm in the extremely low strength material, to 20 mm in the high strength material.

**Table 8-2 Classification System of Cemented Sands adopted for Material Description**

| Term | Definition |
|---|---|
| Uncemented | Clean grains exhibiting soil properties |
| Very weakly cemented | Cement on some grains, collapsing feel under very light finger pressure |
| Weakly cemented | Cement on many grains, collapsing feel under finger pressure, breaks down to individual grains |
| Moderately weakly cemented | Cement on most grains, breaks down to lumps under finger pressure, can crush to individual grains under knife blade |
| Moderately cemented | Cement on most grains, can break off by hand and crush to small lumps under knife blade |
| Well cemented | Practically all grains cemented together, cannot break fragments off by hand, dull sound under hammer |
| Very well cemented | Most primary pores filled with cement, requires firm blow with hammer to break off fragments, rings when struck |

**Table 8-3 Low SPT N Values in cemented materials**

| Borehole | Depth (m) | Soil Description | SPT N Value |
|---|---|---|---|
| R3 | 15 | Sandy clay with gravel of weakly cemented sand | 1 |
| R20 | 12.1 | Very stiff silty clay with iron oxide cementation | 3 |
| R20 | 13.5 | Very loose silty sand, weakly cemented in part | 4 |
| R5 | 9 | Medium dense clayey sand with weakly cemented gravel throughout | 11 |

Gravels were encountered infrequently in the Baxter Formation and where present were generally a minor component in clayey sands and sands. A 0.6m thick horizon of gravelly clay was encountered in borehole S6 at an elevation of -16.8mCD.

**Sherwood Formation**

The Baxter Formation in the Port Area overlies the Sherwood Formation which is present at depths ranging from 6-50 m (-5 to -62 mCD) but is typically present from -15 mCD. The base of this unit has only been proved in eight boreholes in the Port Area; three boreholes in the northern part of the Port Area where it overlies Yallock Formation at around -45 to -50 mCD and south of the BlueScope Steel wharf, offshore from the Old Tyabb reclamation, where it overlies Silurian sandstone, mudstone and siltstone from between -33 and -42 mCD.

Sherwood Formation is typically relatively competent comprising silty sands, sandy silts and silt as described in Section 7.2. This unit appears to be more consistent in terms of vertical variation compared to the overlying Baxter Formation, but there does not appear to be any significant lateral consistency between adjacent boreholes as evident from the geological cross-sections included in Appendix B and C.

As discussed in Section 7.5 this unit includes cemented materials occurring both below the Baxter Formation and towards the base of the unit. The degree of cementation ranges from weakly cemented to well cemented and includes;

- pockets of weakly to well cemented materials
- weakly cemented silt beds (carbonate cementing)
- cemented sand
- cemented shell fragments
- gravel size cemented/mudstone fragments
- weakly cemented sand nodules
- zones of moderately cemented silty sand
- occasional weakly cemented bands (within silty sand)

Grey, dark grey, green and dark green cemented zones up to 5m thick were encountered towards the base of this unit at depths exceeding 40m. Multiple cemented horizons in individual boreholes are also present at higher elevations including;

- Borehole R5 encountered brown low plasticity silt of extremely low to very low strength which was cored from 14 m to 20.5 m depth (-16.55 mCD to -23.05 mCD) and included multiple 100 mm thick silcrete horizons.
- Borehole R20 encountered brown cemented clay of extremely low to medium strength which was cored from 18.8 m to 23.5 m depth (-19.62 to -24.32 mCD).
- Borehole S7 encountered weakly cemented dark green sand from 47 m depth which was cored from 48 m to a termination depth of 55 m and described as being low strength and very weakly cemented.
- Borehole S14 encountered multiple weakly to well cemented horizons, including 200 mm thick horizons described as having a consistency of low strength siltstone, from 8.5 m to 20.5 m depth (-19.2 mCD to -31.2mCD).
- Borehole S17 encountered multiple weakly to well cemented horizons, including well cemented horizons up to 400 mm thick, from 13.5 m to 32.2 m depth (-15.53 mCD to -34.23 mCD).
- Borehole D38 encountered multiple cemented beds at approximately 0.5-1.0 m intervals in very stiff grey silty clay from 11 – 15.6 m depth (-16.08 mCD to -20.68 mCD). Three SPTs were performed in this material with SPT N values of 29, refusal and refusal.

SPT N values of 50 or greater (and N=R) are shown on the geological cross sections in Appendix C above RL -25mCD which includes cemented horizons within the Sherwood Formation. SPT refusal was also encountered in very dense layers within the Sherwood Formation which are not cemented. Similar to the Baxter Formation some tests in cemented materials returned low SPT N values (e.g. N=6 in a weakly cemented horizon in S5 at 13.5m).

Low strength horizons comprising loose and very loose silty sands were encountered in the Sherwood formation in a number of boreholes:

- D5 – 0.7 m thick horizon loose silty sand, originally logged as a stiff sandy silt, from 7m (-15.66 mCD)
- S7 – 1.5 m thick horizon of loose silty sand from 7m (-15.66 mCD)
- S9 – 1.5 m thick horizon of very loose silty sand from 18m (-29.27 mCD)
- S10 – 1.0 m thick horizon of very loose silty sand from 10 m (-16.45 mCD)

Silty calcareous sand was encountered in one borehole R1 from 36 to 43 m depth

Soil horizons in the Sherwood Formation containing soft to firm materials at depth were encountered in three locations in the Port Area:

- R1 - 2.1 m thick horizon of dark grey low plasticity soft silt, which is described as being very soft upon remoulding, from 16 m depth (-15.45 mCD)
- S17 – 0.7 m thick horizon of grey high plasticity firm clayey silt, from 16 m depth (-18.03 mCD)
- 2009 BH01 – 2.35m thick horizon of dark grey, low to medium plasticity, firm silt/clay from 8 m depth (-17.8 mCD)

**Yallock Formation and Older Volcanics**

Yallock Formation was encountered in three boreholes at depths of around 45 – 50 m in the northern part of the Port Area (Boreholes R1, S5 and S8). The base of this unit was only proven in one borehole where it overlaid Older Volcanics. The thickness of the Yallock Formation at this location was approximately 5 m. The materials encountered in the Yallock Formation comprised medium to coarse grained dense sand, medium dense gravelly sand, silty sand and hard clay. Very low strength dark brown to dark grey coal was encountered in borehole S8 and organic silt (peat) was encountered in borehole R1.

The basalt encountered in Borehole R1 ranged from extremely to slightly weathered and was of medium strength. The borehole penetrated 1.5 m of basalt before being terminated and did not prove the base of this unit.

**Silurian sandstone and siltstone**

Silurian sandstone and siltstone was encountered in two boreholes in the Port area as part of the 2013-2014 geotechnical investigation, in the area south of the BlueScope Steel wharf. S14 encountered medium strength siltstone at 29 m depth and S16 encountered interbedded low strength mudstone and high strength siltstone at 21.9 m depth. Siltstone and mudstone were both slightly weathered with a fracture spacing ranging from 20 mm to 1 m.

This is consistent with the interpreted rock levels from the geophysical survey and the 2009 geotechnical investigation which encountered siltstone and sandstone in the same area and at a similar elevation.

#### 8.1.4 North Arm

Ten boreholes were drilled in the North Arm as part of the 2013/2014 investigation between the southern end of Long Island Point and Sandy Point. These boreholes were located to provide general coverage through this part of the North Arm and to target high points in the bathymetry or where the geophysical survey identified possible rock close to seabed or complex geophysical reflectors.

Seabed elevation in these boreholes ranged from -12.2 to -17.0 mCD and the borehole depths ranged from approximately 5.5 m to 11.5 m, targeting material within potential dredge depths.

Recent Quaternary marine deposits are present from seabed and typically comprise very loose to loose siliceous and carbonate, brown, orange and grey, sand and clayey sands containing shells and shell fragments, typically to depths of up to 0.5 m. Pieces of coral at seabed are noted in two boreholes and the factual report states that in places the surface sand is capped with a thin 20 mm thick veneer of coralline, cemented sand. Fine to medium grained angular basalt gravel is present as a minor component in one borehole (N2) at surface. A layer of soft to stiff dark grey to black silty clay of medium plasticity was encountered underlying the surficial sands in borehole N1 to a depth of 2 m. This material included a trace of carbonate shell fragments and from 1 to 2 m depth included very weakly to moderately cemented 5-20 mm thick layers of silica and carbonate shell sand. Loose siliceous carbonate sand with silt and shells was encountered in borehole N6 to a depth of 0.95 m. Loose to dense sand and clayey sand with trace shells and silt (dense sands being at surface) were encountered in borehole N9 to a depth of 3.2 m with approximately 30% shells up to 10mm size present from 3.0m depth.

Baxter Formation is present below the Quaternary deposits to depths of up to 8m. These materials typically comprise; medium to high plasticity, mottled brown, grey, and yellow brown, stiff to hard clay and silty clay, and medium dense to very dense sand. Very dense, dark brown to black clayey gravels containing angular basalt in a matrix of brown sandy clay were encountered in boreholes N4 and N5. Clay and sand materials (in the northern part of the North Arm – N2, N3, N4, N5, and N6) also included angular basalt gravel and gravel sized basalt core stones. These basalt gravels are not typical of the Baxter Formation and were generally not encountered in the Port area. However, it is uncertain whether the presence of angular basalt reflects weathering of basaltic rock, or is a result of reworking of Older Volcanics during deposition of the Baxter Formation. Since basalt rock was not encountered in the area and is not indicated on the geophysical survey it is concluded that these materials are sedimentary.

The Baxter Formation was found to overlie Sherwood Formation in six of the ten boreholes. Residual Silurian mudstone was encountered in borehole N3 at a depth of 4.8 m (-21.06 mCD) and the base of the Baxter Formation was not proved in the other three boreholes.

The Sherwood Formation encountered in the North Arm was typically relatively competent with the exception of a 750 mm thick layer of loose sand present in borehole N4 at a depth of 7.2 m. Materials typically comprised stiff to very stiff silty clay, sandy silt and clayey silt, and pale grey to brown dense to very dense sand with gravel, trace of silt and shell fragments.

The residual mudstone in borehole N3 was present as a hard, medium to high plasticity, light grey to grey silty clay. The borehole was terminated in this unit at a depth of 6.75 m, approximately 2 m into residual mudstone. This is the only location where Baxter Formation has been found to rest directly on Silurian and is also the highest elevation of Silurian within the investigation footprint.

A 200 mm thick black coal horizon was encountered below the Sherwood Formation in borehole N11 at a depth of 11.2m (-23.41m CD). The borehole was terminated in this unit. This material has been assigned as Yallock Formation as coal would not be expected to occur as part of the marine deposited Sherwood Formation, however it is noted that the elevation of the Yallock Formation encountered in other boreholes in the Port Area is significantly deeper at around -50 mCD.

#### 8.1.5 Western Channel and Anchorage

In addition to the 1974 investigation discussed in Section 5.5 two boreholes (WC3 and WC4) were drilled in the Western Channel as part of the 2013/2014 investigation. These two boreholes were located on the eastern side of the Western Channel between the northern side of Phillip Island and Sandy Point in water depths of -15.2 mCD and -17.4 mCD. Borehole WC3 encountered loose Quaternary marine deposits at seabed for 0.6 m overlying brown and grey stiff silty clay and medium dense to very dense silty sand. These materials were of low to medium plasticity and included sand to fine gravel sized shell fragments. The materials underlying the Quaternary deposits are assumed to be Sherwood Formation. Borehole WC4 was drilled approximately 3.5 km southwest of WC3 and encountered basalt from seabed which was highly weathered and very low strength at surface but contained high strength corestones. From 2 m depth the basalt was highly to moderately weathered and of high to very high strength. Defect spacing was typically 40 mm. There was no core recovery in the depth range from 0.8 m to 2.0 m (-18.2 mCD to -19.4 m CD) and the borehole was terminated at 6.1 m depth (23.5 mCD).

The results of the geophysical survey shows significant variation in elevation of the rock reflector in the Western Channel ranging from at or just below seabed, as in the case of borehole WC4, to elevations of around -45 mCD. The rock reflector at borehole WC3 is around -37 mCD which is well below the borehole termination depth of -24 mCD.

The 1974 seabed investigation in the Western Channel also encountered variation in rock elevation with only nine of the 27 boreholes in the Western Channel encountering rock.

Three boreholes were drilled in the Anchorage area of the East Arm approximately 3.2 km from the Western Channel and targeted high spots in the bathymetry with seabed levels ranging from -14.77 mCD to -15.48 mCD.

Boreholes A1 and A2 encountered Quaternary marine deposits comprising loose to medium dense sand and clayey sand overlying basalt which was encountered at 0.7 m depth in borehole A1 and 2.1 m depth in borehole A2. Borehole A1 encountered extremely weathered and extremely low strength basalt to 1.5 m depth overlying highly to moderately weathered basalt of medium to high strength to a termination depth of 9.5 m. Borehole A2 encountered highly weathered and very low to low strength basalt to 5 m depth overlying moderately to slightly weathered basalt of high strength to a termination depth of 8.3 m. Defect spacing in boreholes A1 and A2 ranges from 20 mm to 1 m dipping from 0 to 90 degrees but typically in the range of 10 to 40 degrees. Borehole A3 encountered a thin veneer of Quaternary deposits at surface overlying Sherwood Formation comprising medium dense to dense silty sand to the termination depth of 8 m.

## 8.2 Characteristic Subsoil Properties

This section summarises the range of geotechnical properties from the field and laboratory testing for each geological/stratigraphic unit with the exception of the Yallock Formation, the Quaternary dune deposits and anthropogenic fill which have not been included due to the limited number of tests completed on soil samples from these units. No laboratory testing was undertaken on dune deposits due to the limited thickness and extent of the unit across the terrestrial area. It is anticipated that most future structure foundations will penetrate this unit. Due to the identified variability, structures located on areas underlain by anthropogenic fill deposits will require site specific investigation to determine the properties of the fill at that location. It is anticipated that the Yallock Formation will be below the likely depth of any future structures or their zone of influence.

The field and laboratory test results summarised below include results from both the 2009 and 2013/2014 geotechnical investigations.

The data includes the type and number of tests completed, the range of test results (maximum and minimum), upper and lower quartiles and the mean value for each test. Additional summaries of the test results are included in Appendix D - H and include plots of results by depth and/or RL as appropriate. The data summarised below is not statistically reliable because tests were carried out on selected samples only and the numbers of tests are insufficient for statistical analysis.

The following sections should be read in conjunction with the field and laboratory test results given in the factual reports.

### 8.2.1 Old Tyabb Reclamation Fill

The in situ and laboratory test results from the 2009 investigation for Old Tyabb Reclamation Fill are summarised in Table 8-4 and Table 8-5 below. These results are for fill materials in the top 1 m of the reclamation.

**Table 8-4 Index Properties for Clayey Sand**

| | Liquid Limit (LL) | Plastic Limit (PL) | Plasticity Index (PI) |
|---|---|---|---|
| | (%) | (%) | (%) |
| Value | 30 | 15 | 15 |
| Number of tests | 1 | 1 | 1 |

**Table 8-5 CBR & Moisture Content Results**

| Sample Depth | Moisture Content | CBR |
|---|---|---|
| (m) | % | % |
| 0.2-1.0 | 16.3 | 5.9 |
| 0.5-1.0 | 11.5 | 9.4 |
| 0.6-1.1 | 12.6 | 16.2 |

### 8.2.2 Quaternary deposits (Q3)

The in situ and laboratory test results for Quaternary marine deposits are summarised below. It is noted that sampling of very soft Quaternary clays in the field was not always successful due to low strength of the soils. Also a number of samples that were collected proved unsuitable for testing due to disturbance. The test results reported below in respect to strength and compressibility may therefore not fully reflect the low strength characteristics.

**Index Properties**

A total of 43 Atterberg Limit tests were performed on soil samples recovered from the boreholes. These results are summarised in Table 8-6 and presented on A-Line plots in Appendix H. The testing completed indicates that the fine grained soils within the Quaternary marine deposits generally comprise low to high plasticity clays.

**Table 8-6 Quaternary Marine Deposits - Atterberg Limits**

| | Liquid Limit (LL) | Plastic Limit (PL) | Plasticity Index (PI) |
|---|---|---|---|
| | (%) | (%) | (%) |
| Minimum value | 15 | 12 | 0 |
| Maximum value | 89 | 33 | 67 |
| Mean Value | 45 | 20 | 25 |
| Lower quartile value | 30 | 16 | 13 |
| Upper quartile value | 57 | 22 | 37 |
| Number of tests | 43 | 43 | 43 |

Laboratory test results for moisture content, particle density and dry density are summarised in Table 8-7 and plotted in Appendix H. Results for dry density tests on disturbed samples are not included in the summary below. The average moisture content for clay samples, including nine tests on very soft clays with moisture content of greater than 100%, is 49%, and the average moisture content for sand samples is 25%.

**Table 8-7 Quaternary Marine Deposits - Moisture content, particle density and dry density**

| | Moisture Content | Particle Density | Dry Density |
|---|---|---|---|
| | (%) | (t/m3) | (t/m3) |
| Minimum value | 13 | 2.38 | 0.74 |
| Maximum value | 133 | 2.80 | 1.60 |
| Mean Value | 40 | 2.62 | 1.03 |
| Lower quartile value | 21 | 2.55 | 0.77 |
| Upper quartile value | 45 | 2.70 | 1.39 |
| Number of tests | 130 | 34 | 8 |

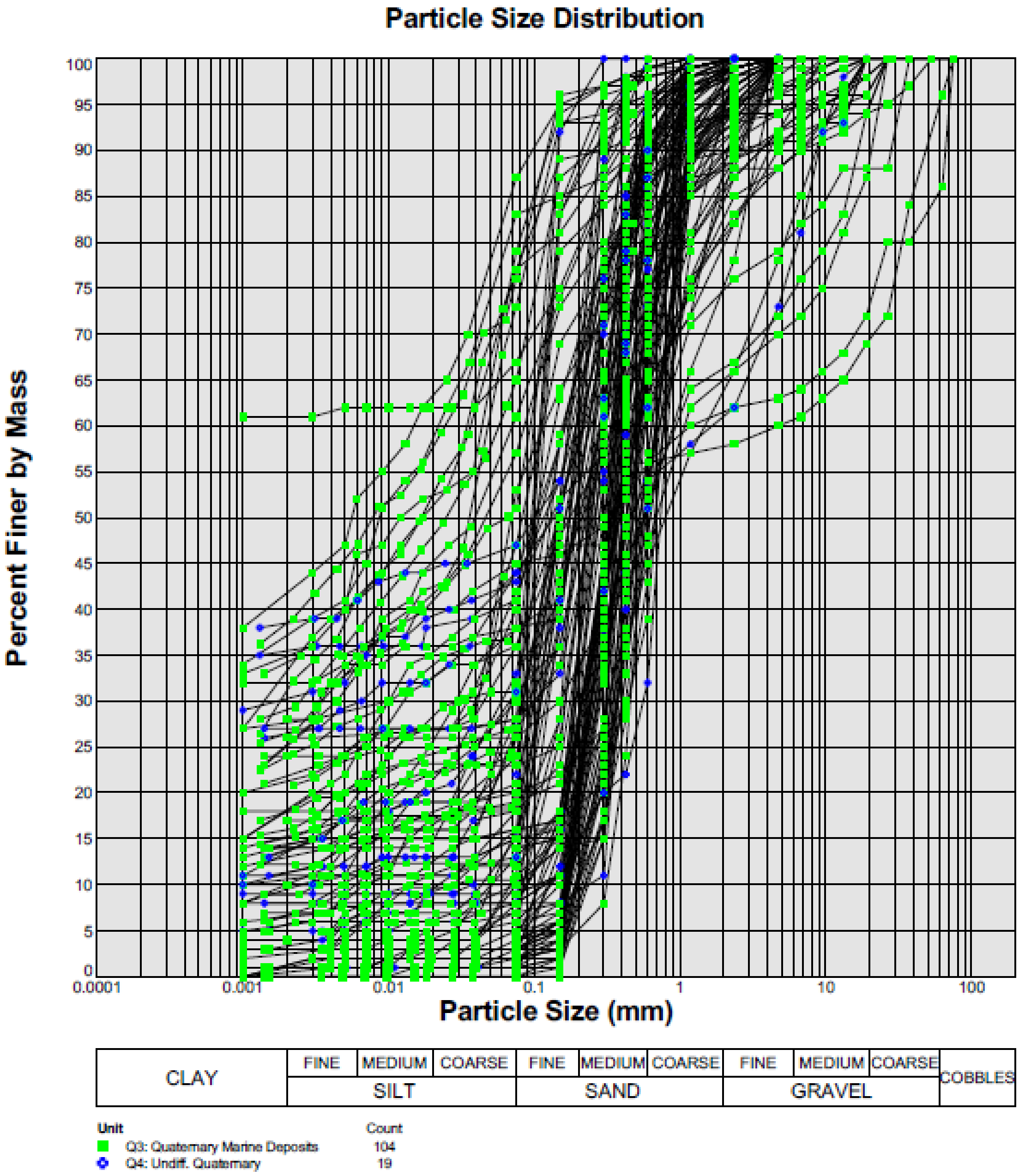
**Particle Size Distribution (PSD)**

In potential dredge areas particle size distribution testing was typically performed in the near surface sediments and at 1.5 m depth intervals to around RL -20 mCD.  Below this depth PSD testing was typically completed at 3 to 5 m depth intervals. In potential reclamation and port structure areas PSD testing was typically performed at 1.5 m depth intervals where low strength and compressible soils were encountered and elsewhere at 3 to 5 m depth intervals. Particle size analysis was typically not scheduled on clay materials resulting in a bias of the PSD curves towards materials which were described in the field as silts and sands. Particle size analysis has however been completed on some clay samples.

PSD curves are given in Figure 8-1 for all Quaternary marine deposits and undifferentiated Quaternary deposits. The materials represented by the PSD curves typically comprise fine to medium grained sand with a fines content ranging from 0% to 50%, and some sandy silts and sandy clays. Materials have been classified in accordance with AS1726 – *Geotechnical Site Investigations* where sands are defined as having more than 50% of the material being larger than 0.075 mm. Sandy materials with a fines content greater than 30 to 40% are likely to behave as a fine grained material in respect to engineering properties, and performance during dredging and reclamation. Approximately 10% of the results shown in Figure 8-1 are classified as silt or clay in accordance with AS 1726.

Additional PSD curves are given in Appendix H and discussed in the following sections.

**Figure 8-1 Quaternary Marine Deposits and Undifferentiated Marine Deposits - PSD Curves**



**SPT Test Results**

Results of the Standard Penetration Tests (SPT) undertaken in the investigation boreholes are presented in Table 8-8 and Appendix G and are also shown on the geological cross-sections along with the material consistency. High SPT N values were recorded at seabed in a number of boreholes however the SPT result does not reflect the consistency of these materials. For example the maximum SPT N value of 47 was recorded in a very loose sand with some shells to 40mm in size.

**Table 8-8 Quaternary Marine Deposits - SPT Test Results**

| | SPT N Value |
|---|---|
| Minimum value | 0 |
| Maximum value | 47 |
| Mean Value | 8 |
| Lower quartile value | 1 |
| Upper quartile value | 10 |
| Number of tests | 83 |

**Undrained Shear Strength**

The in situ testing performed on Quaternary Marine Deposits included in situ shear vane tests, hand shear vane readings on the ends of undisturbed tube samples and single stage unconsolidated undrained triaxial tests (UU) on selected undisturbed samples. The results are summarised in Table 8-9 and in Appendix H. The mean undrained shear strength values for the three test methods represent soils with a consistency ranging from very soft to soft. Soils represented by the lower quartile values are all of very soft consistency, and those represented by the upper quartile value range from very soft to firm. UU test results appear to represent stronger materials compared to the other test methods however this may be a result of scheduled tests having not been completed on weaker soils due to sample disturbance.

**Table 8-9 Quaternary Marine Deposits - Undrained Shear Strength – Results Summary**

| | Undrained Shear Strength (Su) | | |
|---|---|---|---|
| | In Situ Shear Vane | Hand Shear Vane | Undrained unconsolidated triaxial |
| | (kPa) | (kPa) | (kPa) |
| Minimum value | 3.3 | 1 | 5 |
| Maximum value | 18 | 80 | 60 |
| Mean Value | 9 | 15 | 23 |
| Lower quartile value | 4 | 2 | 7 |
| Upper quartile value | 13 | 11 | 29 |
| Number of tests | 6 | 36 | 6 |

**Effective Stress Parameters**

The effective stress parameters of selected samples were determined from consolidated undrained triaxial compression tests with pore water pressure measurement. Cell pressures were selected to cover the stress conditions from the current in-situ stresses to those likely to follow construction. The results from tests performed on undisturbed samples are given in Table 8-10.

**Table 8-10 Quaternary Marine Deposits – Effective Stress Test Results**

| Borehole | Sample Depth | Description | Consistency | Effective Stress Parameters | |
|---|---|---|---|---|---|
| - | (m) | - | - | Cohesion (c') | Friction Angle (□) |
| | | | | (kPa) | (°) |
| S12 | 1.5 | Clay | Very Soft | 1.4 | 20 |
| R18 | 2.5 | Silty Clay | Very Soft | 9 | 10 |
| Mean Value | | | | 5 | 15 |
| Number of tests | | | | 2 | 2 |

**Consolidation Parameters**

Nine consolidation tests have been completed on selected samples recovered as part of the geotechnical investigation. A summary of the test results and interpretation of consolidation parameters are presented in Table 8-11.

