REPORT

Rip Road Reserve Foreshore Rehabilitation

Design Report

Client: Central Coast Council

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Status: 00/D1

Date: 31 October 2019

Royal HaskoningDHV

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Appendices

1 INTRODUCTION

1.1 Background

Rip Road Reserve is located at Blackwell just upstream of Rip Bridge on the southern shoreline of Brisbane Water. The shoreline is in poor condition with failing walls and adhoc protection structures. An Aboriginal shell midden is located at the eastern end of the reserve.

The objective of the investigation is to design a seawall to protect the reserve, landward infrastructure and an Aboriginal shell midden from shoreline erosion. The design is to incorporate suitable pedestrian access and cater for storage and launch/ retrieval of light passive craft.

The project is being commissioned as an action of the Coastal Zone Management Plan (CZMP) for Brisbane Water Estuary (Cardno, 2012) and must therefore meet the aims and objectives of the Plan.

1.2 Aims and objectives of Brisbane Water CZMP

The overarching aims for the management of the Brisbane Water estuary are to:

- Protect, rehabilitate and improve the natural estuarine environment;
- Manage the estuarine environment in the public interest to ensure its health and vitality;
- Improve the recreational amenity of estuarine waters and foreshores;
- Recognise and accommodate natural processes and climate change; and
- Ensure ecologically sustainable development and use of resources.

The objective of the CZMP is to promote estuary management for coastal ecosystem health and community uses of the coastal zone, as opposed to private benefit or for development purposes (Cardno, 2012).

1.3 Design life for the works

The proposed minimum design life for the seawall works is 50 years. This is the life for which the structures would remain fit for use for their intended purposes with appropriate maintenance. Materials for the rip rap revetment have been specified in accordance with *AS2758.6 Aggregates for rock and engineering purposes Part 6: Guidelines for the specification of armourstone*. The guidelines are based on a design life of 50 years.

There is considerable uncertainty regarding the extent of sea level rise in the scientific community. The structures aim to retain the existing reserve level. The seawall an revetment may need to be modified in the future, by raising the crest level, to cater for sea level rise.

1.4 Extent of works

The extent of the rehabilitation works would cover approximately 80 m of foreshore from the end of the new vertical sandstone block wall fronting H5 Rip Road at the western end of the reserve, to a low lying sandstone block wall fronting H504A Orange Grove Road at the eastern end. The extent of the works is shown in **Figure 1** reproduced from the Brief.

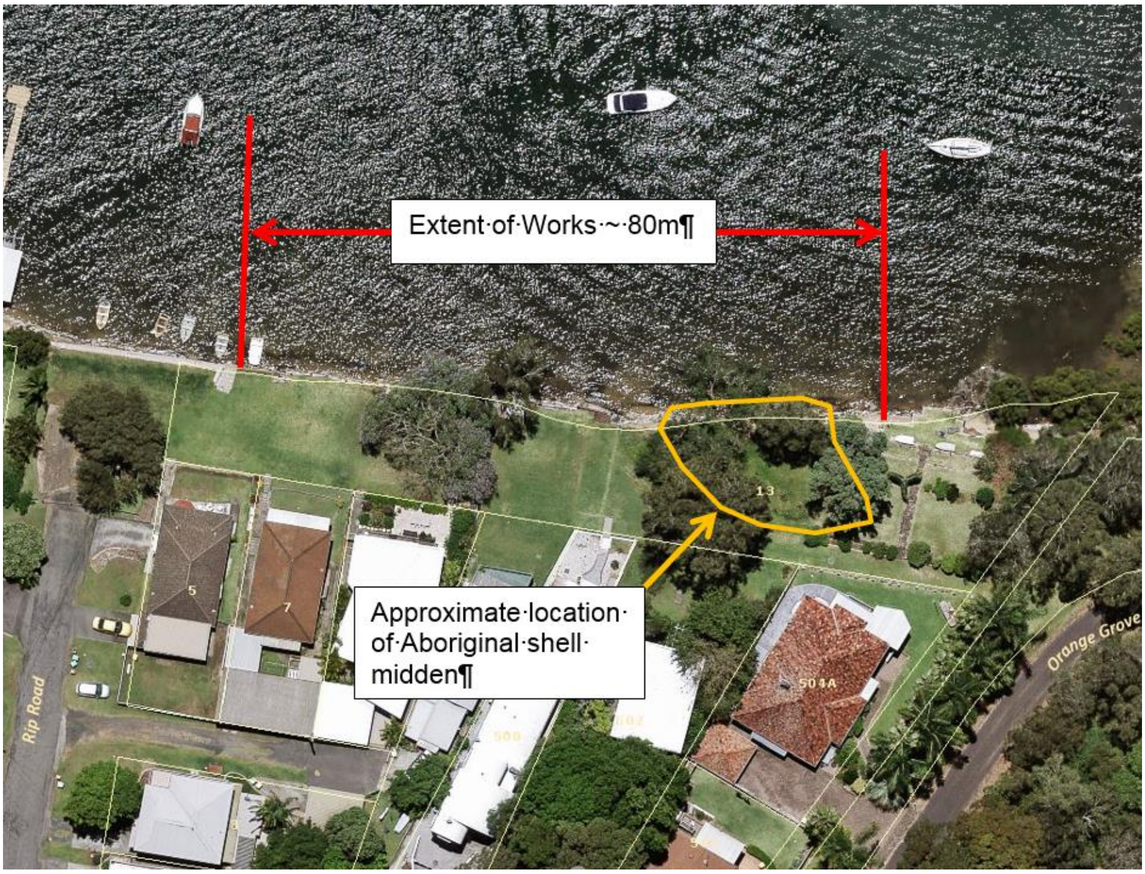

Figure 1 Extent of works (RFP)

1.5 Scope of work

The scope of work was broken down into the following phases:

1. Engagement and project inception
2. Site investigations
3. Design report
4. Detailed design and documentation

This report documents the Design Report scoping item.

1.6 Acknowledgements

We acknowledge the project management assistance of Council's Waterways and Coastal Management Officer Warren Brown.

2 ON SITE INCEPTION MEETING AND DISCUSSIONS

A site inspection and on-site project inception meeting was undertaken on 28 February 2019 between 8.00 am and 9.30 am. The inspection was carried out by Gary Blumberg from RHDHV, accompanied by Warren Brown from CCC and Tracy Howie from the Guringai Tribal Link Aboriginal Corporation. Weather during the inspection was fine, winds were light and scattered showers had fallen in the preceding 24-48 hours. The tide was high at the time of the inspection. Stephen Thorne and Associates and JK Geotechnics respectively commenced their survey and geotechnical investigation work during the inspection and these exercises continued through the day. Selected photos taken during the site inspection are included at **Appendix A**. Various matters were discussed relating to:

- General features, existing seawalls and stormwater outlets
- Groynes and seagrass beds
- Information on the Aboriginal midden area and early development of the reserve
- Agreement on the proposed protective structures and foreshore improvements to be designed.

2.1 General features, existing seawalls and stormwater outlets

Rip Road Reserve is reasonably secluded and is mainly used by occupiers of houses that back onto the reserve.

The site was fully inspected from east to west. The inspection revealed a tree-shaded Aboriginal midden area with its natural eroding foreshore, mature trees, waterside steps, existing privately installed ad-hoc seawalls and stormwater lines, and a substantial block sandstone seawall constructed by Council in 2015 at the western end of the reserve (Work as Ex drawings attached at **Appendix B**). In the eastern half of the reserve three makeshift groynes comprising loose stones and blocks protruded a short distance from the shoreline. Two tree stumps were situated near the shoreline both in the eastern portion of the reserve near the Aboriginal midden area.

Four private seawalls were observed, three timber seawalls and a brick seawall. The western most timber seawall has failed, and the eastern most brick seawall is failing. A new timber seawall was constructed with treated pine timber posts (200x80) backed on their landward side by treated pine whaling panels (200x50). It appeared that there was no/ little filtration behind this seawall with silty water observed escaping from the bottom. The water depth at the base of the new timber seawall was 0.1m at 8.15am, with the waterline measured at 880mm below the top of the seawall.

One major stormwater outlet was located through the block sandstone seawall aligned with the Rip Road reserve. A small private drainage pit with grate and “snorkel” outlet was located just west of the Aboriginal midden. This pit was fully blocked with sand. A 200 dia PVC outlet with flap gate was nearby.

Slight grass burn along the crest of the existing seawall indicated some overtopping into the reserve at high tide.

2.2 Groynes and seagrass beds

Seagrass beds were observed out from the shoreline. The inshore edge of the seagrass beds approximately aligned with the outer ends of the three makeshift groynes and was located approximately 6-7m from the shoreline. The eastern groyne located near the eastern tree stump comprised 50 off 0.3-0.4m dia sandstone rocks protruding above the bed. It was estimated that up to 1-2x that number (50-100 rocks) could be buried directly below the bed at this groyne, thus 100-150 rocks in total were estimated to comprise this eastern groyne. This would equate to a total tonnage of up to approximately 6-9T of rocks at this groyne, and quite possibly similar tonnages at the other two groynes. The protruding groyne rocks were oyster encrusted. It was agreed that construction plant should be limited to working landward of the ends of the groynes to protect the seagrass beds.

2.3 Information on the Aboriginal midden area and early development of the reserve

Registered Aboriginal midden site (45-6-3600) is located along the foreshore and upslope in the eastern corner of the Rip Road Reserve. Shell material had been washed out of the bank below the midden site but recently there had been evidence of some accretion (Warren Brown, pers comm).

Council is of the view that the reserve would have received dredge spoil in years gone by, although this would not have extended to the midden site under the trees which Council considers to be relatively undisturbed. However, the possibility of the site having been affected by excavated material from adjacent housing sites higher up the reserve cannot be discounted. The western tree stump is considered to demarcate the western extent of the designated midden area with insufficient midden materials observed further west. The tree stump at the western groyne was clear of midden material. This area had been checked previously by Council, before growing over with grass as was encountered during the inspection. As part of a previous heritage study, coring below the midden in the face of the bank had encountered bone material, however this was determined to be probably animal.

Guringai Tribal Link Aboriginal Corporation (GTLAC) attended the onsite inception meeting during which the methodology for the geotechnical investigation and design options for the stabilization works were discussed. Agreed drill hole locations for geotechnical testing were within areas containing no visible evidence of midden materials. GTLAC would provide comments on the draft design once completed and supplied by CCC.

2.4 Agreement on the proposed protective structures and foreshore improvements to be designed

It was confirmed that the protective treatment for the site would comprise a sloped rock revetment and subvertical block sandstone seawall. Council confirmed that it would not be necessary to examine other treatments.

It was originally Council’s intention to protect the Aboriginal midden area with a steep seawall and have a sloped revetment protecting the western end of the reserve, but during the inspection it was decided that this should be reversed to better manage impacts to the midden area at the east of the site, and to tie in with the existing Council seawall to the west. It was decided to place the rock revetment directly in front of the midden without any excavation at the toe of the embankment.

A termination detail for the revetment at the eastern end of the site was agreed during the inspection. This required the small stone steps in this area to be relocated slightly to the east.

It was agreed that a new set of steps be constructed at the western end of Council’s block sandstone seawall to replace the existing steps which were estimated to have no more than a 5 year residual life. It was requested by Council that the steps be reconstructed with sandstone blocks incorporating a central timber-rail dinghy skid, similar to that constructed at Ferry Park, Ettalong, about 5 years ago.

It was agreed to remove the other set of dilapidated timber steps and reconstruct these slightly further to the east using sandstone blocks.

The existing timber edged planter bed to the west of the western groyne should not be disturbed if possible, and the tree stump in the foreshore opposite this groyne should be removed. It was agreed that the replacement stone steps be located immediately east of this stump.

It was agreed to incorporate the Basis of Design (BOD) into the Design Report. It was considered that a small 5-10T excavator would be suitable plant to construct the new revetment and seawall. It was agreed that sandy coloured basalt similar to that sourced from Seaham Quarry would be a desirable material for the rock revetment.

3 COLLATION AND REVIEW OF BACKGROUND INFORMATION

The following information has been reviewed and interrogated:

- Plan of Management for Rip Reserve (if it exists)
- Brisbane Water Estuary Processes and Management Study
- MHL Tidal Planes
- AS4997 Guidelines for design of maritime structures
- USACE Shore Protection Manual
- USACE Coastal Engineering Manual
- Historical aerial photography (Council's records)
- BOM wind information (for wind wave hindcast calculations)

Relevant information form the reports is documented herein and has been used to complete the detailed design of the foreshore protection works.

GIS layers were also provided by Council, which were provided to Stephen Thorne and Associates. These layers were also used to develop general arrangements.

4 SITE INVESTIGATIONS

Three site investigations were carried out:

- Engineer site walkover and measurements
- Detailed survey
- Geotechnical investigation
- Acid sulphate soils investigation

4.1 Engineer site walkover and measurements

RHDHV undertook a site walkover with Council's Contact Officer Warren Brown as part of the inception visit (**Section 2**). A working plan was prepared and marked up during this visit. Attention was paid to identifying and locating items and features that would inform the rehabilitation design such as:

- bedrock outcrops
- nature of shoreline sediments
- failing adhoc walls
- unstable natural shoreline areas
- mature trees
- type of passive craft and storage areas
- waterside access zones
- Aboriginal shell midden
- stormwater outlets
- site access restrictions and fences
- site walk lines and footpaths
- overland runoff flow paths

Site access and a preferred Contractors Work Area was also considered during our site walkover.

4.2 Detailed survey

Stephen Thorne and Associates (STA) undertook a detailed survey of the site. Land based survey methods were used. The survey included DBYD search and picking up all relevant visible services. The survey was connected to MGA and AHD using State Survey Control Marks. Survey coordinates were determined for selected boundary points to control plotting accuracy of cadastral boundaries adjacent to the reserve. The survey deliverable comprised Autocad 2000 DWG files with 3D triangular mesh.

The following features were included in the survey:

- Survey points nominal 10m centres through the reserve and concentrated to 5m centres along the shoreline
- Property boundaries from Council GIS
- Water line, with spot levels to -0.5m AHD
- Seawalls and other waterfront structures (serviceable or remnant)
- Rip rap
- Top and bottom of bank
- Beach sand and bedrock outcrops

- Edge concrete/ bitumen/cleared unsealed/grassed
- Major trees (trunk dia, approx. height and canopy)
- Stormwater outlets, size and invert RL
- Fences and gates
- Signage
- Services above and below ground (above ground observation and below ground DBYD)
- All trees and major shrubs
- Dinghies and other water craft on shoreline

A copy of STA's survey for the site is attached in **Appendix C**.

4.3 Geotechnical investigation

JK Geotechnics (JKG) undertook a limited geotechnical investigation for the project. A copy of JKG's report is attached in full at **Appendix D** (JKG, 2019), and their investigation and key findings are summarised below.

The fieldwork comprised four boreholes hand auger drilled to refusal depths between 1.0m and 2.4m. Two holes were drilled in the reserve and two along the foreshore at the toe of the slope. Six Dynamic Cone Penetrometer (DCP) tests were extended to refusal depths between 0.7m and 3.0m. The fieldwork was carried out under the direction of Geotechnical Engineer Joel Babbage.

JKG noted that the reserve was located at the base of a hillside that slopes down to the north at a maximum of 15 degrees to the southern foreshore of Brisbane Water. The waterside site boundary was generally formed by concrete block, masonry, concrete and rendered retaining walls (maximum height about 1.0m).

The 1:100,000 geological map of Sydney indicates the site is underlain by Quaternary age quartz sand with minor shell content, and interdune silt and fine sand Beach ridge system deposits. The boreholes disclosed a generalised subsurface profile comprising fill overlying marine sands and sandy clays followed by natural clays. The refusal of the DCP tests at depths between 1.8m and 3.0m below the reserve surface (DCP1 to DCP3) and depths between 0.7m and 2.0m below the below the foreshore surface (DCP4 to DCP6) have been interpreted to indicate poorly cemented sands although the presence of 'floaters' and/or fragments of previously collapsed seawalls (particularly below the foreshore surface) cannot be discounted. Based on the topographic setting of the site JKG considered it unlikely that DCP refusal indicates bedrock.

Sketch concept designs developed by RHDHV for a subvertical block stone seawall and sloped rock revetment to protect different portions of the shoreline were considered by JKG in the preparation of their advice.

JKG made a number of comments and recommendations relating to contractor insurance and supervision, need for dilapidation reports, excavation conditions, potential ground surface movements, groundwater seepage and tide levels, temporary batter slopes and retention, and shoring design parameters. Earthworks recommendations are also provided including subgrade preparation, subgrade drainage during construction, and a specification for composition and placement of engineered fill. Refer **Appendix D** for this discussion. The Technical Specification for the works would draw from JKG's recommendations.

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4.4 Acid sulphate soils investigation

JK Environments (JKE) undertook an acid sulfate soil (ASS) assessment. The investigation was undertaken in conjunction with the geotechnical assessment by JKG and the results are presented in a separate report, attached in full at **Appendix E** (JKE, 2019). The aims of the assessment were to establish whether actual ASS or potential ASS (PASS) may be disturbed during the proposed development works, and to assess whether an ASSMP is required. The ASS assessment and report were undertaken with reference to the Acid Sulfate Soil Management Advisory Committee (ASSMAC) Acid Sulfate Soil Manual (1998). The JKE investigation and key findings are summarised below.

A review of the Central Coast council LEP indicates that the site is located in a Class 1 ASS risk area. A review of the ASS risk maps prepared by Department of Land and Water Conservation (1997) indicates that the site is located in an area classed as having a ‘high probability’ of encountering ASS. The map indicates that intrusive works (shallow drainage, excavation or clearing) in areas classed as ‘high probability’ can disturb Potential Acid Sulfate Soil (PASS) which pose severe environmental risk.

Field work for this investigation was undertaken on 28 February 2019. Soil samples were collected from four locations in conjunction with the JKG investigation, to a maximum borehole depth of 2.4m, with eight selected soil samples analysed for ASS/PASS. The majority of the soil samples analysed encountered results which exceeded the action criteria adopted for the assessment. Based on these results, and considering the information reviewed for the assessment (risk maps, subsurface conditions, etc), the natural soils at the site are considered to be PASS. Accordingly, an ASSMP would be required to manage these soils during the proposed development works.

During the site inspection, fibre cement fragments (FCF) were encountered along the length of the retaining wall and foreshore area. It was recommended that an emu-bob is conducted to remove the FCF from the surface of the site and any fill to be removed/ excavated as part of the works is classified in accordance with the NSW EPA Waste Classification Guidelines - Part 1: Classifying Waste (2014) and disposed off-site.

5 DESIGN DEVELOPMENT

Design development for the foreshore rehabilitation works included consideration of the following:

- Reserve usage and management
- Shoreline morphology and geotechnical
- Water level
- Wave action
- Shoreline stability
- Stormwater management
- Riparian vegetation
- Aboriginal heritage
- Optional foreshore protection treatments and proposed arrangements

Key considerations in relation to these matters are summarised below.

5.1 Usage and management of Rip Road Reserve

The reserve slopes relatively steeply into the waterway which limits is amenity and usage.

The reserve is used for passive recreation and access to the water by small dinghies, kayaks and other passive craft. We understand from Council that the usage is primarily limited to occupiers of residential properties that border the top of the reserve. Small dinghies are tied back to the shoreline with shoreline slopes too steep for these to be hauled up the bank into the reserve. Council undertakes mowing of the grass and general tending of garden beds and the like. The block sandstone at the western end is a recently built Council asset requiring minimal maintenance. We understand that the other seawall structures, which are generally in varying states of disrepair, are not Council assets.

5.2 Physical site conditions

The survey, geotechnical and ASS investigations indicate the morphology of the site and geotechnical and geochemical considerations (**Section 4**). Other natural conditions and processes of interest to the design development include:

- Water level
- Wave action
- Shoreline stability
- Riparian vegetation

5.2.1 Water level

Predicted tidal planes for Rip Reserve would be approximated by those derived in MHL (2010) for the nearby Brisbane Water site at Koolewong, also located upstream of Rip Bridge (**Table 1**).

Table 1: Tidal planes for Rip Reserve

Tidal Plane		Level (m AHD)
High High Water Summer Solstice	HHWSS	0.64
Mean High Water Springs	MHWS	0.39

Mean Sea Level	MSL	0.08
Mean Low Water Springs	MLWS	-0.22
Indian Spring Low Water	ISLW	-0.40

Source: MHL (2012)

Cardno (2008) reported an indicative peak water levels for Woy Woy, some 2.5km from the site and also upstream of Rip Bridge. Based on elevated ocean levels and freshwater flooding, the following were reported:

- May 1974 storms (nominal 100 year ARI) 1.64m AHD
- 20 year ARI 1.43m AHD
- 10 year ARI 1.37m AHD

The above tidal and flood water levels make no allowance for sea level rise and wave action. In accordance with AS4997 Guidelines for Design of Maritime Structures, maritime facilities including seawalls should be designed to cater for an increase in water level due to sea level rise. AS4997 recommends an allowance of 0.2m for a design life of 50 years, increasing to 0.4m for a design life of 100 years. Various State and Local Government policies in NSW indicate up to a doubling of these allowances.

It is considered reasonable to adopt a design still water level of 1.5m AHD for conditions today, increasing to 1.8m AHD over the 50 year life of the foreshore rehabilitation works.

5.2.2 Wave action

Wave action at the site would be due to wind waves and waves generated by passing vessels.