**Table 8-11 Quaternary Marine Deposits – Consolidation Test Results**

| Borehole | Sample Depth | Description | Consistency | Void Ratio | Preconsolidation pressure | Compression Index | Recompression Index |
|---|---|---|---|---|---|---|---|
| - | (m) | - | - | (e0) | (kPa) | Cc | Cr |
| R18 | 2.5 | Silty Clay | Very Soft | 2.182 | 15 | 0.759 | 0.131 |
| R20 | 0.6 | Silty Clay | Very Soft | 3.51 | - | 1.428 | 0.080 |
| S12 | 1.5 | Clay | Very Soft | 2.21 | 16 | 0.868 | 0.098 |
| S17 | 1.0 | Silty Clay | Very Soft | 2.452 | 7 | 0.689 | 0.062 |
| S17 | 3.5 | Silty Clay | Very Soft | 2.478 | 17 | 0.844 | 0.176 |
| S15 | 0.5 | Silty Clay | Soft | 0.62 | 90 | 0.117 | 0.008 |
| S7 | 2.0 | Silty Sand (10% fines ) | Very loose | 0.464 | 58 | 0.024 | 0.012 |
| R22 | 0.5 | Silty Sand (16% fines) | Very loose | 0.919 | 28 | 0.183 | 0.008 |
| R23 | 0.0 | Clayey Sand (24% fines ) | Very loose | 1.461 | - | 0.335 | 0.023 |

**Organic Content**

Organic content testing was completed on selected samples recovered from the boreholes. A summary of the test results is given in Table 8-12. Organic content testing was typically completed on very loose sand and very soft clay samples recovered from seabed however organic materials were present in Quaternary marine deposits up to depths of 3.5 m in boreholes S17, S12 and R22 which are all located immediately north of the BlueScope Steel Wharf, and correspond with the thickest deposits of very soft to soft clays encountered in the Port Area.

**Table 8-12 Quaternary Marine Deposits – Organic Content – Results Summary**

| | Organic Content (%) |
|---|---|
| Minimum value | 0.03 |
| Maximum value | 7.90 |
| Mean Value | 1.57 |
| Lower quartile value | 0.29 |
| Upper quartile value | 1.70 |
| Number of tests | 25 |

### Durability Testing

Durability testing was completed on selected samples recovered from the boreholes. A summary of the results of chloride, pH, resistivity and sulphate testing is given in Table 8-13. Results for pH include laboratory test results and pH values recorded in the field as reported on the borehole logs.

**Table 8-13 Quaternary Marine Deposits – Durability Tests – Results Summary**

| | Chloride | pH | Resistivity | Sulphate |
|---|---|---|---|---|
| | (mg/kg) | - | (ohm.m) | (mg/kg) |
| Minimum value | 2300 | 6.7 | 1.5 | 77 |
| Maximum value | 19000 | 9.6 | 10 | 960 |
| Mean Value | 7289 | 8 | 4 | 283 |
| Lower quartile value | 4500 | 8 | 2 | 145 |
| Upper quartile value | 9400 | 8 | 4 | 295 |
| Number of tests | 19 | 110 | 19 | 19 |

### Mineralogy

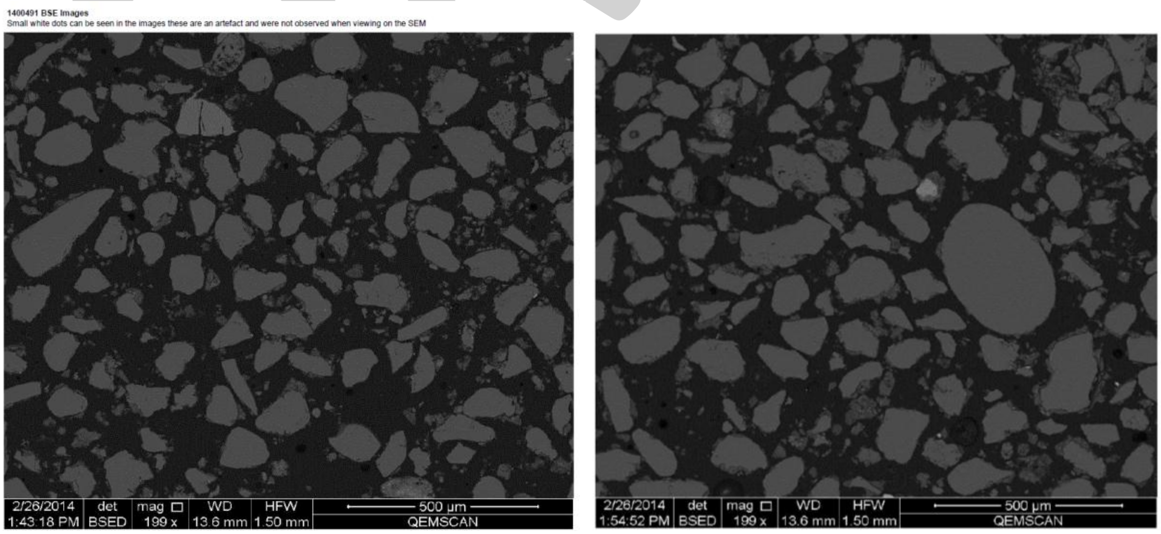
Mineralogical assessment of quartz and carbonate (and other minor minerals) has been completed on selected samples by quantitative x-ray diffraction analysis (XRD). The samples were dried and crushed to 100% passing 2.8 mm and sub samples were pulverised and micro-milled before being loaded into an XRD instrument. Quartz is the dominant mineral in sand materials from Quaternary marine deposits however calcite and aragonite minerals are also present in some samples. Summary results for mineralogical assessment are given in Appendix L.

### Angularity and Roundness

Angularity and roundness assessment has been completed on selected samples using a scanning electron microscope (SEM). The samples were dried and crushed to 100% passing 2.8 mm and sub samples were mounted in an epoxy resin for analysis by SEM which involved taking five photos to illustrate the particle shape.

An example of the SEM analysis results for Quaternary marine deposits is given in Figure 8-2. Additional results for angularity and roundness are given in the factual report on the marine geotechnical investigation.

**Figure 8-2 Quaternary Marine Deposits – SEM Images**



**Abrasion Testing**

Slurry abrasion testing was performed on selected samples recovered from the boreholes. The samples were slurried with distilled water and the weight loss on a cast chromium-iron wear block was recorded after 2, 4 and 6 hours abrasion. The slurry abrasivity (Miller Number) and Slurry Abrasion Response (SAR Number) were measured in accordance with ASTM G75-07 – Standard Test Method for Determination of Slurry Abrasivity (Miller Number) and Slurry Abrasion Response of Materials (SAR Number).

The abrasivity of the slurry is a function of the concentration of the solids in the liquid vehicle and of the following characteristics of the solid particles:

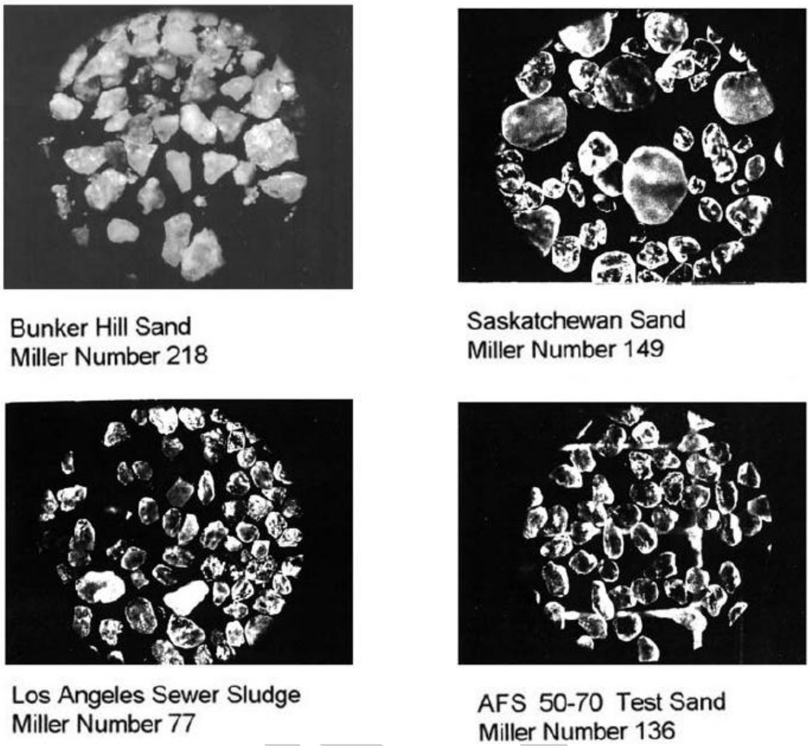
- Hardness
- Size
- Shape
- Size distributions
- Friability

Larger and more angular particles generally yield higher values of the Miller Number. Results for slurry abrasion testing on samples from this unit are given in Table 8-14. Figure 8-3 from ASTM G75-07 shows the particle shape and relative size of several sources of silica sand and the corresponding Miller Number.

**Table 8-14 Quaternary Marine Deposits – Slurry Abrasion Test Results**

| Borehole | Depth | Material | Miller Number | SAR Number |
|---|---|---|---|---|
| - | (m) | - | - | - |
| D1 | 0 | SAND | 154 | 154 |
| D11 | 0 | SILTY SAND | 156 | 156 |
| D12 | 0 | SAND | 137 | 137 |
| D13 | 0 | SILTY SAND | 119 | 119 |
| D18 | 0 | SAND | 104 | 104 |
| N6 | 0 | SAND | 122 | 122 |
| N9 | 0 | SAND | 104 | 104 |
| R21 | 0 | SAND | 120 | 120 |

**Figure 8-3 Miller Number for silica sand**




### 8.2.3 Undifferentiated Quaternary deposits (Q4)

The in situ and laboratory test results for undifferentiated Quaternary deposits are summarised below.

**Index Properties**

A total of 6 Atterberg Limit tests were performed on soil samples recovered from the boreholes. These results are summarised in Table 8-15 and presented on A-Line plots in Appendix H. The testing completed indicates that the fine grained soils within the undifferentiated Quaternary deposits generally comprise intermediate plasticity clays.

**Table 8-15 Undifferentiated Quaternary Deposits - Atterberg Limits**

| | Liquid Limit (LL) | Plastic Limit (PL) | Plasticity Index (PI) |
|---|---|---|---|
| | (%) | (%) | (%) |
| Minimum value | 20 | 10 | 2 |
| Maximum value | 50 | 18 | 36 |
| Mean Value | 38 | 13 | 25 |
| Lower quartile value | 35 | 12 | 23 |
| Upper quartile value | 45 | 14 | 33 |
| Number of tests | 6 | 6 | 6 |

Laboratory test results for moisture content, particle density and dry density are summarised in Table 8-16 and plotted in Appendix H. Results for dry density tests on disturbed samples are not included in the summary below.

Table 8-16 Undifferentiated Quaternary Deposits - Moisture content, particle density and dry density

| | Moisture Content | Particle Density | Dry Density |
|---|---|---|---|
| | (%) | (t/m3) | (t/m3) |
| Minimum value | 4 | 2.53 | 1.58 |
| Maximum value | 30.8 | 2.66 | 1.80 |
| Mean Value | 21 | 2.58 | 1.67 |
| Lower quartile value | 18 | 2.54 | - |
| Upper quartile value | 26 | 2.60 | - |
| Number of tests | 22 | 6 | 3 |

### SPT Test Results

Results of the Standard Penetration Tests (SPT) undertaken in the investigation boreholes are presented in Table 8-17, Appendix G and are also shown on the geological cross-sections along with the material consistency.

Table 8-17 Undifferentiated Quaternary Deposits - SPT Test Results

| | SPT N Value |
|---|---|
| Minimum value | 1 |
| Maximum value | 34 |
| Mean Value | 13 |
| Lower quartile value | 4 |
| Upper quartile value | 19 |
| Number of tests | 15 |

### Undrained Shear Strength

The insitu testing performed on undifferentiated Quaternary deposits included in situ shear vane tests, hand shear vane readings on the ends of undisturbed tube samples and  single stage unconsolidated undrained triaxial tests (UU) on selected undisturbed samples. The results are summarised in Table 8-18 and in Appendix H.

Table 8-18 Undifferentiated Quaternary Deposits - Undrained Shear Strength – Results Summary

| | Undrained Shear Strength (Su) | | |
|---|---|---|---|
| | In Situ Shear Vane | Hand Shear Vane | Undrained unconsolidated triaxial |
| | (kPa) | (kPa) | (kPa) |
| Minimum value | - | 6 | 87 |
| Maximum value | - | 184 | 110 |
| Mean Value | 3.6 | 88 | 98 |
| Lower quartile value | - | 34 | - |
| Upper quartile value | - | 53 | - |
| Number of tests | 1 | 7 | 2 |

### Effective Stress Parameters

The effective stress parameters of selected samples were determined from consolidated undrained triaxial compression tests with pore water pressure measurement.  Cell pressures were selected to cover the stress conditions from the current in-situ stresses to those likely to follow construction. The results from tests performed on undisturbed samples are given in Table 8-19.

**Table 8-19 Undifferentiated Quaternary Deposits – Effective Stress Test Results**

| Borehole | Sample Depth | Description | Consistency | Effective Stress Parameters | |
|---|---|---|---|---|---|
| - | (m) | - | - | Cohesion (c') | Friction Angle (φ') |
| | | | | (kPa) | (°) |
| D14 | 3 | Sand | Very Loose | 0 | 30 |
| Number of tests | | | | 1 | 1 |

### Organic Content

Organic content testing was completed on selected samples recovered from the boreholes. A summary of the test results is given in Table 8-20.

**Table 8-20 Undifferentiated Quaternary Deposits – Organic Content – Results Summary**

| | Organic Content (%) |
|---|---|
| Minimum value | 0.01 |
| Maximum value | 4.00 |
| Mean Value | 1.53 |
| Number of tests | 4 |

### Durability Testing

Durability testing was completed on selected samples recovered from the boreholes. A summary of the results of chloride, pH, resistivity and sulphate testing is given in Table 8-21. Results for pH include laboratory test results and pH values recorded in the field as reported on the borehole logs

**Table 8-21 Undifferentiated Quaternary Deposits – Durability Tests – Results Summary**

| | Chloride | pH | Resistivity | Sulphate |
|---|---|---|---|---|
| | (mg/kg) | - | (ohm.m) | (mg/kg) |
| Minimum value | - | 6.8 | - | - |
| Maximum value | - | 9.1 | - | - |
| Mean Value | 3900 | 7.9 | - | 170 |
| Lower quartile value | - | 7.5 | - | - |
| Upper quartile value | - | 8.1 | - | - |
| Number of tests | 1 | 13 | - | 1 |

### Abrasion Testing

Results for slurry abrasion testing on samples from this unit are given in Table 8-22.

**Table 8-22 Undifferentiated Quaternary Deposits – Slurry Abrasion Test Results**

| Borehole | Depth | Material | Miller Number | SAR Number |
|---|---|---|---|---|
| - | (m) | - | - | - |
| S9 | 1.5 | SAND | 165 | 165 |

### 8.2.4 Baxter Formation (Tb)

The in situ and laboratory test results for the Baxter Formation are summarised below.

**Index Properties**

A total of 219 Atterberg Limit tests were performed on soil samples recovered from the boreholes. These results are summarised in Table 8-23 to Table 8-27 and provided as A-Line plots in Appendix H. Atterberg Limits are used for classification of fine grained soils however tests have also been completed on materials which have been classified as sand in accordance with AS1726 – *Geotechnical Site Investigations* where sands are defined as having more than 50% of the material being larger than 0.075 mm. These sand materials include clayey sands and silty sands with a high fines content and the engineering behaviour of these materials is expected to be similar to that of a fine grained soil where sands have fines content of >30 to 40%. This is also evident from the CPT plots where the soil behaviour type determined from the CPT results often shows silt and clay behaviour types for materials which have been logged as sands. A comparison of CPT plots showing soil behaviour type and adjacent borehole logs are given in Appendix M for boreholes S13, R21, S7 and R16.

Atterberg Limit test results are given below for all materials and for individual material types (sand, silt and clay). The results indicate that the soils within the Baxter Formation generally comprise low to high plasticity clays. The mean results for all material types plot above the A-Line; clays having a high plasticity, silts having intermediate plasticity and sands having low to intermediate plasticity.

**Table 8-23 Baxter Formation - Atterberg Limits – Results Summary (all results - Marine)**

| | Liquid Limit (LL) | Plastic Limit (PL) | Plasticity Index (PI) |
|---|---|---|---|
| | (%) | (%) | (%) |
| Minimum value | 0 | 7 | 0 |
| Maximum value | 163 | 47 | 100 |
| Mean Value | 44 | 19 | 25 |
| Lower quartile value | 30 | 13 | 13 |
| Upper quartile value | 54 | 22 | 35 |
| Number of tests | 219 | 219 | 219 |

**Table 8-24 Baxter Formation - Atterberg Limits – Results Summary (sand materials - Marine)**

| | Liquid Limit (LL) | Plastic Limit (PL) | Plasticity Index (PI) |
|---|---|---|---|
| | (%) | (%) | (%) |
| Minimum value | 16 | 7 | 0 |
| Maximum value | 68 | 41 | 46 |
| Mean Value | 34 | 18 | 16 |
| Lower quartile value | 24 | 14 | 7 |
| Upper quartile value | 43 | 20 | 25 |
| Number of tests | 74 | 74 | 74 |

**Table 8-25 Baxter Formation - Atterberg Limits – Results Summary (silt materials - Marine)**

| | Liquid Limit (LL) | Plastic Limit (PL) | Plasticity Index (PI) |
|---|---|---|---|
| | (%) | (%) | (%) |
| Minimum value | 15 | 12 | 1 |
| Maximum value | 77 | 47 | 48 |
| Mean Value | 46 | 25 | 21 |
| Lower quartile value | 34 | 19 | 11 |
| Upper quartile value | 61 | 28 | 29 |
| Number of tests | 23 | 23 | 23 |

**Table 8-26 Baxter Formation - Atterberg Limits – Results Summary (clay materials - Marine)**

| | Liquid Limit (LL) | Plastic Limit (PL) | Plasticity Index (PI) |
|---|---|---|---|
| | (%) | (%) | (%) |
| Minimum value | 0 | 8 | 4 |
| Maximum value | 163 | 47 | 100 |
| Mean Value | 50 | 18 | 31 |
| Lower quartile value | 37 | 13 | 19 |
| Upper quartile value | 57 | 22 | 38 |
| Number of tests | 116 | 116 | 116 |

**Table 8-27 Baxter Formation - Atterberg Limits – Results Summary (Terrestrial Project Area)**

| | Liquid Limit (LL) | Plastic Limit (PL) | Plasticity Index (PI) |
|---|---|---|---|
| | (%) | (%) | (%) |
| Minimum value | 22 | 11 | 2 |
| Maximum value | 98 | 44 | 68 |
| Mean Value | 51 | 23 | 29 |
| Lower quartile value | 37 | 18.25 | 21.5 |
| Upper quartile value | 61.5 | 26.5 | 35 |
| Number of tests | 35 | 14 | 35 |

Laboratory test results for moisture content, dry density and particle density are summarised in Table 8-28 to Table 8-32 and plotted versus depth in Appendix H. The results are given for all test results for this unit and individually for sands, silts and clays. The results for moisture content include tests completed on SPT samples which may be misleading due to moisture change as a result of the sampling process. The density test results given below are for tests completed on undisturbed samples.

**Table 8-28 Baxter Formation - Moisture Content – Results Summary - Marine**

| | Moisture Content | | | |
|---|---|---|---|---|
| | All | Sand | Silt | Clay |
| | (%) | (%) | (%) | (%) |
| Minimum value | 8.7 | 8.7 | 15.0 | 8.9 |
| Maximum value | 93.5 | 59.5 | 93.5 | 70.5 |
| Mean Value | 27.5 | 24.0 | 38.4 | 30.2 |
| Lower quartile value | 19.1 | 17.6 | 24.5 | 21.7 |
| Upper quartile value | 31.4 | 27.5 | 48.2 | 36.8 |
| Number of tests | 453 | 252 | 42 | 151 |

**Table 8-29 Baxter Formation - Moisture Content – Terrestrial Project Area Results Summary**

| | Moisture Content |
|---|---|
| | (%) |
| Minimum value | 2.4 |
| Maximum value | 54 |
| Mean Value | 21.6 |
| Lower quartile value | 15.2 |
| Upper quartile value | 25.6 |

| | Moisture Content |
|---|---|
| Number of tests | 61 |

**Table 8-30 – Dry Density – Results Summary - Marine**

| | Dry Density | | | |
|---|---|---|---|---|
| | All | Sand | Silt | Clay |
| Minimum value | 0.7 | 1.11 | 0.7 | 0.9 |
| Maximum value | 2.0 | 1.99 | 1.7 | 2.0 |
| Mean Value | 1.5 | 1.54 | 1.3 | 1.5 |
| Lower quartile value | 1.3 | 1.32 | 1.2 | 1.3 |
| Upper quartile value | 1.7 | 1.72 | 1.3 | 1.7 |
| Number of tests | 106 | 25 | 11 | 61 |

**Table 8-31 Baxter Formation – Dry Density – Terrestrial Project Area Results Summary**

| | Dry Density |
|---|---|
| | |
| Minimum value | 1.2 |
| Maximum value | 1.8 |
| Mean Value | 1.5 |
| Lower quartile value | 1.4 |
| Upper quartile value | 1.7 |
| Number of tests | 13 |

**Table 8-32 Baxter Formation – Specific Gravity – Results Summary - Marine**

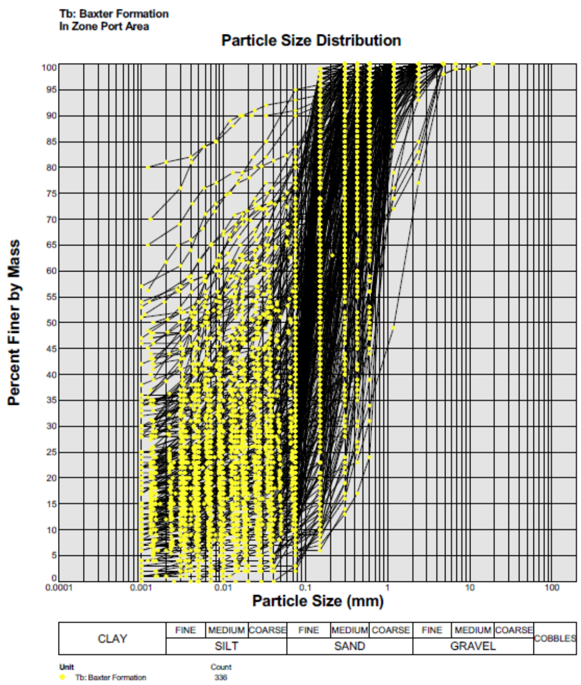
| | Specific Gravity | | | |
|---|---|---|---|---|
| | All | Sand | Silt | Clay |
| Minimum value | 2.17 | 2.17 | 2.40 | 2.24 |
| Maximum value | 2.92 | 2.91 | 2.79 | 2.92 |
| Mean Value | 2.61 | 2.60 | 2.64 | 2.61 |
| Lower quartile value | 2.54 | 2.54 | 2.56 | 2.55 |
| Upper quartile value | 2.68 | 2.64 | 2.75 | 2.69 |
| Number of tests | 99 | 60 | 5 | 32 |

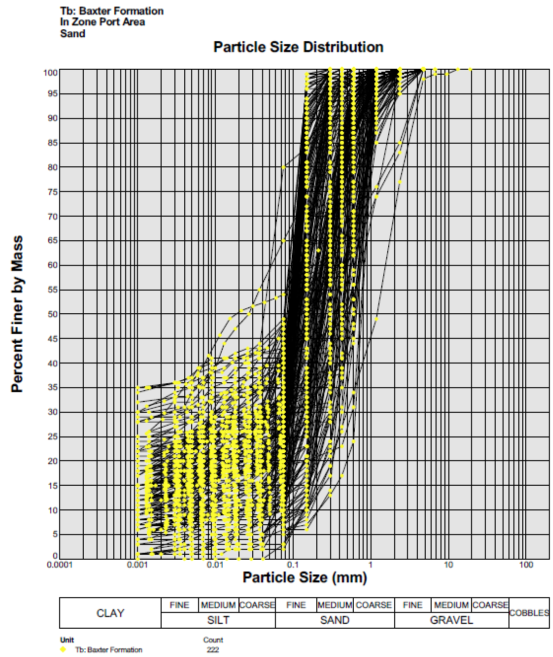
### Particle Size Distribution (PSD)

PSD curves are given in Figure 8-4 for all PSD testing completed on Baxter Formation materials and in Figure 8-5 for all PSD testing completed on Baxter Formation materials described as sand. The PSD curves shown on Figure 8-5 indicate that three of the samples are fine grained soils with more than 50% of the material passing the 0.075 mm sieve however these materials have been described as sand on the borehole logs. Additional PSD curves for Baxter Formation materials described as silt and clay are included in Appendix H. This figure also includes a number of discrepancies where materials plot as a sand but have been described as a silt or clay on the borehole logs. The PSD curves for Baxter Formation show that these materials comprise fine to medium grained sands with a fines content typically ranging from 10% to 45%, and sandy silt and sandy clay materials. As discussed previously PSD testing was typically not scheduled on clay materials and as a result the PSD curves show a bias towards sand materials.

**Figure 8-4 Baxter Formation – All PSD curves - Marine**

**Figure 8-5 Baxter Formation – PSD curves for sand - Marine**




Additional PSD curves are given in Appendix H showing PSD test results for all materials (Quaternary deposits, Baxter Formation and Sherwood Formation) from seabed to -20 mCD on single plots as listed below. Additional figures showing percentage fines versus depth and elevation, and percentage fines histograms for all units are also included in Appendix H.

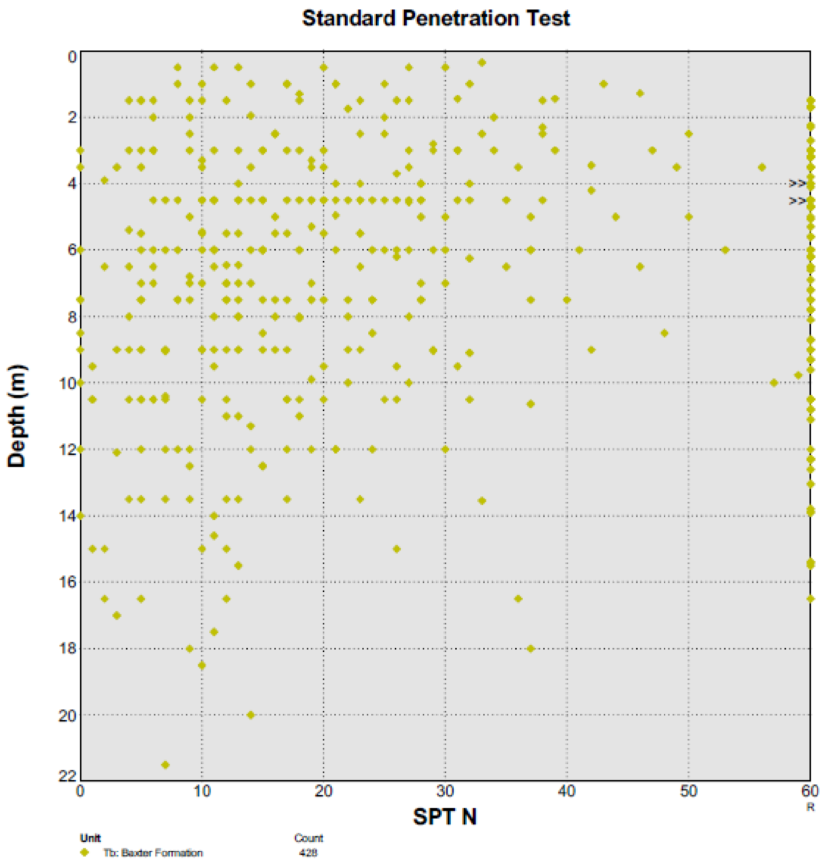
- PSD4 – All materials from seabed to -20 mCD
- PSD 5 – All materials from seabed to -5mCD
- PSD 6 – All materials from -5 mCD to -10 mCD
- PSD 7 - All materials from -10 mCD to -15 mCD
- PSD 8 - All materials from -15 mCD to -20 mCD

These PSD plots show that there is a wide variation in fines content at all depth intervals from seabed to -20 mCD.  Sand materials are typically fine to medium grained with a fines content ranging from 0% to 50%.  This is consistent with the lateral variability and interbedded nature of materials within this depth range as shown on the borehole logs and geological cross-sections.

### SPT Test Results

Results of the Standard Penetration Tests (SPT) undertaken in the investigation boreholes are plotted versus depth in Figure 8-6 and are given in Appendix G plotted versus RL. SPT N values equal to or greater than 50 and SPT refusal are highlighted on the geological cross section given in Appendix C. These materials typically represent very dense sands and hard clays and silts, and in some instances represent cemented materials. SPT refusal values have not been extrapolated to provide an equivalent SPT N value as the reason for refusal is not known and extrapolation may be therefore misleading. For example refusal may be on a very dense sand layer or a relatively thin cemented horizons.  SPT N values less than or equal to 5 are also highlighted on these cross sections and typically represent very loose to loose sand materials. SPTs were typically completed on coarse grained soils and on fine grained soil of very stiff to hard consistency.

**Figure 8-6 Baxter Formation – SPT N-Value vs Depth - Marine**




As discussed in the factual report on the marine geotechnical investigation there were instances where low SPT N values were recorded in fine grained soils which would usually indicate that the soil is of very soft or soft consistency. However subsequent tactile examination of the soil recovered in the SPT sampler and/or comparison with adjacent undisturbed samples has identified these soils as being of firm to very stiff consistency. It is noted that the correlation between SPT N value and consistency can vary depending on plasticity, sensitivity and fissuring state. Low SPT N values can also be as a result of drilling disturbance. The factual report states that SPT N data should not be relied upon in isolation from other available data to infer soil strength where low SPT N values are recorded in fine grained soils. Table 8-33 below is reproduced from the factual report on the marine geotechnical investigation and presents a list of boreholes and tests that recorded low SPT N values (N=5 or less) in fine grained soil with the logged consistency which was based upon visual and tactile assessment of the collected soil sample. Fourteen of these tests were completed on Baxter Formation materials and the remainder on Sherwood Formation materials.

**Table 8-33 Boreholes recording N=5 or less in fine grained soils**

| Borehole | Depth (m) | Material Type | Undrained shear strength (from visual and tactile assessment of the collected soil sample) | Seating Blows | N Value |
|---|---|---|---|---|---|
| D32r | 10.50 | Sandy Clay | Stiff | 1 | 1 |
| R10 | 17.00 | Sandy Clay | Firm | 11 | 3 |
| R16 | 3.00 | Silt | Firm | 2 | 0 |
| R16 | 3.5 | Silt | Firm | 0 | 0 |
| R16 | 10.50 | Silty Clay | Stiff | 0 | 0 |
| R18 | 19.50 | Sandy Clay | Stiff | 3 | 3 |
| R18 | 21.00 | Silty Clay | Stiff | 0 | 1 |
| R20 | 12.10 | Silty Clay | Very Stiff | 1 | 3 |
| R3 | 15.00 | Sandy Clay | Firm | 0 | 1 |
| R5 | 10.50 | Silty Clay | Firm/Stiff | 0 | 5 |
| R5 | 12.00 | Silty Clay | Stiff | 0 | 1 |
| R6 | 7.50 | Silty Clay | Very Stiff | 1 | 0 |
| R8 | 10.50 | Clayey Silt | Firm | 0 | 4 |
| R8 | 12.00 | Clay | Stiff | 1 | 3 |
| S1 | 12.00 | Silt | Stiff | 0 | 5 |
| S1 | 13.50 | Silt | Very Stiff | 1 | 5 |
| S4 | 10.50 | Clayey Silt | Stiff | 0 | 1 |
| S5 | 9.00 | Sandy Clay | Stiff | 3 | 3 |
| S8 | 10.00 | Silty Clay | Stiff | 0 | 0 |

In addition to above table the factual report on the marine geotechnical investigation also provides a table of SPT N values for materials which have been classified as coarse grained soils in accordance with AS 1726 which represent materials that may behave as fine grained soil. It is stated in the factual report that based on field tactile assessments the relative density term provided are unlikely to reflect the actual soil state and engineering response of these materials. This table is reproduced as Table 8-34 below.

**Table 8-34 Low SPT N values in coarse grained soils – based on AS1726**

| Borehole | Depth (m) | Material Type | Relative Density Term (based upon SPT N Value) | Seating Blows | N Value |
|---|---|---|---|---|---|
| D33R | 7.50 | Silty Sand | Loose | 2 | 5 |
| D36R | 7.50 | Clayey Sand | Loose | 0 | 5 |
| G2 | 22.00 | Silty Sand | Loose | 0 | 5 |
| R13 | 6.00 | Clayey Sand | Very Loose | 0 | 0 |
| R18 | 9.00 | Silty Sand | Loose | 2 | 4 |
| R18 | 15.00 | Silty Sand | Very Loose | 0 | 2 |
| R18 | 16.50 | Silty Sand | Very Loose | 4 | 2 |
| R22 | 10.50 | Silty Sand | Loose | 2 | 5 |
| R8 | 9.00 | Clayey Sand | Loose | 1 | 55 |
| S1 | 16.50 | Clayey Sand | Loose | 1 | 5 |

The borehole log for R18 shows approximately 9 m of silty sand, sandy clay and silty clay between 14.5 and 24.2 m depth with low SPT N values ranging from 1 to 3 as highlighted in the above tables. Sand materials have been described as very loose and the clay materials as stiff. These materials were logged as silt and clay with a consistency of stiff to very stiff on the field logs. The low SPT N values and inconsistency with the strength

terms used to describe these materials prompted drilling of an adjacent borehole (R18A) over the same depth interval. SPT N values in borehole R18A, which was approximately 1.5m from borehole R18, recorded SPT N values ranging from 7 to 14 in medium dense sands and stiff to hard silt, sandy clay and silty clay.

**Undrained Shear Strength**

The in situ testing performed on Baxter Formation included in situ shear vane tests, hand shear vane readings on the ends of undisturbed tube samples, and single stage unconsolidated undrained triaxial tests (UU) performed on selected undisturbed samples. The results of these tests are summarised in Table 8-35 to Table 8-39 and are plotted versus depth and RL in Appendix H. Testing has been completed on sand samples as the engineering behaviour of sand materials with a high fines content (clayey sand and silty sand) is expected to be similar to that of a fine grained soil where fines content is > than 30 to 40%. The results are given for all tests completed on this unit and individually for sand, silt and clay materials. The range of undrained shear strength represented by the lower and upper quartile values is equivalent to a consistency of stiff to very stiff. The results also indicate that the undrained shear strength of sand, silt and clay materials within the Baxter Formation are all similar. The minimum UU result of 8 kPa was for a very soft silty clay encountered in borehole S16 at a depth of 3m and the maximum UU result of 234 kPa was for a very stiff silt at a similar depth of 3.5m.

Where possible the residual shear strength of the soil was also measured using the hand shear vane and was obtained by rotating the vane until the recorded value levelled off to a constant value. Where recorded, the residual shear strength values are shown on the borehole logs. The sensitivity of materials within the Baxter Formation was typically in the range of 2-12 where:

$$Sensitivity = \frac{\text{Undisturbed shear strength}}{\text{Remoulded shear strength}}$$

**Table 8-35 Baxter Formation – Undrained Shear Strength – Results Summary (all results - Marine)**

| | Undrained Shear Strength (Su) | | |
|---|---|---|---|
| | In Situ Shear Vane | Hand Shear Vane | Undrained unconsolidated triaxial |
| | (kPa) | (kPa) | (kPa) |
| Minimum value | - | 10 | 8 |
| Maximum value | - | 600 | 234 |
| Mean Value | 72 | 120 | 94 |
| Lower quartile value | - | 70 | 60 |
| Upper quartile value | - | 163 | 122 |
| Number of tests | 1 | 200 | 97 |

**Table 8-36 Baxter Formation – Undrained Shear Strength – Results Summary (sand materials - Marine)**

| | Undrained Shear Strength (Su) | | |
|---|---|---|---|
| | In Situ Vane | Hand Shear Vane | Undrained unconsolidated triaxial |
| | (kPa) | (kPa) | (kPa) |
| Minimum value | - | 10 | 38 |
| Maximum value | - | 280 | 204 |
| Mean Value | - | 108 | 106 |
| Lower quartile value | - | 70 | 72 |
| Upper quartile value | - | 133 | 136 |
| Number of tests | - | 40 | 25 |

**Table 8-37 Baxter Formation – Undrained Shear Strength – Results Summary (silt materials - Marine)**

| | Undrained Shear Strength (Su) | | |
|---|---|---|---|
| | In Situ Vane | Hand Shear Vane | Undrained unconsolidated triaxial |
| | (kPa) | (kPa) | (kPa) |
| Minimum value | - | 10 | 8 |
| Maximum value | - | 188 | 234 |
| Mean Value | 72 | 96 | 99 |
| Lower quartile value | - | 61 | 58 |
| Upper quartile value | - | 120 | 116 |
| Number of tests | - | 26 | 11 |

**Table 8-38 Baxter Formation – Undrained Shear Strength – Results Summary (clay materials - Marine)**

| | Undrained Shear Strength (Su) | | |
|---|---|---|---|
| | In Situ Vane | Hand Shear Vane | Undrained unconsolidated triaxial |
| | (kPa) | (kPa) | (kPa) |
| Minimum value | - | 10 | 10 |
| Maximum value | - | 600 | 192 |
| Mean Value | - | 132 | 88 |
| Lower quartile value | - | 80 | 54 |
| Upper quartile value | - | 176 | 124 |
| Number of tests | - | 130 | 57 |

**Table 8-39 Baxter Formation – Undrained Shear Strength – Terrestrial Project Area Results Summary**

| | Undrained unconsolidated triaxial |
|---|---|
| | Su (kPa) |
| Minimum value | 60 |
| Maximum value | 230 |
| Mean Value | 128 |
| Lower quartile value | 75 |
| Upper quartile value | 150 |
| Number of tests | 13 |

### Effective Stress Parameters

The effective stress parameters of selected samples were determined from consolidated undrained triaxial compression tests with pore water pressure measurement (CUPP). Cell pressures were selected to cover the stress conditions from the current in-situ stresses to those likely to follow construction. The results from tests performed on undisturbed samples are given in Table 8-40 and Table 8-41. The results given below are for triaxial tests on undisturbed samples. One CUPP test was completed on a SPT sample in silty sand which had a friction angle of 35° and cohesion of 5kPa.

The results show that sand, silt and clay materials within the Baxter Formation have a similar mean effective friction angle. The clay and silt materials have a similar cohesion while sand has a mean cohesion of around 4kPa as a result of the significant fines content in these materials.

**Table 8-40 Baxter Formation – Effective Stress Parameters – Marine**

| Borehole | Sample Depth | Description | Consistency | Effective Stress Parameters | |
|---|---|---|---|---|---|
| - | (m) | - | - | Cohesion (c') | Friction Angle (□) |
| | | | | (kPa) | (°) |
| R18A | 15 | Sand | MD | 0 | 32 |
| D32r | 8.5 | Silty Sand | MD | 4 | 33 |
| R10 | 8 | Silty Sand | MD | 5 | 35 |
| S5 | 7.5 | Silty Sand | VSt | 13 | 32 |
| S10 | 6 | Clayey Sand | MD | 0 | 33 |
| R13 | 6.5 | Clayey Sand | L | 3 | 31 |
| S16r | 6 | Sandy Silt | VSt | 14 | 33 |
| R18A | 21 | Sandy Clay | VSt | 0 | 32 |
| R12 | 10.5 | Silty Clay | St | 0.5 | 32 |
| S14r | 3.5 | Silty Clay | St | 18 | 35 |
| S16r | 3 | Silty Clay | VS | 20 | 32 |
| S7 | 10.5 | Clay | VSt | 6 | 28 |

**Table 8-41 Baxter Formation – Effective Stress Parameters – Marine**

| | Effective Stress Parameters | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| | Friction Angle (°) | | | | Cohesion (kPa) | | | |
| | All | Sand | Silt | Clay | All | Sand | Silt | Clay |
| Minimum value | 26 | 31 | - | 26 | 0 | 0 | - | 0.0 |
| Maximum value | 35 | 33 | - | 35 | 22 | 13 | - | 22 |
| Mean Value | 32 | 32 | 33 | 31 | 8 | 4 | 13.5 | 11 |
| Lower quartile value | 32 | 32 | - | 29 | 0.4 | 0 | - | 2 |
| Upper quartile value | 33 | 33 | - | 32 | 15 | 4 | - | 20 |
| Number of tests | 12 | 5 | 1 | 6 | 12 | 5 | 1 | 6 |

### Unconfined Compressive Strength

Unconfined compressive strength (UCS) testing was undertaken on one cored sample of silty clay at a depth of 15.5 m (-16.32mCD) in Borehole R20. The soil description states that the soil includes iron oxide cemented corestones up to 100 mm diameter. The silty clay had a UCS of 4.7MPa and overlaid extremely low to high strength brown, ferruginised siltstone cemented clay.

### Consolidation Parameters

22 consolidation tests have been completed on selected samples recovered as part of the geotechnical investigation. A summary of the test results and interpretation of consolidation parameters are presented in Table 8-55.

**Table 8-42 Baxter Formation – Consolidation Test Results - Marine**

| Borehole | Sample Depth | Description | Consistency | Void Ratio | Preconsolidation pressure | Compression Index | Recompression Index |
|---|---|---|---|---|---|---|---|
| - | (m) | - | - | (e0) | (kPa) | Cc | Cr |
| S16r | 3.0 | Silty Clay | Very Soft | 1.253 | 90 | 0.332 | 0.057 |
| R8 | 9.5 | Clayey Sand | Firm | 0.825 | 150 | 0.093 | 0.007 |
| S11 | 13.5 | Sandy Clay | Firm - Stiff | 1.996 | 250 | 0.758 | 0.036 |

| Borehole | Sample Depth | Description | Consistency | Void Ratio | Preconsolidation pressure | Compression Index | Recompression Index |
|---|---|---|---|---|---|---|---|
| R12 | 0.5 | Silty Clay | Stiff | 0.508 | 61 | 0.062 | 0.019 |
| R12 | 9.0 | Silty Clay | Stiff | 1.211 | 300 | 0.393 | 0.022 |
| R14 | 9.0 | Silty Clay | Stiff | 0.801 | 160 | 0.164 | - |
| S1 | 3.5 | Clay | Stiff | 0.92 | 175 | 0.237 | 0.044 |
| S1 | 18.5 | Silty Clay | Stiff | 1.819 | 290 | 0.719 | 0.055 |
| S14r | 3.5 | Silty Clay | Stiff | 1.083 | 220 | 0.359 | 0.027 |
| R18A | 21.0 | Sandy Clay/Silty Sand | Very Stiff to stiff | 1.17 | 375 | 0.686 | 0.077 |
| R2 | 2.0 | Silty Clay | Very Stiff | - | - | - | - |
| R18A | 17.0 | Silt | Very Stiff | 0.666 | - | 0.044 | 0.013 |
| S16 | 6.0 | Sandy Silt | Very Stiff | 1.769 | 240 | 0.826 | 0.043 |
| R9 | 3.0 | Silty Clay | Very Stiff | 0.574 | 160 | 0.073 | - |
| R15 | 4.5 | Silty Clay | Very Stiff | 0.412 | 175 | 0.068 | 0.02 |
| R20 | 3.0 | Silty Sand | Hard | 0.217 | 170 | 0.032 | - |
| R13 | 7.5 | Clayey Sand | Loose | 0.794 | 150 | 0.11 | 0.008 |
| R11 | 7.0 | Clayey Sand | Loose-Medium Dense | 0.639 | 200 | 0.156 | 0.008 |
| R18A | 15.0 | Sand | Medium Dense | 0.804 | 115 | 0.072 | 0.016 |
| R9 | 7.5 | Silty Sand | Medium Dense | 1.746 | 300 | 0.697 | 0.065 |
| D32r | 8.5 | Silty Sand | Medium Dense | 0.85 | 140 | 0.144 | 0.017 |
| S15 | 9.0 | Silty Sand | Medium Dense | 0.64 | 180 | 0.097 | 0.023 |
| S6 | 2.0 | Sandy Clay | Very Stiff | 0.663 | 250 | 0.324 | 0.019 |

**Permeability Testing**

A total of 11 constant head permeability tests have been carried out on selected samples recovered as part of the geotechnical investigation (Borehole G1, G2, G3, S10, R14, S5). Eight tests were completed on undisturbed samples and three tests on remoulded SPT samples. A summary of the tests results is given in Table 8-43.

**Table 8-43 Baxter formation – Permeability Test Results - Marine**

| | Coefficient of Permeability | | | |
|---|---|---|---|---|
| | All | Sand | Sand - Remoulded | Clay |
| | (m/s) | (m/s) | (m/s) | (m/s) |
| Minimum value | 2.00E-10 | 2.00E-10 | 2.00E-10 | 9.00E-10 |
| Maximum value | 1.00E-07 | 1.00E-07 | 5.00E-08 | 2.00E-08 |
| Mean Value | 1.78E-08 | 2.86E-08 | 1.81E-08 | 6.98E-09 |
| Number of tests | 11 | 4 | 3 | 4 |

### Organic Content

Organic content testing was completed on selected samples recovered from the boreholes. A summary of the test results is given in Table 8-44. An organic content of 20% was present in a sample of dark brown medium dense clayey sand from borehole D30 at a depth of 5 m, which was described on the borehole log as 'appears organic'. Several of the materials which are described on the borehole logs as appearing to be organic, as discussed in Section 8.1.3, had an organic content in the range 2-14%.

**Table 8-44 Baxter Formation – Organic Content – Results Summary - Marine**

| | Organic Content (%) |
|---|---|
| Minimum value | 0.0 |
| Maximum value | 20.0 |
| Mean Value | 2.3 |
| Lower quartile value | 0.1 |
| Upper quartile value | 2.7 |
| Number of tests | 47 |

### Durability Testing

Durability testing was completed on selected samples recovered from the boreholes. A summary of the results of chloride, pH, resistivity and sulphate testing is given in Table 8-45 Results for pH include laboratory test results and pH values recorded in the field as reported on the borehole logs.

**Table 8-45 - Baxter formation – Durability Testing – Results Summary- Marine**

| | Chloride | pH | Resistivity | Sulphate |
|---|---|---|---|---|
| | (mg/kg) | - | (ohm.m) | (mg/kg) |
| Minimum value | 1100 | 3.7 | 2.7 | 34 |
| Maximum value | 9000 | 9.5 | 2900 | 1000 |
| Mean Value | 3868 | 7.2 | 167 | 197 |
| Lower quartile value | 2800 | 6.8 | 5 | 77 |
| Upper quartile value | 4200 | 7.7 | 7 | 215 |
| Number of tests | 31 | 130 | 31 | 31 |

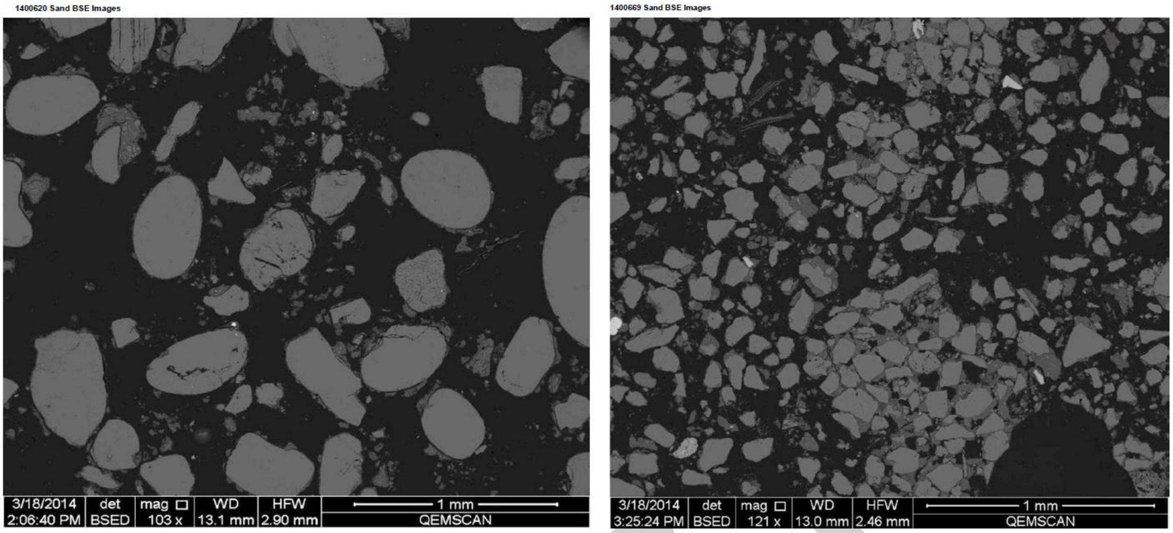
### Mineralogy

Mineralogical assessment of quartz and carbonate (and other minor minerals) has been completed on selected samples by quantitative x-ray diffraction analysis (XRD). The samples were dried and crushed to 100% passing 2.8 mm and sub samples were pulverised and micro-milled before being loaded into an XRD instrument. Quartz is the dominant mineral in sand materials from the Baxter Formation. Summary results for mineralogical assessment are given in Appendix L.

### Angularity and Roundness

An example of the SEM analysis results for two samples (sand and silty sand) from the Baxter Formation is given in Figure 8-7 below. Additional results for angularity and roundness are given in the factual report on the marine geotechnical investigation.

**Figure 8-7 Baxter Formation – SEM Images (BH S11–4.5m–Sand & BH R19-6.0m-Silty Sand) - Marine**




### Abrasion Testing

Results for slurry abrasion testing on samples from this unit are given in Table 8-46.

**Table 8-46 Baxter Formation – Slurry Abrasion Test Results - Marine**

| Borehole | Depth | Material | Miller Number | SAR Number |
|---|---|---|---|---|
| - | (m) | - | - | - |
| D1 | 7.5 | SILTY SAND | 167 | 167 |
| D11 | 4.0 | SILTY SAND | 140 | 140 |
| D12 | 3.0 | SAND | 155 | 155 |
| D14 | 8.0 | SANDY SILT | 142 | 142 |
| D9 | 4.5 | SILTY SAND | 127 | 127 |
| G2 | 5.5 | SAND | 154 | 154 |
| G2 | 11.5 | SILTY SAND | 215 | 215 |
| S10 | 4.5 | CLAYEY SAND | 120 | 120 |
| S13 | 6.5 | CLAYEY SAND | 71 | 71 |

### Petrographic Analysis

Petrographic analysis was performed on two samples of siltstone (logged as SILTY CLAY and *SILTSTONE CEMENTED CLAY*) recovered from borehole R20 at a depth of 15.0 and 15.5m – within the Baxter Formation unit. The petrographic analysis was performed by thin section for identification of lithology and associated petrogenesis/paragenesis. The laboratory description for both samples stated that they consisted of poorly competent, massive, argillaceous, friable, intensely ferruginized siltstone with silt sized grains cemented together with ferruginized clays. For engineering purposes both samples have been classified as follows:

- Ferruginized bioclastic siltstone
- Components of poorly sorted and rounded, sand sized ferruginous shelly material and undifferentiated biogenic clasts (23-27%)
- Carrying about 1% to 2% of rounded sand grains of quartz and minor feldspar
- Cemented by ferruginized clays
- Soft
- Weak

### 8.2.5 Sherwood Formation (Tm)

**Index Properties**

A total of 99 Atterberg Limit tests were performed on soil samples recovered from the boreholes. These results are summarised in Table 8-47 and plotted in Appendix H. The testing completed indicates that the soils within the Sherwood Formation generally comprise low to intermediate plasticity silts and high plasticity clays.