Wind waves are caused by winds blowing across a water surface. The height and period of the waves depend on the wind speed, the distance over which the wind blows, and the water depth. A design 50 year ARI wind wave hindcast prediction for the site based on long-term wind statistics for the Sydney region and using methods in accordance with AS1170.2 Structural Design Actions, Wind Actions and CERC (2002) are as follows:

- Fetch distance and direction 1.7 km N
- Terrain category 1
- Significant wave height (Hs) 0.67 m
- Peak wave period (Tp) 2.3 s

It is noted that the design wind wave would approach the shoreline with a small degree of obliquity.

Boat waves are generated by passing vessels. Boat wave conditions depend largely on the hull size and shape, whether the vessel is planing or not, water depth, and distance from the sailing line. Based on extensive experience with boat wave assessments, the recommended incident design boat wave condition for the site is:

Maximum wave height (H_{max}) = 0.6 m
Wave Period (T) = 2-3 s

Boats would not be operating during severe wind events. It is considered reasonable to adopt a design incident significant wave height of 0.67 m for the foreshore rehabilitation works.

5.2.3 Shoreline stability

The shoreline is unprotected, steep and probably eroding over the eastern 40m of the site. Seawalls occur further to the west. These are light structures in varying states of disrepair, except for the more substantial block sandstone seawall at the western end of the site. The foreshore rehabilitation works are required to protect the 80m of shoreline shown in **Figure 1**.

The beach and inshore bed areas appear reasonably stable. The undated SIX Maps (NSW Spatial Services) image of the reserve shows a slight sediment buildup beside the rock groynes suggesting some W to E longshore sediment transport (**Figure 2**).

There is no particular net longshore transport of sediment evident at the groynes.

5.2.4 Stormwater management

One major stormwater outlet is located through the block sandstone seawall aligned with the Rip Road reserve. A small private drainage pit with grate and “snorkel” outlet is located just west of the Aboriginal midden, and there is a 200 dia PVC outlet with flap gate nearby.

The new Council owned block sandstone seawall and its major stormwater outlet would not be disturbed by the works. All other stormwaters are understood to be private facilities. These would be temporarily diverted and reinstated, like-for-like or better, as part of the works.

5.2.5 Riparian vegetation

The shoreline of the site is devoid of any significant marine vegetation. While mangroves occur at the eastern end of the reserve, no mangroves occur along the foreshore to be rehabilitated. A seagrass edge is visible in places encroaching to within 7m of the high water mark as noted during the site inspection (**Section 2.2**). However available seagrass mappings for Brisbane Water do not show any seagrass beds in the vicinity of the reserve (West et al, 1985; and Jelbart and Ross, 2006), hence the significance of the beds observed would be expected to be low.

Casuarina occur just above the high water mark towards the eastern end of the site.

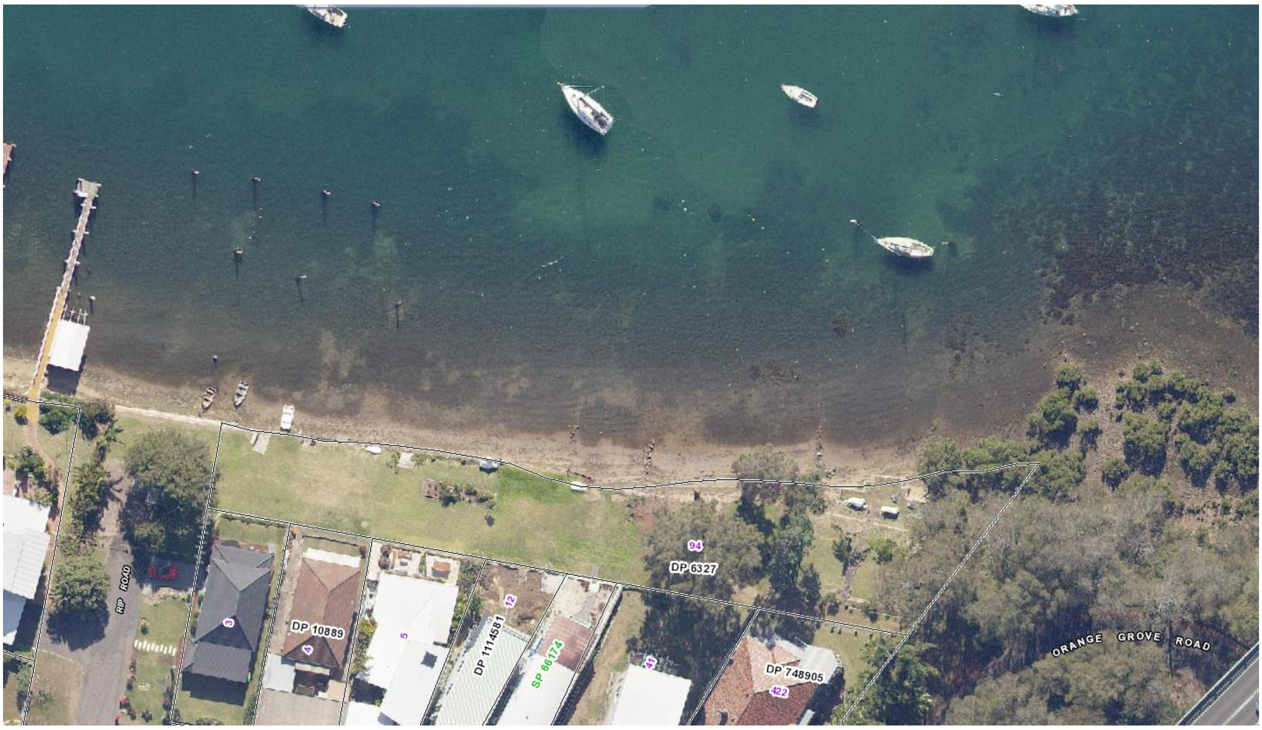

Figure 2 SIX Maps image of Rip Road Reserve showing property boundaries. Sediment buildup beside rock groyne structures indicate W to E net longshore sediment transport

5.3 Aboriginal Heritage

Registered Aboriginal midden site (45-6-3600) is located along the foreshore and upslope in the eastern corner of the Rip Road Reserve. Shell material had been washed out of the bank below the midden site but recently there had been evidence of some accretion (**Section 4.1**). No further information on the Aboriginal midden site was provided.

The design of the foreshore stabilisation works would avoid any disturbance to the Aboriginal midden site.

5.4 Foreshore protection options and proposed concepts

Pre tender discussions with Council identified a near-vertical shore protection solution to protect the Aboriginal shell midden, and a sloped revetment to protect the foreshore of the reserve further to the west. During the inception site walkover it was agreed that this arrangement should be reversed.

A steep protection solution would require a deeper and more substantial toe potentially destabilising the midden, whereas leaving the embankment fully intact and covering it with rock would achieve a far better outcome for this area. The far western end of the reserve is protected with a relatively recent subvertical block sandstone seawall designed by Council and tying in to this structure with a similar profile would provide longshore and cross shore continuity in this section of the reserve. Providing the two foreshore protection types, sloped revetment in the east and subvertical seawall in the west, and joining these in the vicinity of the failing brick seawall opposite the eastern boundary of Lot 12 DP 1114581, was considered to deliver the optimum foreshore protection arrangement for the reserve.

It was agreed that the western block sandstone seawall would comprise the same block sizes and profile as the existing Council block sandstone seawall, subject of course to confirming the wall structural design and stability. The eastern sloped rock revetment would essentially involve covering the existing natural embankment with rock with minimal excavation limited to securing the toe of the structure. It was also agreed that no further options need be considered.

Rock is the material of choice for revetments around Brisbane Water. Gosford Quarries has supplied suitable sandstone for many projects on the Central Coast, and suitable basalt and rhyodacite are also quarried locally. Council has recently expressed a preference for the buff coloured basalt which it has sourced at reasonable cost from the Seaham area near Raymond Terrace. It is proposed that suitable sandstone would be used for the block seawall and suitable basalt for the rock revetment.

Two options are available for the revetment design; a conventional two layer design (armour and underlayer) or a more widely graded rip-rap design. In discussion with Council it was agreed to develop a rip-rap design for the site. The same rock size range is achieved but with a saving of around 20% of the total rock tonnage. The rip rap design proposed for Rip Road Reserve is summarised in **Table 2**.

Table 2: Design parameters for proposed rip rap revetment

Design parameters	Value	Comments
Significant wave height and peak period	0.67m, 2.3s	Since H_{max} <1.5 m rip rap design permissible from CERC (1984). Refer **Section 5.2.2**
Slope	1:2	Relatively flat slope but similar to existing embankment thereby avoiding the need to reshape the embankment and potentially disturb shell middens
Minimum design rock dry density	2.5 T/m^3	About 5% lower than typical dry density to allow for supply variations
Median rock mass and mass range	53kg (20-210kg)	Based on Hudson assessment with K_D=2.2 (CERC, 2003)
Minimum rip rap thickness	640mm	Allows for greater of 2x D50 and 1.25x Dmax as per CERC (1984)
Design scour level	700mm below bed level	Measured at the back of the revetment
Crest level	2.1m AHD	600mm freeboard for design 1.5m AHD SWL applying today, and 300mm freeboard for design 1.8m AHD SWL adopted over 50 year planning timeframe. Crest level can be increased by adding rock to mitigate overtopping in the future if required

The sandstone seawall would comprise individual stone blocks which readily meet requirements for hydraulic (inundation and wave) stability. The structure would be grouted with the thickness of grout suitably thick to accommodate tolerance of the sandstone blocks. A crest level of 2.0m AHD is adopted for this structure which is equivalent to the average crest level of the existing block seawall, but notably exceeds the crest levels of the existing vertical seawalls in the reserve by approximately 0.5m. It is predicted that the design maximum wave runup level at the proposed block sandstone seawall would be 2.5m AHD today, increasing to approximately 2.8m AHD in 50 years time. The predicted 5 year ARI wave runup level occurring today is estimated at 1.93m AHD. Management of the expected wave runup at the block sandstone seawall is considered appropriate. As for the sloping rock revetment, management of additional wave runup due to sea level rise would be simply achieved by adding a blockwork course along the crest as required. The seawall would be designed to accommodate any future capping course up to say 0.5m in height.

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Four sets of steps currently occur in the reserve from west to east as follows:

- 2m wide concrete and brick stone steps opposite Lot 3 DP10889
- 2.1m wide failed steps comprising timber, stone and tyres opposite Lot 4 DP10889
- 2.2m wide timber steps opposite Lot 5 DP10889
- 800mm wide stone steps adjacent to stone seawall opposite Lot 422 DP

The steps are generally in poor condition or have failed, and there is a need to repair and reorganise all of these the accessways. Council would also like a dinghy skid to be included.

At the inception walkover it was agreed that the 3 existing sets of steps in the western half of the reserve be rationalised to two; a new set of sets incorporating a dinghy skid between the existing block sandstone seawall and the western end of the new bock sandstone seawall (opposite Lot 3 DP10889), and a new set of steps opposite Lot 12 DP1114581 located adjacent to and immediately east of the groyne in this location. Council has requested that the dinghy skid access (rails within a set of steps) be of a design which is similar to the arrangement which RHDHV designed for Ferry Park, Ettalong, in 2013. A copy of this design is shown in **Figure 3**.

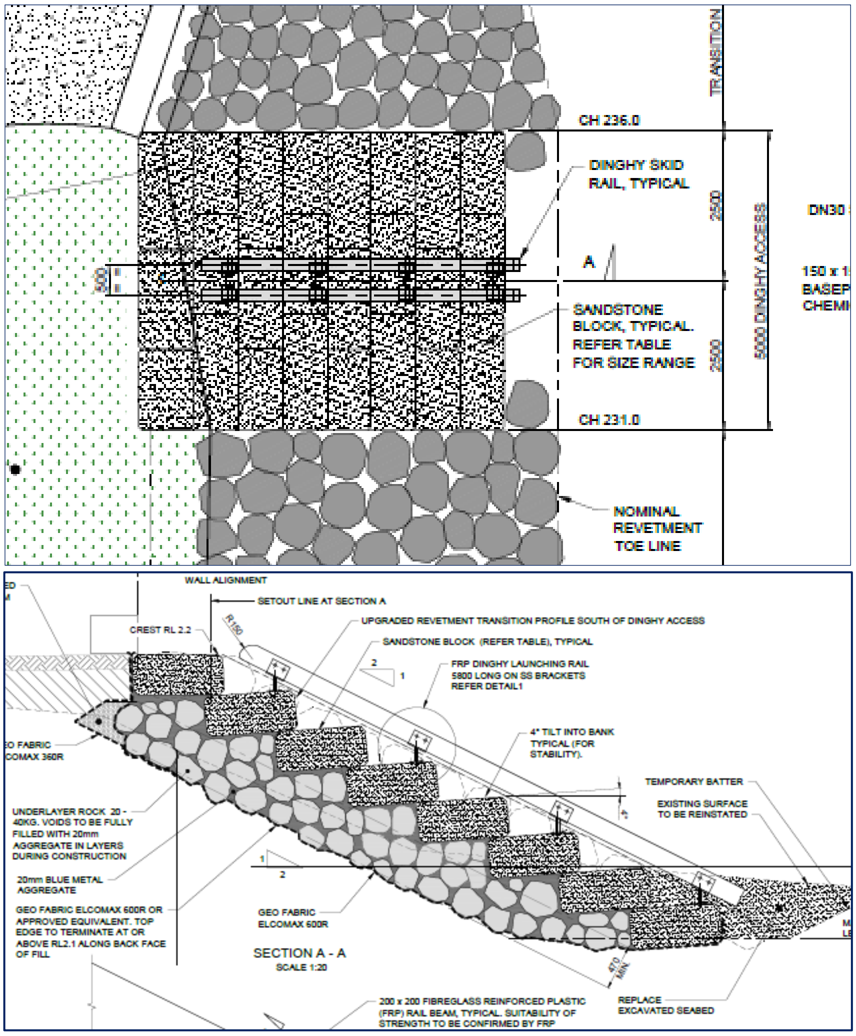

Figure 3: Dinghy skid design provided at Ferry Park, Ettalong (RHDHV, 2013)

6 CONSTRUCTION FACTORS

Construction access to the site would be from land only. There is good access into Rip Road Reserve from Rip Road.

The foreshore margin of the reserve would be heavily disturbed to construct the rock revetment and sandstone seawall. The level of disturbance would be greater than that recently experienced at Elfin Hill due to the significantly more elevated foreshore, and larger volumes of materials required. The Aboriginal midden site would be barricaded during the works and no access into the zone would be permitted.

The protection works would be progressed generally from east to west. The rock revetment would be constructed using excavators mobilised by road into the reserve and tracked along the shoreline at low tide. Rocks would be stockpiled in the reserve and ferried along the shoreline at low tide. A slot would be excavated to place the toe of the revetment, with the revetment built back to the west and up the embankment and from east to west. No plant would be permitted to track above the crest of the existing embankment opposite the midden site, and low tide access along the shoreline for all land-based plant would be kept inshore of a line joining the outer ends of the three existing groynes extending broadly parallel along the foreshore. It is expected that a silt curtain would be deployed to contain turbidity to the construction site.

Public access to the reserve would be severely limited by the works, however it should be possible to preserve a 3m corridor running along the backyard boundary fences at the top of the reserve joining Rip Road with the back of the Aboriginal midden and the landscaped paved areas in the east of the reserve.

Royal HaskoningDHV

7 SAFETY-IN-DESIGN

In accordance with Work Health and Safety Regulation 2011, safety in design considerations have been integrated into the design process for the project.

The Safe Design of Structures Code of Practice (WorkCover, 2014) defines safe design as the integration of control measures early in the design process to eliminate or, if this is not reasonably practicable, minimise risks to health and safety throughout the life of the structure being designed. The safe design of a structure will always be part of a wider set of design objectives, including practicability, aesthetics, cost and functionality. These sometimes competing objectives need to be balanced in a manner that does not compromise the health and safety of those who work on or use the structure over its life.

The Code of Practice notes that safe design begins at the concept development phase of a structure when making decisions about:

- the design and its intended purpose;
- materials to be used;
- possible methods of construction, maintenance, operation, demolition or dismantling and disposal; and,
- what legislation, codes of practice and standards need to be considered and complied with.

Consideration of these points is included in the Safety in Design Risk Register presented in **Appendix F.** The Risk Register considers risk during 3 key phases of the project, which are:

1. Safety in Design
2. Construction Risk
3. Operational Risk

8 REFERENCES

Acid Sulfate Soil Management Advisory Committee (ASSMAC) (1998)
Acid Sulfate Soil Manual
ISBN 0734700008, 26 August 1998

Cardno (2012)
Coastal Zone Management Plan for Brisbane Water Estuary
Prepared for Gosford City Council, July 2012
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Cardno (2008)
Brisbane Water Estuary Processes Study
Report prepared for Gosford City Council and Department of Environment and Climate Change
LJ2255/R2262, 6 March 2008

Department of Land and Water Conservation (1997)
ASS Risk Maps

Jelbart JE and Ross PM (2006)
Examination of the Seagrass and Associated Fauna in Gosford Local Government Area (GLGA) including Correa Bay and Patonga Creek
University of Western Sydney, report for Gosford City Council

JK Geotechnics (2019)
Geotechnical Investigation for Proposed Foreshore Rehabilitation at Rip Road Reserve, Blackwall NSW
32217rrpt for RHDHV, 18 June 2019

JK Environments (2019)
Acid Sulfate Soil Assessment, Proposed Foreshore Rehabilitation, Rip Road Reserve, Blackwal NSW
E32217BTlet_ASS for RHDHV,17 April 2019

MHL (2012)
OEH NSW tidal planes analysis, 1990-2010 harmonic analysis
Report No. MHL2053, PW Report No. 11009, ISBN 9780 7347 4435 4
MHL File No. EDP8-2172, First published October 2012

NSW EPA (2014)
Waste Classification Guidelines - Part 1: Classifying Waste

USACE (1984)
Shore Protection Manual
Vicksberg, USA

USACE (2003)
Coastal Engineering Manual
Vicksburg, USA

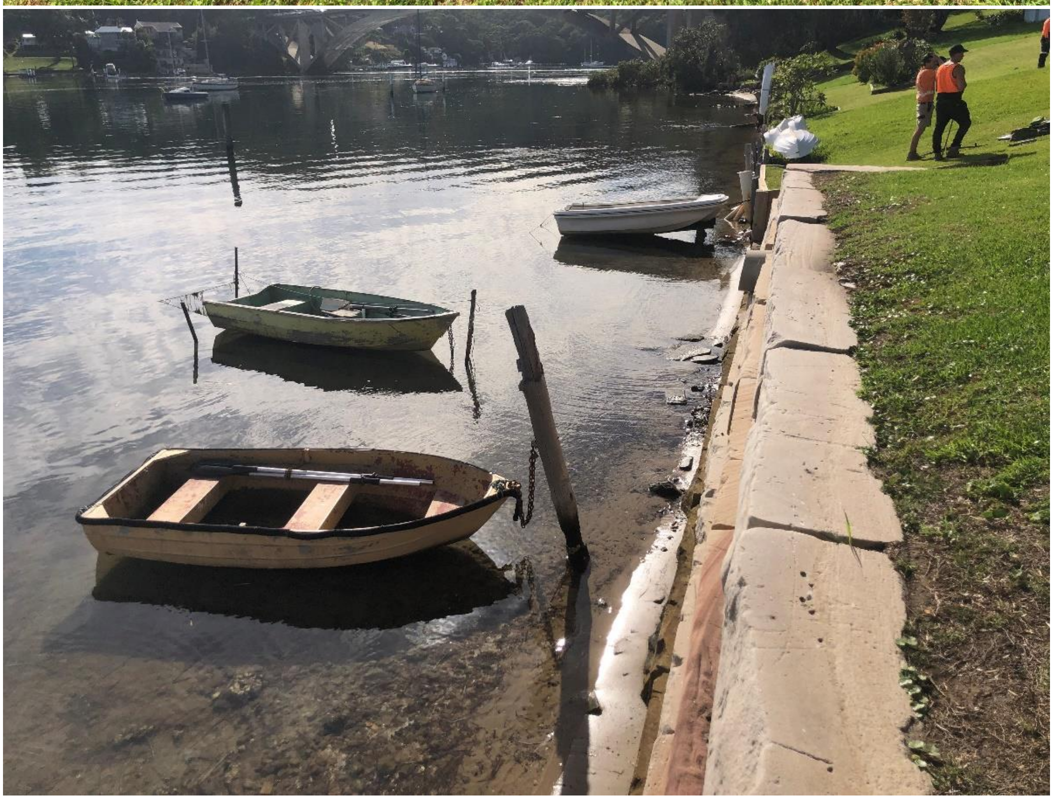
WorkCover(2014)
Safe Design of Structures Code of Practice