**Table 8-47 Sherwood Formation – Atterberg Limit – Results Summary - Marine**

| | Liquid Limit (LL) | Plastic Limit (PL) | Plasticity Index (PI) |
|---|---|---|---|
| | (%) | (%) | (%) |
| Minimum value | 20 | 10 | 0 |
| Maximum value | 105 | 54 | 73 |
| Mean Value | 48 | 30 | 18 |
| Lower quartile value | 43 | 25 | 7 |
| Upper quartile value | 54 | 35 | 24 |
| Number of tests | 99 | 99 | 99 |

**Table 8-48 Sherwood Formation – Atterberg Limit – Terrestrial Project Area Results Summary**

| | Liquid Limit (LL) | Plastic Limit (PL) | Plasticity Index (PI) |
|---|---|---|---|
| | (%) | (%) | (%) |
| Minimum value | 37 | 22 | 4 |
| Maximum value | 70 | 40 | 37 |
| Mean Value | 50 | 30 | 20 |
| Lower quartile value | 44 | 26 | 15 |
| Upper quartile value | 58 | 33 | 26 |
| Number of tests | 14 | 14 | 14 |

Laboratory test results for moisture content, particle density and dry density are summarised in Table 8-49 and Table 8-50 and plotted in Appendix H.

**Table 8-49 Sherwood Formation – Density test – Results Summary- Marine**

| | Moisture Content | Particle Density | Dry Density |
|---|---|---|---|
| | (%) | (t/m3) | (t/m3) |
| Minimum value | 9.8 | 1.88 | 1.05 |
| Maximum value | 89.5 | 3.00 | 1.67 |
| Mean Value | 45 | 2.65 | 1.22 |
| Lower quartile value | 40 | 2.60 | 1.11 |
| Upper quartile value | 51 | 2.73 | 1.29 |
| Number of tests | 245 | 39 | 39 |

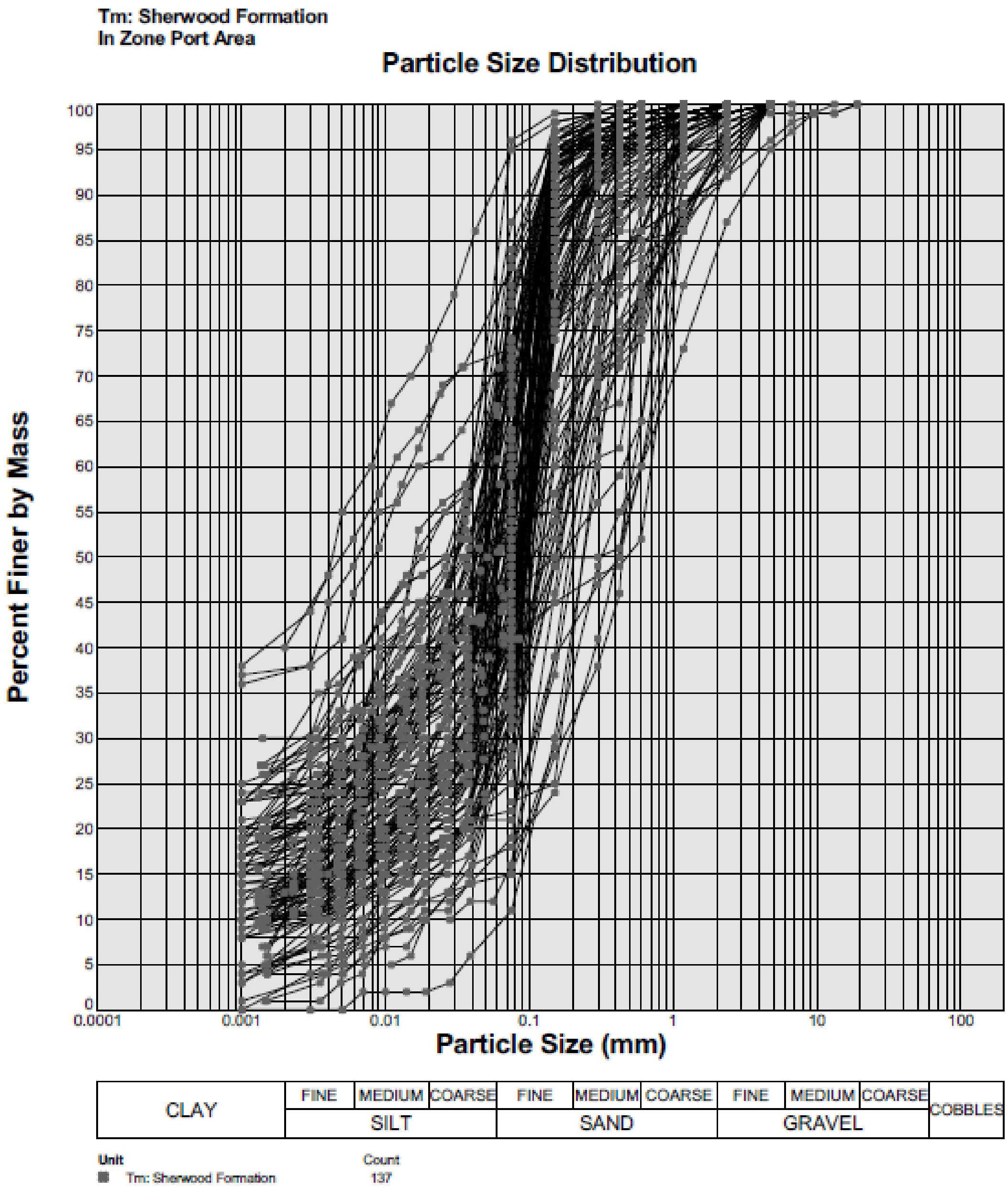
**Table 8-50 Sherwood Formation – Density test – Terrestrial Project Area Results Summary**

| | Moisture Content | Dry Density |
|---|---|---|
| | (%) | (t/m3) |
| Minimum value | 23.0 | 1.0 |
| Maximum value | 63.1 | 1.2 |
| Mean Value | 47.1 | 1.1 |
| Lower quartile value | 45.9 | - |
| Upper quartile value | 51.4 | - |
| Number of tests | 21 | 3 |

**Particle Size Distribution (PSD)**

PSD curves are given in Figure 8-8 for all particle size analysis completed on Sherwood Formation materials. The PSD curves for Sherwood Formation show that these materials comprise fine to medium grained sands with a fines content typically ranging from 20% to 50%, and sandy silt and sandy clay materials. As discussed previously PSD testing was typically not scheduled on clay materials and as a result the PSD curves show a bias towards sand materials.

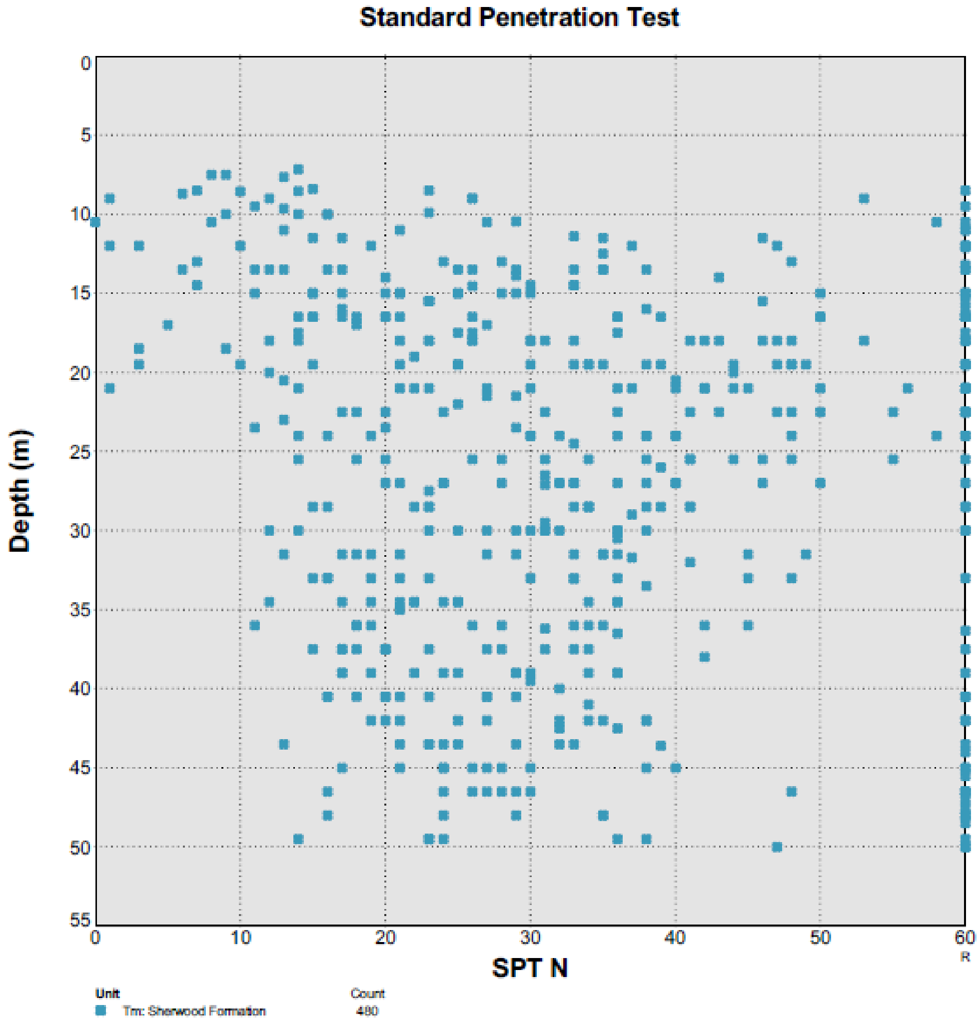
**Figure 8-8 Sherwood Formation – All PSD curves - Marine**




## SPT Test Results

Results of the Standard Penetration Tests (SPT) undertaken in the investigation boreholes are plotted versus depth in Figure 8-9 and are given in Appendix G plotted versus RL. SPT N values are also shown on the geological cross-sections. SPT refusal values have not been extrapolated to provide an equivalent SPT N value as the reason for refusal is not known and extrapolation may be therefore misleading. For example refusal may be on a very dense sand layer or a relatively thin cemented horizons.

**Figure 8-9 Sherwood Formation – SPT N values vs depth - Marine**




## Undrained Shear Strength

The in situ testing performed on Sherwood Formation included hand shear vane readings on the ends of undisturbed tube samples, and single stage unconsolidated undrained triaxial tests (UU) performed on selected undisturbed samples. The results of these tests are summarised in Table 8-51 and are plotted versus depth and RL in Appendix H.

**Table 8-51 Sherwood Formation - Undrained Shear Strength – Results Summary - Marine**

| | Undrained Shear Strength (Su) | |
|---|---|---|
| | Hand Shear Vane | Undrained unconsolidated triaxial |
| | (kPa) | (kPa) |
| Minimum value | 22 | 43 |
| Maximum value | 240 | 348 |
| Mean Value | 124 | 169 |
| Lower quartile value | 81 | 81 |
| Upper quartile value | 160 | 250 |
| Number of tests | 43 | 31 |

UU tests were predominantly performed on fine grained soils typically low to intermediate plasticity silts and high plasticity clays.

One in-situ shear vane test was performed on the Sherwood Formation and had a peak vane strength of 556kPa. This result is noted in the factual report as being unreliable as over-sampling of vane test interval recorded soft silt which is not consistent with the test result which is indicative of hard material.

Based on the hand shear vane results the sensitivity of materials within the Sherwood Formation is typically in the range of 2-6.

**Effective Stress Parameters**

The effective stress parameters of selected samples were determined from consolidated undrained triaxial compression tests with pore water pressure measurement. Cell pressures were selected to cover the stress conditions from the current in-situ stresses to those likely to follow construction. The results from tests performed on undisturbed samples are given in Table 8-52. Four of the five tests comprised three single stage/multi-stage tests and one test was a single stage test on sand where cohesion was assumed to be zero. The mean and upper quartile values for cohesion are skewed by two high values of 35 kPa and 76kPa on samples of clayey silt and silty sand.

In addition to tests on undisturbed samples four tests were completed on SPT samples. These materials had a cohesion of zero and friction angle ranging from 25 to 38 degrees.

**Table 8-52 Sherwood Formation - Effective Stress Parameters – Results Summary – Marine**

| Borehole | Sample Depth | Description | Consistency | Effective Stress Parameters | |
|---|---|---|---|---|---|
| - | (m) | - | - | Cohesion (c') | Friction Angle (ϕ) |
| | | | | (kPa) | (°) |
| S11 | 34.5 | Sand | Dense | 0 | 45 |
| R14 | 12 | Silt | Stiff | 0 | 32 |
| S15 | 34.5 | Silt | Hard | 0 | 38 |
| S17r | 16.5 | Clayey Silt | Firm | 35 | 35 |
| S8 | 13.5 | Sandy Silt | Very Stiff | 76 | 27 |
| Mean Value | | | | 22 | 35 |
| Number of tests | | | | 5 | 5 |

**Point Load Strength Index**

Point load strength index tests (diametrical) were undertaken on selected samples of cemented materials in the Sherwood Formation. The results are summarised in Table 8-53 and indicate that the cemented materials have an equivalent rock strength ranging from extremely low to high strength.

**Table 8-53 Sherwood Formation – Point load tests results on cemented horizons - Marine**

| Borehole | Depth | Soil Description | Point Load Is(50) |
|---|---|---|---|
| - | (m) | - | (MPa) |
| R5 | 14 | Silcrete Horizon in Silt | 2.00 |
| R5 | 15.6 | Silcrete Horizon in Silt | 1.70 |
| 0R5 | 16.8 | Silcrete Horizon in Silt | 1.40 |
| R20 | 20.5 | Cemented Clay | 1.30 |
| R20 | 22 | Cemented Clay | 0.03 |
| S7 | 48.3 | Cemented Sand | 0.12 |
| S7 | 48.4 | Cemented Sand | 0.80 |

### Permeability Testing

A total of nine constant head permeability tests have been carried out on selected samples recovered as part of the geotechnical investigation. All of these tests have been completed on ‘G’ boreholes in the North Port Area with five tests being completed on undisturbed samples and four tests on remoulded SPT samples. A summary of the tests results is given in Table 8-54.

**Table 8-54 Sherwood Formation – Permeability Test Results – Results Summary - Marine**

| | Coefficient of Permeability (m/s) | |
|---|---|---|
| | Undisturbed | Remoulded |
| Minimum value | 8.00E-10 | 4.00E-09 |
| Maximum value | 4.00E-07 | 4.00E-08 |
| Mean Value | 1.09E-07 | 2.35E-08 |
| Number of tests | 5 | 4 |

### Consolidation Parameters

Four consolidation tests have been completed on selected samples recovered as part of the geotechnical investigation. A summary of the test results and interpretation of consolidation parameters are presented in Table 8-55.

**Table 8-55 Sherwood Formation – Consolidation Test Results - Marine**

| Borehole | Sample Depth | Description | Consistency | Void Ratio | Preconsolidation pressure | Compression Index | Recompression Index |
|---|---|---|---|---|---|---|---|
| - | (m) | - | - | (e0) | (kPa) | Cc | Cr |
| R1 | 16.50 | Silt | Soft | 1.126 | 110 | 0.197 | 0.017 |
| R1 | 17.50 | Silt | Soft | 1.093 | 150 | 0.300 | 0.022 |
| R4 | 7.50 | Clayey Sand | Medium Dense | 1.118 | 240 | 0.515 | 0.060 |
| R5 | 12.50 | Sandy Clay | Stiff | 1.104 | 300 | 0.582 | 0.031 |

### Organic Content

Organic content testing was completed on selected samples recovered from the boreholes. A summary of the test results is given in Table 8-56

**Table 8-56 Sherwood Formation – Organic Content – Results Summary - Marine**

| | Organic Content (%) |
|---|---|
| Minimum value | 0.3 |
| Maximum value | 14 |
| Mean Value | 2.8 |
| Lower quartile value | 0.9 |
| Upper quartile value | 2.6 |
| Number of tests | 12 |

### Durability Testing

Durability testing was completed on selected samples recovered from the boreholes. A summary of the results of chloride, pH, resistivity and sulphate testing is given in Table 8-57. Results for pH include laboratory test results and pH values recorded in the field as reported on the borehole logs.

**Table 8-57 Sherwood Formation – Durability Tests – Results Summary - Marine**

| | Chloride | pH | Resistivity | Sulphate |
|---|---|---|---|---|
| | (mg/kg) | - | (ohm.m) | (mg/kg) |
| Minimum value | 480 | 6.4 | 2.6 | 12 |
| Maximum value | 9200 | 9.1 | 1800 | 420 |
| Mean Value | 3141 | 8.0 | 75 | 108 |
| Lower quartile value | 1650 | 7.5 | 5 | 54 |
| Upper quartile value | 3900 | 8.5 | 13 | 125 |
| Number of tests | 35 | 66 | 35 | 35 |

**Petrographic Analysis**

Petrographic analysis was performed on one sample of Silcrete (logged as a SILCRETE horizon within SILT) recovered from borehole R5 from 14.0 to 14.1m depth which is within the Sherwood Formation. Petrographic examination in thin section confirmed that at least 80 volume % of the sample consists of massive essentially cryptocrystalline gradational to patchy microcrystalline to vaguely oolitic/spherulitic carbonate, with minor diffuse optically-brownish 'fine' staining throughout more extensive clouded-yellowish carbonate, which was identified as siderite. Various aspects of the petrography are illustrated in Figure 8-10.

**Figure 8-10 Sherwood Formation – Silcrete Horizon – Petrographic Analysis - Marine**

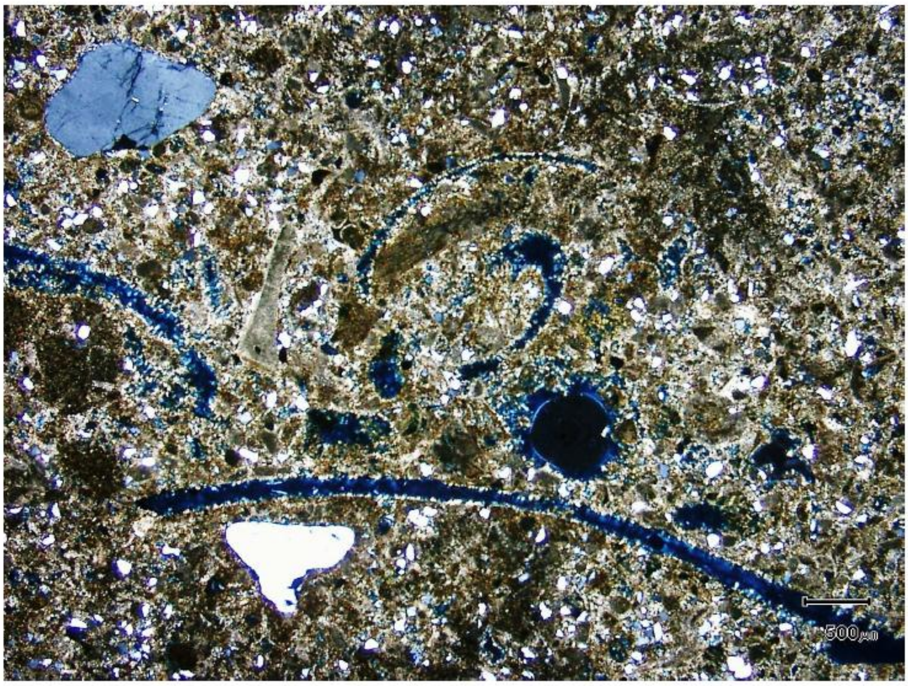



**Fig 1** **P0H, 1010–01290**

Transmitted light, thin section (TS), crossed nicols (x nic). Lowest magnification (X20). Typical of whole sample, massive microcrystalline siderite, crowded with scattered fine fossils/fossil fragments, disseminated white very fine quartz sand grains, grey and white. Blue epoxy filling voids of leached-out large thin and curved ex-fossils.

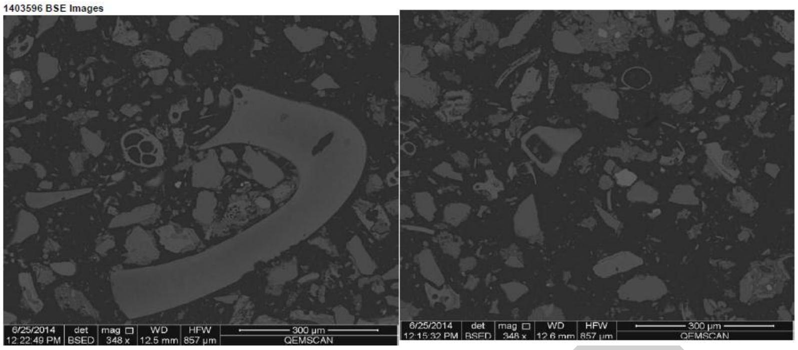
**Mineralogy**

Mineralogical assessment of quartz and carbonate (and other minor minerals) has been completed on selected samples by quantitative x-ray diffraction analysis (XRD). The samples were dried and crushed to 100% passing 2.8 mm and sub samples were pulverised and micro-milled before being loaded into an XRD instrument. Sherwood Formation typically includes quartz, calcite and aragonite minerals. Summary results for mineralogical assessment are given in Appendix L.

### Angularity and Roundness

An example of the SEM analysis results for Sherwood Formation is given in Figure 8-11 below. Additional results for angularity and roundness are given in the factual report on the marine geotechnical investigation.

**Figure 8-11 Sherwood Formation – SEM Images (BH S8–30m-Sandy Silt with trace shells)**




## 8.2.6 Older Volcanics (Tvo)

Laboratory testing was undertaken on selected samples from boreholes in the Western Channel and Anchorage. The results below do not include laboratory testing from material in Borehole R1 which has been logged as basalt.

Point load strength index tests (diametrical and axial) and unconfined compressive strength (UCS) testing was undertaken on selected basalt core samples. The results are summarised in Table 8-58 and indicate that the rock strength of basalt ranges from very low to medium strength with a mean rock strength of medium strength.

**Table 8-58 Older Volcanics – Point Load and UCS Tests – Results Summary**

| | Point Load Is(50) | | UCS |
|---|---|---|---|
| | Diametral (MPa) | Axial (MPa) | (MPa) |
| Minimum value | 0.04 | 0.10 | 0.3 |
| Maximum value | 0.57 | 0.44 | 19.7 |
| Mean Value | 0.26 | 0.20 | 7.0 |
| Number of tests | 4 | 4 | 5 |

### Mineralogy

Mineralogical assessment of quartz and carbonate (and other minor minerals) has been completed on selected samples by quantitative x-ray diffraction analysis (XRD). The samples were dried and crushed to 100% passing 2.8 mm and sub samples were pulverised and micro-milled before being loaded into an XRD instrument. Results for mineralogical assessment are given in Appendix L.

### Abrasivity

CERCHAR abrasivity testing was undertaken on selected samples of basalt from boreholes A1 and A2. The Cerchar abrasivity index and classification results are given in Table 8-59 and the classification scale is given in Table 8-60. The results from both tests indicate that Older Volcanic basalt is slightly abrasive.

Table 8-59 Older Volcanics – CERCHAR Abrasivity Test results

| Borehole | Depth (m) | RL (mCD) | Weathering | CERCHAR Abrasivity Index | CERCHAR Abrasivity classification |
|---|---|---|---|---|---|
| A1 | 5.59 | -20.36 | MW | 0.81 | Slightly abrasive |
| A2 | 2.3 | -17.68 | HW | 0.72 | Slightly abrasive |

Table 8-60 CERCHAR Abrasivity Test Classification Scale

| CERCHAR Abrasivity Classification | CERCHAR Abrasivity Index |
|---|---|
| Not abrasive | <0.3 |
| Not very abrasive | 0.3-0.5 |
| Slightly abrasive | 0.5-1.0 |
| Medium abrasiveness to abrasive | 1.0-2.0 |
| Very abrasive | 2.0-4.0 |
| Extremely abrasive | 4.0-6.0 |

**Petrographic Analysis**

Petrographic analysis was performed on one sample of Older Volcanics basalt rock (logged as highly weathered – moderately weathered basalt) recovered from borehole WC4 at a depth of 2.35m. The petrographic analysis was performed by thin section for identification of lithology and associated petrogenesis/paragenesis. The laboratory description stated that the sample consisted of poorly competent, massive, ferruginized and argillized igneous rock of basaltic style. For engineering purposes the rock has been summarised as follows:

- Argillized olivine basalt
- Intensely altered
- Slightly porous (5%)
- Carrying about 69% soft, weak minerals
- Soft
- Weak

### 8.2.7 Silurian

Point load strength index tests (axial and diametrical) and unconfined compressive strength (UCS) testing was undertaken on selected siltstone and sandstone core samples from the Port Area. The results are summarised in Table 8-61 and indicate that the rock strength of Silurian sandstone, siltstone and mudstone ranges from very low to very high strength with a mean rock strength of high strength.

Table 8-61 Silurian - Point Load and UCS Test – Results Summary

| | Point Load Is(50) | | UCS |
|---|---|---|---|
| | Diametral (MPa) | Axial (MPa) | (MPa) |
| Minimum value | 0.12 | 0.07 | 11.6 |
| Maximum value | 6.16 | 8.07 | 62.7 |
| Lower quartile value | 0.36 | 0.19 | 13 |
| Upper quartile value | 2.52 | 1.73 | 35 |
| Mean Value | 1.82 | 1.37 | 28 |
| Number of tests | 27 | 20 | 4 |

## 8.3 Groundwater

### 8.3.1 General

The hydrogeological regime of Western Port Bay is to be the subject of studies undertaken by the environmental assessment work streams and is outside the scope of the geotechnical interpretive report. However, for completeness, a summary of aquifers identified in the area, together with a summary of groundwater monitoring results carried out as part of the 2013/ 2014 field investigations are given below.

### 8.3.2 Regional Hydrostratigraphy

The aquifers identified within the stratigraphic profile are (from shallowest to deepest):

**Undifferentiated Quaternary**

Lakey *et al* (1980) indicate that the surficial dune deposits can form low yielding aquifers of potable quality. Fluvial sediments (shoe string sands) can form higher yielding aquifers, however groundwater salinity in these zones can be more variable.

**Baxter Formation (Western Port Group)**

Fluvial and paludal sands, gravels and clays, often carbonaceous deposited in a marine to fluvial environment. These sediments are widely developed throughout the Western Port Basin. The heterogeneous nature of the deposits result in significant anisotropy in hydraulic conductivity within this unit.

**Sherwood Formation (Western Port Group)**

Sands and limestones of marine origin. Within the Western Port Basin the coarse zones within the formation can be high yielding aquifers with considerable development potential. The heterogeneous nature of the deposits result in significant anisotropy in hydraulic conductivity within this unit.

**Yallock Formation (Western Port Group)**

Sands and gravels, often ligneous, deposited in a fluvial and paludal environment. The heterogeneous nature of the deposits result in significant anisotropy in hydraulic conductivity within this unit.

**Older Volcanics**

Often found with a thick, clay or highly weathered capping, the volcanics tend to function as a semi-confined to confined aquifer within the basin. The clay / weathered horizon forms an aquitard separating this formation from the shallower Tertiary and Quaternary sediments.

**Childers Formation**

Sands and gravels, often ligneous, deposited in a fluvial and paludal environment.

**Palaeozoic Rocks**

The basement rocks of the Westernport Basin are Silurian – Ordovician in age and comprised of mudstone, slate and sandstone.

### 8.3.3 Conceptual Hydrogeological Model

Lakey *et al* (1981) described the Western Port basin as a 'leaky-confined, horizontally stratified aquifer system', with each unit capable of development. In the terrestrial project area, the Western Port Group aquifer system (comprising the Late Tertiary age formations) is the main aquifer. Lakey *et al* (1981) report that 'the basin is considered to have a limited annual recharge component that is controlled by direct infiltration into small areas of the outcropping Tertiary aquifer, along the eastern and western basin boundaries, and also via infiltration into the Quaternary age sediments in the northeast along the Bunyip and Tarago rivers. Lakey *et al* (1980) suggests that much of the Western Port Group is in hydraulic continuity.

Groundwater flow was also characterised by Lakey *et al* (1981) which indicated that flow is predominantly coastwards (eastwards) into Western Port Bay. Groundwater flow in the Basin has been altered by anthropogenic development and increased extraction, largely for irrigation purposes. The threat of saline intrusion into the system prompted the declaration of the Koo Wee Rup Water Supply Protection Area.

### 8.3.4 Groundwater Levels

Groundwater levels measured following the terrestrial investigation are presented in Table 8-62.

**Table 8-62 Groundwater Levels**

| Borehole ID | Casing Elevation (m CD) | Depth to groundwater (m below top of casing) | Groundwater Level (m CD) |
|---|---|---|---|
| GA14-BH1GW | 15.53 | 6.15 | 9.38 |
| GA14-BH2GW | 4.91 | 5.35 | -0.44 |
| GA14-BH3GW | 11.06 | 3.50 | 7.56 |
| GA14-BH4GW | 12.60 | 3.30 | 9.30 |
| GA14-BH5GW | 9.30 | 5.65 | 3.65 |
| GA14-BH6GW | 12.00 | dry | - |
| GA14-BH7GW | 14.48 | 3.40 | 11.08 |
| GA14-BH8GW | 4.03 | 1.15 | 2.88 |
| GA14-BH9GW | 15.42 | 2.55 | 12.87 |
| GA14-BH10GW | 6.60 | 1.60 | 5.00 |
| GA14-BH11GW | 16.26 | 6.55 | 9.71 |
| GA14-MB01 | 15.59 | 2.40 | 13.19 |
| GA14-MB02 | 4.99 | 2.60 | 2.39 |
| GA14-MB03 | 10.85 | 3.20 | 7.65 |
| GA14-MB04 | 12.55 | 3.05 | 9.50 |
| GA14-MB05 | 9.32 | dry | - |

Initial monitoring data from installed data-loggers presented in the Terrestrial Factual Report indicate potential separated upper and lower aquifers in the northern part of the Terrestrial Project Area (GA14-MB01/GA14-BH1GW and GA14-MB02/GA14-BH2GW) which are not evident in monitoring results obtained elsewhere on the site.

Groundwater monitoring is ongoing and will primarily be used to inform future hydrogeological studies. A summary of the available groundwater monitoring data is given in Appendix J.

Although not encountered in any of the investigation test sites, where relatively thick deposits of permeable dune sands overlies the clayey strata of the Baxter Formation there is the potential for a transient ‘perched water table’ condition to develop.

# 9.0 Geotechnical Considerations for Port Expansion

## 9.1 General

Previous sections of this report have presented a summary of ground conditions present across the site including general discussions on spatial distribution of various soil and rock types within the port area. The purpose of this section of the report is to provide a general discussion on geotechnical considerations relating to dredging, reclamation, ground improvements, and wharf and ancillary structures.

It is noted that the scope and layout of proposed port development has not been determined at this stage, and various layout options are still being considered. The following discussions are therefore preliminary and general in nature, and are not intended to be exhaustive. It is expected that during review of development options and construction methodology / staging as part of other work packages/ work stream studies, further geotechnical risks and constraints will be identified that will need to be assessed and refined.

## 9.2 Dredging

It is expected that all port options will require significant dredging to accommodate the range of design vessels being considered. Dredging is expected to be required in the Port Area, North Arm and possibly in the Western Channel and Anchorage. It is probable that dredging of the Port Area will be undertaken in stages to support staged development of the port.

Materials expected to be present within the dredge depth are discussed in the following sections. For the purposes of this assessment the dredge depth has been assumed to be from seabed to -20 mCD. This is a nominal depth which has been adopted for the assessment of ground conditions and does not reflect the design dredge depth which is expected to be less than -20 mCD. The assessment of ground conditions is generally not sensitive to changes in dredge depth in the order of a few metres.

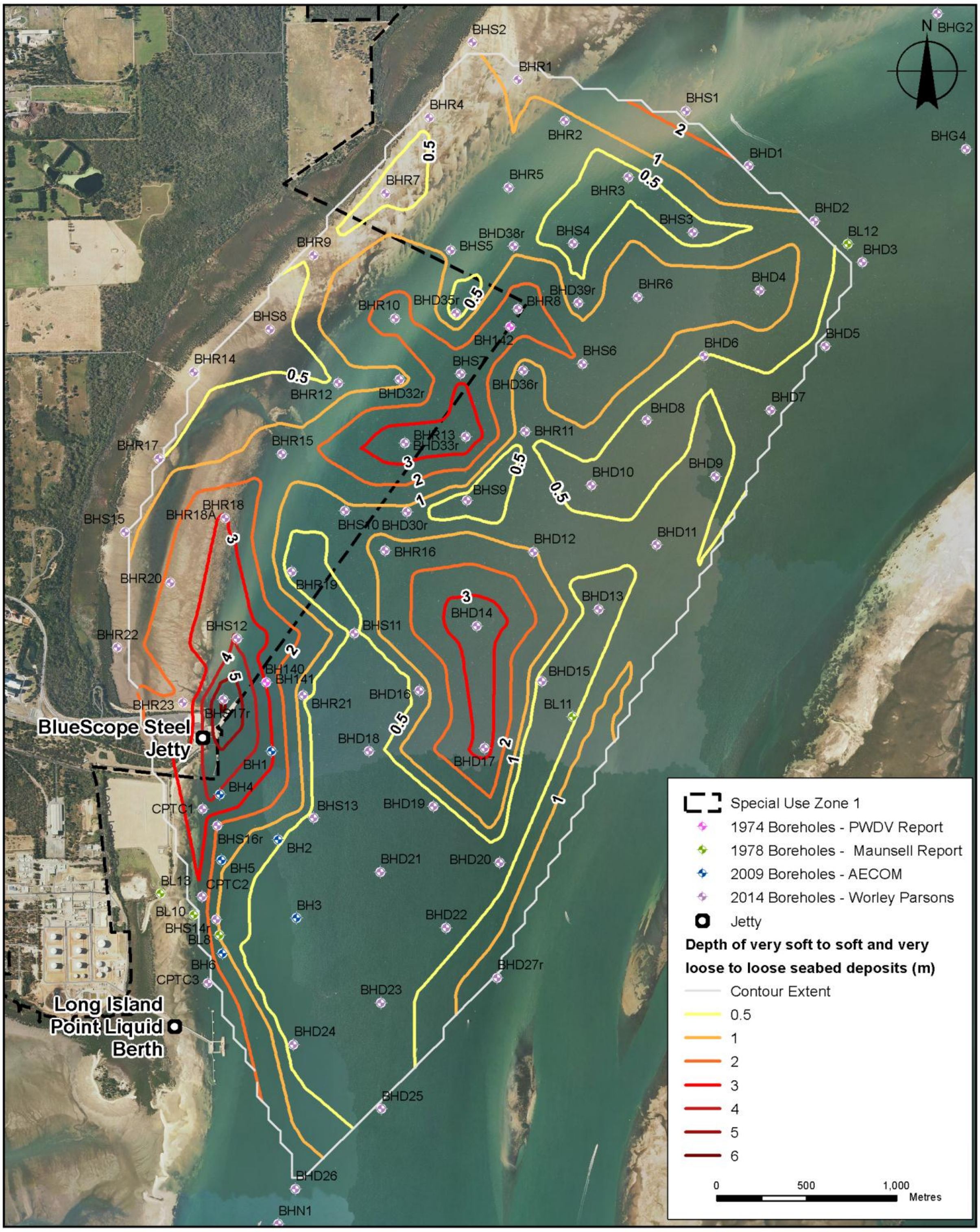
Specific studies on dredge methodology, assessment of dredge volumes and dredge materials management are being undertaken as part of other work packages or other work stream activities and are therefore not discussed in detail in this section.

### 9.2.1 Materials to be dredged

**Port Area**

Materials to be dredged within the Port Area consist of a surficial layer of weak Quaternary marine deposits comprising loose and very loose sand and soft and very soft clays and silts. The thickness of these weak materials ranges from 0 m to 6.3 m but is typically less than 1.5 m. The greatest thicknesses occur in the area north of the BlueScope Steel jetty where they are up to 6.3 m thick and between the BlueScope Steel jetty and the Long Island Point jetty where the thickness is around 3 to 4 m. A contour plot for the Port Area showing the depth of very soft to soft and very loose to loose deposits from seabed is given in Figure 9-1. This information is also provided in Appendix K showing depths at each borehole and differentiates between fine grained and coarse grained soils.

**Figure 9-1 Port Area – Depth in metres of very soft to soft and very loose to loose seabed deposits**




G:\31\31439\GIS\Maps\Deliverables\DE\20141021_Geotech Report\AGH-CEP0-DE-FIG-0103-DepthSeabedDeposits_Insert_B.mxd VicMap (DEPI, 2014), Port of Hastings Development Authority

Baxter Formation underlies the Quaternary deposits and comprises clayey sand, sandy clay, silty sand and silty clay and is typically of stiff to very stiff or medium dense to dense consistency/relative density. The materials within this unit exhibit vertical and lateral variability and depending on the dredging methods employed may be difficult to selectively dredge. Fine grained materials are predominantly clays and range from low to high plasticity. Sands are typically fine to medium grained and contain a high percentage of fines typically ranging from 10% to 50%.

The Baxter Formation also contains frequent weak horizons at depth as highlighted on the geological cross-section given in Appendix C. Cemented horizons are also present as discussed in Section 8.1.

The Sherwood Formation underlies the Baxter Formation and typically comprises silt and silty sand within the expected dredge depth. Sands are typically fine to medium grained, medium dense to dense and contain a high percentage of fines typically ranging from 20% to 50%. Silts are typically low to intermediate plasticity and of stiff to hard consistency.

**North Arm**

Potential dredge materials in the North Arm also comprise a surficial layer of weak Quaternary marine deposits similar to the Port Area up to depths of 0.5 m. Baxter Formation is present below the Quaternary deposits to depths of up to 8m. The consistency of the Baxter Formation materials is typically stiff to hard or medium dense to very dense. Clayey gravels containing angular basalt gravel are present in some boreholes as discussed in Section 8.1.4. Similar to the Port Area the Baxter Formation is underlain by Sherwood Formation (with the exception of borehole N3) which typically comprises stiff to very stiff silty clay, sandy silt and clayey silt, and dense to very dense sand with gravel. Residual mudstone was present in one borehole in the North Arm as a hard, medium to high plasticity, silty clay, at an RL of -21mCD which is expected to be below future dredge levels.

**Western Channel and Anchorage**

Materials within potential dredge depths in the Western Channel and Anchorage include a surficial layer of weak Quaternary marine deposits, stiff silty clay, medium dense to very dense silty sands and extremely weathered to moderately weathered basalt, ranging from extremely low to high strength, as shown on the cross sections given in Appendix B. The cross sections for the Western Channel and Anchorage also show the elevation of the rock reflector as interpreted from the geophysical survey.

### 9.2.2 Dredging Equipment

There are three main techniques commonly used in the dredging industry—trailing suction hopper dredging (TSHD), cutter suction dredging (CSD), and mechanical dredging (grab or backhoe).

In the Western Channel and Anchorage weathered basalt rock may be present within the dredge depth. Depending on the strength and fracture spacing of these materials, drill and blast techniques may be required to loosen materials prior to dredging.

**Trailing Suction Hopper Dredging**

Given that TSHDs excavate sea bed materials by suction, the technique is best suited to dredging of unconsolidated materials such as loose and soft soil, such as the Quaternary deposits which are present at the Hastings site from seabed.

For stiffer materials such as those present in the Baxter and Sherwood Formation, the trailing suction drag heads may be fitted with ripping tools and blades to loosen and slice consolidated materials, thus enabling the removal by suction into the pump and discharge system. However, highest production rates will be achieved in loose, fine to medium grained sands and soft clays and muds. Low production rates are expected for dredging in stiffer clays, and very dense sands. TSHD may not be practical in more competent hard clays or soils with partially cemented horizons.

**Cutter Suction Dredging**

The CSD technique is suitable for dredging a wide variety of materials ranging from soft clays to very dense sands and even soft rock (with unconfined compressive strength less than 25MPa).

For the Hastings site CSD is likely to be a suitable technique for dredging of the variable Baxter Formation sandy and silty clays, silty and clayey sands, and silts. Production rates may be reduced in stronger soils and where materials are cemented.

Where the Baxter Formation comprises stiff to hard clayey materials, or sands with high fines content (i.e. sandy soil with cohesive engineering behaviour), dredging using a CSD and pumping into a reclamation site is expected to result in the formation of clay balls which are lumps of original clay that have not disintegrated. The part of the clay that disintegrates forms a clay slurry as it mixes with the process water which leads to a heterogeneous fill with clay balls that settle close to the end of the pipeline and slurry that partly remains within the macro pores in between the clay balls. As the clay is likely to be dredged in combination with more sandy layers the resulting clay balls may be encapsulated in a sand matrix. Ground improvement is expected to be required to achieve competent ground conditions in reclamation where cutter suction dredging has been undertaken for clays.

Based on the current understanding of the Baxter Formation, it is expected that selective dredging of specific material types e.g. sands as opposed to clays, may not be possible in many instances, due to the interbedded nature and variable particle size distribution of the soils, and due to lateral variability.

**Mechanical Dredging**

Mechanical dredging typically involves the use of either a mechanical grab or back-hoe excavator for removal of sea-bed materials. It is a relatively slow and low-productivity process.

Dredging by backhoe excavator involves the use of a barge-mounted long reach excavator for the removal of sea bed material. Similarly a mechanical grab dredge lowers a clamshell grab to the seabed to excavate the material.

Advantages of using grab or backhoe equipment is that a wide variety of materials can be dredged ranging from soft to hard clays, very dense sand and weak rock. Dredge materials can also be selectively excavated, provided a good soil model is available for the dredging area, and can be visually observed to assess suitability for reuse. Excavated soils can also retain much of their in situ soil structure and are not fluidised as is the case for cutter suction or trailing suction dredging, resulting in a more compact material at the placement site.

## 9.3 Reclamation

Some port development options may include reclamation. The final reclamation footprint is unknown and will depend on the overall configuration of the port, position of the wharf line and the draft of the design vessel, all of which are being assessed as part of other work packages or other work stream activities.

It is assumed that the overall reclamation footprint will include discrete reclamation areas which will support staged expansion of the port. Dredged material for reclamation will need to be placed in areas contained within bund walls. The bund walls would need to be designed to retain the often slurry-like output from dredging in discrete areas, protect the reclamation from erosion and wave attack, and retain and manage the tail water from the dredge material by forming settling ponds. The bund walls will need to be constructed prior to the commencement of reclamation works. Dredge materials would likely be unsuitable for bund wall construction due to their fine grained nature, the high fines content of sandy soils present, , and the difficulty associated with selectively dredging suitable materials. It is therefore most likely that bund walls may have to be constructed using imported granular fill or other suitable materials. Alternatively temporary or permanent structures such as sheet pile walls or permanent wharf structures such as bulkhead walls, or concrete counterfort units could be investigated in-lieu of external bund walls. Similarly external bund walls could be incorporated into permanent seawall structures.

Soft ground under bund walls will most likely need to be excavated and replaced with suitable fill prior to bund construction. Quaternary deposits comprising sandy materials and the top of the Baxter Formation are expected to provide a suitable foundation for construction of bund walls.

Dredged materials placed in reclamations will need to be treated prior to use of the reclaimed land for port operations. The type of ground improvement will depend on the materials being dredged, the dredging method and equipment and the placement methods used. Typical dredging methods are discussed in the previous section and placement methods may include the following:

**Placement Under Water**

- Dumping by TSHD and barges
- Pumping through a spreader or diffuser at the end of a pipeline
- Dumping or placing by grab

**Placement Above Water**

- Discharging via a pressure pipeline from cutter suction dredgers or TSHDs
- Placement of mechanically dredged materials by truck or other means, followed by spreading and compaction
- Placement of land sourced borrow materials by truck or other means followed by spreading and compaction

In addition to the dredging and general placement methods, there are several variables associated with control and management of dredged materials within the reclamation which will influence the geotechnical properties of the reclaimed material. These variables may include pumping distances, pipe sizes, pipeline arrangements, number and arrangement of discharge points, nature of the dredged material, thickness of lifts, and geometry of the reclamation area, all of which are dependent on the selected placement method.

Given the high percentage of fines in the dredge materials it is expected that fines will tend to segregate during the deposition process and placement and control of dredged materials will need to ensure that flocculated fines are not segregated within the reclamation. Management of dredge materials will need to avoid encapsulating fines in reclamation materials, and/ or avoid the formation of a layered system comprising alternating layers of fine and coarse materials. Where dredged materials contain a high fines content or where very soft or soft deposits (Quaternary deposits or soft horizons in the Baxter Formation) are dredged these materials may need to be removed from the reclamation. This may require a methodology which promotes segregation of the unsuitable and suitable fraction of the material.

Since the existing seabed has a variable thickness of very soft marine clays present, it is anticipated that difficulties would be experienced when placing reclamation fill without displacement or bearing failure of the low strength soils. In view of this, and so as to avoid entrapment of displaced very soft soils beneath and within the reclamation fill, prior removal of these materials by dredging should be considered.

Dependent on the type and location of proposed wharf structures and perimeter seawalls, the presence of weaker layers within the Baxter Formation that may impact on perimeter stability may need to be considered. If lower strength soils are present that could impact on the stability of dredged slopes then excavation and removal or local ground improvement may need to be considered. Based on current borehole data, the presence of weaker ground within the Baxter Formation does not appear to be extensive, but local horizons of weaker ground are present.

## 9.4 Ground Improvement

Following reclamation, it is anticipated that ground improvement works will need to be undertaken. The exact nature and scope of ground improvement works would be dependent on a number of considerations including:

- Methods of dredging and reclamation
- Actual materials that are to be dredged and used as reclamation fill
- Time available to carry out the ground improvement works
- The level and distribution of live loading
- Construction staging
- The sensitivity of specific facilities or operations to differential settlement
- Preference by owners and/ or operators for maintenance and or repair of facilities

- Technical and economic merits of ground improvement options

It is noted that the assessment of the need for, and selection of appropriate ground improvement methodology, is to be undertaken as part of other work package studies.

Notwithstanding the above, it is anticipated that ground improvement will be required in the areas listed below and that different techniques may be implemented in different areas of the site depending on ground conditions, types of materials to be improved, and the loading and settlement criteria for that area.

- To improve the density of dredged materials within the reclamation, in particular where materials have been placed under water. Loosely placed hydraulic sand, if not compacted, could potentially be subject to excessive post construction settlement, slope instability due to its low placement density e.g. in the sea wall area, and potentially susceptible to liquefaction during earthquake events.
- To improve the density and shear strength of soft and loose intertidal sediments left in place beneath future reclaimed areas. These low strength soils will be compressible when loaded, and if not improved, may present a risk of post construction differential settlement that could adversely affect performance of facilities e.g. pavement, gantry beams, buildings, services, drainage etc. An alternative to leaving these soft and loose deposits in place and later treating them with ground improvement would be to remove them by dredging prior to reclamation, or displace them laterally during filling operations, and progressively remove the soils along the displacement front.
- To improve the level of compaction of the Old Tyabb reclamation fill which was probably placed in an uncontrolled manner (i.e. without systematic compaction), and has been proven to be soft/ loose in the lower levels.

It is noted that the in situ Baxter Formation soils beneath the port area are generally relatively competent. Extensive ground improvement of these materials is unlikely to be required. However, local lower strength strata has been identified within the Baxter Formation at some locations, which if present beneath perimeter bunds, or at locations of critical or settlement sensitive structures, may mean that some form of ground improvement is required.

### 9.4.1 Ground Improvement Techniques

There are numerous techniques that can be used to improve the properties of the ground where sites are underlain by soft or loose natural soils or fills. Many of the available ground improvement techniques are implemented by specialist ground improvement contractors, some using proprietary equipment. Other techniques simply involve adding fill to a site to pre-load the ground to a level in excess of anticipated post construction loading, and then allowing the site to consolidate.

Several commonly used ground improvement options that may be appropriate for the proposed site are described below, together with a brief commentary on where such options may be applicable.

**Vibroflotation**

Vibroflotation or vibrocompaction involves the use of a vibrating probe that can penetrate granular soil to depths of over 30 metres (see Figure 9-2 – Schematic of Vibroreplacement process). The vibrations of the probe cause the grain structure to collapse thereby densifying the soil surrounding the probe. In order to effectively treat and densify a loose soil, the vibroflot is raised and lowered in a grid pattern.

Vibroreplacement is a combination of vibroflotation with the introduction of a gravel backfill resulting in stone columns, which increases the amount of densification, provides a degree of reinforcement of weaker soils, and can potentially provide an effective means of drainage. Vibroreplacement can also be used to improve the strength of softer cohesive materials whereby granular columns are installed at suitably close centres to reinforce the weaker soils. This method is not suited in very soft soils which have a very low undrained shear strength since the lateral support for the stone columns may be insufficient.

The range of soils for which the vibroflotation method is suitable is given in Figure 9-3 which indicates performance in medium and coarse sand and fine and medium gravel deposits. However, vibroflotation may not be suitable in fine sands and in soils with more than 10 to 20% silt and clay content. Since granular Baxter Formation soils comprise fine sands often with a relatively high fines content, depending on the resultant

particle size distribution after reclamation filling, the ability to densify reclamation sands derived from the Baxter Formation using vibroflotation may be marginal.

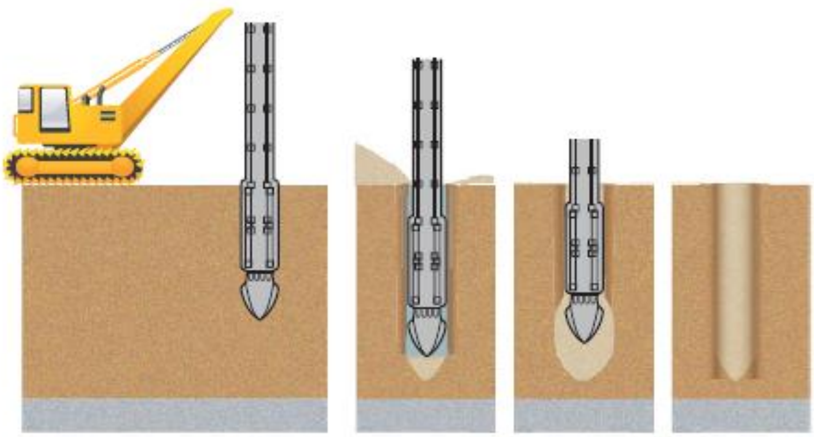


Figure 9-2 – Schematic of Vibroreplacement process

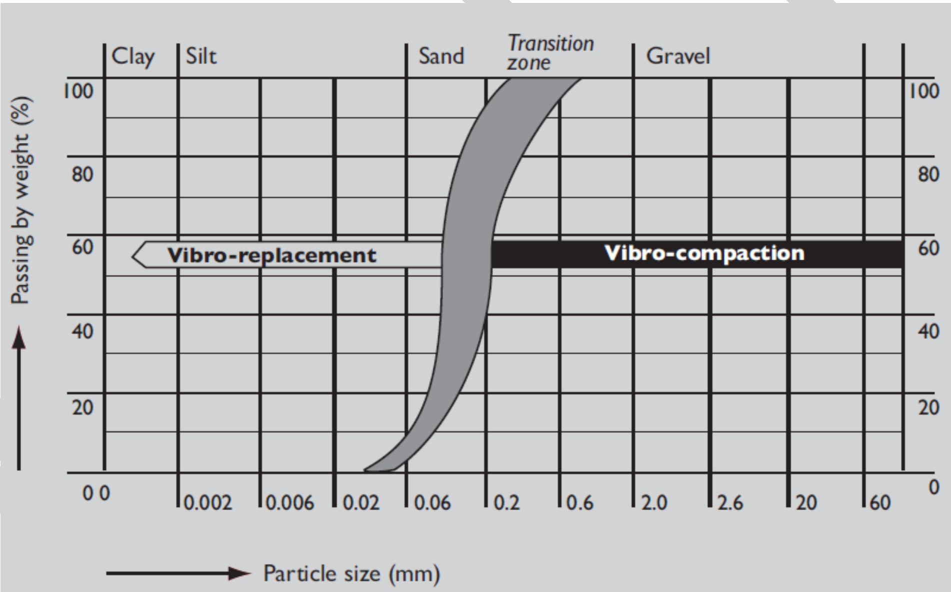



Figure 9-3 Field of application for vibro methods

**Dynamic Compaction**

Densification by dynamic compaction is performed by dropping a heavy weight of steel or concrete in a grid pattern from heights of 10 to 30 m. It can provide an economical way of improving soil for mitigation of liquefaction hazards and strengthening shallow weaker soils. Local liquefaction can be initiated beneath the drop point making it easier for the sand grains to densify. As can be seen from Figure 9-4 the process is invasive; and the surface of the soil would require compaction with addition of granular fill following dynamic compaction.