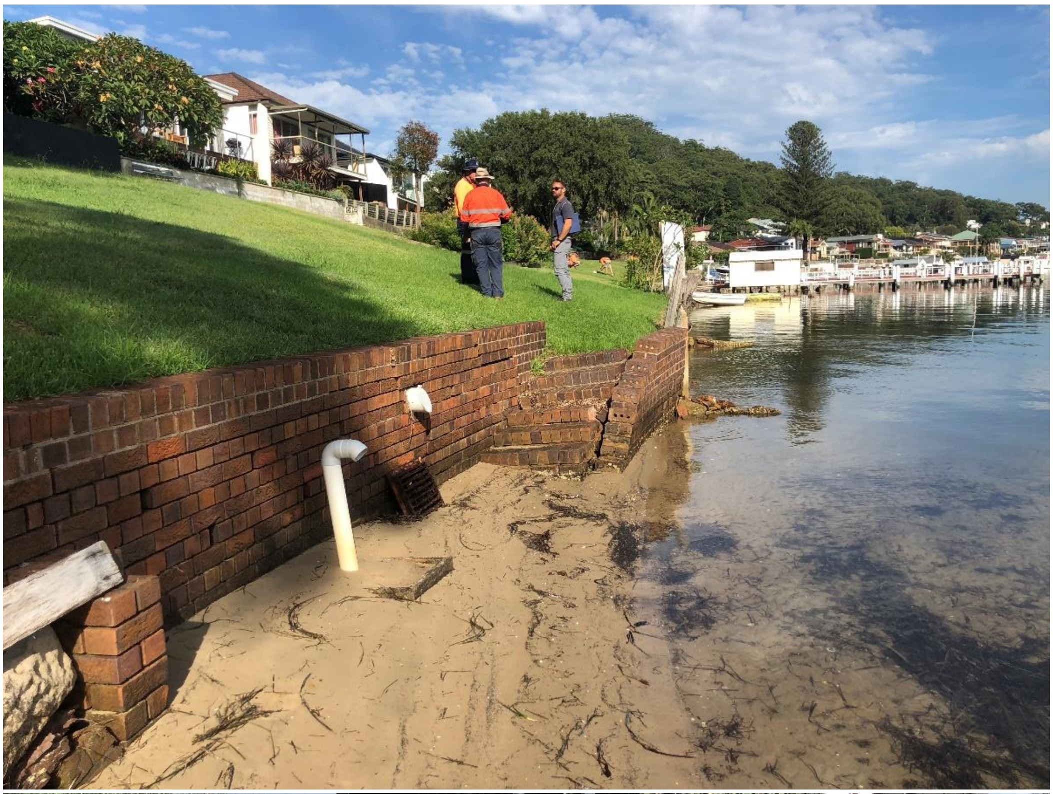
Royal HaskoningDHV

ISBN 978-0-642-78546-6 [PDF], July 2014

Appendix A

Site inspection photos 28/2/19





Appendix B

Rip Road Blackwall,
Rock Retaining Wall Construction (CCC, 2015)

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W. A. E

RIP ROAD, BLACKWALL

ROCK RETAINING WALL CONSTRUCTION

LOCALITY SKETCH

NOT TO SCALE

PREPARED BY:
PROJECT IMPLEMENTATION UNIT
INFRASTRUCTURE PLANNING
GOSFORD CITY COUNCIL

					REFERENCE FILES	RECOMMENDED-	SHEET 1
					SURVEY: DESIGN:		
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1	FOR APPROVAL	DB		26/03/2015	DESIGNED:CD		
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PROJECT No: P283

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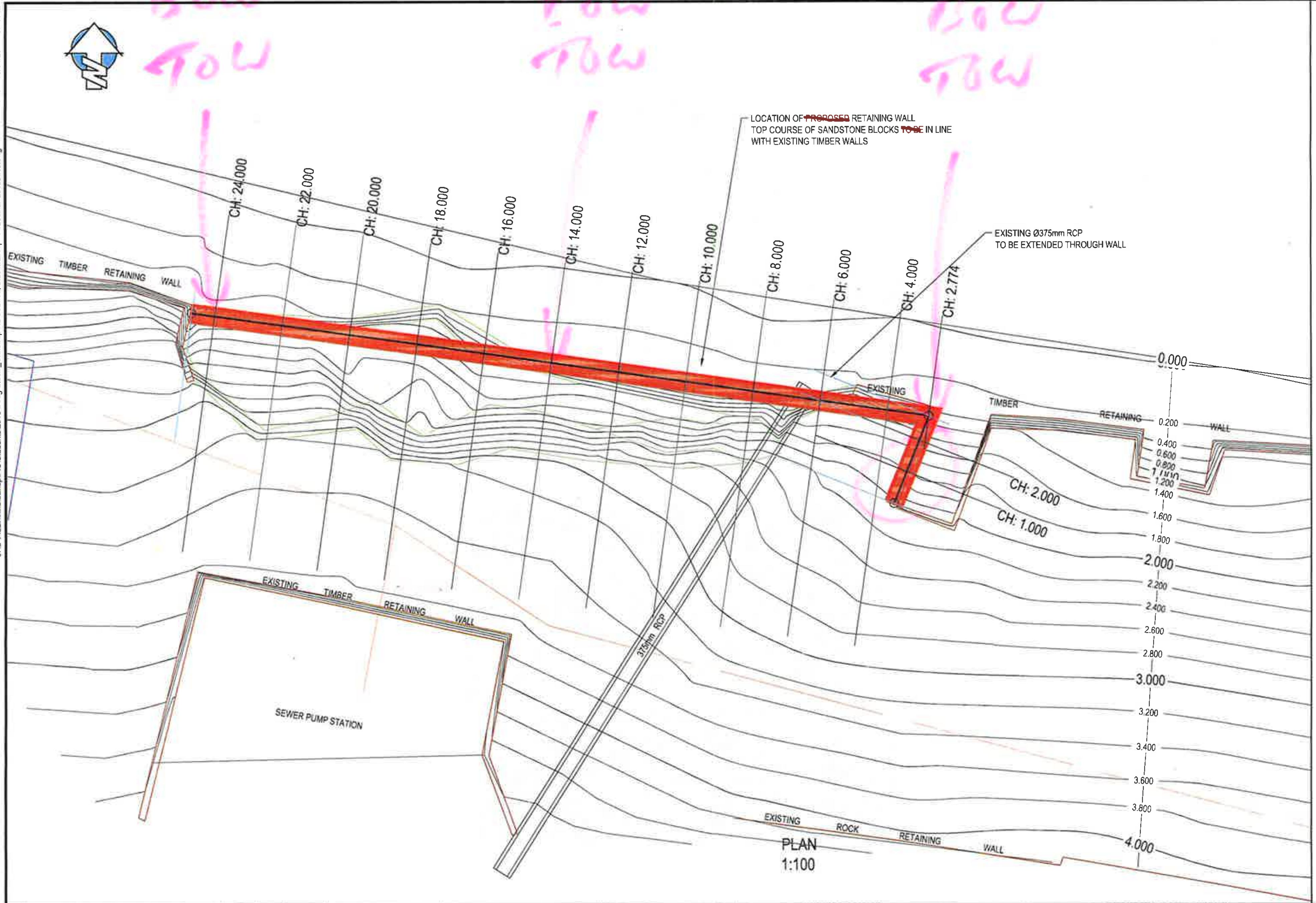

REVISION: -

ECM IR No:

PROJECT / CONTRACTS No: P283

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				DESIGN:	INFRASTRUCTURE PLANNING	APPROVED-	
				SURVEYED: RM	GOSFORD CITY COUNCIL		
1	FOR APPROVAL	DB		26/03/2015 DESIGNED: CD			COORDINATE SYSTEM: M.G.A.
O	REVISION DETAILS	DRAWN	APPR	DATE CERTIFIED:			HEIGHT DATUM: A.H.D.

GOSFORD CITY COUNCIL

RIP ROAD, BLACKWALL

PROPOSED ROCK RETAINING WALL CONSTRUCTION

LAYOUT

SHEET 2 OF 11

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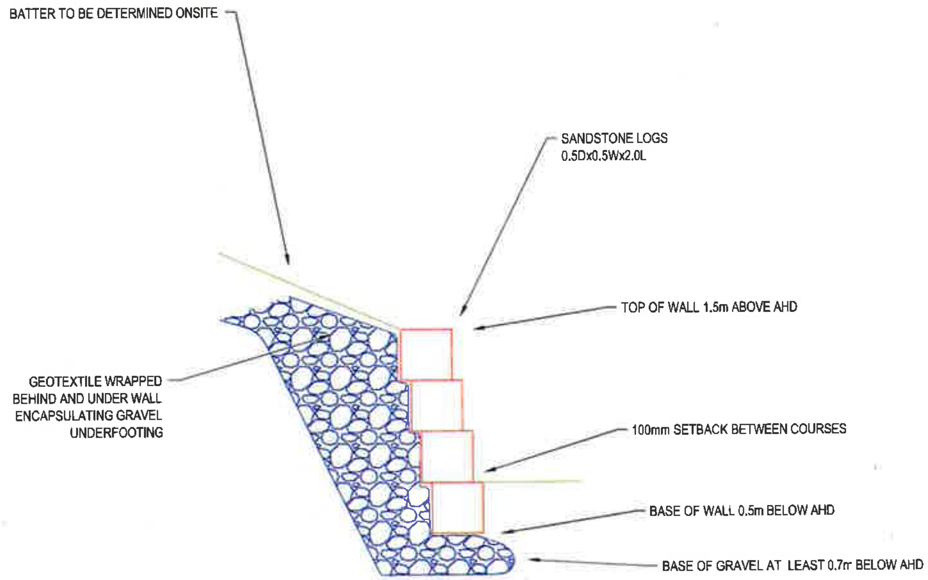

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ECM IR No:

PROJECT / CONTRACTS No: P283

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PROJECT IMPLEMENTATION UNIT
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GOSFORD CITY COUNCIL

RECOMMENDED-

APPROVED-

BASE SCALE FOR A3 PLAN
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COORDINATE SYSTEM: M.G.A. | HEIGHT DATUM: A.H.D.

GOSFORD CITY COUNCIL

RIP ROAD, BLACKWALL
PROPOSED ROCK RETAINING WALL CONSTRUCTION DETAIL

SHEET 3 OF 11

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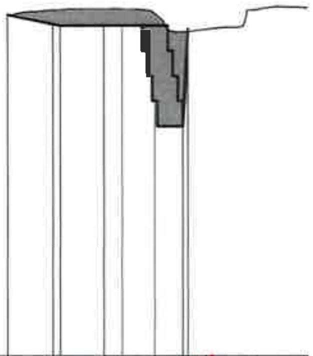

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NATURAL LEVEL	1.71	1.81	1.83	1.735	1.38	1.42	1.81	1.85
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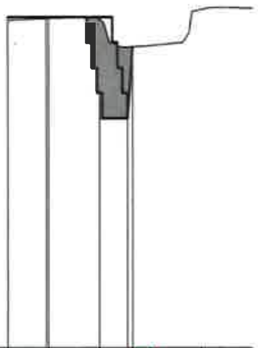

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DESIGN LEVEL	1.50	2.06 1.50	0.000	0.92	0.40		
NATURAL LEVEL	1.44	1.48	1.465	0.92	0.94	1.65	1.67
OFFSET	-1.80	-1.00	0.00	0.65	1.00	2.00	3.00

CHAINAGE 02.00

NOTE: REMOVE AND REPLACE EXISTING TIMBER WALL AND RETURN ON WEST SIDE OF STEPS, BUT LEAVE STEPS IN PLACE

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BASE SCALE FOR A3 PLAN
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COORDINATE SYSTEM: M.G.A. HEIGHT DATUM: A.H.D.

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GOSFORD CITY COUNCIL
RIP ROAD, BLACKWALL
PROPOSED ROCK RETAINING WALL CONSTRUCTION
SECTION

SHEET 4 OF 11

REVISION: -
ECM IR No:
PROJECT / CONTRACTS No: P283

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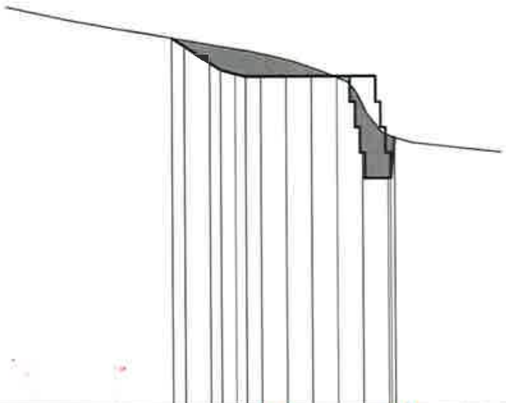

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DESIGN LEVEL					2.26	1.77	1.500	1.50	2.05 1.50	0.000	0.30	0.35		
NATURAL LEVEL	2.88	2.66	2.47	2.30	2.26	2.13	2.009	1.95	1.67	0.945	0.30	0.20	0.09	
OFFSET	-7.00	-6.00	-5.00	-4.00	-3.75	-3.00	-0.65	-2.00	-1.00	0.00	0.63	1.00	2.00	3.00

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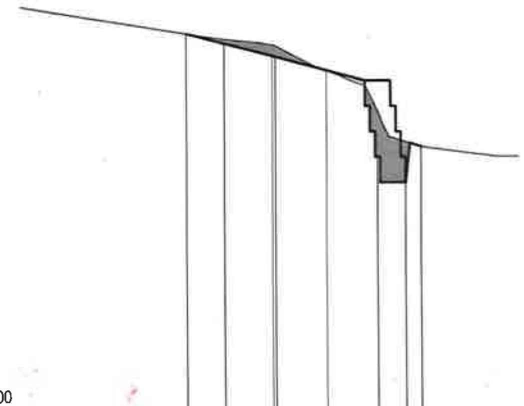

DATUM: -5.00

DESIGN LEVEL		2.421	1.988	2.05 0.000	0.215 0.31
NATURAL LEVEL	2.62	2.421	2.201	0.903	0.216
OFFSET	-5.00	-3.75	-2.06	0.00	0.85

CHAINAGE 06.00

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PROJECT / CONTRACTS No: P283

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BASE SCALE FOR A3 PLAN
1:100
COORDINATE SYSTEM: M.G.A. HEIGHT DATUM: A.H.D.

GOSFORD CITY COUNCIL

GOSFORD CITY COUNCIL
RIP ROAD, BLACKWALL
PROPOSED ROCK RETAINING WALL CONSTRUCTION
LAYOUT

SHEET 5 OF 11

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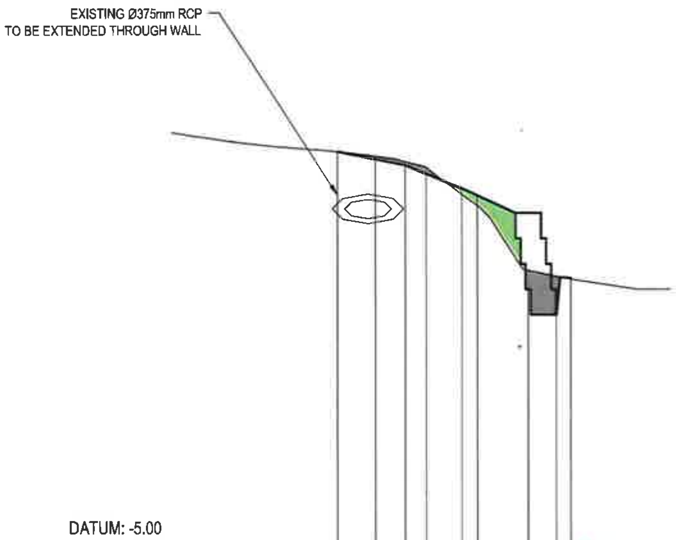

DATUM: -5.00

DESIGN LEVEL		2.701	2.440	1.985	2.05 1.500	0.000	0.218 0.28
NATURAL LEVEL	2.81	2.701	2.510	1.867	0.378	0.259	0.200
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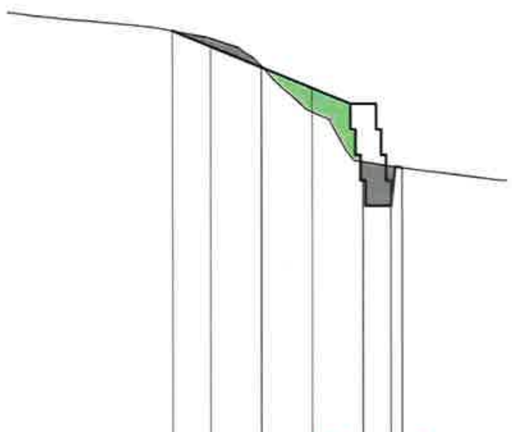
CHAINAGE 08.00



DATUM: -5.00

DESIGN LEVEL					2.93	2.63	2.22	1.81	2.04 0.00	0.24	0.26		
NATURAL LEVEL	3.29	3.18	3.09	2.98	2.93	2.79	2.25	1.35	0.37	0.25	0.22	0.09	
OFFSET	-7.00	-6.00	-5.00	-4.00	-3.75	-3.00	-2.00	-1.00	0.00	0.77	1.00	2.00	3.00

CHAINAGE 10.00

REVISION: -

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PROJECT / CONTRACTS No: P283

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BASE SCALE FOR A3 PLAN
1:100
COORDINATE SYSTEM: M.G.A. HEIGHT DATUM: A.H.D.

GOSFORD CITY COUNCIL
RIP ROAD, BLACKWALL
PROPOSED ROCK RETAINING WALL CONSTRUCTION
LAYOUT

SHEET 6 OF 11

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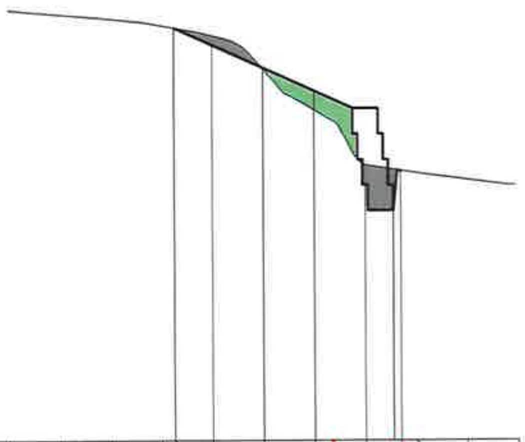
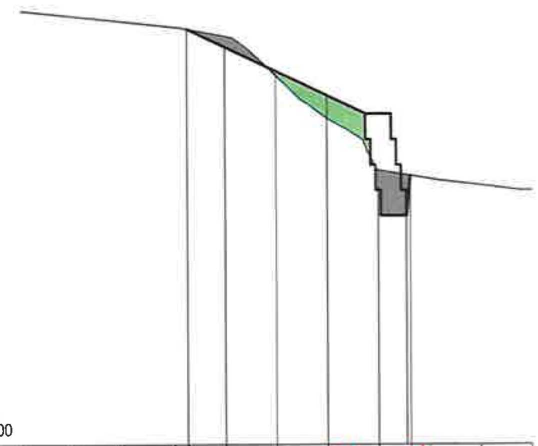


DATUM: -5.00

DESIGN LEVEL					3.08	2.74	2.29	1.84	2.04 0.00	0.27	0.25		
NATURAL LEVEL	3.44	3.34	3.26	3.13	3.08	2.92	2.28	1.47	0.39	0.29	0.24	0.11	
OFFSET	-7.00	-6.00	-5.00	-4.00	-3.75	-3.00	-2.00	-1.00	0.00	0.70	1.00	2.00	3.00

CHAINAGE 12.00



DATUM: -5.00

DESIGN LEVEL					3.17	2.82	2.34	1.86	2.04 0.000	0.29	0.24		
NATURAL LEVEL	3.54	3.43	3.32	3.21	3.17	3.03	2.25	1.42	0.397	0.29	0.23	0.10	0.00
OFFSET	-7.00	-6.00	-5.00	-4.00	-3.75	-3.00	-2.00	-1.00	0.00	0.63	1.00	2.00	3.00

CHAINAGE 14.00

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PROJECT / CONTRACTS No: P283

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BASE SCALE FOR A3 PLAN
1:100
COORDINATE SYSTEM: M.G.A. HEIGHT DATUM: A.H.D.

GOSFORD CITY COUNCIL

GOSFORD CITY COUNCIL
RIP ROAD, BLACKWALL
PROPOSED ROCK RETAINING WALL CONSTRUCTION
LAYOUT

SHEET 7 OF 11

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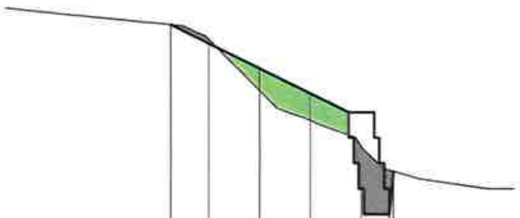

DATUM: -5.00

DESIGN LEVEL					3.22	2.86	2.36	1.87	2.04 0.000	0.34	0.24		
NATURAL LEVEL	3.55	3.44	3.35	3.25	3.22	2.94	1.94	1.34	0.820	0.34	0.24	0.07	0.00
OFFSET	-7.00	-6.00	-5.00	-4.00	-3.75	-3.00	-2.00	-1.00	0.00	0.63	1.00	2.00	3.00

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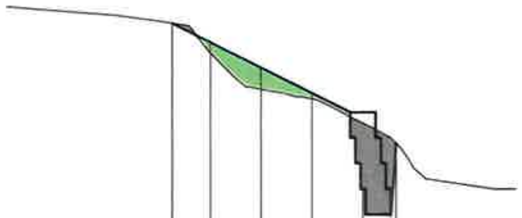

DATUM: -5.00

DESIGN LEVEL					3.25	2.87	2.38	1.88	2.05 0.00	0.90	0.24		
NATURAL LEVEL	3.57	3.49	3.41	3.28	3.25	2.67	1.95	1.77	1.29	0.90	0.40	0.09	0.00
OFFSET	-7.00	-6.00	-5.00	-4.00	-3.75	-3.00	-2.00	-1.00	0.00	0.65	1.00	2.00	3.00

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REVISION: -

ECM IR No:

PROJECT / CONTRACTS No: P283

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PROJECT IMPLEMENTATION UNIT
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GOSFORD CITY COUNCIL

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BASE SCALE FOR A3 PLAN
1:100
COORDINATE SYSTEM: M.G.A. | HEIGHT DATUM: A.H.D.

GOSFORD CITY COUNCIL
RIP ROAD, BLACKWALL
PROPOSED ROCK RETAINING WALL CONSTRUCTION
LAYOUT

SHEET 8 OF 11

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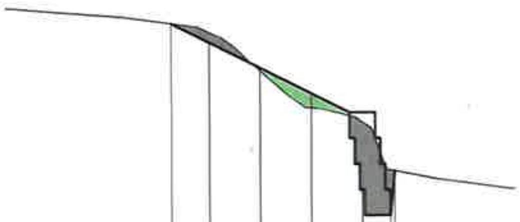

DATUM: -5.00

DESIGN LEVEL					3.27	2.89	2.39	1.88	2:05 0.00	0.37	0:24		
NATURAL LEVEL	3.59	3.51	3.42	3.30	3.27	3.09	2.33	1.59	1.31	0.37	0.30	0.14	0.00
OFFSET	-7.00	-6.00	-5.00	-4.00	-3.75	-3.00	-2.00	-1.00	0.00	0.64	1.00	2.00	3.00

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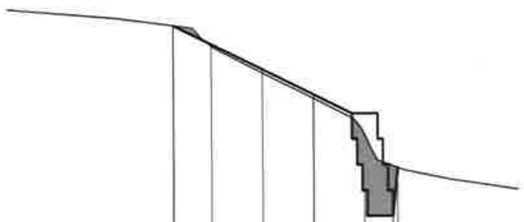

DATUM: -5.00

DESIGN LEVEL					3.25	2.88	2.38	1.88	2:06 0.00	0.45	0:24		
NATURAL LEVEL	3.58	3.50	3.41	3.29	3.25	2.85	2.31	1.60	1.08	0.45	0.35	0.16	0.01
OFFSET	-7.00	-6.00	-5.00	-4.00	-3.75	-3.00	-2.00	-1.00	0.00	0.64	1.00	2.00	3.00

CHAINAGE 22.00

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PROJECT / CONTRACTS No: P283

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PROJECT IMPLEMENTATION UNIT
INFRASTRUCTURE PLANNING
GOSFORD CITY COUNCIL

RECOMMENDED-

APPROVED-

BASE SCALE FOR A3 PLAN

1:100

COORDINATE SYSTEM: M.G.A. HEIGHT DATUM: A.H.D.

GOSFORD CITY COUNCIL

RIP ROAD, BLACKWALL
PROPOSED ROCK RETAINING WALL CONSTRUCTION
LAYOUT

SHEET 9 OF 11

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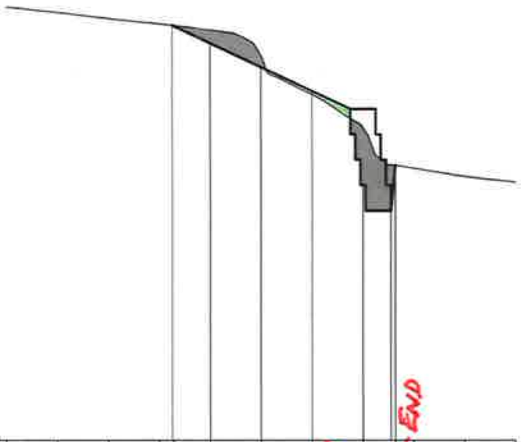

DATUM: -5.00

DESIGN LEVEL					3.16	2.80	2.33	1.86	2·07 0.000	0.40	1·94 END		
NATURAL LEVEL	3.52	3.41	3.29	3.18	3.16	3.06	2.54	1.80	1.133	0.40	0.34	0.17	0.08
OFFSET	-7.00	-6.00	-5.00	-4.00	-3.75	-3.00	-2.00	-1.00	0.00	0.64	1.00	2.00	3.00

CHAINAGE 24.00

REVISION: -

ECM IR No:

PROJECT / CONTRACTS No: P283

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PROJECT IMPLEMENTATION UNIT
INFRASTRUCTURE PLANNING
GOSFORD CITY COUNCIL

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APPROVED-

BASE SCALE FOR A3 PLAN

1:100

COORDINATE SYSTEM: M.G.A. HEIGHT DATUM: A.H.D.

GOSFORD CITY COUNCIL

RIP ROAD, BLACKWALL
PROPOSED ROCK RETAINING WALL CONSTRUCTION
LAYOUT

SHEET 10 OF 11

Total Volume Table							
Chainage	Cut Area	Fill Area	Cut Vol	Fill Vol	Cum Cut Vol	Cum Fill Vol	Net Vol
1.000	2.145m2	0.000m2	0.000m3	0.000m3	0.000m3	0.000m3	0.000m3
2.000	1.129m2	0.044m2	1.637m3	0.022m3	1.637m3	0.022m3	1.615m3
2.774	1.540m2	0.000m2	0.556m3	-0.034m3	2.193m3	-0.012m3	2.205m3
4.000	1.698m2	0.020m2	1.985m3	0.012m3	4.178m3	0.000m3	4.178m3
6.000	0.951m2	0.041m2	2.648m3	0.061m3	6.827m3	0.061m3	6.766m3
8.000	0.589m2	0.492m2	1.550m3	0.533m3	8.376m3	0.594m3	7.783m3
10.000	0.703m2	0.806m2	1.302m3	1.298m3	9.678m3	1.892m3	7.786m3
12.000	0.747m2	0.689m2	1.449m3	1.495m3	11.127m3	3.387m3	7.740m3
14.000	0.713m2	0.692m2	1.460m3	1.380m3	12.587m3	4.768m3	7.819m3
16.000	0.756m2	1.104m2	1.469m3	1.795m3	14.056m3	6.563m3	7.493m3
18.000	1.169m2	0.769m2	1.925m3	1.872m3	15.981m3	8.435m3	7.546m3
20.000	1.191m2	0.352m2	2.360m3	1.120m3	18.342m3	9.555m3	8.786m3
22.000	0.872m2	0.197m2	2.064m3	0.548m3	20.405m3	10.104m3	10.302m3
24.000	1.288m2	0.134m2	2.160m3	0.331m3	22.565m3	10.434m3	12.131m3

NOTE: APPROXIMATE VALUES ONLY, NO ALLOWANCE FOR COMPACTION, IMPORTED MATERIAL, ETC

REVISION: -

ECM IR No:

PROJECT / CONTRACTS No: P283

1	FOR APPROVAL	DB		26/03/2015
	REVISION DETAILS	DRAWN	APPR	DATE

REFERENCE FILES
SURVEY: ECMIR-13938536
DESIGN:
SURVEYED: RM
DESIGNED: CD
CERTIFIED:

PREPARED BY-
PROJECT IMPLEMENTATION UNIT
INFRASTRUCTURE PLANNING
GOSFORD CITY COUNCIL

RECOMMENDED-

APPROVED-

BASE SCALE FOR A3 PLAN
1:100
COORDINATE SYSTEM: M.G.A. HEIGHT DATUM: A.H.D.

GOSFORD CITY COUNCIL

GOSFORD CITY COUNCIL
RIP ROAD, BLACKWALL
PROPOSED ROCK RETAINING WALL CONSTRUCTION
LAYOUT

SHEET 11 OF 11

Appendix C

Site survey by Stephen Thorne and Associates

STEPHEN THORNE & ASSOCIATES
POSTAL ADDRESS: PO BOX 1315, GOSFORD NSW 2250 TEL: (02) 4323 1255
No. 3/127 ERINA STREET, GOSFORD NSW 2250
registered surveyors
TSS Total Surveying Solutions Pty Ltd
A.C.N. 603 458 546
A. B.N. 53 603 458 546
Email: stsurvey@bigpond.net.au

Our ref: 190353
Your ref:

5 March 2019

Royal Haskoning DHV
Level 14, 56 Berry Street
NORTH SYDNEY NSW 2060

Attn: Gary Blumberg
Technical Director, Coastal Maritime & Aviation

Re: Detail & Contour Survey Of The Rip Road Reserve, Blackwall.

Dear Gary,

Further to your instructions we have surveyed part of the Rip Road Reserve at Blackwall in the Local Government Area of Central Coast.

We have connected our survey to the Australian Height Datum and the Map Grid of Australia co-ordinate system and located all relevant topographical and physical features on and adjacent to the subject land as specified in your email correspondence and site instructions.

A detail and contour plan has been prepared at a scale of 1:200 suitable for design purposes and as a base plan for submission to Council with a development application.

We advise that we have forwarded an electronic copy of the survey plan in a DWG file and PDF copy to you by email.

We thank you for your instructions in this matter and now enclose our tax invoice for your attention.

Yours faithfully,
STEPHEN THORNE & ASSOCIATES
Per:

Stephen Thorne

Stephen R Thorne
Surveyor Registered Under
The Surveying and Spatial Information Act 2002

KEY

—w—w—w—	DENOTES COUNCIL WATER MAIN
—s—s—s—	DENOTES COUNCIL SEWER MAIN
—T—T—T—	DENOTES TELSTRA CABLES (SCALED LOCATION ONLY)
—E—E—E—	DENOTES AERIAL ELECTRICAL CABLES
●	DENOTES POWER POLE

CAUTIONS :

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

www.dialbeforeyoudig.com.au
DIAL 1100
BEFORE YOU DIG

CAD FILE: 190353_DETAIL.dwg
THIS PLOT: 04-03-2019 08:13
REFERENCE: 190353.000

SCALE: 1:200 @ A1 SHEET

DATUM: Australian Height Datum

DATE
04/03/2019

STEPHEN THORNE AND ASSOCIATES
TSS Total Surveying Solutions Pty Ltd
3/127 ERINA STREET, GOSFORD. 2250
Phone (02)4323-1255 Email stsurvey@bigpond.net.au
P.O. BOX 1315 GOSFORD NSW 2250
A.C.N. 603 458 546 A.B.N. 53 603 458 546

DETAIL AND CONTOUR SURVEY OVER
LOT 94 D.P. 6327
RIP ROAD RESERVE RIP ROAD,
BLACKWALL

ORIGIN OF LEVEL SSM 52182 RL=9.039 AHD

4 0 4 8 12 16 20
SCALE OF METRES 1:200

amendments

PLAN NUMBER
190353

ISSUE
A

SHEET 1 OF 1 SHEETS

Appendix D

Geotechnical investigation by JK Geotechnics

REPORT TO

ROYAL HASKONINGDHV

ON

GEOTECHNICAL INVESTIGATION

FOR

PROPOSED FORESHORE REHABILITATION

AT

RIP ROAD RESERVE, BLACKWALL, NSW

Date: 18 June 2019

Ref: 32217Rrpt

JKGeotechnics

www.jkgeotechnics.com.au

Date: 18 June 2019
Report No: 32217Rrpt
Revision No: 0

Report prepared by:
Paul Roberts
Principal Associate | Engineering Geologist

For and on behalf of
JK GEOTECHNICS
PO BOX 976
NORTH RYDE BC NSW 1670

DOCUMENT REVISION RECORD

Report Reference	Report Status	Report Date
32217Rrpt	Draft Report	18/6/19

This report (which includes all attachments and annexures) has been prepared by JK Geotechnics (JKG) for its Client, and is intended for the use only by that Client.

This Report has been prepared pursuant to a contract between JKG and its Client and is therefore subject to:
a) JKG's proposal in respect of the work covered by the Report;
b) The limitations defined in the Client's brief to JKG;
c) The terms of contract between JK and the Client, including terms limiting the liability of JKG.
If the Client, or any person, provides a copy of this Report to any third party, such third party must not rely on this Report, except with the express written consent of JKG which, if given, will be deemed to be upon the same terms, conditions, restrictions and limitations as apply by virtue of (a), (b), and (c) above.

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Table of Contents

1 INTRODUCTION

This report presents the results of a geotechnical investigation for the proposed foreshore rehabilitation at Rip Road Reserve, Blackwall. A site location plan is presented as Figure 1. The investigation was commissioned by Gary Blumberg (Royal HaskoningDHV [RHDHV]) in an email dated 8 February 2019. The commission was on the basis of our fee proposal (Ref P48398R) dated 12 November 2018.

We have been provided with the following information:

- A site survey plan (Ref: 190353[A] dated 4 March 2019) prepared by Stephen Thorne and Associates. The survey datum is the Australian Height Datum (AHD).
- A copy of the survey plan annotated by RHDHV.
- Sketch sections of the proposed foreshore rehabilitation works prepared by RHDHV dated 29 May 2019.

Based on a review of the provided information, we understand that the foreshore rehabilitation works will include:

- A western portion (46m long) comprising a sandstone block seawall supported on a concrete footing founded at RL -0.48m AHD. The crest of the seawall will be formed at RL2m AHD. The sandstone blocks will be 1.8m x 0.5m x 0.5m size. The seawall will be constructed within temporary batters formed at 1 Vertical (V) in 2 Horizontal (H), and the landward side of the excavation will be supported with a temporary cantilever shoring system designed for a maximum vertical retained height of 1.5m. The maximum excavation depth will be about 2.0m below the reserve surface and about 0.6m below the foreshore surface. Seawall backfill will comprise 'broken and demolished sandstone' with blue metal gravel drainage wrapped in Texcel 400R with '*general fill to reinstate the remainder*' of the excavated foreshore slope with the final retained surface formed at about 15°.
- An eastern portion (37m long) comprising a basalt rock revetment situated immediately seaward of the aboriginal midden site. The rock revetment will be formed with a seaward face sloping typically at 1V in 1.5H, and a crest level of RL2.1mAHD. The revetment will be formed over the existing foreshore slope and undulations in the slope profile will be infilled with engineered fil (less than 2% fines and maximum particle size less than 0.15m) to form flat the basal profile of the underlayer. The interface between the original foreshore slope and the revetment materials will be covered with TEXCEL 600R. The underlayer (maximum 0.27m thick) will comprise of 6kg to 45kg rocks (0.15m to 0.3m size), and the armour layer (0.59m thick) will comprise 45kg to 150kg rocks (0.3m to 0.5m size). The base of the seaward end of the revetment will be formed at a design scour level of about RL -0.4mAHD.
- New stone steps (3.0m wide) will be provided at the eastern end of the revetment and towards the eastern end of the seawall. In addition, a 'dinghy skid' and steps will be provided at the western end of the seawall.

The purpose of the investigation was to obtain geotechnical information on the subsurface conditions as a basis for comments and recommendations on excavation, temporary cut batters, seawall and temporary shoring design parameters, footing design, earthworks and drainage.

2 INVESTIGATION PROCEDURE

The fieldwork for the investigation was carried out on 28 February 2019 and was limited by access constraints to the use of portable hand-held equipment. The fieldwork comprised:

- Four boreholes (BH1, BH3, BH5 and BH6) hand auger drilled to refusal depths between 1.0m and 2.4m (BH1, BH3 and BH5) and to 1.0m depth (BH6). BH1 and BH3 were situated in the reserve and BH5 and BH6 were situated along the foreshore at the toe of the slope.
- Six Dynamic Cone Penetration (DCP) tests (DCP1 to DCP6) carried out adjacent to the boreholes and at two additional locations. The DCP tests were extended to refusal depths between about 0.7m and 3.0m (DCP1 to DCP4) and to depths of about 1.4m (DCP5) and 2.0m (DCP6).

Prior to the commencement of the fieldwork, the test locations were scanned for the presence of buried services by a specialist sub-contractor.

The test locations are shown on the attached Figure 2 and were set out using a handheld GPS device. Figure 2 is based on aerial imagery sourced from '*Nearmap*'. The approximate surface RL's at the test locations were interpolated between spot levels shown on the provided survey plan.

The compaction of the fill, strength of the natural silty clays and relative density of the natural sands were assessed from the DCP blow counts, augmented by hand penetrometer tests on cohesive soil samples recovered from the hand auger. The refusal depth of the DCP tests can also provide an indicative depth to bedrock, though we note that refusal can also occur on obstructions in fill, 'floaters' and other hard layers.

Groundwater observations were made in the boreholes during and on completion of hand auger drilling. No longer-term groundwater monitoring has been carried out.

Further details of the methods and procedures employed in the investigation are presented in the attached Report Explanation Notes.

The fieldwork for the investigation was carried out under the direction of our geotechnical engineer (Joel Babbage) who was present full-time on site, and set out the test locations, directed the buried services scan, logged the encountered subsurface profile, and nominated in-situ testing and sampling. The borehole logs (which include groundwater observations) and the DCP test results sheets are attached, together with a glossary of logging terms and symbols used.

Geotechnical laboratory testing was not carried out as it was deemed unnecessary.

JK Environments, our specialist environmental division, have completed an Acid Sulfate Soil Assessment Report (Ref: E32217BTlet_ASS, dated 17 April 2019) and an Acid Sulfate Soil Management Plan (Ref. E32217BTlet_ASSMP, dated 17 April 2019). These reports should be read in conjunction with this report.

3 RESULTS OF INVESTIGATION

3.1 Site Description

The site is located at the base of a hillside that slopes down to the north at a maximum of 15° to a portion of the southern foreshore of Brisbane Water. The western end of the site is accessed from the northern end of Rip Road and we understand that an Aboriginal midden is located over the eastern end of the site.

At the time of the fieldwork the site comprised a grass surfaced reserve area that sloped down to the north at between about 10° and 30° and had a stepped profile with a moderately steep portion situated mid-slope. The reserve surface extended east and west beyond the site boundaries.

The toe of the foreshore slope was supported by generally dilapidated timber, sandstone and brick seawalls (typically 1.0m height) with intermittent sandstone and brick steps. At some locations, sections of timber seawall appeared to be in good condition and have been assumed to represent areas of relatively recent seawall repair.

Over selected sections of the foreshore slope, where seawall collapse had occurred, geofabric had been placed over the sandy soil slope. Small to large trees were scattered across the site (particularly over the eastern end) and occasional trees where located at the toe of the foreshore slope adjacent to sections of collapsed seawall. A sandy beach lined the toe of the foreshore seawalls and gently sloped down to the north.

The southern site boundary was generally formed by concrete block, sandstone masonry, concrete and rendered retaining walls (maximum height about 1.0m) which supported the seaward ends of the neighbouring landscaped yard area. Neighbouring one and two storey brick and timber houses were set-back at least 4.0m from the southern site boundary. Based on a cursory inspection from within the site, the neighbouring buildings and structures were generally in reasonably good condition.

3.2 Subsurface Conditions

The 1:100,000 geological map of Sydney indicates the site is underlain by Quaternary age quartz sand with minor shell content, and interdune silt and fine sand Beach ridge system deposits. The boreholes have disclosed a generalised subsurface profile comprising fill overlying marine sands and sandy clays followed by natural clays. Reference should be made to the attached borehole logs for specific details at each location. A summary of the pertinent subsurface characteristics is presented below.

Fill

Silty clayey sand fill was encountered in BH1 and BH3 and extended to respective depths of 0.5m and 0.7m below the reserve surface. Based on the DCP test results the fill was assessed to be poorly compacted. Assuming a similar subsurface profile below the reserve surface, the results of DCP2 have been interpreted to indicate similar poorly compacted sandy fill.

Marine Soils Sands and Sandy Clays
The marine soils comprised a banded sequence of sands and clays were encountered below the fill in BH1 and BH3 and from the surface in BH5 and BH6.

In the landward boreholes (BH1 and BH3) the clay, sandy clay and silty clay were assessed to be of variable plasticity (low to high) and of firm or stiff to very stiff strength. The results of DCP1 and DCP3 indicated that the relative density of the sands was loose to medium dense. Assuming a similar subsurface profile, the results of DCP2 have been interpreted to indicate a similar banded sequence of sands and clays. Refusal??

In the foreshore boreholes (BH5 and BH6) the clay and sandy clay were respectively assessed to be of high and low to medium plasticity. The clays in BH5 were of stiff strength and the clays in BH6 were initially very soft strength improving to firm strength with depth. The results of DCP5 and DCP6 indicated that the relative density of the sands was very loose or loose to medium dense. Assuming a similar subsurface profile, the results of DCP4 have been interpreted to indicate a similar banded sequence of sands and clays.

The refusal of the DCP tests at depths between 1.8m and 3.0m below the reserve surface (DCP1 to DCP3) and depths between 0.7m and 2.0m below the below the foreshore surface (DCP4 to DCP6) have been interpreted to indicated poorly cemented sands although the presence of 'floaters' and/or fragments of previously collapsed seawalls (particularly below the foreshore surface) cannot be discounted. Based on the topographic setting of the site we consider it unlikely that DCP refusal indicates bedrock.

Groundwater
The landward boreholes (BH1 and BH3) were 'dry' during, and on completion of, hand auger drilling. In the boreholes along the foreshore (BH5 and BH6), standing water levels were recorded at respective depths 0.3m and 0.7m on completion of hand auger drilling. In addition, BH5 collapsed to 0.7m depth on completion of hand auger drilling. The standing water levels corresponded well with the tidal water level. No longer-term groundwater monitoring has been carried out.

4 COMMENTS AND RECOMMENDATIONS

4.1 Site Preparation

4.1.1 General

The works will need to be completed using suitably experienced (and insured) contractors and supervised by a suitably qualified engineer.

Prior to works commencing, consideration should be given to preparing detailed dilapidation reports on the seaward sides of the private residential properties forming the southern boundary of the reserve. The property owners should be asked to confirm that the reports present a fair record of existing conditions as the reports may assist the clients in pursuing any claims against the contractor for damage.