The effective depth of dynamic compaction is dependent on the nature of materials to be compacted, and the height, weight, and spacing of drop, and would be subject to site trials. Typical effective depths would be in the order of 5 to 10 m in granular soils, up to around 5 m in cohesive soils, but may be ineffective in very soft and soft clays.

Dynamic compaction could be suitable for densification and compaction of dredged materials in the main reclamation area, existing reclamation fills in the Old Tyabb reclamation and possibly the underlying loose

intertidal sediments. However, dynamic compaction may not be fully effective in improving fills and sediments where they have significant clay content, particularly at depth. Dynamic compaction would not be suitable for treatment of areas below water.

Figure 9-4 - Dynamic Compaction

### Pre-load Surcharge

Pre-load surcharge would involve placement of fill above the reclamation platform up to a level where pre-loading is in excess of final loading. Once the pre-load is placed the area is allowed to consolidate.

In granular materials settlement is rapid as porewater can readily escape from the soil as load is placed. However, where fill is placed over thick deposits of soft clay, settlement can continue for considerable time (years) due to slow drainage of excess pore water pressures. Installation of strip drains typically at 1 to 2 m centres shortens the drainage path and speeds consolidation. Figure 9-5 is a schematic of pre-load surcharge with vertical drains.

Preload surcharge may be a suitable ground improvement technique to consolidate and strengthen soft and loose fills and underlying intertidal sediments in the Old Tyabb reclamation. Based on typical magnitude of port loading (40 to 60 kPa) the thickness of pre-load surcharge needed may be in the order of 3 to 4 m.

Since soft clays deposits beneath the Old Tyabb reclamation would appear to be relatively thin and sandy layers are present, it may be feasible to implement a “rolling” pre-load surcharge regime without the need for vertical drains.

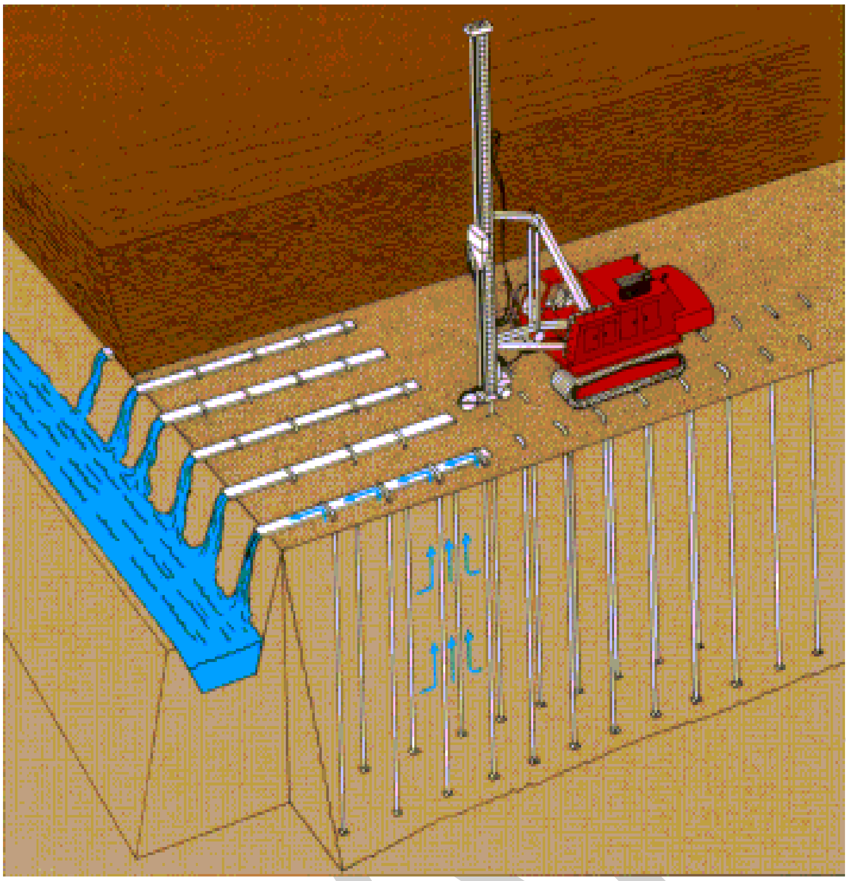


Figure 9-5 – Schematic of pre-load surcharge with vertical drains

**Vacuum Consolidation**

An alternative form of applying surcharge loading to an area is to adopt vacuum consolidation. This method involves placing an airtight membrane over the surface to be treated and applying a vacuum. This can generate a preload of up to a theoretical maximum of 100kPa (equivalent to a 5m high embankment) but in practice this value will be less as a result of losses in the system.

The vacuum loading is generated by electric pumps removing water from a surface drainage layer connected to vertical wick drains. This accelerates the rate of consolidation of the soft clay significantly. Figure 9-6 is a schematic of vacuum consolidation.

One of the advantages of this method over traditional surcharge methods is that it exerts an isotropic load without introducing shear stresses in the subsoil and therefore can be used where traditional surcharge is not possible for edge stability reasons.

Vacuum consolidation may be used as a suitable technique to accelerate the rate of consolidation of soft clay or silty clay layers, and may be used in combination with pre-load surcharge. In sands or sandy clays cut-off walls into non-permeable layers may be required to seal off the vacuum.

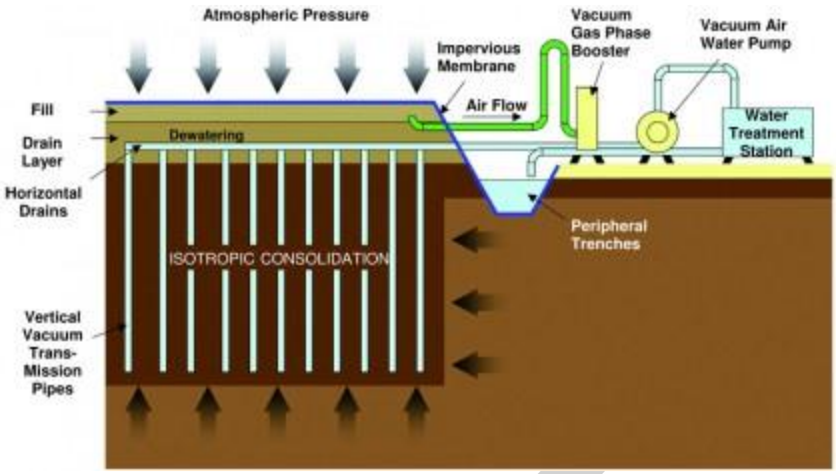



**Figure 9-6 Schematic of Vacuum Consolidation**

### Deep Soil Mixing

Deep soil mixing (DSM) involves in-situ mixing of soil with admixtures which chemically react with the soil to increase strength and stiffness, and reduce permeability and compressibility. The stabilising additives are injected into the soil in dry or slurry form through hollow rotating mixing shafts tipped with various cutting and mixing tools resulting in the formation of stiff columns within the soil. The method can be used in stratified soils including sands, soft clays and silts and can be applied up to depths of 50 m.

Deep soil mixing could be suitable for improvement of dredged materials in the main reclamation area, existing reclamation fills in the Old Tyabb reclamation and possibly the underlying loose intertidal sediments.

Deep soil mixing has the benefit of not extracting soils, however depending on the adopted method some spoil may be generated during the mixing process which would need to be disposed of. This method can comprise single columns or multiple installations forming walls, blocks, or grids to improve the ground and support specific facilities as illustrated in Figure 9-7. Depending on the column patterns a load transfer platform may be required at surface to transfer load to the columns using crushed rock and high strength geotextile or geogrid.

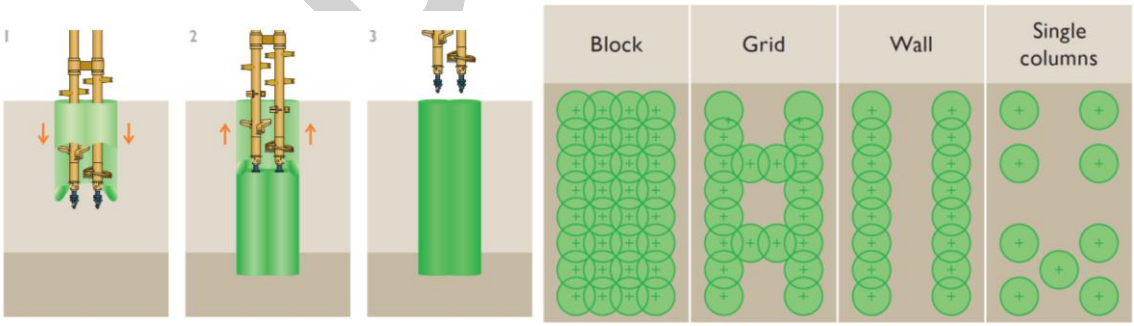



**Figure 9-7 Examples of Deep Soil Mixing column patterns**

### Controlled Modulus Columns

Controlled Modulus Columns (CMCs) can be used to strengthen the ground by installation of rigid inclusions. This method differs from DCM techniques in that the method uses a displacement auger and pumping concrete or grout during auger extraction.

Where CMCs are used to support facilities this typically requires a load transfer platform at surface to transfer load to the CMCs using crushed rock and high strength geotextile or geogrid. CMCs can be reinforced and used as displacement piles to support structural loads or to resist bending.

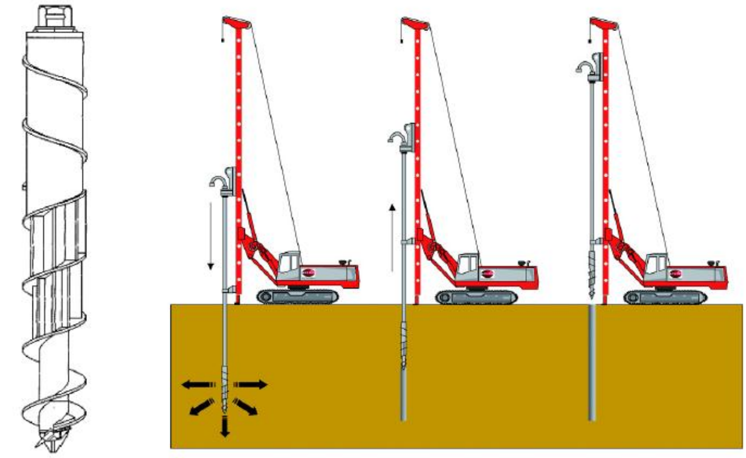



**Figure 9-8 Installation process for CMCs and Load Transfer Platform**

### Impact Rolling

Impact rolling involves the use of a non-circular roller to impart a high level of energy into the ground when towed at relatively high speed (10 to 12 km/hr). A number of proprietary 3, 4, or 5 sided rollers are available on the market (see Figure 9-9). The benefit of impact rolling is that it can improve ground to a greater depth than compaction with conventional earthworks equipment, with reported depths of influence of up to 4 m, but with 2 m being more typical.

Impact rolling is unlikely to be suitable for improving deeper uncontrolled fills, and is not suitable for soft sediments. However, impact rolling could be effective in compacting the Old Tyabb reclamation fill, in conjunction with pre-load surcharging, as a preparatory treatment of the exiting fill platform for pavement support.

**Figure 9-9 - High Energy Impact Rolling using a 3 sided roller**

## 9.5 Wharf Structures

This section discusses a range of structure types that may be considered for quay structures for both the land-backed and non-land backed schemes. A number of key geotechnical considerations associated with each option are also discussed. Multiple structure types and hybrids may be used depending on the port layout and associated ground profiles. It is noted that determination of suitable wharf structure options, including soil structure interaction analysis, and assessment of slope stability and pile capacities are to be undertaken as part of separate work package studies.

**Piled Quay Deck over a Rock Revetment Slope**

A piled quay deck would comprise a reinforced concrete deck supported on vertical and raker piles, with revetted embankment under the wharf. Driven tubular steel piles are commonly used for suspended deck structures due to their ability to support high axial loads and bending, but other options such as driven reinforced concrete piles can be used.

The quay deck would typically be constructed from precast or in-situ reinforced concrete beams, with precast deck panels and an in-situ topping slab, supported on tubular steel piles. Piles (for a piled quay deck and for other structures where piles are required) would be driven to a nominated level, a specified driving resistance or to refusal on rock depending on the location of the quay line, in order to achieve required geotechnical design capacity in accordance with AS2159 "Pile design and Installation". Pile drivability studies and a pile testing programme using static load tests and Pile Dynamic Analyser (PDA) testing with CAPWAP would typically be undertaken to inform the pile design, and to confirm pile capacities have been achieved during construction. Typical skin friction and end bearing parameters for preliminary assessment are provided in Section 9.10.

In order to achieve lateral stability and ensure piles are founded in competent ground suitable to support design loading piles for quay structures would need to be founded a suitable distance below dredge level. It is noted that within both the Baxter and Sherwood Formations at the Hastings site cemented soils are present through which it may be difficult to drive piles. This may result in heavy driving or potentially requiring other means of pile advancement such as pre-boring or mucking out of piles during the driving process.

Revetment slopes under the deck are expected to be in the order of 1V:2H, and subject to slope stability analysis, depending on the nature and consistency of the soils within and beneath the revetment. Excavation and replacement of soft and loose soils that may influence stability may be required, if present.

**Reinforced Concrete Caisson**

Concrete caissons are a form of gravity structure which consist of open-topped precast concrete cells which are usually constructed in the dry, typically at a slipway or in a dry-dock, which are then floated to their final location and sunk into position on to pre-prepared rock fill/granular base

The quay structure is formed by a series of caisson units positioned side by side and jointed together. In situ concrete keyed joints are normally used between rectangular caissons, placed within vertical recesses formed in the outer walls of each caisson, and are required to accommodate placing tolerances and differential settlement between units.

After positioning the caissons are usually filled with sand or granular fill and the upper portion of the caisson constructed in-situ which provides for a continuous beam along the quay line which is usually designed to incorporate the front crane rail.

This type of structure requires competent strata at foundation level to support the high bearing pressures and to minimise differential settlement between the caisson units. High allowable bearing capacities may not be available for the Baxter and Sherwood Formations, and depending on the location of the quay line, excavation of a trench along the alignment of the quay may be required to significant depth to remove lower strength materials below dredged seabed level to provide a suitable foundation for the rockfill base. Alternatively, weaker foundation soils could be treated or reinforced in-situ to improve its bearing capacity, but this is expected to be an expensive option.

**Counterfort Wall**

Counterfort walls are another form of gravity structure commonly used for quay construction and consist of precast concrete wall segments which include a base slab and cantilever wall strengthened with counterforts monolithic with the back of the wall slab and base slab. Similar to the caisson option, this type of structure is usually constructed in the dry and delivered to site using barges and handled using barge mounted cranes.

Counterfort walls are also placed in segments onto a pre-prepared granular base and jointed in a similar manner to the concrete caisson option, however the number of joints required is usually significantly more than that for a caisson system. The wall would be backfilled with imported granular fill.

Similar to the caisson option this type of structure also requires competent strata at foundation level to support the high bearing pressures and to minimise differential settlement between the counterfort units. Removal of unsuitable material and excavation of a trench along the alignment of the quay would be required to depths similar to the caisson option.

Both this system and the caisson system are forms of gravity structures. Counterforts utilise the self-weight of the retained material to provide stability and resistance against overturning, whereas the caisson option utilises the self-weight of fill material retained within the caisson units to provide stability and resistance against overturning. The caisson system is less susceptible to loss of retained material and differential settlement and is generally considered to be a more robust solution for quay wall construction when compared to a counterfort wall solution.

**Bulkhead Wall**

This option comprises an anchored bulkhead wall constructed along the quay line. The bulkhead wall would typically be constructed using either large diameter tubular steel piles driven to sufficient depth for toe restraint and to support crane rail loading, with alternate piles potentially driven to shallower depths, or a combi-wall system comprising a combination of tubular steel piles driven to sufficient depth for toe restraint with infill sheetpile sections. The piles are clutched together as they are driven and laterally restrained by a series of steel tie rods and a sheet pile anchor wall or anchor piles. Alternative forms of construction for bulkhead walls include concrete diaphragm walls.

The front crane rail is located over the tubular steel piles which transfer the vertical loads to foundation level, while an independent row of piles and crane rail beam would be required for the rear crane rail.

Granular materials would typically be used immediately behind the bulkhead wall to reduce lateral loads on the wall and reduce the magnitude of tidal lag from one side of the wall to the other, with dredged material then placed behind this zone of granular material. Dredge materials from the Baxter or Sherwood Formation could potentially be used in the area immediately behind the bulkhead wall but would require ground improvement.

Bulkhead walls can be driven from floating plant with subsequent filling behind the wall after installation, or alternatively bulkhead piles could be installed within areas previously reclaimed or within existing ground (eg. Basin option), with subsequent re-excavation and dredging in front of the wall. Should fills be used behind the wall that require ground improvement, or if in situ low strength soils are left in place that require ground improvement, then the staging of treatment would need to be undertaken so as to avoid adverse impact or disturbance of the wall and tie back system.

Alternatively a suspended deck supported on piles could be used in conjunction with a bulkhead wall option which would reduce the volume of high quality imported fill materials and/or reduce the extent of ground improvement required behind the bulkhead wall. Incorporation of a suspended deck also avoids the potential for differential settlement across the wharf between areas where dredged materials have been placed and ground improved, and areas where granular materials have been placed immediately behind the wall.

An important issue associated with this type of quay structure is the potential for differential settlement where fill material tends to settle behind the main bulkhead wall capping beam, and the associated stresses which this settlement can create in the tie-rods supporting the wall. This issue may be overcome by careful detailing of the connection between the bulkhead wall and tie-rods including provision of flexible joints, and placing tie rods in oversize tubes to isolate then from ground movement.

For the bulkhead wall and gravity type wharf structures the rear crane rail will be an independent structure typically constructed from a series of tubular steel piles (or other suitable pile types) driven to achieve required geotechnical pile capacities. A reinforced concrete crane rail beam would span between the piles. Alternative pile type for installation using land based equipment may include reinforced bored cast in situ, precast concrete driven piles or continuous flight auger (CFA) piles. Previous comments in respect to pile installation/ driving studies and pile testing regime would be required to inform the pile design. The presence of cemented horizons potentially causing piles to terminate at shallow depth would also need to be considered when selecting pile type and assessing design lengths.

**Seawalls**

Seawalls may be required at the boundaries of potential reclamation areas or for formalisation of existing seawalls. The seawalls provide a transition from the potential reclamation to the existing bed level and are also designed to prevent overtopping and inundation of the reclamation area due to wave action.

The stability of seawalls would need to take into account ground conditions within the proximity of the slope and similar to revetment slopes excavation and replacement of soft and loose soils beneath the seawall may be required.

Characteristic engineering parameters for use in the design of wharf structures are given in Section 9.10. As the port layout is developed, design stratigraphic profiles and corresponding design parameters can be produced. It is expected that this will be completed as part of future work packages. Examples of how these parameters can be applied to the boreholes to develop a design profile are given in Appendix M.

## 9.6 Ancillary Structures and Facilities – Terrestrial Project Area

### 9.6.1 General

It is anticipated that a number of ancillary structures will be required to support the port development, which will be situated landside within the Special Use Zone (SUZ1). The exact nature of the infrastructure, loadings and the associated individual development footprints are yet to be determined, but are likely to include warehouse structures, container storage areas, office buildings, factories, road and rail infrastructure, intermodal facilities etc.

Typically the soil profile across the majority of the SUZ1 is underlain at shallow depth by stiff to very stiff clay soils belonging to the Baxter Formation (Tb). This formation offers suitable founding strata for spread footings systems (pad, strip footings or slab systems).

Some localised areas of the SUZ1 associated with local drainage lines and coastal fringes contain Quaternary alluvium deposits. The bearing capacity support of footings situated in this unit can generally be expected to be less than the Tb unit.

Borehole GA14-BH8GW, which was drilled close to the existing coast line, indicated weak soils mainly described as organic loose silty sand extending to approximately 1.5 m depth. For these areas deeper footing systems (i.e. piers) or local ground replacement / improvement may be required depending on the structural loading requirements.

Borehole GA-BH5-GW and CPT probe GA14-CPT08 describe clayey sand and sandy clay soil types overlying a weak layer of organic silty sand present between approximately 5.5 m to 7.2 m depth. Based on the material descriptions and the surrounding topography, which includes the adjacent Tyabb reclamation area it is possible that the soil layers in the upper part of the bore are representative of fill placed over the natural weak alluvium (organic silty sand). Layers of saturated very loose to medium dense sand are recorded within GA-BH5-GW, which based on their low relative density are considered to be potentially liquefiable under seismic loading and warrant further analysis.

### 9.6.2 Soil Reactivity

Based on the results of the investigations it is concluded a site classification of Class M (moderate soil reactivity) can be adopted with respect to AS 2870 – 2011. This classification is based on the predominant Baxter Formation (Tb) soil unit, which generally consists of natural sandy clay and clayey sand soil types.

It should be noted that the Class M classification deals with residential slabs and footings (AS 2870). This classification is not technically correct for the type of industrial structures proposed and therefore is given as a guide only with respect to seasonal shrink/ swell movement. For buildings supported by shallow spread footings seasonal surface movements due to reactive founding soils may be up to approximately 40 mm.

The classification of CLASS M does not take into account abnormal moisture conditions arising from improper maintenance or drainage in proximity to the foundation zone. Where such conditions prevail, structures will have an increased likelihood of damage due to shrinkage and heave effects.

For localised areas underlain by weak Quaternary alluvium or uncontrolled fill, further engineering assessment will be required for footing design.

Placement of imported engineered filling, the introduction and/ or removal of vegetation and trees, and the installation of service trenches and drainage, amongst other considerations, may alter the site reactivity and the potential for shrink-swell movement recommendations given above.

### 9.6.3 Spread Footings

Shallow spread footings (pads, strip footings and slab edge beams) may be founded into the naturally occurring sand and clay soil types of the Baxter Formation (Tb).

Footings should be designed to accommodate the soil reactivity as described in Section 9.6.2

The following allowable bearing pressures may be adopted for concept design purposes in this formation:

Stiff or better Clay soils = 150 kPa

Very stiff clay soils or medium dense to dense sands = 200 kPa.

The minimum founding depth for spread footings based on the prevailing geotechnical conditions is likely to be less than 1 m, however footings should also penetrate all surficial sub-soil layers and any localised uncontrolled fill deposits.

Site specific geotechnical investigations should be carried out for individual structures once the design loads and footprints are known, and founding recommendations adjusted accordingly.

### 9.6.4 Piled Footings

In areas containing significant thicknesses of weak soil or for highly loaded or settlement sensitive structures it is recommended that deep foundations consisting of piled footings be used.

Bored pile foundations should penetrate not less than 500mm or 3 pile diameters (whichever is greater) into the natural materials to achieve the preliminary allowable bearing capacities given in Table 9-1.

**Table 9-1 Preliminary Allowable Bearing Capacities for Bored Piles**

| Minimum Founding Depth | Allowable End Bearing | Allowable Skin Friction | Natural Founding Strata |
|---|---|---|---|
| (m) | (kPa) | (kPa) | - |
| 3.0 | 300 | 30 | Very stiff Clay |
| 3.0 | 900 | 20 | Dense to very dense Sand |

The bearing capacities given in this section are indicative only for relatively shallow piles or piers and should only be used for preliminary purposes.

The bearing capacities given assume the pile excavation is clean and free of clay smear and loose material. A camera inspection of the pile shaft and base should be completed prior to concrete pouring to confirm the pile base and sides are clean.

No skin friction should be adopted in the top 1.0m depth of the soil profile due to the likelihood of shrinkage of reactive clay soil away from the pile shaft.

The above bearing capacities assume a pile spacing of not less than 3 pile diameters.

Settlement of piles is likely to be in the order of 1% of the pile diameter at working load. The settlement estimate does not include elastic shortening due to compression in the pile shaft.

Driven piles are also considered appropriate for this site. Working loads for driven piles will typically be governed by the shaft dimension and structural capacity.

Where driven piles are used the design bearing capacity should be confirmed during pile installation using Pile Dynamic Analyser (PDA) testing with CAPWAP analysis in accordance with AS2159.

## 9.7 Ancillary Structures and Facilities – Reclamation Area

### 9.7.1 General

Depending on port terminal layout and proposed method or methods of operation, it is anticipated that areas of land reclamation may be used for:

- Heavy duty port pavements for container handling and stacking
- Port interchange zones such as truck grids, rail sidings etc.
- Roadways and transitions to wharf structures
- Ancillary buildings and infrastructure including buried services and drainage

The exact nature of the infrastructure, loadings and the reclamation development footprints are yet to be determined. The nature and consistency of reclamation fill prior to and/ or following implementation of ground improvement activities is also not known. Specific requirements for individual applications and circumstances would therefore need to be assessed on a case by case basis.