With regard to the JK Environments reports we note the following:

- During the site inspection, fibre cement fragments (FCF) were encountered along the length of the retaining wall and foreshore area. It was recommended that an emu-bob was conducted to remove the FCF from the surface of the site and any fill to be removed/excavated as part of the rehabilitation works be classified in accordance with the NSW EPA Waste Classification Guidelines - Part 1: Classifying Waste (2014)[1] and disposed off-site.
- The natural soils at the site were considered to be Potential Acid Sulfate Soils (PASS) and an Acid Sulfate Soil Management Plan (ASSMP) is required to manage the soils during the works.

The JK Environments reports should be referred to for further advice.

4.1.2 Excavation Conditions

Excavation recommendations provided below should be completed by reference to the Safe Work Australia Code of Practice *'Excavation Work'*, dated July 2015.

Bulk excavations locally required to achieve design subgrade levels for the proposed seawall and revetment will extend to maximum depths of about 2.0m below the reserve surface and about 0.6m below the foreshore surface. The excavations will extend through the banded clayey and sandy soil profile. The excavations are expected to be readily completed using tracked excavators but with possible localised over-excavation to remove any obstructions. Any topsoil or root affected soils should be stripped and separately stockpiled for re-use in landscape areas as such soils are not suitable for re-use as engineered fill.

Care will need to be exercised in order to maintain the stability of the adjacent sections of existing seawalls to the east and west that will remain. We recommend that test pits be excavated close to adjacent sections of seawalls. The test pits should be inspected by the geotechnical, coastal and structural engineers, in order to confirm footing details, foundation materials and the nature and form of temporary and permanent support measures.

4.1.3 Potential Ground Surface Movements

Due to the presence of poorly compacted fill and loose/very loose natural sands, which we expect will extend across the general area, we advise that sudden stop/start movements of tracked excavators and dropping of items causing ground impacts should be avoided in order to reduce transmission of ground vibrations to the adjacent sections of buildings and structures neighbouring the site.

4.1.4 Groundwater Seepage and Tidal Levels

Groundwater inflow is expected within the excavations within the soil profile, due to tidal fluctuations. Consideration of appropriate sequencing of the works in relation to tidal levels will be required.

[1] NSW EPA, (2014). *Waste Classification Guidelines, Part 1: Classifying Waste.* (referred to as Waste Classification Guidelines 2014)

In general, we expect any groundwater inflows to be of small volume and managed by infiltration into the generally sandy subgrade. Inspection and monitoring of groundwater seepage during excavations is recommended, so that any unexpected conditions, which may be revealed, can be incorporated into the drainage design.

The Highest Astronomical Tide (HAT) is about RL0.64mAHD and excavations over the toe area of the proposed revetment and seawall will extend below tidal water levels and some instability can be expected; further advice is presented in Section 4.2.1 below.

4.2 Temporary Batter Slopes and Retention

4.2.1 General

The proposed temporary excavation batters of 1V in 2H detailed on the sketches are considered feasible for the sandy soils above the groundwater levels. Where clayey soils are encountered, steeper temporary excavation batters of 1V in 1H are appropriate. These temporary batter slopes are only expected to be accommodated over the landward and seaward sides of the proposed work and may undermine the adjacent sections of seawalls to the east and west that will remain. Further advice is provided in Section 4.2.2 below.

To facilitate backfilling (and proof rolling) in order to reinstate the areas landward of the proposed revetment and seawall, we recommend that the landward batter slope be formed with a stepped profile (within an overall slope of 1V in 2H) to facilitate the use of compaction equipment; see Section 4.3 below.

We note that the bulk excavations over the seaward side of the works will extend below the tidal groundwater level and will affect the stability of the excavation sides. Allowance should be made for use of sand bags to support temporary batters close to, and below, the groundwater levels.

4.2.2 Temporary Retention and Shoring

The provided sketches indicate that the landward side of temporary excavations for the proposed seawall works will require "*temporary shoring*" with a maximum retained height of 1.5m. The form of the temporary shoring has not been detailed and may comprise engineer designed cantilever sheet piles or a contiguous piled wall. The piles would need to extend to sufficient depth below bulk excavation level (BEL) to satisfy stability considerations.

We forewarn that there are likely to be potentially damaging vibrations associated with installation of sheet piles particularly for adjacent high level footings founded in the sandy soils, and they may not be preferred. Further advice from the contractor will be required in this regard if sheet piles are proposed.

The potentially collapsible nature of the sandy soils, particularly where groundwater seepage is encountered, could cause adjacent ground surface movements which extend beyond the site boundaries. However, collapse could occur before they were installed and as such we do not recommend their use. Contiguous bored piles drilled using hand auger methods or an auger attachment could be used but temporary liners would be required.

A grout injected (CFA) contiguous piled wall could also be considered. Decompression of the sandy soils may occur if any groundwater is encountered, and a site trial in the centre of the site would need to be undertaken under the direction of a geotechnical engineer to assess potential decompression. Alternatively, decompression effects would be satisfactorily controlled by using double rotary CFA piles, which includes a casing system to support the drill hole.

With regard to contiguous piles, allowance must be made for making good gaps between the piles in order to reduce the loss of retained soils and consequent inducement of adjacent ground surface movements.

With regard to the adjacent sections of existing seawall to the east and west that will remain, measures such as localised sand bagging, temporary propping using stiff formwork timbers and props, underpinning footings etc may be suitable. This can be better assessed following inspection of the test pits described in Section 4.1.2 above.

4.2.3 Retention and Shoring Design Parameters

The following earth pressure coefficients and subsoil parameters may be adopted for the design of the seawall, temporary retention measures and/or any underpins supporting a soil profile:

- For design of temporary shoring (propped formwork, cantilever sheet piles, piled walls etc) and/or any underpins supporting a soil profile, we recommend the use of a triangular lateral earth pressure distribution with an 'at rest' earth pressure coefficient (k_o) of 0.55 for the retained profile, assuming a horizontal backfill surface.
- For design of the seawall, we recommend the use of a triangular lateral earth pressure distribution with an 'active' earth pressure coefficient (k_a) of 0.4 for the retained profile, assuming a backfill surface sloping at a maximum of 15^o.
- A bulk unit weight of $20kN/m^3$ and $10kN/m^3$ should respectively be adopted for the retained profile above and below groundwater level.
- Any surcharge affecting the walls (e.g. nearby footings, compaction stresses, construction loads etc) should be allowed for in the design using the above appropriate earth pressure coefficient.
- The seawall should be designed as drained and provision made for permanent and effective drainage of the ground behind the wall. We note that the provided sketches detail blue metal gravel drainage wrapped in Texcel 400R, to act as a filter against subsoil erosion. The subsoil drains should discharge into the stormwater system.
- Underpins supporting a soil profile (if required) and piled walls (if adopted) may need to be designed as permanently drained and further advice can be provided following inspection of test pits at the affected property boundaries. Any drainage would need to be appropriately discharged. Drainage would be expected to comprise PVC pipes should be installed at nominal 1.2m horizontal spacing just above the adjacent surface level. Holes will need to be drilled to allow installation of the pipes and/or use of gaps between contiguous piles. The end of the pipe penetrating the retained soils behind the retention system

must be wrapped in a non-woven geotextile fabric, such as Bidim A34, to act as a filter against subsoil erosion.
- Lateral restraint of temporary cantilever piles founded in the soil profile below adjacent surface levels and the seawall founded below foreshore level may be provided by the passive pressure of the soil below these levels. A 'passive' earth pressure coefficient, K_p, of 1.5 may be adopted, using a triangular pressure distribution and provided a Factor of Safety of at least 2 is used in order to reduce the high deflections that are associated with achieving a full passive case.

4.3 Earthworks

The following earthworks recommendations should be complemented by reference to AS3798–2007 "Guidelines on Earthworks for Commercial and Residential Developments".

4.3.1 Subgrade Preparation

Prior to placement of fill to reinstate the reserve area, preparation of the soil subgrade should consist of the following:

- Following completion of bulk excavations and installation of the revetment and seawall, proof rolling may be completed using a vibrating plate compactor (attached to an excavator or hand held) or, if space permits, with at least eight passes of a static (non-vibratory) smooth drum roller of at least 2 tonnes deadweight. The sandy subgrade should be thoroughly moistened prior to proof rolling.
- The final pass of proof rolling should be carried out under the direction of an experienced geotechnical engineer for the detection of unstable or soft areas which should be removed and replaced with engineered fill (if required), as outlined in Section 4.3.3 below. In some instances a bridging layer may be required and further advice is provided in section 4.3.3 below.
- Care should also be taken when using vibrating equipment not to cause damage to any adjacent structures. The vibrations should be qualitatively monitored by site personnel. If there is any cause for concern then proof-rolling should cease and further advice sought. Alternatively, where appropriate, the static (non-vibration) mode may be used.
- Sections of clay subgrade that contain shrinkage cracks should be lightly watered and rolled until the shrinkage cracks disappear.

4.3.2 Subgrade Drainage During Construction

Clayey subgrades may be found to be unstable/soft if proper site drainage is not maintained during construction. It is therefore important to provide good drainage in order to promote run-off and reduce ponding. Earthworks platforms should be graded to maintain cross-falls during construction. If the clays are exposed to periods of rainfall, softening may result and site trafficability will be poor. If softening occurs, the subgrade should be over-excavated to below the depth of moisture softening, and replaced with engineered fill (see below). Trafficability may be improved by the use of a sacrificial surface layer of crushed demolition rubble.

4.3.3 Engineered Fill

Fill required to reinstate reserve areas and unstable areas of subgrade should comprise engineered fill.

Engineered fill suitable for use as '*General Fill*' as noted on the provided sketches should be free from organic materials, other contaminants and deleterious substances and have a maximum particle size not exceeding 40mm. The expected sands and clays sourced from the excavations will be suitable for engineered fill, provided they are thoroughly mixed, with any coarse gravel and cobble size material removed. However, any very soft to soft and/or over wet clays will not be suitable for use as engineered fill and must be separately stockpiled. We reiterate the warning regarding the presence of FCF in Section 4.1.1 above.

The engineered fill should be compacted using the above mentioned roller in layers of maximum 100mm loose thickness to a density between 98% and 102% of Standard Maximum Dry Density (SMDD) and within 2% of their Standard Optimum Moisture Content (SOMC). The density may be reduced to 95% of SMDD if the designer considers that settlement of the reserve surface can be tolerated.

Backfill to the seawall should also comprise engineered fill. The provided information indicates that such backfill will comprise '*broken and demolished sandstone*'. The backfill should be well graded following crushing of the sandstone. Alternatively, well graded imported granular materials such as demolition rubble would be suitable for this purpose. The retaining wall backfill materials must also free of deleterious substances and has a maximum particle size not exceeding 40mm. It is expected that at least a portion of this material will require importing. Such graded granular fill should be compacted to at least 98% of Standard Maximum Dry Density (SMDD) and within 2% of their Standard Optimum Moisture Content (SOMC). The compaction requirement may be reduced to 95% of SMDD, where settlement can be tolerated. Such fill should be compacted in horizontal layers as described above. Care will be required to ensure excessive compaction stresses are not transferred to the seawall.

The engineered fill required to infill existing foreshore slope undulations and form flat the basal profile of the revetment underlayer has been detailed to comprise material with less than 2% fines and a maximum particle size less than 0.15m placed over TEXCEL 600R geotextile. Our recommendation for such select granular material is to use a well graded and high strength material, such as crushed concrete, igneous rock, steel furnace slag or high strength sandstone, with a maximum particle size not exceeding 75mm, and with less than 10% by weight passing a 0.075mm sieve. A single layer of maximum 0.4m thickness may be placed and compacted using a large static smooth drum roller (say 12 tonne size) or by tracking with a large excavator. Following compaction the surface should be inspected by a geotechnical engineer.

If areas of poor subgrade are exposed then a bridging layer comprising a 0.3m thick layer of coarse gravel and cobbles (of 75mm to 300mm nominal size) should then be placed on the exposed base and pushed into the very soft clay/very loose sands with the bucket of a large tracked excavator, say of at least 20 tonne size. This material must be angular, of high strength such as crushed igneous rock, concrete or high strength sandstone, and must be well graded, subsequent layers should be added and pushed in until no further penetration occurs. A high strength woven geotextile such as Mirafi PET 200-50 should then be placed over this prepared surface, then the well graded granular fill described above then placed over the bridging layer.

The poor subgrade areas could be over-excavated and replaced with engineered fill but this would be difficult in the foreshore environment, particularly below the water level.

Density tests should be carried out at the frequencies outlined in AS3798. At least Level 2 testing of earthworks should be carried out in accordance with AS3798. Any areas of insufficient compaction will require reworking.

4.4 Seawall Foundation Soils

We expect very loose and loose sands and very soft and soft clays to be present at the base of the seawall footing excavation and they will have a limited bearing capacity (typically around 20kPa to 50kPa). Consideration may need to be given to providing a bridging layer as outlined in Section 4.3.3 above in order to provide a suitable foundation for the seawall.

Geotechnical inspection (including DCP testing) of the exposed seawall foundation will be required in order to confirm the bearing capacity of the foundation soils and need for a bridging layer.

4.5 Further Geotechnical Input

The following summarises the scope of further geotechnical work recommended within this report. For specific details reference should be made to the relevant sections of this report.

- Dilapidation surveys of neighbouring residences to the south.
- Geotechnical inspection of exposed sub-grade.
- Geotechnical inspection and DCP testing of the seawall foundation soils.
- Density testing of engineered fill and pavement materials.

5 GENERAL COMMENTS

The recommendations presented in this report include specific issues to be addressed during the construction phase of the project. As an example, special treatment of soft spots may be required as a result of their discovery during proof-rolling, etc. In the event that any of the construction phase recommendations presented in this report are not implemented, the general recommendations may become inapplicable and JK Geotechnics accept no responsibility whatsoever for the performance of the structure where recommendations are not implemented in full and properly tested, inspected and documented.

The long term successful performance of floor slabs and pavements is dependent on the satisfactory completion of the earthworks. In order to achieve this, the quality assurance program should not be limited to routine compaction density testing only. Other critical factors associated with the earthworks may include subgrade preparation, selection of fill materials, control of moisture content and drainage, etc. The satisfactory control and assessment of these items may require judgment from an experienced engineer. Such judgment often cannot be made by a technician who may not have formal engineering qualifications and experience. In order to identify potential problems, we recommend that a pre-construction meeting be held so that all parties involved understand the earthworks requirements and potential difficulties. This meeting should clearly define the lines of communication and responsibility.

Occasionally, the subsurface conditions between and below the completed boreholes and DCP tests may be found to be different (or may be interpreted to be different) from those expected. Variation can also occur with groundwater conditions, especially after climatic changes. If such differences appear to exist, we recommend that you immediately contact this office.

This report provides advice on geotechnical aspects for the proposed civil and structural design. As part of the documentation stage of this project, Contract Documents and Specifications may be prepared based on our report. However, there may be design features we are not aware of or have not commented on for a variety of reasons. The designers should satisfy themselves that all the necessary advice has been obtained. If required, we could be commissioned to review the geotechnical aspects of contract documents to confirm the intent of our recommendations has been correctly implemented.

A waste classification will need to be assigned to any soil excavated from the site prior to offsite disposal. Subject to the appropriate testing, material can be classified as Virgin Excavated Natural Material (VENM), General Solid, Restricted Solid or Hazardous Waste. Analysis takes seven to 10 working days to complete, therefore, an adequate allowance should be included in the construction program unless testing is completed prior to construction. If contamination is encountered, then substantial further testing (and associated delays) should be expected. We strongly recommend that this issue is addressed prior to the commencement of excavation on site.

This report has been prepared for the particular project described and no responsibility is accepted for the use of any part of this report in any other context or for any other purpose. If there is any change in the proposed development described in this report then all recommendations should be reviewed. Copyright in this report is the property of JK Geotechnics. We have used a degree of care, skill and diligence normally exercised by consulting engineers in similar circumstances and locality. No other warranty expressed or implied is made or intended. Subject to payment of all fees due for the investigation, the client alone shall have a licence to use this report. The report shall not be reproduced except in full.

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BOREHOLE LOG

Borehole No.
1
1/1

Client: ROYAL HASKONINGDHV
Project: PROPOSED FORESHORE REHABILITATION
Location: RIP ROAD RESERVE, BLACKWALL, NSW

Job No.: 32217R **Method:** HAND AUGER **R.L. Surface:** ≈ 1.7m
Date: 28/2/19 **Datum:** AHD
Plant Type: N/A **Logged/Checked by:** J.D.B./P.R.

Groundwater Record	SAMPLES (ES, U50, DB, DS)	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
		REFER TO DCP TEST RESULTS	0		-	FILL: Silty clayey sand, fine to medium grained, dark brown, with root fibres.	M			APPEARS POORLY COMPACTED
			0.5		CL	Sandy CLAY: low plasticity, brown, fine to medium grained sand.	w>PL	F	60 40 70	BEACH RIDGE DEPOSITS
			1		SC	Clayey SAND: fine to coarse grained, brown and grey mottled red brown.	M	L-MD		
AFTER 4 HRS			1.5		CI-CH	CLAY: medium to high plasticity, grey and brown, trace of fine to medium grained sand.		St-VSt	160 190 200	
ON COMPLETION			2							
					CI	Sandy CLAY: medium plasticity, light grey, fine to medium grained sand.			250 280 290	
			2.5			END OF BOREHOLE AT 2.40m				HAND AUGER REFUSAL
			3							
			3.5							

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BOREHOLE LOG

Borehole No. **3**

1/1

Client: ROYAL HASKONINGDHV

Project: PROPOSED FORESHORE REHABILITATION

Location: RIP ROAD RESERVE, BLACKWALL, NSW

Job No.: 32217R **Method:** HAND AUGER **R.L. Surface:** ≈ 2.8m

Date: 28/2/19 **Datum:** AHD

Plant Type: N/A **Logged/Checked by:** J.D.B./P.R.

Groundwater Record	SAMPLES (ES / U50 / DB / DS)	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DRY ON COMPLETION		REFER TO DCP TEST RESULTS	0		-	FILL: Silty clayey sand, fine to medium grained, dark brown, with root fibres.	M			APPEARS POORLY COMPACTED
			0.5							
					CH	Silty CLAY: high plasticity, orange brown, trace of fine to medium grained sand.	w<PL	(St-VSt)		TOO FRIABLE FOR HP TESTING BEACH RIDGE DEPOSITS
			1							
			1.5			as above, but light grey.	w>PL	VSt	280 320 310	
						END OF BOREHOLE AT 1.60m				HAND AUGER REFUSAL
			2							
			2.5							
			3							
			3.5							

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BOREHOLE LOG

Borehole No. **5**

1/1

Client: ROYAL HASKONINGDHV

Project: PROPOSED FORESHORE REHABILITATION

Location: RIP ROAD RESERVE, BLACKWALL, NSW

Job No.: 32217R **Method:** HAND AUGER **R.L. Surface:** ≈ 0.1m

Date: 28/2/19 **Datum:** AHD

Plant Type: N/A **Logged/Checked by:** J.D.B./P.R.

Groundwater Record	SAMPLES (ES, U50, DB, DS)	Field Tests	Depth (m)	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
		REFER TO DCP TEST RESULTS	0	SP	SAND: fine to medium grained, brown and grey.	M	L-MD		BEACH RIDGE DEPOSITS
ON COMPLETION					SAND: fine to coarse grained, grey, with shells.	W			
			0.5				VL		
				CL-CI	Sandy CLAY: low to medium plasticity, orange brown, with shells and medium to coarse grained ironstone gravel.	w>PL	(St)		
				CH	CLAY: high plasticity, light grey, with fine to medium grained sand.	w>PL	St	160 180 170	
			1		END OF BOREHOLE AT 1.0m				HAND AUGER REFUSAL
			1.5						
			2						
			2.5						
			3						
			3.5						

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BOREHOLE LOG

Borehole No.
6

Client: ROYAL HASKONINGDHV

Project: PROPOSED FORESHORE REHABILITATION

Location: RIP ROAD RESERVE, BLACKWALL, NSW

Job No.: 32217R **Method:** HAND AUGER **R.L. Surface:** ≈ 0.2m

Date: 28/2/19 **Datum:** AHD

Plant Type: N/A **Logged/Checked by:** J.D.B./P.R.

Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
					REFER TO DCP TEST RESULTS	0		SP	SAND: fine to medium grained, brown.	M	(VL)		
								CL-CI	Sandy CLAY: low to medium plasticity, orange brown.	w>PL	(VS)		BEACH RIDGE DEPOSITS
						0.5							
ON COMPLETION								CH	CLAY: high plasticity, light grey.		(VS)		
											(F)		
						1			END OF BOREHOLE AT 1.0m				
						1.5							
						2							
						2.5							
						3							
						3.5							

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DYNAMIC CONE PENETRATION TEST RESULTS

Client:	ROYAL HASKONINGDHV
Project:	PROPOSED FORESHORE REHABILITATION
Location:	RIP ROAD RESERVE, BLACKWALL, NSW

Job No.	32217R	Hammer Weight & Drop: 9kg/510mm
Date:	28-2-19	Rod Diameter: 16mm
Tested By:	J.D.B.	Point Diameter: 20mm

Test Location	1	2	3	Test Location		2	
Surface RL	≈1.7m	≈1.6m	≈2.8m	Surface RL		≈1.6m	
Depth (mm)	Blows per 100mm Penetration			Depth (mm)	Blows per 100mm Penetration		
0 - 100	1	SUNK	1	3000-3100		10/30mm	
100 - 200	2		1	3100-3200		REFUSAL	
200 - 300	1		1	3200-3300			
300 - 400	2			3300-3400			
400 - 500	1	2	2	3400-3500			
500 - 600	2	1	6	3500-3600			
600 - 700	2	2	7	3600-3700			
700 - 800	3	3	8	3700-3800			
800 - 900	3	4	6	3800-3900			
900 - 1000	4	3	6	3900-4000			
1000 - 1100	5	5	12	4000-4100			
1100 - 1200	4	3	9	4100-4200			
1200 - 1300	4	3	8	4200-4300			
1300 - 1400	5	5	11	4300-4400			
1400 - 1500	3	5	7	4400-4500			
1500 - 1600	5	8	6	4500-4600			
1600 - 1700	5	7	7	4600-4700			
1700 - 1800	5	7	8	4700-4800			
1800 - 1900	7	7	5/0mm	4800-4900			
1900 - 2000	8	6	REFUSAL	4900-5000			
2000 - 2100	13	8		5000-5100			
2100 - 2200	15	7		5100-5200			
2200 - 2300	14	7		5200-5300			
2300 - 2400	25/50mm	7		5300-5400			
2400 - 2500	REFUSAL	9		5400-5500			
2500 - 2600		9		5500-5600			
2600 - 2700		7		5600-5700			
2700 - 2800		9		5700-5800			
2800 - 2900		7		5800-5900			
2900 - 3000		11		5900-6000			

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

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DYNAMIC CONE PENETRATION TEST RESULTS

Client:	ROYAL HASKONINGDHV							
Project:	PROPOSED FORESHORE REHABILITATION							
Location:	RIP ROAD RESERVE, BLACKWALL, NSW							
Job No.	32217R			Hammer Weight & Drop: 9kg/510mm				
Date:	28-2-19			Rod Diameter: 16mm				
Tested By:	J.D.B.			Point Diameter: 20mm				
Test Location	**4**	**5**	**6**	Test Location				
Surface RL	≈0.5m	≈0.1m	≈0.2m	Surface RL				
Depth (mm)	Blows per 100mm Penetration			Depth (mm)	Blows per 100mm Penetration			
0 - 100	SUNK	5	SUNK	3000-3100				
100 - 200	9	3		3100-3200				
200 - 300	3	4		3200-3300				
300 - 400	2	5		3300-3400				
400 - 500		2		3400-3500				
500 - 600				3500-3600				
600 - 700	7	11		3600-3700				
700 - 800	16/0mm	14		3700-3800				
800 - 900	REFUSAL	20		3800-3900				
900 - 1000		20	3	3900-4000				
1000 - 1100		11	7	4000-4100				
1100 - 1200		22	9	4100-4200				
1200 - 1300		13	15	4200-4300				
1300 - 1400		30	16	4300-4400				
1400 - 1500		END	18	4400-4500				
1500 - 1600			15	4500-4600				
1600 - 1700			16	4600-4700				
1700 - 1800			19	4700-4800				
1800 - 1900			24	4800-4900				
1900 - 2000			20/50mm	4900-5000				
2000 - 2100			END	5000-5100				
2100 - 2200				5100-5200				
2200 - 2300				5200-5300				
2300 - 2400				5300-5400				
2400 - 2500				5400-5500				
2500 - 2600				5500-5600				
2600 - 2700				5600-5700				
2700 - 2800				5700-5800				
2800 - 2900				5800-5900				
2900 - 3000				5900-6000				

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

AERIAL IMAGE SOURCE: MAPS.AU.NEARMAP.COM, 27 DEC 2018.

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

Title:	**SITE LOCATION PLAN**
Location:	RIP ROAD RESERVE BLACKWALL, NSW
Report No: 32217R	Figure No: 1

JKGeotechnics

PLOT DATE: 11/06/2019 10:36:14 AM DWG FILE: S:\6 GEOTECHNICAL\6F GEOTECHNICAL JOBS\32000'S\32217R BLACKWALL\CAD\32217R.DWG

AERIAL IMAGE SOURCE: MAPS.AU.NEARMAP.COM, 27 DEC 2018.

LEGEND

- HAND AUGER AND DCP TEST
- DCP TEST

0 3 6 9 12 15

SCALE 1:300 @A3 METRES

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

Title:	TEST LOCATION PLAN
Location:	RIP ROAD RESERVE BLACKWALL, NSW
Report No: 32217R	Figure No: 2

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PLOT DATE: 11/06/2019 10:36:31 AM DWG FILE: S:\6 GEOTECHNICAL JOBS\32000'S\32217R BLACKWALL\CAD\32217R.DWG

REPORT EXPLANATION NOTES

INTRODUCTION

These notes have been provided to amplify the geotechnical report in regard to classification methods, field procedures and certain matters relating to the Comments and Recommendations section. Not all notes are necessarily relevant to all reports.

The ground is a product of continuing natural and man-made processes and therefore exhibits a variety of characteristics and properties which vary from place to place and can change with time. Geotechnical engineering involves gathering and assimilating limited facts about these characteristics and properties in order to understand or predict the behaviour of the ground on a particular site under certain conditions. This report may contain such facts obtained by inspection, excavation, probing, sampling, testing or other means of investigation. If so, they are directly relevant only to the ground at the place where and time when the investigation was carried out.

DESCRIPTION AND CLASSIFICATION METHODS

The methods of description and classification of soils and rocks used in this report are based on Australian Standard 1726:2017 *'Geotechnical Site Investigations'*. In general, descriptions cover the following properties – soil or rock type, colour, structure, strength or density, and inclusions. Identification and classification of soil and rock involves judgement and the Company infers accuracy only to the extent that is common in current geotechnical practice.

Soil types are described according to the predominating particle size and behaviour as set out in the attached soil classification table qualified by the grading of other particles present (eg. sandy clay) as set out below:

Soil Classification	Particle Size
Clay	< 0.002mm
Silt	0.002 to 0.075mm
Sand	0.075 to 2.36mm
Gravel	2.36 to 63mm
Cobbles	63 to 200mm
Boulders	> 200mm

Non-cohesive soils are classified on the basis of relative density, generally from the results of Standard Penetration Test (SPT) as below:

Relative Density	SPT 'N' Value (blows/300mm)
Very loose (VL)	< 4
Loose (L)	4 to 10
Medium dense (MD)	10 to 30
Dense (D)	30 to 50
Very Dense (VD)	> 50

Cohesive soils are classified on the basis of strength (consistency) either by use of a hand penetrometer, vane shear, laboratory testing and/or tactile engineering examination. The strength terms are defined as follows.

Classification	Unconfined Compressive Strength (kPa)	Indicative Undrained Shear Strength (kPa)
Very Soft (VS)	≤ 25	≤ 12
Soft (S)	> 25 and ≤ 50	> 12 and ≤ 25
Firm (F)	> 50 and ≤ 100	> 25 and ≤ 50
Stiff (St)	> 100 and ≤ 200	> 50 and ≤ 100
Very Stiff (VSt)	> 200 and ≤ 400	> 100 and ≤ 200
Hard (Hd)	> 400	> 200
Friable (Fr)	Strength not attainable – soil crumbles	

Rock types are classified by their geological names, together with descriptive terms regarding weathering, strength, defects, etc. Where relevant, further information regarding rock classification is given in the text of the report. In the Sydney Basin, 'shale' is used to describe fissile mudstone, with a weakness parallel to bedding. Rocks with alternating inter-laminations of different grain size (eg. siltstone/claystone and siltstone/fine grained sandstone) is referred to as 'laminite'.

SAMPLING

Sampling is carried out during drilling or from other excavations to allow engineering examination (and laboratory testing where required) of the soil or rock.

Disturbed samples taken during drilling provide information on plasticity, grain size, colour, moisture content, minor constituents and, depending upon the degree of disturbance, some information on strength and structure. Bulk samples are similar but of greater volume required for some test procedures.

Undisturbed samples are taken by pushing a thin-walled sample tube, usually 50mm diameter (known as a U50), into the soil and withdrawing it with a sample of the soil contained in a relatively undisturbed state. Such samples yield information on structure and strength, and are necessary for laboratory determination of shrink-swell behaviour, strength and compressibility. Undisturbed sampling is generally effective only in cohesive soils.

Details of the type and method of sampling used are given on the attached logs.

INVESTIGATION METHODS

The following is a brief summary of investigation methods currently adopted by the Company and some comments on their use and application. All methods except test pits, hand auger drilling and portable Dynamic Cone Penetrometers require the use of a mechanical rig which is commonly mounted on a truck chassis or track base.

Test Pits: These are normally excavated with a backhoe or a tracked excavator, allowing close examination of the insitu soils and 'weaker' bedrock if it is safe to descend into the pit. The depth of penetration is limited to about 3m for a backhoe and up to 6m for a large excavator. Limitations of test pits are the problems associated with disturbance and difficulty of reinstatement and the consequent effects on close-by structures. Care must be taken if construction is to be carried out near test pit locations to either properly recompact the backfill during construction or to design and construct the structure so as not to be adversely affected by poorly compacted backfill at the test pit location.

Hand Auger Drilling: A borehole of 50mm to 100mm diameter is advanced by manually operated equipment. Refusal of the hand auger can occur on a variety of materials such as obstructions within any fill, tree roots, hard clay, gravel or ironstone, cobbles and boulders, and does not necessarily indicate rock level.

Continuous Spiral Flight Augers: The borehole is advanced using 75mm to 115mm diameter continuous spiral flight augers, which are withdrawn at intervals to allow sampling and insitu testing. This is a relatively economical means of drilling in clays and in sands above the water table. Samples are returned to the surface by the flights or may be collected after withdrawal of the auger flights, but they can be very disturbed and layers may become mixed. Information from the auger sampling (as distinct from specific sampling by SPTs or undisturbed samples) is of limited reliability due to mixing or softening of samples by groundwater, or uncertainties as to the original depth of the samples. Augering below the groundwater table is of even lesser reliability than augering above the water table.

Rock Augering: Use can be made of a Tungsten Carbide (TC) bit for auger drilling into rock to indicate rock quality and continuity by variation in drilling resistance and from examination of recovered rock cuttings. This method of investigation is quick and relatively inexpensive but provides only an indication of the likely rock strength and predicted values may be in error by a strength order. Where rock strengths may have a significant impact on construction feasibility or costs, then further investigation by means of cored boreholes may be warranted.

Wash Boring: The borehole is usually advanced by a rotary bit, with water being pumped down the drill rods and returned up the annulus, carrying the drill cuttings. Only major changes in stratification can be assessed from the cuttings, together with some information from "feel" and rate of penetration.

Mud Stabilised Drilling: Either Wash Boring or Continuous Core Drilling can use drilling mud as a circulating fluid to stabilise the borehole. The term 'mud' encompasses a range of products ranging from bentonite to polymers. The mud tends to mask the cuttings and reliable identification is only possible from intermittent intact sampling (eg. from SPT and U50 samples) or from rock coring, etc.

Continuous Core Drilling: A continuous core sample is obtained using a diamond tipped core barrel. Provided full core recovery is achieved (which is not always possible in very low strength rocks and granular soils), this technique provides a very reliable (but relatively expensive) method of investigation. In rocks, NMLC or HQ triple tube core barrels, which give a core of about 50mm and 61mm diameter, respectively, is usually used with water flush. The length of core recovered is compared to the length drilled and any length not recovered is shown as NO CORE. The location of NO CORE recovery is determined on site by the supervising engineer; where the location is uncertain, the loss is placed at the bottom of the drill run.

Standard Penetration Tests: Standard Penetration Tests (SPT) are used mainly in non-cohesive soils, but can also be used in cohesive soils, as a means of indicating density or strength and also of obtaining a relatively undisturbed sample. The test procedure is described in Australian Standard 1289.6.3.1–2004 (R2016) *'Methods of Testing Soils for Engineering Purposes, Soil Strength and Consolidation Tests – Determination of the Penetration Resistance of a Soil – Standard Penetration Test (SPT)'*.

The test is carried out in a borehole by driving a 50mm diameter split sample tube with a tapered shoe, under the impact of a 63.5kg hammer with a free fall of 760mm. It is normal for the tube to be driven in three successive 150mm increments and the 'N' value is taken as the number of blows for the last 300mm. In dense sands, very hard clays or weak rock, the full 450mm penetration may not be practicable and the test is discontinued.

The test results are reported in the following form:

- In the case where full penetration is obtained with successive blow counts for each 150mm of, say, 4, 6 and 7 blows, as

  N = 13
  4, 6, 7

- In a case where the test is discontinued short of full penetration, say after 15 blows for the first 150mm and 30 blows for the next 40mm, as

  N > 30
  15, 30/40mm

The results of the test can be related empirically to the engineering properties of the soil.

A modification to the SPT is where the same driving system is used with a solid 60° tipped steel cone of the same diameter as the SPT hollow sampler. The solid cone can be continuously driven for some distance in soft clays or loose sands, or may be used where damage would otherwise occur to the SPT. The results of this Solid Cone Penetration Test (SCPT) are shown as 'N_c' on the borehole logs, together with the number of blows per 150mm penetration.

Cone Penetrometer Testing (CPT) and Interpretation: The cone penetrometer is sometimes referred to as a Dutch Cone. The test is described in Australian Standard 1289.6.5.1–1999 (R2013) *'Methods of Testing Soils for Engineering Purposes, Soil Strength and Consolidation Tests – Determination of the Static Cone Penetration Resistance of a Soil – Field Test using a Mechanical and Electrical Cone or Friction-Cone Penetrometer'*.

In the tests, a 35mm or 44mm diameter rod with a conical tip is pushed continuously into the soil, the reaction being provided by a specially designed truck or rig which is fitted with a hydraulic ram system. Measurements are made of the end bearing resistance on the cone and the frictional resistance on a separate 134mm or 165mm long sleeve, immediately behind the cone. Transducers in the tip of the assembly are electrically connected by wires passing through the centre of the push rods to an amplifier and recorder unit mounted on the control truck. The CPT does not provide soil sample recovery.

As penetration occurs (at a rate of approximately 20mm per second), the information is output as incremental digital records every 10mm. The results given in this report have been plotted from the digital data.

The information provided on the charts comprise:

- Cone resistance – the actual end bearing force divided by the cross sectional area of the cone – expressed in MPa. There are two scales presented for the cone resistance. The lower scale has a range of 0 to 5MPa and the main scale has a range of 0 to 50MPa. For cone resistance values less than 5MPa, the plot will appear on both scales.
- Sleeve friction – the frictional force on the sleeve divided by the surface area – expressed in kPa.
- Friction ratio – the ratio of sleeve friction to cone resistance, expressed as a percentage.

  The ratios of the sleeve resistance to cone resistance will vary with the type of soil encountered, with higher relative friction in clays than in sands. Friction ratios of 1% to 2% are commonly encountered in sands and occasionally very soft clays, rising to 4% to 10% in stiff clays and peats. Soil descriptions based on cone resistance and friction ratios are only inferred and must not be considered as exact.

Correlations between CPT and SPT values can be developed for both sands and clays but may be site specific.

Interpretation of CPT values can be made to empirically derive modulus or compressibility values to allow calculation of foundation settlements.

Stratification can be inferred from the cone and friction traces and from experience and information from nearby boreholes etc. Where shown, this information is presented for general guidance, but must be regarded as interpretive. The test method provides a continuous profile of engineering properties but, where precise information on soil classification is required, direct drilling and sampling may be preferable.

There are limitations when using the CPT in that it may not penetrate obstructions within any fill, thick layers of hard clay and very dense sand, gravel and weathered bedrock. Normally a 'dummy' cone is pushed through fill to protect the equipment. No information is recorded by the 'dummy' probe.

Flat Dilatometer Test: The flat dilatometer (DMT), also known as the Marchetti Dilometer comprises a stainless steel blade having a flat, circular steel membrane mounted flush on one side.

The blade is connected to a control unit at ground surface by a pneumatic-electrical tube running through the insertion rods. A gas tank, connected to the control unit by a pneumatic cable, supplies the gas pressure required to expand the membrane. The control unit is equipped with a pressure regulator, pressure gauges, an audio-visual signal and vent valves.

The blade is advanced into the ground using our CPT rig or one of our drilling rigs, and can be driven into the ground using an SPT hammer. As soon as the blade is in place, the membrane is inflated, and the pressure required to lift the membrane (approximately 0.1mm) is recorded. The pressure then required to lift the centre of the membrane by an additional 1mm is recorded. The membrane is then deflated before pushing to the next depth increment, usually 200mm down. The pressure readings are corrected for membrane stiffness.

The DMT is used to measure material index (I_D), horizontal stress index (K_D), and dilatometer modulus (E_D). Using established correlations, the DMT results can also be used to assess the 'at rest' earth pressure coefficient (K_o), over-consolidation ratio (OCR), undrained shear strength (C_u), friction angle (ϕ), coefficient of consolidation (C_h), coefficient of permeability (K_h), unit weight (γ), and vertical drained constrained modulus (M).

The seismic dilatometer (SDMT) is the combination of the DMT with an add-on seismic module for the measurement of shear wave velocity (V_s). Using established correlations, the SDMT results can also be used to assess the small strain modulus (G_o).

Portable Dynamic Cone Penetrometers: Portable Dynamic Cone Penetrometer (DCP) tests are carried out by driving a 16mm diameter rod with a 20mm diameter cone end with a 9kg hammer dropping 510mm. The test is described in Australian Standard 1289.6.3.2–1997 (R2013) *'Methods of Testing Soils for Engineering Purposes, Soil Strength and Consolidation Tests – Determination of the Penetration Resistance of a Soil – 9kg Dynamic Cone Penetrometer Test'*.

The results are used to assess the relative compaction of fill, the relative density of granular soils, and the strength of cohesive soils. Using established correlations, the DCP test results can also be used to assess California Bearing Ratio (CBR).

Refusal of the DCP can occur on a variety of materials such as obstructions within any fill, tree roots, hard clay, gravel or ironstone, cobbles and boulders, and does not necessarily indicate rock level.

Vane Shear Test: The vane shear test is used to measure the undrained shear strength (C_u) of typically very soft to firm fine grained cohesive soils. The vane shear is normally performed in the bottom of a borehole, but can be completed from surface level, the bottom and sides of test pits, and on recovered undisturbed tube samples (when using a hand vane).

The vane comprises four rectangular blades arranged in the form of a cross on the end of a thin rod, which is coupled to the bottom of a drill rod string when used in a borehole. The size of the vane is dependent on the strength of the fine grained cohesive soils; that is, larger vanes are normally used for very low strength soils. For borehole testing, the size of the vane can be limited by the size of the casing that is used.

For testing inside a borehole, a device is used at the top of the casing, which suspends the vane and rods so that they do not sink under self-weight into the 'soft' soils beyond the depth at which the test is to be carried out. A calibrated torque head is used to rotate the rods and vane and to measure the resistance of the vane to rotation.

With the vane in position, torque is applied to cause rotation of the vane at a constant rate. A rate of 6° per minute is the common rotation rate. Rotation is continued until the soil is sheared and the maximum torque has been recorded. This value is then used to calculate the undrained shear strength. The vane is then rotated rapidly a number of times and the operation repeated until a constant torque reading is obtained. This torque value is used to calculate the remoulded shear strength. Where appropriate, friction on the vane rods is measured and taken into account in the shear strength calculation.

LOGS

The borehole or test pit logs presented herein are an engineering and/or geological interpretation of the subsurface conditions, and their reliability will depend to some extent on the frequency of sampling and the method of drilling or excavation. Ideally, continuous undisturbed sampling or core drilling will enable the most reliable assessment, but is not always practicable or possible to justify on economic grounds. In any case, the boreholes or test pits represent only a very small sample of the total subsurface conditions.

The terms and symbols used in preparation of the logs are defined in the following pages.

Interpretation of the information shown on the logs, and its application to design and construction, should therefore take into account the spacing of boreholes or test pits, the method of drilling or excavation, the frequency of sampling and testing and the possibility of other than 'straight line' variations between the boreholes or test pits. Subsurface conditions between boreholes or test pits may vary significantly from conditions encountered at the borehole or test pit locations.

GROUNDWATER

Where groundwater levels are measured in boreholes, there are several potential problems:

- Although groundwater may be present, in low permeability soils it may enter the hole slowly or perhaps not at all during the time it is left open.
- A localised perched water table may lead to an erroneous indication of the true water table.
- Water table levels will vary from time to time with seasons or recent weather changes and may not be the same at the time of construction.
- The use of water or mud as a drilling fluid will mask any groundwater inflow. Water has to be blown out of the hole and drilling mud must be washed out of the hole or 'reverted' chemically if reliable water observations are to be made.

More reliable measurements can be made by installing standpipes which are read after the groundwater level has stabilised at intervals ranging from several days to perhaps weeks for low permeability soils. Piezometers, sealed in a particular stratum, may be advisable in low permeability soils or where there may be interference from perched water tables or surface water.

FILL

The presence of fill materials can often be determined only by the inclusion of foreign objects (eg. bricks, steel, etc) or by distinctly unusual colour, texture or fabric. Identification of the extent of fill materials will also depend on investigation methods and frequency. Where natural soils similar to those at the site are used for fill, it may be difficult with limited testing and sampling to reliably assess the extent of the fill.

The presence of fill materials is usually regarded with caution as the possible variation in density, strength and material type is much greater than with natural soil deposits. Consequently, there is an increased risk of adverse engineering characteristics or behaviour. If the volume and quality of fill is of importance to a project, then frequent test pit excavations are preferable to boreholes.

LABORATORY TESTING

Laboratory testing is normally carried out in accordance with Australian Standard 1289 *'Methods of Testing Soils for Engineering Purposes'* or appropriate NSW Government Roads & Maritime Services (RMS) test methods. Details of the test procedure used are given on the individual report forms.

ENGINEERING REPORTS

Engineering reports are prepared by qualified personnel and are based on the information obtained and on current engineering standards of interpretation and analysis. Where the report has been prepared for a specific design proposal (eg. a three storey building) the information and interpretation may not be relevant if the design proposal is changed (eg. to a twenty storey building). If this happens, the Company will be pleased to review the report and the sufficiency of the investigation work.

Reasonable care is taken with the report as it relates to interpretation of subsurface conditions, discussion of geotechnical aspects and recommendations or suggestions for design and construction. However, the Company cannot always anticipate or assume responsibility for:

- Unexpected variations in ground conditions – the potential for this will be partially dependent on borehole spacing and sampling frequency as well as investigation technique.
- Changes in policy or interpretation of policy by statutory authorities.
- The actions of persons or contractors responding to commercial pressures.
- Details of the development that the Company could not reasonably be expected to anticipate.

If these occur, the Company will be pleased to assist with investigation or advice to resolve any problems occurring.

SITE ANOMALIES

In the event that conditions encountered on site during construction appear to vary from those which were expected from the information contained in the report, the Company requests that it immediately be notified. Most problems are much more readily resolved when conditions are exposed rather than at some later stage, well after the event.

REPRODUCTION OF INFORMATION FOR CONTRACTUAL PURPOSES

Where information obtained from this investigation is provided for tendering purposes, it is recommended that all information, including the written report and discussion, be made available. In circumstances where the discussion or comments section is not relevant to the contractual situation, it may be appropriate to prepare a specially edited document. The Company would be pleased to assist in this regard and/or to make additional report copies available for contract purposes at a nominal charge.

Copyright in all documents (such as drawings, borehole or test pit logs, reports and specifications) provided by the Company shall remain the property of Jeffery and Katauskas Pty Ltd. Subject to the payment of all fees due, the Client alone shall have a licence to use the documents provided for the sole purpose of completing the project to which they relate. Licence to use the documents may be revoked without notice if the Client is in breach of any obligation to make a payment to us.

REVIEW OF DESIGN

Where major civil or structural developments are proposed or where only a limited investigation has been completed or where the geotechnical conditions/constraints are quite complex, it is prudent to have a joint design review which involves an experienced geotechnical engineer/engineering geologist.

SITE INSPECTION

The Company will always be pleased to provide engineering inspection services for geotechnical aspects of work to which this report is related.

Requirements could range from:

i) a site visit to confirm that conditions exposed are no worse than those interpreted, to

ii) a visit to assist the contractor or other site personnel in identifying various soil/rock types and appropriate footing or pile founding depths, or

iii) full time engineering presence on site.

SYMBOL LEGENDS

SOIL

- FILL
- TOPSOIL
- CLAY (CL, CI, CH)
- SILT (ML, MH)
- SAND (SP, SW)
- GRAVEL (GP, GW)
- SANDY CLAY (CL, CI, CH)
- SILTY CLAY (CL, CI, CH)
- CLAYEY SAND (SC)
- SILTY SAND (SM)
- GRAVELLY CLAY (CL, CI, CH)
- CLAYEY GRAVEL (GC)
- SANDY SILT (ML, MH)
- PEAT AND HIGHLY ORGANIC SOILS (Pt)

ROCK

- CONGLOMERATE
- SANDSTONE
- SHALE/MUDSTONE
- SILTSTONE
- CLAYSTONE
- COAL
- LAMINITE
- LIMESTONE
- PHYLLITE, SCHIST
- TUFF
- GRANITE, GABBRO
- DOLERITE, DIORITE
- BASALT, ANDESITE
- QUARTZITE

OTHER MATERIALS

- BRICKS OR PAVERS
- CONCRETE
- ASPHALTIC CONCRETE

CLASSIFICATION OF COARSE AND FINE GRAINED SOILS

Major Divisions		Group Symbol	Typical Names	Field Classification of Sand and Gravel	Laboratory Classification	
Coarse grained soil (more than 65% of soil excluding oversize fraction is greater than 0.075mm)	GRAVEL (more than half of coarse fraction is larger than 2.36mm	GW	Gravel and gravel-sand mixtures, little or no fines	Wide range in grain size and substantial amounts of all intermediate sizes, not enough fines to bind coarse grains, no dry strength	≤ 5% fines	$C_u > 4$ $1 < C_c < 3$
		GP	Gravel and gravel-sand mixtures, little or no fines, uniform gravels	Predominantly one size or range of sizes with some intermediate sizes missing, not enough fines to bind coarse grains, no dry strength	≤ 5% fines	Fails to comply with above
		GM	Gravel-silt mixtures and gravel-sand-silt mixtures	'Dirty' materials with excess of non-plastic fines, zero to medium dry strength	≥ 12% fines, fines are silty	Fines behave as silt
		GC	Gravel-clay mixtures and gravel-sand-clay mixtures	'Dirty' materials with excess of plastic fines, medium to high dry strength	≥ 12% fines, fines are clayey	Fines behave as clay
	SAND (more than half of coarse fraction is smaller than 2.36mm)	SW	Sand and gravel-sand mixtures, little or no fines	Wide range in grain size and substantial amounts of all intermediate sizes, not enough fines to bind coarse grains, no dry strength	≤ 5% fines	$C_u > 6$ $1 < C_c < 3$
		SP	Sand and gravel-sand mixtures, little or no fines	Predominantly one size or range of sizes with some intermediate sizes missing, not enough fines to bind coarse grains, no dry strength	≤ 5% fines	Fails to comply with above
		SM	Sand-silt mixtures	'Dirty' materials with excess of non-plastic fines, zero to medium dry strength	≥ 12% fines, fines are silty	N/A
		SC	Sand-clay mixtures	'Dirty' materials with excess of plastic fines, medium to high dry strength	≥ 12% fines, fines are clayey	

Major Divisions		Group Symbol	Typical Names	Field Classification of Silt and Clay			Laboratory Classification
				Dry Strength	Dilatancy	Toughness	% < 0.075mm
ine grained soils (more than 35% of soil excluding oversize fraction is less than 0.075mm)	SILT and CLAY (low to medium plasticity)	ML	Inorganic silt and very fine sand, rock flour, silty or clayey fine sand or silt with low plasticity	None to low	Slow to rapid	Low	Below A line
		CL, CI	Inorganic clay of low to medium plasticity, gravelly clay, sandy clay	Medium to high	None to slow	Medium	Above A line
		OL	Organic silt	Low to medium	Slow	Low	Below A line
	SILT and CLAY (high plasticity)	MH	Inorganic silt	Low to medium	None to slow	Low to medium	Below A line
		CH	Inorganic clay of high plasticity	High to very high	None	High	Above A line
		OH	Organic clay of medium to high plasticity, organic silt	Medium to high	None to very slow	Low to medium	Below A line
	Highly organic soil	Pt	Peat, highly organic soil	–	–	–	–

Laboratory **Classification** Criteria

A well graded coarse grained soil is one for which the coefficient of uniformity $Cu > 4$ and the coefficient of curvature $1 < C_c < 3$. Otherwise, the soil is poorly graded. These coefficients are given by:

$$C_U = \frac{D_{60}}{D_{10}} \quad and \quad C_C = \frac{(D_{30})^2}{D_{10}\,D_{60}}$$

Where D_{10}, D_{30} and D_{60} are those grain sizes for which 10%, 30% and 60% of the soil grains, respectively, are smaller.

NOTES:

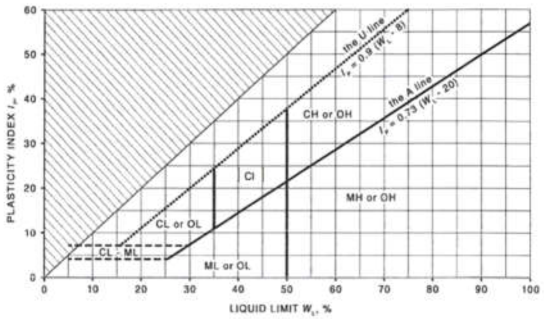
1 For a coarse grained soil with a fines content between 5% and 12%, the soil is given a dual classification comprising the two group symbols separated by a dash; for example, for a poorly graded gravel with between 5% and 12% silt fines, the classification is GP-GM.

2 Where the grading is determined from laboratory tests, it is defined by coefficients of curvature (C_c) and uniformity (C_u) derived from the particle size distribution curve.

3 Clay soils with liquid limits > 35% and ≤ 50% may be classified as being of medium plasticity.

4 The U line on the Modified Casagrande Chart is an approximate upper bound for most natural soils.

Modified Casagrande Chart for Classifying Silts and Clays according to their Behaviour



LOG SYMBOLS

Log Column	Symbol	Definition
Groundwater Record		Standing water level. Time delay following completion of drilling/excavation may be shown.
	C	Extent of borehole/test pit collapse shortly after drilling/excavation.
		Groundwater seepage into borehole or test pit noted during drilling or excavation.
Samples	ES U50 DB DS ASB ASS SAL	Sample taken over depth indicated, for environmental analysis. Undisturbed 50mm diameter tube sample taken over depth indicated. Bulk disturbed sample taken over depth indicated. Small disturbed bag sample taken over depth indicated. Soil sample taken over depth indicated, for asbestos analysis. Soil sample taken over depth indicated, for acid sulfate soil analysis. Soil sample taken over depth indicated, for salinity analysis.
Field Tests	N = 17 4, 7, 10	Standard Penetration Test (SPT) performed between depths indicated by lines. Individual figures show blows per 150mm penetration. 'Refusal' refers to apparent hammer refusal within the corresponding 150mm depth increment.
	N_c = 5 7 3R	Solid Cone Penetration Test (SCPT) performed between depths indicated by lines. Individual figures show blows per 150mm penetration for 60° solid cone driven by SPT hammer. 'R' refers to apparent hammer refusal within the corresponding 150mm depth increment.
	VNS = 25 PID = 100	Vane shear reading in kPa of undrained shear strength. Photoionisation detector reading in ppm (soil sample headspace test).
Moisture Condition (Fine Grained Soils)	w > PL $w \approx$ PL w < PL $w \approx$ LL w > LL	Moisture content estimated to be greater than plastic limit. Moisture content estimated to be approximately equal to plastic limit. Moisture content estimated to be less than plastic limit. Moisture content estimated to be near liquid limit. Moisture content estimated to be wet of liquid limit.
(Coarse Grained Soils)	D M W	DRY – runs freely through fingers. MOIST – does not run freely but no free water visible on soil surface. WET – free water visible on soil surface.
Strength (Consistency) Cohesive Soils	VS S F St VSt Hd Fr ( )	VERY SOFT – unconfined compressive strength ≤ 25kPa. SOFT – unconfined compressive strength > 25kPa and ≤ 50kPa. FIRM – unconfined compressive strength > 50kPa and ≤ 100kPa. STIFF – unconfined compressive strength > 100kPa and ≤ 200kPa. VERY STIFF – unconfined compressive strength > 200kPa and ≤ 400kPa. HARD – unconfined compressive strength > 400kPa. FRIABLE – strength not attainable, soil crumbles. Bracketed symbol indicates estimated consistency based on tactile examination or other assessment.
Density Index/ Relative Density (Cohesionless Soils)		**Density Index (I_D) Range (%)** — **SPT 'N' Value Range (Blows/300mm)**
	VL	VERY LOOSE — ≤ 15 — 0 – 4
	L	LOOSE — > 15 and ≤ 35 — 4 – 10
	MD	MEDIUM DENSE — > 35 and ≤ 65 — 10 – 30
	D	DENSE — > 65 and ≤ 85 — 30 – 50
	VD	VERY DENSE — > 85 — > 50
	( )	Bracketed symbol indicates estimated density based on ease of drilling or other assessment.
Hand Penetrometer Readings	300 250	Measures reading in kPa of unconfined compressive strength. Numbers indicate individual test results on representative undisturbed material unless noted otherwise.

Log Column	Symbol	Definition
Remarks	'V' bit	Hardened steel 'V' shaped bit.
	'TC' bit	Twin pronged tungsten carbide bit.
	T_{60}	Penetration of auger string in mm under static load of rig applied by drill head hydraulics without rotation of augers.
	Soil Origin	The geological origin of the soil can generally be described as:
		RESIDUAL – soil formed directly from insitu weathering of the underlying rock. No visible structure or fabric of the parent rock.
		EXTREMELY WEATHERED – soil formed directly from insitu weathering of the underlying rock. Material is of soil strength but retains the structure and/or fabric of the parent rock.
		ALLUVIAL – soil deposited by creeks and rivers.
		ESTUARINE – soil deposited in coastal estuaries, including sediments caused by inflowing creeks and rivers, and tidal currents.
		MARINE – soil deposited in a marine environment.
		AEOLIAN – soil carried and deposited by wind.
		COLLUVIAL – soil and rock debris transported downslope by gravity, with or without the assistance of flowing water. Colluvium is usually a thick deposit formed from a landslide. The description 'slopewash' is used for thinner surficial deposits.
		LITTORAL – beach deposited soil.

Classification of Material Weathering

<table>
<tr><th colspan="2">Term</th><th colspan="2">Abbreviation</th><th>Definition</th></tr>
<tr><td colspan="2">Residual Soil</td><td colspan="2">RS</td><td>Material is weathered to such an extent that it has soil properties. Mass structure and material texture and fabric of original rock are no longer visible, but the soil has not been significantly transported.</td></tr>
<tr><td colspan="2">Extremely Weathered</td><td colspan="2">XW</td><td>Material is weathered to such an extent that it has soil properties. Mass structure and material texture and fabric of original rock are still visible.</td></tr>
<tr><td>Highly Weathered</td><td rowspan="2">Distinctly Weathered (Note 1)</td><td>HW</td><td rowspan="2">DW</td><td>The whole of the rock material is discoloured, usually by iron staining or bleaching to the extent that the colour of the original rock is not recognisable. Rock strength is significantly changed by weathering. Some primary minerals have weathered to clay minerals. Porosity may be increased by leaching, or may be decreased due to deposition of weathering products in pores.</td></tr>
<tr><td>Moderately Weathered</td><td>MW</td><td>The whole of the rock material is discoloured, usually by iron staining or bleaching to the extent that the colour of the original rock is not recognisable, but shows little or no change of strength from fresh rock.</td></tr>
<tr><td colspan="2">Slightly Weathered</td><td colspan="2">SW</td><td>Rock is partially discoloured with staining or bleaching along joints but shows little or no change of strength from fresh rock.</td></tr>
<tr><td colspan="2">Fresh</td><td colspan="2">FR</td><td>Rock shows no sign of decomposition of individual minerals or colour changes.</td></tr>
</table>

NOTE 1: The term 'Distinctly Weathered' is used where it is not practicable to distinguish between 'Highly Weathered' and 'Moderately Weathered' rock. 'Distinctly Weathered' is defined as follows: *'Rock strength usually changed by weathering. The rock may be highly discoloured, usually by iron staining. Porosity may be increased by leaching, or may be decreased due to deposition of weathering products in pores'*. There is some change in rock strength.

Rock Material Strength Classification

Term	Abbreviation	Uniaxial Compressive Strength (MPa)	Guide to Strength	
			Point Load Strength Index $Is_{(50)}$ (MPa)	Field Assessment
Very Low Strength	VL	0.6 to 2	0.03 to 0.1	Material crumbles under firm blows with sharp end of pick; can be peeled with knife; too hard to cut a triaxial sample by hand. Pieces up to 30mm thick can be broken by finger pressure.
Low Strength	L	2 to 6	0.1 to 0.3	Easily scored with a knife; indentations 1mm to 3mm show in the specimen with firm blows of the pick point; has dull sound under hammer. A piece of core 150mm long by 50mm diameter may be broken by hand. Sharp edges of core may be friable and break during handling.
Medium Strength	M	6 to 20	0.3 to 1	Scored with a knife; a piece of core 150mm long by 50mm diameter can be broken by hand with difficulty.
High Strength	H	20 to 60	1 to 3	A piece of core 150mm long by 50mm diameter cannot be broken by hand but can be broken by a pick with a single firm blow; rock rings under hammer.
Very High Strength	VH	60 to 200	3 to 10	Hand specimen breaks with pick after more than one blow; rock rings under hammer.
Extremely High Strength	EH	> 200	> 10	Specimen requires many blows with geological pick to break through intact material; rock rings under hammer.

Abbreviations Used in Defect Description

Cored Borehole Log Column		Symbol Abbreviation	Description
Point Load Strength Index		• 0.6	Axial point load strength index test result (MPa)
		x 0.6	Diametral point load strength index test result (MPa)
Defect Details	– Type	Be	Parting – bedding or cleavage
		CS	Clay seam
		Cr	Crushed/sheared seam or zone
		J	Joint
		Jh	Healed joint
		Ji	Incipient joint
		XWS	Extremely weathered seam
	– Orientation	Degrees	Defect orientation is measured relative to normal to the core axis (ie. relative to the horizontal for a vertical borehole)
	– Shape	P	Planar
		C	Curved
		Un	Undulating
		St	Stepped
		Ir	Irregular
	– Roughness	Vr	Very rough
		R	Rough
		S	Smooth
		Po	Polished
		Sl	Slickensided
	– Infill Material	Ca	Calcite
		Cb	Carbonaceous
		Clay	Clay
		Fe	Iron
		Qz	Quartz
		Py	Pyrite
	– Coatings	Cn	Clean
		Sn	Stained – no visible coating, surface is discoloured
		Vn	Veneer – visible, too thin to measure, may be patchy
		Ct	Coating ≤ 1mm thick
		Filled	Coating > 1mm thick
	– Thickness	mm.t	Defect thickness measured in millimetres

Appendix E

Acid sulphate soils assessment by Environmental Investigation Services

17 April 2019

Ref: E32217BTlet_ASSMP

Central Coast Council
PO Box 21
GOSFORD NSW 2250

Attention: Mr Warren Brown

ACID SULFATE SOIL MANAGEMENT PLAN
PROPOSED FORESHORE REHABILITATION
RIP ROAD RESERVE, BLACKWALL, NSW

1 INTRODUCTION

Central Coast Council ('the client') commissioned Environmental Investigation Services (EIS)[1] to prepare an acid sulfate soil management plan (ASSMP) for the proposed foreshore rehabilitation at Rip Road Reserve, Blackwall, NSW. The site is part of Lot 94 in DP6327. The site location is shown on Figure 1 and the management plan is confined to the proposed development area as shown on Figure 2.

The ASSMP was prepared generally in accordance with a JK Geotechnics proposal (Ref: P48398R) of 12 November 2018 and written acceptance by Central Coast Council via email of 27 March 2019.

1.1 Proposed Development Details

Based on the information provided, EIS understand that the proposed development includes foreshore rehabilitation works to the public reserve. We assume that minor excavation works will be required to prepare the site.

2 SITE INFORMATION / BACKGROUND

The site is located in a residential part of Blackwall, NSW. The site is located along the northern boundary of Rip Road Reserve where it converges with Brisbane Water. To the immediate south of the site is the remainder of Rip Road Reserve and residential properties. The interface of the reserve and the water is built up in sections with brick and/or timber retaining walls. Parts of the brick retaining walls are eroded and unstable.

[1] Environmental consulting division of Jeffery & Katauskas Pty Ltd (J&K)

In February 2019, EIS undertook an acid sulfate soil (ASS) assessment at the site. Soil samples were collected from four locations across the site to a maximum depth of 2.4m below ground level (BGL). The natural soils at the site generally comprised of sandy clay, clayey sand, sand or silty clay.

During the site inspection, fibre cement fragments (FCF) were encountered along the length of the retaining wall and foreshore area. It was recommended that an emu-bob was conducted to remove the FCF from the surface of the site and any fill to be removed/excavated as part of the rehabilitation works be classified in accordance with the NSW EPA Waste Classification Guidelines - Part 1: Classifying Waste (2014)[2] and disposed off-site.

Eight soil samples from varying depths were analysed using the sPOCAS method. The majority of the soil samples analysed encountered results which exceeded the action criteria adopted for the assessment (0.03% w/w peroxide oxidisable sulfur). Based on the results, and considering the information reviewed for the assessment (ASS risk maps, subsurface conditions, etc), the natural soils at the site were considered to be PASS and an ASSMP is required to manage the soils during the proposed development.

2 ACID SULFATE SOIL MANAGEMENT PLAN (ASSMP)

2.1 Introduction

The most effective management strategy for dealing with PASS is to avoid disturbing the material. If this is not a viable option then the ASSMP should be implemented.

The objective of the ASSMP is to reduce the potential on-site and off-site environmental impacts associated with disturbance of PASS identified at the site. The ASSMP has been prepared generally in accordance with the ASS Manual 1998. Reference has also been made to the Queensland Acid Sulfate Soil Technical Manual v 3.8 (2002)[3].

The following issues are addressed in the ASSMP:
- Strategies for the management of PASS during development;
- Implementation of a soil and groundwater monitoring program; and
- Contingency procedures to be implemented in the event of the failure of management strategies.

2.2 Extent of Management

The investigation identified PASS in the natural soils at the site. Overlying fill material does not require treatment/management in relation to the generation of ASS conditions. However separating the fill soil from the underlying natural soil may not be practical, in which case all of the excavated soil should be considered to be PASS.