Notwithstanding the above, the following general comments are made in respect to infrastructure support. It is noted that the following list is not intended to be exhaustive and it is expected that other considerations would need to be taken into account for specific facilities. It is noted that underlying soils of the Baxter and Sherwood Formations are relatively competent, with only local weaker horizons (layers). Comments below in respect to ground improvement are therefore mainly related to improvement of reclamation fills, and of existing Quaternary deposits should they be left in place below the reclamation. Many of the issues discussed in the comments below are also relevant to the onshore deposition of dredged materials.

- Reclamation fills derived from the Baxter and Sherwood Formations would unlikely be suitable to support port infrastructure without ground improvement. The design of the port development would therefore need to consider appropriate ground improvement and/ or structural support on deep foundations to minimise future maintenance and operational disruption.
- Depending on the loading scenarios, the nature and thickness of reclamation fill, and the method or methods of ground improvement, it is likely that design of structures and facilities will need to accommodate some level of post construction and differential settlements.
- The time needed to successfully implement ground improvement options taking account of port development staging will need to be carefully considered. Some options such as preload surcharge may take a number of years to implement over a large area. Use of a rolling surcharge approach over smaller areas could be considered to minimise the required fill volume.
- The design of dredging and reclamation operations would need to take into account access needs for equipment to undertake ground improvement and piling works. This may require construction of suitable temporary pavement or platforms to facilitate safe access for heavy plant.
- Allowance would need to be made for differential settlement between piled and non-piled structures. This may include provision of approach slabs or allowance for maintenance and re-levelling. All buried services that pass between piled and non-piled areas would also need to be flexible to accommodate differential movement.

- Allowance would need to be made for differential settlement between areas subject to ground improvement and areas not improved. Introduction of transition zones in areas of ground improvement may be incorporated into the design to reduce effects of abrupt differential movements.
- Surface water drainage and surface gradients would need to be designed to take into account both operational tolerances of equipment, and to accommodate anticipated settlement and differential settlement without functional impairment. This may include adopting relatively high gradients to reduce risk of ponding and reverse flows, and by making allowance for maintenance and re-levelling.
- Buried stormwater drainage would need to be designed incorporating flexible connections and short pipe lengths. The system will also need to accommodate potential rotation between pipes and pits without damage or leakage. Incorporating oversized pipe to reduce the effects of differential settlement may be required (to account for sagging of pipes and resultant sedimentation and loss of cross sectional area).
- Where piles are used to support structures in areas where post construction settlement is expected, pile design would need to take account of negative skin friction.
- Ground improvement may be required to address the risk of liquefaction under earthquake conditions in areas where sands and silts are used as reclamation fill. This may be important to minimise potential damage, for example, in areas of quay structures or areas where critical infrastructure is present.
- Ground improvement for container stacking and handling areas would need to take into account construction and operational tolerances. Areas of container stacking will impose significant loading onto the ground which will not only result in settlement beneath the stack, but also result in downdrag settlement and lateral displacement of immediately adjacent areas. Rail mounted gantry cranes and automated stacking crane (ASC) systems typically have tight tolerances in respect to longitudinal gradient, differential settlement between adjacent rails, and wheel gauge. Maintaining alignment of containers within stacks is also important for operational tolerance and operational efficiency. Therefore the design of foundation support and ground improvement systems beneath container stacking, container handling equipment, and in interchange areas will need careful consideration. Positive support such as piled systems or controlled modulus columns may need to be considered for settlement sensitive operations.

A possible cost effective design approach in some circumstances may be to accept that settlement/ differential settlement will occur and allow for maintenance and re-levelling or re-profiling of the site to maintain areas within operational tolerances. Obviously for such an approach allowance would need to be made for disruption to operations and associated costs.

## 9.8 Pavements

### 9.8.1 Pavement Types

The proposed layout and operating system as related to proposed pavement types, has not been determined.

Depending on the proposed operations it is expected that new pavements at the site will comprise combinations of the following:

- Heavy duty port pavements for container handling and stacking areas using rubber tyred equipment, this may comprise concrete, asphalt or segmental concrete block paving at the surface.
- Heavy duty port pavements as above for interchange zones such as truck grids, rail sidings etc.
- Heavy duty road type pavements for truck areas comprising thick deep strength asphalt or full depth asphalt or alternatively, rigid concrete pavements.
- Light duty road type pavements for (non port truck) access to, and parking for, areas such as administration, operations and maintenance buildings.

Heavy duty port pavements would be expected to accommodate plant such as reach stackers, straddle carriers, forklifts, terminal tractors, trucks, rubber tyre gantries (RTGs) and static loads such as container stacks and possibly dry bulk.

If an Auto Stacking Crane (ASC) system were to be adopted, it would be expected that the crane rails would be supported by structural elements, although it is possible that ballast systems could also be used. For ASC operations, heavy duty pavements would be required for the end interchanges, whilst the midblock container stacking areas should be designed based on geotechnical requirements, taking into account operational and maintenance considerations.

Traffic loadings on each pavement area shall be assessed when the operations and plant types are known.

The method of pavement design will depend on the selected pavement type and loading conditions.

#### 9.8.2 Pavement Subgrade Conditions

Pavements will be required to be supported on one or more of the following:

- On reclaimed land, including
  - Near shore inter-tidal area
  - Offshore area on reclaimed material
- On land/terrestrial areas

The design and locations of reclaimed areas is yet to be determined, and the material types used for the reclamation could vary. If materials for the reclamation are sourced from nearby dredging works, the materials may comprise combinations of sand, silt and clay, although the nature of the dredged material and suitability for use may depend on the method of dredging used and the location/depth of dredging.

It is likely that ground improvement will be required to support heavy duty pavements on reclaimed areas and also in parts of the terrestrial area where lower strength (say firm or lower consistency) subgrades are present. Ground improvement may be required for improving subgrade stiffness and/or to reduce post construction settlement.

Limited geotechnical site investigations have been undertaken in the terrestrial project area, and the subgrade considerations below are based on the available information from the Golder Associates report dated 19 June 2014. The upper soils within the terrestrial area that are likely to be required to form the subgrade for pavements are generally expected to comprise soils of the Tertiary age Baxter Formation Unit. A variable thickness of materials may overlie the Baxter Formation at some locations, including:

- Fill material;
- Fluvial deposits;
- Alluvial deposits;
- Salt marsh deposits;

The stiffness of these overlying materials varies but in some cases may be very low (such as for the salt marsh deposits) or variable (for uncontrolled fil materials).

The Baxter Formation materials are likely to predominantly comprise stiff or better clay/silt materials or medium dense or better sand materials. There may be some locations where the consistency of the cohesive Baxter Formation soils is less than stiff, or where the density of sandy soils is less than medium dense. The preliminary investigation data indicates firm clay materials of the Baxter Formation at some locations, particularly where the existing surface is below about +5 mCD.

The Baxter Formation comprises clay materials with Plasticity Index in the order of 20% to 50%. This indicates that some of the clay materials may be expansive in nature and thus be sensitive to volume change via shrinking or swelling when subject to changes in moisture content. No laboratory soaked California Bearing Ratio (CBR) and associated swell testing, nor Shrink-Swell index testing has been undertaken at the site. It is expected that further investigations and testing will be undertaken to confirm assumed design subgrade CBR values.

The following design CBR considerations may apply to pavements on terrestrial areas:

- Materials overlying the Baxter Formation:

  - It is assumed that uncontrolled fill materials, and any soft/weak soils not meeting the specified CBR requirements will be improved or removed;
  - Adopt CBR 3% and undertake ground improvement if required to achieve CBR 3% to the required depth.
- Sand of the Baxter Formation (subject to thickness of layer) - adopt CBR 5%;
- Clay and silt of the Baxter Formation – adopt CBR range of 3% to 5% depending on the conditions encountered. Ground treatment/improvement may be required to achieve a minimum CBR of 3%, particularly where clay or silt of lower than stiff consistency is encountered.
- Clay and silt of the Baxter Formation should be assumed to be expansive for the purposes of pavement design unless localised testing of CBR swell or suitable other testing can demonstrate otherwise. For some pavement types, this may require the inclusion of specific earthworks or pavement layers, such as low permeability capping layers.
- Where imported fills are required to be placed above the existing soils to achieve design levels, the design CBR value for such materials should be based on the CBR of imported material taking into account the CBR of the underlying in situ materials.
- It is possible that existing site soils, including the Baxter Formation and overlying near surface units may be suitable for excavation and use as fill beneath pavements. The design CBR for such materials will be as per the CBR for the in-situ materials, as outlined above for sand, clay and silt of the Baxter Formation.

The following design CBR considerations may apply to pavements on reclaimed land:

- Where reclaimed materials comprise dredged Baxter Formation materials, the design CBR for the reclamation will depend on the method of dredging, treatment and placement of the materials, and will also depend on ground improvement undertaken. A CBR value in the range of 2% to 4% would likely be appropriate for dredged materials, on the basis that materials with a CBR of less than 2% are unlikely to be suitable without further ground improvement to improve stiffness. Depending on the nature and extent of ground improvement undertaken this may enable the use of design CBR values higher than 4%.
- It is assumed that ground improvement may be required to be undertaken to limit post construction settlement to desirable limits. This should take into account not only long term requirements for pavement surface levels and shape but also considerations such as interfaces with fixed (piled) structures, maintenance expectations and drainage system functionality and design.
- Where reclamations are constructed of imported materials then the design CBR shall be based on that assessed to be suitable for the imported materials.
- Where at all possible, for the purposes of assisting in providing a stiff and stable subgrade for pavement construction, the highest quality/stiffness materials should be placed towards the top of the reclamation.

For some of the types of pavements expected to be required at the site, the design of the subgrade may require multiple layers of different CBR to be considered. Different loading conditions and design methodologies will require consideration of different base depths to which subgrade stiffness should be considered.

Some design methods do not use CBR as an input, rather they use elastic modulus. Appropriate conversions between CBR and modulus should be made taking into account the duration of loading and the impact on the stiffness response of the subgrade.

Based on the available information, groundwater is not expected to be a significant concern for pavement design and construction at the site.

## 9.9 Durability

The following information is based on AS2159 – *Piling Design and Installation* and will be reviewed as part of the overall durability and design life requirements of the project. Soil aggressivity is assessed for the potential to adversely impact on concrete and steel components that are in contact with the ground.

The classification for concrete piles fully submerged in seawater is moderate and for elements in the intertidal or splash zone the classification is severe. For steel piles in seawater the exposure classification is also severe.

For on-shore buried concrete members the exposure classification based on chemical compositions of the soil (AS2159, table 6.4.2 (C)) should be considered.

Selected soil samples were tested for aggressivity (pH, chloride, sulphate and electrical conductivity) to aid in the exposure classification for the durability of concrete and steel foundation systems. The results of this testing are summarised below.

- Chloride analysis (in groundwater) returned values of 10 mg/kg to 3,420 mg/kg, which is not considered significant with regards to concrete and steel durability (non-aggressive).
- Sulphate soil analysis returned values of 10 mg/kg to 1,120 mg/kg, which is not considered significant with regards to concrete durability (non-aggressive).
- Electrical conductivity results indicated between 3 and 625 µS/cm in a range of soil types.
- Testing revealed alkaline to acidic soil conditions (pH = 4.9 to 8.7).

Based on these test results, an exposure classification of **Mild** is deemed appropriate with respect to concrete and steel durability (AS 2159).

For buried concrete and steel members, based on the laboratory test results, an exposure classification of **B2** should be adopted in accordance with AS 2758.1 (Table A2).

It is noted that the samples tested are representative of a wide range of depth intervals and the summarised chemical anolyte information varies considerable across the site. Specific testing is recommended for individual structures and the information presented here should be considered indicative only.

The above recommendations do not include special considerations for localised ground conditions containing acid sulphate soils. Such conditions possibly existing within the intertidal zone and within localised drainage lines containing Quaternary alluvium.

Durability requirements for wharf structures including steel piles, concrete piles and reinforced concrete elements will be addressed as part of other work packages.

## 9.10 Characteristic Engineering Parameters

Characteristic engineering parameters for each of the main stratigraphic units are given in Table 9-2. These parameters have been derived from the results of in situ and laboratory testing, CPT results and engineering experience with similar materials/geological units. Given the vertical and lateral variation within each of the main stratigraphic units, characteristic parameters are given for the range of soil types and consistency expected to be present within those units. As the port layout is developed, design stratigraphic profiles can be produced and the range of engineering parameters refined. It is expected that this will be completed as part of future work packages. Examples of how these parameters can be applied to the boreholes to develop a design profile are given in Appendix M.

These parameters are appropriate for use in the concept design of foundations, assessment of slope stability, retaining wall design and reclamation design. Sensitivity assessment would need to be undertaken as part of design assessment and analysis to assess impact of parameter variability.

| Unit | Soil Type | Consistency | Bulk unit weight | Undrained shear strength | Effective stress parameters | | Consolidation Parameters | | | | Permeability | Poisson's Ratio | | Young's Modulus/Rock Mass Modulus | | Ultimate End Bearing | Ultimate Shaft Adhesion | Spring Constant |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | γb | $s_u$ | c' | ϕ' | Initial voids ratio | Compression Index | Recompression Index | Co-efficient of consolidation | - | $\nu'$ | $\nu_u$ | E' | Eu | fb | fs | k |
| | | | (kN/m³) | (kPa) | (kPa) | (°) | (e0) | (Cc) | (Cr) | (Cv) ($m^2$/yr) | (m/day) | - | - | (MPa) | (MPa) | (kPa) | (kPa) | kPa/m |
| QUATERNARY (Q3-Q4) | Clays and Silts | Very Soft | 14 | 2.0+1.5z | 0 | 17 | 2.5 | 0.9 | 0.14 | 0.5 | 1.0E-05 | 0.3 | 0.49 | 1 | 1.4 | - | - | - |
| | | Soft | 16 | 12-17.5 | 0 | 20 | 1.5 | 0.6 | 0.09 | 0.5 | 1.0E-05 | 0.3 | 0.49 | 2 | 3 | - | - | 4,000 |
| | | Firm | 17 | 25-37.5 | 2 | 24 | 1.2 | 0.35 | 0.05 | 0.5 | 1.0E-05 | 0.3 | 0.49 | 5 | 7 | - | - | 8,000 |
| | Sands | Very Loose | 16 | - | 0 | 25 | - | - | - | - | 1.0E-02 | 0.3 | - | 5 | - | - | - | 5,000 |
| | | Loose | 18 | - | 0 | 27 | - | - | - | - | 1.0E-02 | 0.3 | - | 10 | - | - | - | 10,000 |
| | | Medium Dense | 18 | - | 0 | 30 | - | - | - | - | 1.0E-02 | 0.3 | - | 35 | - | - | - | 35,000 |
| BAXTER FORMATION (Tb) SHERWOOD FORMATION (Tm ) | Clays and Silts | Soft | 16 | 12-17.5 | 0 | 20 | 1.5 | 0.6 | 0.09 | 0.5 | 1.0E-05 | 0.3 | 0.49 | 2 | 3 | - | - | 4000 |
| | | Firm | 18 | 25-37.5 | 2 | 24 | 1.2 | 0.35 | 0.05 | 10 | 1.0E-04 | 0.3 | 0.49 | 5 | 7.0 | - | 30 | 8,000 |
| | | Stiff | 19 | 50-75 | 5 | 26 | 1.0 | 0.35 | 0.05 | 10 | 1.0E-04 | 0.3 | 0.49 | 15 | 21 | 560 | 50 | 16,000 |
| | | Very Stiff | 19 | 100-150 | 7.5 | 28 | 0.8 | 0.35 | 0.05 | 10 | 1.0E-04 | 0.3 | 0.49 | 40 | 57 | 900 | 70 | 30,000 |
| | | Hard | 20 | 200-300 | 10 | 28 | 0.7 | 0.35 | 0.05 | 10 | 1.0E-04 | 0.3 | 0.49 | 80 | 114 | 2250 | 125 | 60,000 |
| | Sands | Very Loose | 16 | - | 0 | 25 | - | - | - | - | 1.0E-02 | 0.3 | - | 5 | - | - | - | 5,000 |
| | | Loose | 17 | - | 0 | 27 | - | - | - | - | 1.0E-02 | 0.3 | - | 10 | - | - | - | 10,000 |
| | | Medium Dense | 18 | - | 0 | 30 | - | - | - | - | 1.0E-02 | 0.3 | - | 35 | - | 20*σ' (4,800) | 0.37*σ' (80) | 35,000 |
| | | Dense | 19 | - | 0 | 35 | - | - | - | - | 1.0E-02 | 0.3 | - | 60 | - | 40*σ' (9,600) | 0.46*σ' (95) | 60,000 |
| | | Very Dense | 20 | - | 0 | 38 | - | - | - | - | 1.0E-02 | 0.3 | - | 100 | - | 50*σ' (12,000) | 0.56*σ' (115) | 100,000 |
| SILURIAN | XW | Extremely weathered | 20 | - | 30 | 32 | - | - | - | - | - | 0.3 | - | 100 | - | 2500 | 250 | - |
| | HW | Highly weathered | 22 | - | 70 | 35 | - | - | - | - | - | 0.25 | - | 500 | - | 7500 | 450 | - |
| | MW | Moderately Weathered | 24 | - | 200 | 38 | - | - | - | - | - | 0.2 | - | 2000 | - | 12,500 | 700 | - |
| | SW | Slightly Weathered | 24 | - | 1000 | 42 | - | - | - | - | - | 0.2 | - | 8000 | - | 15,000 | 900 | - |

Table 9-2 Characteristic Engineering Design Parameters

**Notes:**

1) The above parameters are for concept design purposes and are generally applicable for the entire site. Changes to individual parameters for specific locations may be required.

2) Parameters for pile design:

   (a) Skin friction and end bearing is ignored in very loose to loose sands and very soft to soft silts and clays and all Quaternary deposits

   (b) Geotechnical reduction factors need to be applied to ultimate geotechnical pile capacities in accordance with AS2159

   (c) Pile driveability studies and a pile test programme would be required to confirm piling parameters

(d) Skin friction and end bearing values are for driven piles. Piling parameters may differ depending on pile type and method of installation.

(e) For driven pipe piles in cohesionless soils unit skin friction and end bearing are based on the recommendations given in API-RP-2A where:

   (i) Unit skin friction fs = K* σ'* tan δ. K= 0.8 has been adopted

   (ii) Open-ended driven pipe piles are assumed to be plugged however large diameter pipe piles driven in dense to very dense sand may not plug.

   (iii) Unit end bearing fb = σ' *Nq where Nq is a dimensionless bearing capacity factor and σ' = effective overburden pressure

   (iv) Limiting unit skin friction and end bearing values are given in brackets

3) Sensitivity assessment should be undertaken to assess the impact of parameter variability for the permeability of sand materials due to the variable fines content of these materials.

4) The spring constant values are provided for pile footing design. This parameter is not an intrinsic property of a material and is a function of the loaded area and modulus. The tabulated values are approximate and should be used with caution.

5) The Young's modulus parameter is not an intrinsic property of a material and is function of soil strength and applied level of stress on the soil. The tabulated values are approximate and should be used with caution.

6) Laboratory test data for Baxter Formation has given a wide range of consolidation parameters. Sensitivity analysis to assess the impact of parameter variability should be undertaken for compression and recompression index values considering the range of values given in Section 8.2.

7) Coefficient of consolidation values (Cv) can vary considerably depending on mass permeability and soil fabric.

8) Analysis for soil structure interaction and assessment of pile capacities should include sensitivity analysis to assess impact of parameter variability.

9) Additional design recommendations are given in the following report sections:

- Section 9.7 - Foundation recommendations for soil reactivity, spread footings and piled footings for ancillary structures and facilities in the terrestrial project area
- Section 9.8 - Recommendations for pavement design on terrestrial areas and on reclaimed land
- Section 9.9 - Durability requirements

The design recommendations given in the above sections should be read in conjunction with the discussion also included in these sections.

## 9.11 Further Investigation

The 2013/2014 investigations have provided valuable information to inform the business case however given the relatively widely spaced grid pattern (500 m centres) it is expected that further investigations would need to be undertaken during future design stages. These investigations may include:

- additional terrestrial and marine investigation borehole drilling and CPTs as the terminal footprint and location is developed
- additional sediment sampling and laboratory characterisation of soils to inform detailed dredging studies
- further geophysical survey including seismic refraction to assist in determining engineering properties of subsurface soils and rocks

## 9.12 Instrumentation and Monitoring

**General**

As part of the proposed port development instrumentation and monitoring activities would be required to assist in design development, to monitor performance during construction and to provide long term monitoring data. Such activities may have very long lead in times and require monitoring for a number of years. Instrumentation and monitoring plans would need to be developed as part of port planning activities so that the information is obtained to inform detailed design and construction activities.

These activities may include:

- Dredging trials
- Trial embankments to inform the design of surcharge options
- Pile testing and monitoring
- Quality control testing to verify reclamation construction e.g. CPTs, density tests, and compaction tests
- Settlement monitoring during reclamation filling and/ or as part of ground improvement design and implementation
- Measurement of porewater pressures of soils within and beneath reclamation fills
- Installation of inclinometers to monitor lateral movement and confirm stability of edge structures and batters
- Settlement monitoring and assessment of lateral movement of specific structures or loaded areas
- Settlement monitoring of pavements or other areas to assess maintenance requirements or intervention triggers if maintenance and repair regimes are adopted

**Old Tyabb Reclamation**

The Old Tyabb reclamation is one area where construction of a trial embankment would assist in ground improvement planning. The existing Old Tyabb reclamation area, filled with materials derived from cutter suction dredging, contains deposits of weak compressible alluvium trapped below a layer of poorly compacted reclamation filling. Similar deposits of soft alluvium are known to be present at sea bed and in the coastal land environment which will need to be dealt with as part of the Port of Hasting redevelopment.

If left in place without treatment the existing compressible materials pose a significant geotechnical risk in terms of settlement and poor support for reclamation fill and port structures.

Settlements in the order of hundreds of millimetres are possible if this layer is left in place, with further long term settlement 'creep' occurring over many years. Such problems are akin to long term ground movement issues experienced in the vicinity of the Port of Melbourne.

As discussed in previous sections of this report, ground improvement to minimise adverse settlement impacts can be accomplished through surcharging with or without vertical drains, as a passive long term method, or through more intensive methods such as stone columns, deep cement mixing etc.

Settlement mitigation using surcharge methods requires knowledge of the compressibility of the soil for settlement predictions, and requires monitoring. An instrumented trial embankment in the Old Tyabb reclamation area to simulate the engineering loads will allow better prediction of the rate and magnitude of settlement. Construction of a trial embankment would be beneficial to enable designers to assess whether surcharging is an effective option and what this will mean to project delivery time tables (example – a surcharge option may be significantly cheaper than an 'intensive' option such as stone columns but be too restrictive on the development timetable and hold up follow on activities).

Geotechnical instrumentation incorporated into the trial embankment design should include as a minimum:

- Settlement pins;
- Settlement plates;
- Extensometers; and
- Vibrating wire piezometers.

Typical dimensions of the trial embankment are likely to will be in the order of 15 m x 30 x 5m (e.g. Approximately. 2500 $m^3$ of earthworks). In order to properly assess the ground response to surcharge loading for various circumstances, the embankment should include a separate area containing vertical drains. This will allow a better understanding of the consolidation rates where drains are used to accelerate the settlement response.

The construction and measurement of the embankment settlement is a long lead item as monitoring needs to occur over a significant time span (months to years) to understand the soil response to loading (i.e. short and long term magnitudes and settlement rate trends).

It is recommended that trial embankment construction be considered early in the project design program to allow appropriate decisions to be made regarding ground settlement magnitudes and selection of appropriate ground improvement methods. For planning purposes, it would be expected that monitoring of a trial embankment would be undertaken for a 1 to 2 year period.

# 10.0 3D Geological Model Development

A 3D Geological Model of the project area has been developed using the available investigation data, and interpreted stratigraphy. The model has been developed using implicit geological modelling software. The model includes borehole, test pit and geophysical information. In addition to the model surfaces and volumes, a digital database of all investigation data has been collated. The 3D Geological Model is presented and reported separately from this report.

The purpose of the model is to aid visualisation and interrogation of ground conditions within the project area and to be used in conjunction with this report to provide guidance for the design going forward.

# 11.0 Summary and Conclusions

## 11.1 General

The Victorian Government has identified the Port of Hastings as the suitable site for additional capacity for international container trade as the Port of Melbourne reaches capacity, expected by early to mid-2020s.

Prior to 2013, only limited existing site investigation information was available in the Port of Hastings area. Therefore, in 2013 PoHDA commissioned terrestrial and marine geotechnical investigations, and a marine geophysical survey, to obtain subsurface information to inform planning and business case studies for the proposed Port of Hastings expansion.

In the preparation of this Geotechnical Interpretative Report, previous and recently completed geotechnical and geophysical investigation works has been reviewed and summarised to provide a preliminary assessment of ground conditions within the development area/project site.

At the time of writing this report the nature and layout of the port expansion, and the footprint of proposed wharf and terminal areas are still under consideration, and is to be subject of other work package studies which would assess required operational layout, dredge and reclamation methodology, dredge materials management, ground improvement requirements, and wharf and terminal studies. Discussions presented in this report in respect to geotechnical considerations for the Project are therefore general in nature, and will require further review and assessment as the project definition is refined.

## 11.2 Geology and Geomorphology

Published geological information indicates that the stratigraphy of the western side of Western Port (which encompasses the Project site) consists of:

- Silurian marine sedimentary basement rock underlying the area
- Older Volcanics weathered basalt present to the south of French Island and Stoney Point in the Western Channel and Anchorage
- Yallock Formation encountered at depth overlying the Older Volcanics and the Silurian rocks
- Sherwood Formation also overlying the Older Volcanics or the Silurian rock, and the Yallock Formation.
- Baxter Formation which disconfomably overlies the Sherwood Formation and elsewhere disconformably overlies the Older Volcanics and unconformably overlies the Silurian rock.
- Quaternary deposits including onshore dune sands, alluvium, and swamp and lagoon sediments, and offshore soft and loose marine deposits overlying the Baxter Formation
- Anthropogenic deposits comprising terrestrial fill, and land reclamation fill at the BlueScope Steel wharf and the Old Tyabb reclamation.

The predominant geological structural features in the Hastings area are northeast trending folds and faults, the most significant of which is the active Selwyn Fault that runs along the eastern edge of Port Philip. Despite the presence of nearby faults, the occurrence of large seismic events in the area is considered to be highly unlikely, with a relatively low seismic hazard determined to be equivalent to an acceleration of around 0.1 g.

The bathymetry of Western Port reflects the dominance of tidal movement and wave and current dispersion around Phillip Island and French Island. Sediment in the bay is dominated by sand in the entrances and broader channels, with high concentrations of mud in embayments and at the head of the tidal channels. There is often considerable local variation in sediment type, and stratigraphic (vertical) variation is common.

Coastlines around the bay include active rock and earth cliffs, shore platforms, inactive cliffs and bluffs, sandy beaches and mangrove and salt marsh fringed shores.

European settlement impacted on much of the shoreline of Western Port, including removal of mangrove and salt marsh, alteration of foreshore and near shore topography by construction of jetties and mooring sites, the covering of tidal flats and dredging for shipping access, and by draining former swampland and replacing

natural channels with excavated drains. The developments at the ESSO Long Island Point and BlueScope Steel wharves have further altered the shoreline.

## 11.3 Site Description

The Port of Hastings is an operational commercial port. The shipping channel to the port consists of a two-way channel from the west of Phillip Island to Sandy Point, and a one way channel from north of Sandy Point to the existing berths, and includes approaches, swing basin, anchorage and berth pocket.

Previous dredging works were completed in the 1960s and 1970s in the Western Port area. The material dredged is reported to be variable silt, sandy clay, clay and isolated boulders of varying sizes. Dredging equipment included cutter suction dredgers, bucket dredgers and grab-dredgers. Drilling and blasting of rock is also reported to have been undertaken at Stony Point.

Study of aerial photographs indicates a progressive change of land use in the Hastings area with intense development commencing from the late 1960s, including modification to the shore profile, construction of sea defences, construction of port facilities for ESSO and BlueScope Steel, and land reclamation.

Outside of the Long Island Point port environs and heavy industry areas, land use within the Terrestrial Project Area SUZ1 is predominantly pastoral, with local interspersed wooded areas.

## 11.4 Site Investigations

**General**

Previous site investigations in the Hastings area include investigations in and around the ESSO Long Island Point (LIP) wharf, BlueScope Steel wharf, Crib Point and Stony Point wharfs, and along the existing navigation channel within VRCA waters.

In 2013/ 2014 marine geophysical survey, and marine and terrestrial geotechnical investigations have been undertaken to supplement existing information and to inform the current phase of design development.

**Geophysics**

The marine geophysical survey was undertaken concurrently with the marine geotechnical investigation, and comprised bathymetric survey, side scan sonar, sub-bottom profiling, and multi-channel seismic reflection.

Subsurface materials were identified based on the specific seismic “signature” and amplitude correlated to borehole data.

Interpretation of the geophysics has provided the following:

- The interface between soil and rock has been identified with reasonable clarity due to sufficient density contrast.
- The subtle changes in stratigraphy within soil deposits were not clearly identifiable based on correlation of geophysics results and the boreholes and CPTs, particularly in the Port Area.
- Indications are that the rock levels in the Port Area are significantly deeper than the potential dredge level.
- The depth to rock in the North Arm Channel is interpreted to be generally deeper than the proposed dredge level (-20 m LAT).
- In the Western Channel, the shallowest rock can be expected at the north west corner of the Western Channel with the rock levels shallower than -18 m LAT.
- The rock levels in Western Channel central area is shallowest in the central area western zone with rock indicated above -20 m LAT.
- A rock reflector has been mapped across the Anchorage with depths ranging from -12m LAT in the central northern area to depth greater than -20 m LAT towards the south west and south east ends.

**Marine and Terrestrial Geotechnical Investigations**

- Limited terrestrial investigations comprised the drilling of 18 No. boreholes to depths of between 15 and 50 m (including 11 No. boreholes with groundwater monitoring wells), excavation of 7 No. shallow test pit, Cone Penetration Tests (CPTs) at 8 No. locations, together with associated insitu and laboratory testing. The terrestrial investigations were confined to the area owned by BlueScope Steel.
- The marine geotechnical investigation included drilling and associated insitu and laboratory testing at 94 No. locations across the Port Area (75 No.), North Arm (10 No.), Western Channel (2 No.), Anchorage (3 No.) the North Port area (4 No.) A total 21 No. CPTs were also undertaken coincident with borehole locations in the Port Area.
- The Port Area marine boreholes were typically drilled on a wide grid pattern on 500 m centres to around -23 m CD in areas of potential dredging, with deeper holes to 50 m depth at selected locations at around 1000 m spacing in areas where potential wharf or other structures may be considered.

Laboratory testing has been undertaken to characterise the physical, chemical and engineering properties of the underlying soil and rock. An opportunistic environmental sampling and testing program has also been undertaken in the marine area, the results of which are reported under separate cover.

The site geology was found to be generally consistent with the published geological data. Characteristic subsoil properties have been assessed for each geological/stratigraphic unit encountered within the project area based on field and laboratory test results. Tabulated and graphical summaries of test results have been presented.

In summary investigations have encountered:

*Old Tyabb Reclamation* fill comprises variable deposits of sand and clay, and in places was observed to include cobble sized lumps of clay in a sand/sandy clay matrix interpreted as being remnant “balling” of material from cutter suction dredge operations.

*BlueScope Steel Reclamation* fill on the approach to the BlueScope Steel Wharf comprises firm to stiff sandy clays and medium dense clayey sands.

*Dune Sands (Q1)* comprising local thin deposits of loose to dense silty sands and sands locally encountered in the terrestrial SUZ1 area.

*Alluvium (Q2)* was inferred to be locally present in terrestrial areas. This material was disturbed during cultural heritage investigations which preceded the terrestrial investigation and as a result of this disturbance the consistency could not be determined.

*Quaternary marine deposits (Q3)* are present at sea bed comprising thin layers (generally < 1 to 4 m thick) of loose sand and very soft to firm clays and silts of low to high plasticity. Some samples had very high moisture content typical of normally consolidated recent deposits. Undrained shear strength of cohesive Quaternary deposits were variable with normally consolidated clays present typically exhibiting strengths of < 5 kPa, for which sampling often proved unsuccessful due to the very soft consistency.

*Undifferentiated Quaternary (Q4)* interpreted to be derived from reworking of underlying Tertiary deposits and was locally present in offshore areas which comprise similar materials to the underlying Baxter Formation but also contained shell fragments and were of lower strength.

*Baxter Formation (Tb)* is extensively present at shallow depth across the terrestrial and marine areas of the proposed Port Area. A large proportion of material to be dredged would comprise Baxter Formation soils. Material descriptions and particle size distribution curves indicate variable interbedded sands, silts and clays. The clays range from low to high plasticity. Of note with regard to dredge and reclamation characteristics is that sands, where present, are typically fine grained and commonly contain 10 to 50% clay and silt content. Materials classified as sands in accordance with AS1726 but with high silt and clay content are likely to behave in a similar manner to cohesive soils. The Baxter Formation is over consolidated and relatively competent with clays typically being of stiff and very stiff, or medium dense to dense consistency, although some weaker horizons with organic content are present. Cemented layers (with SPT refusal) are present within the unit, with occasional layers of extremely low to high strength rock reported. By inspection of borehole logs in the Port

Area (at 500 m centres), no apparent lateral continuity of material types between adjacent boreholes could be identified.

*Sherwood Formation (Tm)* occurring at depth across the terrestrial and marine areas generally comprises low to intermediate plasticity silts and high plasticity clays with many samples plotting below the A line when compared to the overlying Baxter Formation. Dredging to depths of -18 to -20 mCD would encounter this material. Soil descriptions and particle size distribution curves indicate variable interbedded sands, silts and clays. The Sherwood Formation is over consolidated and relatively competent typically being of stiff to hard, or medium dense to very dense consistency. Cemented layers (with SPT refusal) are common within the unit, with point load strengths of up to 2 MPa recorded. Differentiation of the Sherwood Formation from the Baxter Formation is difficult in some cases.

*Older Volcanics Group (Tvo)* A limited number of boreholes (5 No.) have been drilled in areas where Older Volcanics basalt are known to occur, in the Western Channel and the Anchorage. 8 No. scheduled boreholes in the Western Channel were not drilled as part of the current drilling campaign due to operational difficulties with the jack up barge. The basalt encountered proved to be variable in respect to weathering and strength. Laboratory testing undertaken on selected samples indicated point load strengths varying from 0.04 to 0.57 MPa (4 No. tests), and unconfined compressive strength of between 0.3 and 19.7 MPa. It is noted that basalt is typically highly variable and rock strength higher than encountered by the limited number of boreholes is likely to be present. Although dredging in the Western Channel and Anchorage will be limited i.e. comprising local removal of “high spots” only, the geophysical survey does indicate that basalt rock is present within potential dredge depths.

*Silurian Sandstone and Siltstones (S)* is present as a variably weathered basement rock and was encountered in the Port Area south of the BlueScope Steel wharf at depths ranging from 22 m to 47 m below sea bed (i.e. well below the depth of possible dredging). Laboratory testing undertaken on selected samples indicated point load strengths varying from 0.07 to 8.7 MPa (27 No. tests), and unconfined compressive strength of between 11.6 and 62.7 MPa.

## 11.5 Geotechnical Considerations for Port Expansion

**General**

At the time of writing this report, the scope and layout of the port infrastructure and associated facilities have not been finalised. Various options in respect to location and layout are being considered.

Discussions on geotechnical considerations for future port development, with regard to dredging, reclamation, ground improvements, and wharf and ancillary structures, are therefore by necessity preliminary in nature

**Dredging**

All port development options will require dredging to accommodate the range of design vessels being considered. The majority of dredging will occur in the Port Area which will predominately involve dredging of surficial low strength Quaternary marine deposits (typically less than 1.5 m thick) and Baxter and Sherwood Formation clayey sand, sandy clay, silty sand and silty clay. The Baxter and Sherwood Formations exhibit vertical and lateral variability and depending on the dredging methods employed may be difficult to selectively dredge. Frequent low strength cemented horizons are present in the Baxter Formation which may influence selection of dredging equipment.

Local dredging and removal of high spots in the Western Channel and Anchorage (where required) will include dredging of surficial Quaternary deposits, and more competent stiff clays and medium dense to very dense silty sands with gravels, and extremely weathered to moderately weathered basalt, ranging from extremely low to high strength.

The three main dredging techniques commonly used within the industry are trailing suction hopper dredging (TSHD), cutter suction dredging (CSD), and mechanical dredging (grab or backhoe). In the Western Channel and Anchorage where weathered basalt rock may be present, local drill and blast techniques may be required to loosen materials prior to dredging.

Assessment of appropriate dredging and reclamation operation is outside the scope of this report and is to be covered by other work package studies. However, it is expected that a combination of dredging techniques would likely be used for the Port of Hastings expansion, including TSHD of Quaternary materials, and CSD and potentially backhoe dredging of the Baxter and Sherwood Formations. CSD of cohesive materials and pumping into reclamation will result in the formation of clay balls. Low strength clay and silt slurry generated during the dredging process would need to be separated from the reclamation fill to prevent entrapment. CSD dredging of sandier soils would result in fluidised material pumped to the reclamation, where sand fractions would settle. Silt and clay slurry from such operations would also need to be segregated from the reclamation fill. Ground improvement and densification of reclamation fills derived from the Baxter and Sherwood Formations materials would be required.

**Ground Improvement**

The requirement for and method of ground improvement would be dependent on methods of dredging, materials to be used as reclamation fill, time available to carry out the ground improvement works, loading and settlement tolerance of facilities, acceptability of maintenance of facilities, and construction staging requirements.

Assessment of ground improvement requirements and methodology is outside the scope of the current report, and is to be undertaken as part of other work packages.

It is anticipated that ground improvement will be required to:

1) improve the consistency/ density of dredged materials within the reclamation, in particular where materials have been placed under water
2) to improve in situ soft or loose Quaternary soils if left in place beneath reclamation fill, and
3) to improve the existing Old Tyabb reclamation.

The in situ Baxter and Sherwood Formation soils beneath the port area are generally relatively competent and the need for extensive ground improvement is unlikely, although lower strength horizons are present and if they occur at the location of perimeter bunds, or beneath critical or settlement sensitive structures, some form of ground improvement would need to be considered.

Numerous techniques can be used to improve the properties of the ground. Commonly used ground improvement techniques include vibroflotation, vibroreplacement (forming sand or stone columns), dynamic compaction, pre-load surcharge with or without vertical drains, vacuum consolidation, deep soil mixing, controlled modulus columns, and high energy impact rolling.

**Wharf Structures**

Determination of suitable wharf structure options, including soil structure interaction analysis, and assessment of slope stability and pile capacities are outside the scope of this report and are to be undertaken as part of separate work package studies. Key geotechnical considerations associated with possible structure options include:-

*Piled Quay Deck*

A piled quay deck would comprise a reinforced concrete deck supported on vertical and raker piles, with revetted embankment under the wharf. Driven tubular steel piles are commonly used for suspended deck structures, but other options such as driven reinforced concrete piles could be used. Piles would be driven to required depths, a specified driving resistance or to refusal on rock, in order to achieve required geotechnical design capacity in accordance with AS2159 ”Pile design and Installation”. Pile drivability studies and a pile testing programme would typically be undertaken to inform the pile design, and to confirm pile capacities during construction. Piles would need to be founded a suitable distance below dredge level to achieve fixity and so as not to compromise slope stability. Cemented materials are present in the Baxter and Sherwood Formations which could hinder pile installation, and may require heavy driving, or potentially the need for pile advancement by pre-boring or the mucking out of open tube piles during the driving process.

Stable revetment slope angles under the deck would depend on the nature and consistency of the soils within and beneath the revetment. Excavation and replacement of soft and loose soils that may influence stability may be required, if present.

*Concrete Caissons*

Concrete caissons are gravity structures comprising open-topped precast concrete cells constructed in the dry and floated and sunk into position on to pre-prepared rock fill/granular base. Keyed joints are used to accommodate placement tolerances and differential settlement between units. Units are filled with sand or granular fill. Caissons require competent foundations to support high bearing pressures which may require excavation of a trench along the quay line within the Baxter or Sherwood Formations to place a rockfill base. Rear crane rails would require independent support on piles. An alternative pile type for installation using land based equipment may include reinforced bored cast in situ, driven precast concrete piles or continuous flight auger (CFA) piles.

*Counterfort Wall*

Counterfort walls are another form of gravity structure and consist of precast concrete wall segments with base slab and cantilever wall strengthened with counterforts.  Similar to the caisson option, this type of structure is usually constructed in the dry and delivered to site using barges and sunk onto a pre-prepared granular base. Similar to the caisson option this type of structure would likely require excavation of a trench along the quay line backfilled with rockfill to provide a sound base.

*Bulkhead Wall*

A bulkhead wall would typically be constructed using either large diameter tubular steel piles driven to sufficient depth for toe restraint and to provide adequate vertical capacity for crane rail support (with alternate piles potentially driven to shallower depths), or a combi-wall system comprising a combination of tubular steel piles driven to sufficient depth for toe restraint with infill sheetpile sections. Concrete diaphragm walls are another alternative bulkhead wall option. The bulkhead wall would be laterally restrained by tie rods and a sheet pile anchor wall or anchor piles. An independent row of piles would be required for the rear crane rail. Imported granular fill material would typically be used to fill immediately behind the bulkhead wall. Bulkhead walls can be driven from floating plant with subsequent filling behind the wall after installation, or could be installed on reclaimed land. Importantly tie-rods supporting the wall would need careful detailing including flexible joints and placement of tie rods within oversize tubes to isolate them from ground movement caused by settlement or compression due to ground improvement of fill materials placed behind the bulkhead wall.

**Ancillary Structures and Facilities**

Areas of land reclamation formed as part of the port expansion may be used for heavy duty port pavement and container handling/ stacking, interchange zones such as truck grids and rail sidings, and ancillary buildings and infrastructure including buried services and drainage. The exact nature of the infrastructure, loadings, and the reclamation development footprints are yet to be determined. The nature and consistency of reclamation fill prior to and/ or following implementation of ground improvement is also not known. The following would need to be taken into consideration as part of assessment studies

Reclamation fills derived from the Baxter and Sherwood Formations would require ground improvement and/ or the use of deep foundations to support equipment and facilities.

Following ground improvement some level of long term creep settlement and differential settlement will still occur. Facilities would need to take anticipated post construction settlement into account.

Reclamation timing and port development staging would need to consider long lead in times for ground improvement, which could take a number of years to implement.

Allowance would need to be made for differential settlement between piled and non-piled structures, including provision of approach slabs, allowance for maintenance re-levelling, and provision of flexibility for buried services.

Surface water drainage and surface gradients would need to be designed to take into account both operational tolerances of equipment, and to accommodate anticipated settlement.

Piles would need to account for negative skin friction resulting from post construction settlement. Installation difficulties associated with piling through cemented horizons would need to be considered.

Ground improvement for container stacking and handling areas would need to take into account construction and operational tolerances so as to maintain to gradient and alignment. Positive support such as piles or controlled modulus columns may need to be considered for settlement sensitive equipment and operations.

A possible design approach could be to accept settlement, and allow for maintenance and site re-levelling for operational tolerances.

**Pavements**

Depending on the proposed operations it is expected that new pavements will comprise combinations of heavy duty port pavements for container handling and stacking (concrete, asphalt or segmental block paving), heavy duty port pavements for interchange zones such as truck grids and rail sidings, heavy duty road type pavements for truck areas such as thick deep strength asphalt, full depth asphalt, or rigid concrete pavements, and light duty road type pavements.

Heavy duty port pavements would be expected to accommodate plant such as reach stackers, straddle carriers, forklifts, terminal tractors, trucks, rubber tyre gantries (RTGs) and static loads such as container stacks and possibly dry bulk.

If an Automated Stacking Crane (ASC) system is adopted, crane rails would be supported by structural elements, although it is possible that ballast systems could also be used. For ASC operations, heavy duty pavements would be required for the end interchanges, with midblock container stacking areas designed based on geotechnical requirements.

Pavements will be founded on a combination of reclaimed land, and on land/ terrestrial areas. Ground improvement will be required to support heavy duty pavements on reclaimed areas and where lower strength subgrades are present. Design CBR for reclaimed fill would be dependent on the reclamation materials used but may be in the order of 2% to 4%. Design CBR for terrestrial areas is expected to be in the order of 3 to 5% for Baxter Formation soils subject to further investigations.

**Durability**

Soil aggressivity for concrete and steel components that are in contact with the ground is based on AS2159 – *Piling Design and Installation.* The classification for concrete piles fully submerged in seawater is moderate and for elements in the intertidal or splash zone the classification is severe. For steel piles in seawater the exposure classification is also severe.

Selected soil samples have been tested for aggressivity (pH, chloride, sulphate and electrical conductivity). Based on the chemical testing results in section 4.3, an exposure classification of Mild is deemed appropriate with respect to concrete and steel durability (AS-2159). For buried concrete and steel members, an exposure classification of B2 could be adopted in accordance with AS 2758.1 (Table A2). It is noted that these recommendations do not include special considerations for localised ground conditions containing acid sulphate soils. Such conditions possibly exist within the intertidal zone and within localised drainage lines containing Quaternary alluvium. Durability requirements for wharf structures including steel piles, concrete piles and reinforced concrete elements will be addressed as part of other work packages.

## 11.6 Characteristic Engineering Parameters

Characteristic engineering parameters for each of the main stratigraphic units have been provided, derived from the results of in situ and laboratory testing and engineering experience with similar materials/geological units. Given the vertical and lateral variation within each of the main stratigraphic units, and the variability of laboratory and in situ test results, characteristic parameters are given for the range of soil types and consistency expected to be present within those units. These parameters are considered adequate for concept design however it is expected further investigation and testing will be required for future design stage studies. As the port layout is developed, design stratigraphic profiles can be produced and the range of engineering parameters refined as part of future work packages. Sensitivity assessment would need to be undertaken as part of such design assessments and analysis.

## 11.7 3D Geological Model Development

A 3D Geological Model of the project area has been created using the investigation data and interpreted stratigraphy. The model has been developed using implicit geological modelling software. The model includes borehole, test pit and geophysical information and provides a visualisation tool for the data. In addition to the modelling of surfaces and volumes, a digital database of all investigation data has been collated. The 3D Geological Model is presented and reported separately from this report.

## 11.8 Conclusions and Recommendations

The recently completed 2013/2014 geotechnical investigation drilling and CPTs in conjunction with the geophysical survey have provided valuable information to inform current and forthcoming business case studies, and provide a better understanding of geotechnical conditions and risks across the area of potential Port of Hastings expansion.

The investigations have indicated that low strength recent marine clay deposits below sea bed within the Port Area are relatively thin (< 1 to 4 m). Within the BlueScope Steel terrestrial SUZ1 area and across most of the offshore Port Area relatively competent Baxter Formation soils are present at shallow depth (near surface or within a few meters of seabed). Baxter Formation soils are deemed to be suitable founding strata on which to place reclamation fill. The Baxter Formation and the underlying Sherwood Formation (and deeper Silurian bedrock where present), are also expected to be suitably competent to found quay structures and associated facilities supported on appropriately designed foundations.

Dredging in the Port Area would involve excavation of recent marine clays and loose sands, together with the underlying more competent and variable sandy/ silty clays, clayey and silty sands, and silts, of the Baxter and Sherwood Formations. Recent marine clays would be unsuitable as reclamation fill. It may be beneficial to remove low strength recent marine deposits in areas of reclamation prior to filling. Where reclamation filling using dredged Baxter and Sherwood Formation soils is proposed, careful management of fluidised silts and clays derived from trailing suction and/ or cutter suction dredging processes would be required, so that unconsolidated slurry does not become entrapped within the reclamation fill. Backhoe dredging could be considered for cohesive soils in some circumstances with the benefit that in situ moisture content and soil consistency can be largely retained.

Bedrock is not expected to be encountered during dredging in the Port Area or the North Arm although local dredging and removal of high spots in the Western Channel and Anchorage would likely encounter weathered basalt.

Geotechnical considerations identified in the Geotechnical Interpretive Report would need to be taken into account in work package studies in respect to dredging and reclamation, ground improvement, and wharf and terminal development. As the port layout is developed, design stratigraphic profiles would need to be produced and the range of proposed engineering parameters refined. Sensitivity assessment would need to be undertaken as part of such design assessments and analysis.

Whilst the subsurface drilling and CPTs undertaken in 2013/ 2014 have provided valuable information to inform current and forthcoming business case studies, since the investigations were on a relatively wide spaced grid pattern (500 m centres), it should be recognised that further investigations would need to be undertaken during future design stage studies. This may include additional closer spaced and targeted terrestrial and marine investigation borehole drilling and CPTs, closer spaced sediment sampling and laboratory characterisation of soils to inform detailed dredging studies, the undertaking of further geophysical survey including seismic refraction to assist in determining engineering properties of subsurface soils and rocks, and a trial embankment on the Old Tyabb reclamation.

The key findings of the geotechnical and geophysical investigations as described in the Geotechnical Interpretative Report are given below:

- Bedrock is not expected to be encountered during dredging in the Port Area or the North Arm, however dredging of cemented materials will be required. Dredge materials in the Port Area and the North Arm are not expected to pose significant challenges for conventional dredging plant and equipment. Localised

dredging and removal of high spots in the Western Channel and Anchorage would likely encounter weathered basalt rock.

- Within the BlueScope Steel terrestrial SUZ1 area and across most of the offshore Port Area relatively competent Baxter Formation soils are present at shallow depth which are deemed to provide suitable founding strata on which to place reclamation fill. Relatively thin low strength recent deposits are present below sea bed within the Port Area. It may be beneficial to remove these materials in areas of reclamation prior to filling.
- The dredge materials, predominantly comprising Baxter and Sherwood Formation soils, exhibit vertical and lateral variability. As a result of this variability it may not be possible to selectively dredge these sand and clay deposits. Reclamation using dredged Baxter and Sherwood Formation soils would require careful management to prevent unconsolidated slurry from becoming entrapped within the reclamation fill. Ground improvement would be required for reclamation fills derived from these materials to densify soils and reduce post reclamation settlement. Recent soft marine clays would be unsuitable as fill for reclamation.
- Foundation materials are expected to be suitably competent to support quay structures and associated facilities using conventional forms of construction including piled bulkhead walls or piled quay decks. Piles for wharf structures are expected to be driven to refusal on bedrock or a specified driving resistance in Baxter or Sherwood Formation soils. Foundations for gravity structures may require construction of a rockfill base along the quay line within the Baxter or Sherwood Formations due to the high bearing pressures associated with these structure types.

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# Appendix A Borehole Location Plans

# Appendix B Cross-sections – Stratigraphy

# Appendix C Cross Sections to -25mCD - Port Area

# Appendix D In Situ and Field Test Data – Terrestrial

# Appendix E In Situ and Field Test Data – Marine

# Appendix F Laboratory Test Summary Tables

# Appendix G Laboratory Test Summary Plots – Terrestrial

# Appendix H Laboratory Test Summary Plots- Marine

# Appendix I Geophysical Survey - Outputs

# Appendix J Groundwater Monitoring Data Summary

# Appendix K Depth of Soft and Loose Soil at Seabed - Port Area

# Appendix L Mineralogy Results Summary

# Appendix M Borehole – CPT Comparison

# Appendix N Example Design Profiles