[2] NSW EPA, (2014). *Waste Classification Guidelines, Part 1: Classifying Waste.* (referred to as Waste Classification Guidelines 2014)
[3] Queensland Department of Natural Resources and Mines, (2002). *Queensland Acid Sulfate Soil Technical Manual. Soil Management Guidelines* version 3.8.

2.3 Management Options for ASS/PASS

Management options for ASS/PASS have been outlined and evaluated by EIS in the following table:

Table 2-1: Management of ASS/PASS

Option	Details	EIS Evaluation of Applicability
Option A: Disposal of PASS beneath the water table at a landfill	Immediate transport of natural PASS to landfill for disposal beneath the water table. A number of conditions have to be satisfied for burial beneath the water table to be viable. This option is not suitable for fill material or natural soil that has been impacted by contaminants.	Potential option for the natural soil provided the material is free of contamination. Classification in accordance with the NSW EPA Waste Classification Guidelines (2014)[4] would be required.
Option B: Treatment of PASS, waste classification and disposal to landfill	PASS is excavated and neutralised with lime. A waste classification is assigned for the off-site disposal of the treated PASS to landfill.	Most viable and preferred option considering proposed development details.
Option C: Treatment of PASS and on-site re-use.	PASS is excavated and neutralised with lime. The treated material is re-used on site with adequate capping. This option is not suitable for PASS that has been impacted by contaminants.	Not the preferred option for this project as material is not required for filling.

2.4 Preferred Option for Management of ASS/PASS

As outlined in the above table, the most viable and therefore the preferred option for managing ASS/PASS during the proposed development works is Option B (treatment of ASS/PASS, followed by waste classification and off-site disposal). The management procedure for Option B is outlined in the following subsection.

Procedures for the remaining two options are included in Section 2.5 for reference purposes. These options could be considered further in consultation with a suitably qualified environmental consultant and the relevant contractors if required by the client.

2.4.1 Treatment, Waste Classification and Disposal to Landfill (Option B)

Potential acid generation is typically managed by the addition of lime to neutralise acid that may be generated during and after the excavation works. The treated material should then be assigned a waste classification in accordance with the NSW EPA Waste Classification Guidelines - Part 1: Classifying Waste

[4] NSW EPA, (2014). *Waste Classification Guidelines, Part 1: Classifying Waste.* (referred to as Waste Classification Guidelines 2014)

(2014)[5] and Waste Classification Guidelines Part 4: Acid Sulfate Soils (2014)[6], and disposed of to a NSW EPA licensed landfill facility.

The procedures outlined in the following table should be implemented for this option:

Table 2-2: Management Procedure for Option B

Procedure	Details
<u>Step 1</u>: Lime selection	A slightly alkaline, low solubility product such as agricultural lime should be used. This form of lime is chemically stable and any excess lime takes a significant period of time (years) to influence soil pH beyond the depth of mixing. The lime particles eventually become coated with an insoluble layer of ferrihydrite ($Fe[OH]_3$) that inhibits further reaction. Long term alteration of groundwater conditions is not expected to occur as a result of the use of lime during the proposed development works.
<u>Step 2</u>: Set up treatment area/s	A treatment area for the mixing of excavated soil with agricultural lime should be established. As this site is relatively small this could consist of dusting the treatment surface with lime. The purpose of this guard layer is to minimise the risk of acidic water leaching from the base of the treatment area into the groundwater. Alternatively the treatment could be undertaken in a skip bin. An earthworks strategy should be prepared to ensure that sufficient space is available on–site to accommodate treatment of the PASS.
<u>Step 3</u>: Manage water run-off	Installation of detention tanks or construction of ponds may not be viable on this site therefore all the stockpiles should be covered with builders plastic or similar during rain to prevent the water coming into contact with the stockpiled material. If skip bins are used, bunding should not be necessary. However, the bins should be covered to prevent them from filling with rainwater. The application of neutralising agents into natural water bodies or water courses should be avoided unless carefully planned and approved by council and relevant authorities.
<u>Step 4</u>: Excavation & handling	PASS disturbed during development works should be immediately transferred to the designated treatment area and spread out in 150mm to 300mm thick layers. If possible the layers should be allowed to dry in order to aid the mixing process. The layers should then be interspersed with the appropriate amount of lime to aid in the effective mixing of lime and soil. Lime should be applied to the excavated material within the treatment area as soon as possible. If circumstances prevent the spreading and treatment of the material, the surface area of the stockpile should be minimised by forming a relatively high coned shape and avoiding 'spreading-out' of the stockpile. This will limit the surface area exposed to oxidation. Water

[5] NSW EPA, (2014). *Waste Classification Guidelines, Part 1: Classifying Waste.* (Part 1 of the Waste Classification Guidelines 2014)
[6] NSW EPA, (2014). *Waste Classification Guidelines, Part 4: Acid Sulfate Soils.* (Part 4 of the Waste Classification Guidelines 2014)

Procedure	Details
	infiltration should be minimised by covering the stockpile during wet weather. This will limit the formation and transport of acid leachate due to rainfall. The stockpile should be bunded to prevent erosion of the PASS and any movement of potentially acid leachate. Upstream surface runoff water should also be diverted around the stockpile.
Step 5: Lime treatment & pH testing	The laboratory analysis results have indicated that approximately 65kg lime per tonne of soil is required to adequately stabilise the PASS. An excavator or other suitable equipment (as deemed appropriate by the excavation contractor) should be used to thoroughly mix the lime through the soil. Alternatively use of a pug mill may be considered dependent upon the volume of soil to be treated in a timely fashion. The pH of the soil should be checked using the test method(s) outlined in the ASS Manual 1998 (Methods 21A and or 21Af) to confirm that PASS have been neutralised by lime addition. If required, additional lime should be added to the soil and additional mixing undertaken. Following treatment with lime the pH of the soil should be in the 5.5 to 8.5 range.
Step 6: Monitoring by qualified personnel	Monitoring should be undertaken by qualified personnel to ensure the mixing is undertaken to a suitable extent as the success of the neutralisation method relies on the effectiveness of the mixing process.
Step 7: Waste classification and off-site disposal	Following treatment the material should be tested and assigned a waste classification in accordance with the Parts 1 and 4 of the Waste Classification Guidelines 2014. All neutralised material should be disposed of off-site to a NSW EPA landfill licensed to accept treated PASS/ASS.

2.5 Alternative Management Options for ASS/PASS

As outlined in Sections 2.3 and 2.4, Option B is considered to be the most viable and appropriate option for managing ASS/PASS during the proposed development works. An outline of the management requirements for the remaining two options have been provided below for reference purposes. These options could be considered further in consultation with a suitably qualified environmental consultant and the relevant contractors if required by the client.

2.5.1 Disposal of PASS beneath the Water Table at a Landfill (Option A)

Natural soil classed as PASS may be disposed of below the water table at a landfill facility without lime treatment provided that the following conditions are met:

- The material is disposed below the water table within **24 hours** of excavation;
- The material meets the definition of 'virgin excavated natural material' (VENM) under the *Protection of the Environment Operations Act* (1997[7]), even though it contains sulfidic ores;

[7] *Protection of Environment Operations Act,* NSW Government, 1997 (POEO Act 1997)

- The receiving landfill is licensed by the NSW EPA to dispose of PASS below the water table; and
- The material meets the highly stringent pH criteria.

The procedures outlined in the following table should be implemented for this option:

Table 2-3: Management Procedure for Option A

Procedure	Details
Step 1: Contact Landfill	Prior to commencement of excavation works, the landfill should be contacted and the necessary approvals should be obtained for disposal.
Step 2: Excavation & Handling	Natural soil classed as PASS should be excavated/disturbed in stages. PASS must be kept wet at all times during excavation and subsequent handling, transport and storage until they can be disposed of safely.
Step 3: pH testing	The pH of the soil should be checked using the test method(s) outlined in the ASS Manual 1998 (Methods 21A and or 21Af). The pH of each load and the time of extraction should be recorded and forwarded to the landfill. If the pH **is less than 5.5** then the material is not suitable for burial beneath the water and Option B should be implemented.
Step 4: Transport	Provided that the pH of the excavated PASS is **not less than 5.5** the material can be loaded onto trucks and transported immediately to the landfill. Prior to burial the landfill will check the pH of each load. Any loads that do not meet the acceptance pH criteria will be turned away.

2.5.2 Treatment of PASS and On-site Re-use (Option C)

Potential acid generation is typically managed by the addition of lime to neutralise acid that may be generated during and after the excavation works. The treated material may be re-used on-site provided it is capped and not left exposed. The procedures outlined in the following table should be implemented for this option:

Table 2-4: Management Procedure for Option C

Procedure	Details
Steps 1 to 6	As outlined for Option B.
Step 7: On-site Re-use	Treated PASS should not be spread over sensitive areas (e.g. mangroves) or directly adjacent to waterways. The area where the treated PASS is going to be placed should be cleared and, if present, the turf should be removed. The area should be dusted with lime. The neutralised PASS should then be spread across the placement area in layers. Care should be taken not to disturb the underlying soil. On completion, the surface of the neutralised PASS should be dusted with additional lime prior to capping. A suitable capping layer (such as a clay liner or crushed sandstone)

	should be placed over the neutralised PASS. The finished surface should be turfed or paved to minimise the potential for erosion.

2.6 Groundwater Seepage and Dewatering

The procedure for managing water seepage and dewatering during works in the reserve and foreshore is outlined in the following table:

Table 3 5: Procedure for Managing Water Seepage and Monitoring

Procedure	Details
Step 1: Minimise the depth of dewatering	Where possible the material in the foreshore being excavated should be kept submerged to reduce the generation of ASS and/or acidic conditions. Where excavation works are in the foreshore bank, works should be staged over short durations to reduce the time and volume of PASS exposed to oxidation.
Step 2: Approvals for Groundwater Disposal	Reference should be made to the local council, NSW Office of Water / WaterNSW, Sydney Water and other relevant authority's approval requirements for further information in relation to disposal of water to either the sewer or stormwater systems.
Step 2: pH Testing and Neutralisation	Any water pumped from excavations in the foreshore bank should be placed in a portable tank, or appropriate holding facility, where samples can be obtained for testing. Prior to commencing site works a baseline pH should be established for the adjacent waterway. The pH of the adjacent waterway should be measured at the start and end of the working day. If the pH of the adjacent waterway deviates more than +/- 1pH unit from the baseline value, an experienced environmental consultant should be contacted immediately. The cause of the pH deviation should be established and corrective action taken. The water should be in the pH range of 6.5 to 8.5 (Schedule 5 of Protection of the Environment Operations (General) Regulation 2009[8]). If the pH is outside of this range, treatment will be necessary prior to disposal. Based on the disposal option chosen for the development, additional screening for contaminants may be required by the relevant authorities prior to disposal.
Step 3: On-going groundwater monitoring	In the event that extended pumping of water is necessary during the construction period, the quality of the groundwater should be monitored on a regular basis over the entire construction period. The pH should be measured and recorded on a regular basis. Immediate advice is to be sought from an experienced consultant if the pH at any location is not within 10% of the initial pH at the commencement of pumping. If required, corrective action should be taken as soon as possible. Laboratory analysis will be required on water samples as part of the corrective action to assess the quantity of neutralising agents required if treatment is necessary.

[8] NSW Government, (2009). *Protection of Environment Operations (General) Regulation,* Schedule 5 Prescribed matter for the definition of water pollution (page 124) (POEO Regulation 2009)

2.7 Contingency Plan

In the event the results of soil neutralisation or groundwater monitoring tests indicate a significant change in acidic conditions, the contingency plan should be implemented.

If soil monitoring indicates the presence of significantly more acidic material than expected or water monitoring indicates that the pH of the pumped water has become significantly more acidic, all excavation works should be placed on hold until further action is taken to limit the oxidation of PASS in the development area. Contingency works will be undertaken as follows:

- The depth to groundwater (i.e. the extent of de-watering) in the area of excavation will be measured;
- The pH of soils exposed to oxygen within the excavation will be measured to establish the source of the acidic conditions;
- Material found to be acidic will be excavated and neutralised in accordance with the methods presented in Section 2.4.1;
- Where suitable, in-place treatment involving lime addition and mixing may by adopted; and
- In the event unacceptable acidic levels are recorded by the groundwater monitoring, installation of a neutralisation trench (or similar) may be required to intercept and treat acidic groundwater prior to discharge. This could consist of an excavation filled with a sand/lime mixture designed to filter, intercept and treat groundwater flowing across the trench.

2.8 Disposal Information

The costs associated with the treatment and off-site disposal of PASS can be significant and may affect project viability. These costs should be assessed at an early stage of the project to avoid significant future unexpected additional costs.

Section 143 of the POEO Act1997 states that if waste is transported to a place that cannot lawfully be used as a waste facility for that waste, then the transporter and owner of the waste are each guilty of an offence. The transporter and owner of the waste have a duty to ensure that the waste is disposed of in an appropriate manner. EIS accepts no liability whatsoever for the unlawful disposal of any waste from any site.

3 LIMITATIONS

The report limitations are outlined below:

- EIS accepts no responsibility for any unidentified ASS or PASS issues at the site. Any unexpected problems/subsurface features that may be encountered during development works should be inspected by an environmental consultant as soon as possible;
- This report has been prepared based on site conditions which existed at the time of the investigation; scope of work and limitation outlined in the EIS proposal; and terms of contract between EIS and the client (as applicable);
- The conclusions presented in this report are based on investigation of conditions at specific locations, chosen to be as representative as possible under the given circumstances, visual observations of the site and immediate surrounds and documents reviewed as described in the report;

- Subsurface soil and rock conditions encountered between investigation locations may be found to be different from those expected. Groundwater conditions may also vary, especially after climatic changes;
- The investigation and preparation of this report have been undertaken in accordance with accepted practice for environmental consultants, with reference to applicable environmental regulatory authority and industry standards, guidelines and the assessment criteria outlined in the report;
- Where information has been provided by third parties, EIS has not undertaken any verification process, except where specifically stated in the report;
- EIS accept no responsibility for potentially asbestos containing materials that may exist at the site. These materials may be associated with demolition of pre-1990 constructed buildings or fill material at the site;
- EIS have not and will not make any determination regarding finances associated with the site;
- Additional investigation work may be required in the event of changes to the proposed development or landuse. EIS should be contacted immediately in such circumstances;
- Material considered to be suitable from a geotechnical point of view may be unsatisfactory from a soil contamination viewpoint, and vice versa;
- This report has been prepared for the particular project described and no responsibility is accepted for the use of any part of this report in any other context or for any other purpose;
- Copyright in this report is the property of EIS. EIS has used a degree of care, skill and diligence normally exercised by consulting professionals in similar circumstances and locality. No other warranty expressed or implied is made or intended. Subject to payment of all fees due for the investigation, the client alone shall have a licence to use this report;
- If the client, or any person, provides a copy of this report to any third party, such third party must not rely on this report except with the express written consent of EIS; and
- Any third party who seeks to rely on this report without the express written consent of EIS does so entirely at their own risk and to the fullest extent permitted by law, EIS accepts no liability whatsoever, in respect of any loss or damage suffered by any such third party.

If you have any questions concerning the contents of this letter please do not hesitate to contact us.

Kind Regards

Katrina Taylor
Senior Environmental Scientist

Adrian Kingswell
Principal Consultant

Appendices:

Appendix A: Report Figures
Appendix B: ASS Assessment Report Tables
Appendix C: Laboratory Reports & Chain of Custody Documents

Appendix A: Report Figures

AERIAL IMAGE SOURCE: MAPS.AU.NEARMAP.COM, 27 DEC 2018.

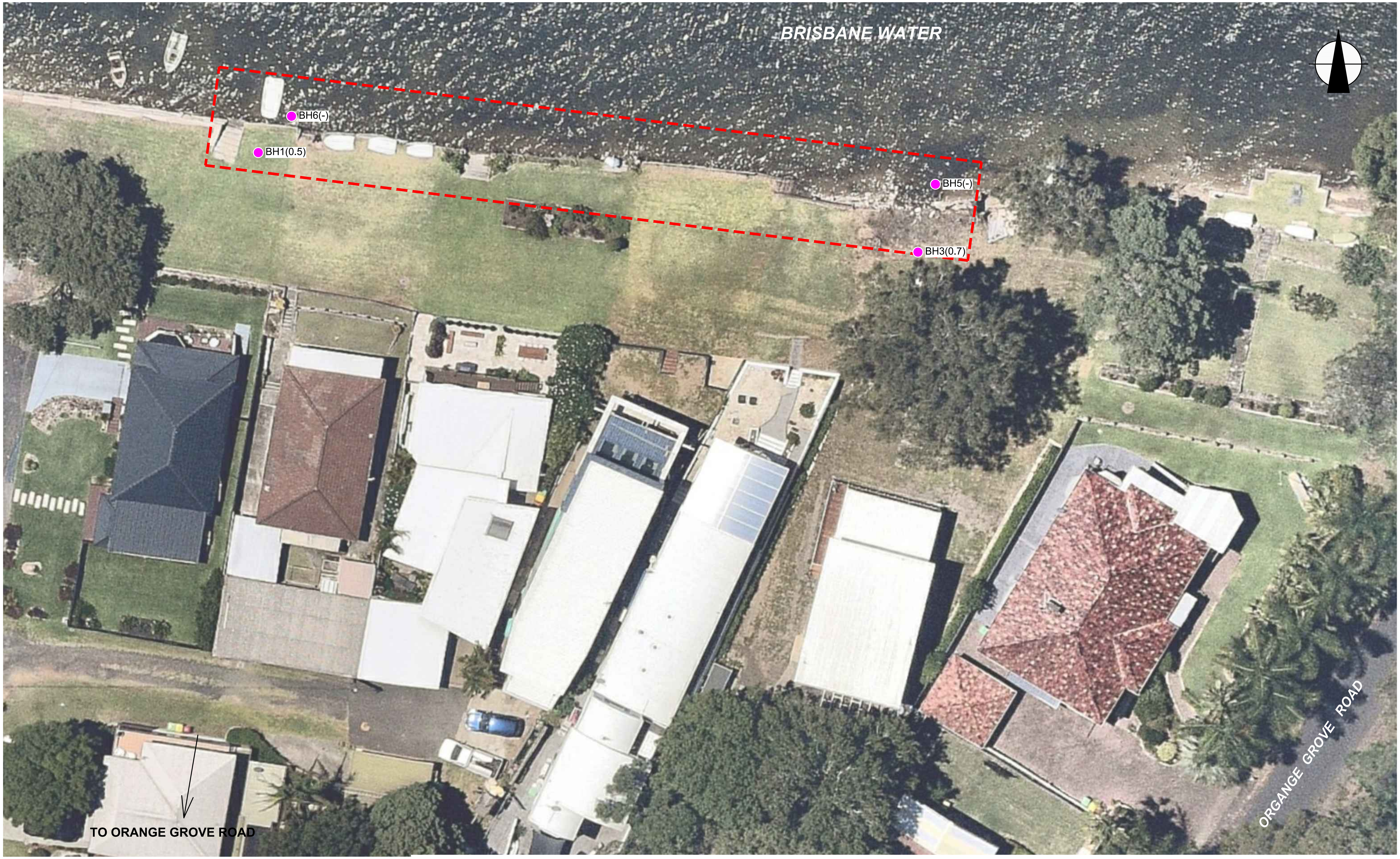
This plan should be read in conjunction with the Environmental report.

Title:	SITE LOCATION PLAN
Location:	RIP ROAD RESERVE BLACKWALL, NSW
Report No: E32217BT	Figure No: 1

JKEnvironments

PLOT DATE: 5/04/2019 12:48:11 PM DWG FILE: S:\5 EIS\5C EIS JOBS\32000\E32217BT BLACKWALL\CAD\E32217BT.DWG

AERIAL IMAGE SOURCE: MAPS.AU.NEARMAP.COM, 27 DEC 2018.



PLOT DATE: 5/04/2019 12:52:43 PM DWG FILE: S:\5 EIS\5C EIS JOBS\32000'S\E32217BT BLACKWALL\CAD\E32217BT.DWG

LEGEND

APPROXIMATE SITE BOUNDARY

BH (Fill Depth) BOREHOLE LOCATION, NUMBER AND DEPTH OF FILL (m)

0 3 6 9 12 15

SCALE 1:300 @A3 METRES

This plan should be read in conjunction with the Environmental report.

Title:	SAMPLE LOCATION PLAN
Location:	RIP ROAD RESERVE BLACKWALL, NSW
Report No: E32217BT	Figure No: 2

JKEnvironments

Appendix B: ASS Assessment Report Tables

ABBREVIATIONS AND EXPLANATIONS

Abbreviations used in the Tables:

NA: Not Analysed
NC: Not Calculated
NL: Not Limiting
NSL: No Set Limit
pH_{KCL} : pH of filtered 1:20, 1M KCL extract, shaken overnight
pH_{ox} : pH of filtered 1:20 1M KCl after peroxide digestion
PQL: Practical Quantitation Limit
SAC: Site Assessment Criteria
S_{Cr}: Chromium reducible sulfur
S_{POS}: Peroxide oxidisable Sulfur
TAA: Total Actual Acidity in 1M KCL extract titrated to pH6.5
TPA: Total Potential Acidity, 1M KCL peroxide digest
TSA: Total Sulfide Acidity (TPA-TAA)

TABLE A
SUMMARY OF LABORATORY RESULTS - ACID SULFATE SOIL ANALYSIS (sPOCAS)

		Analysis	pH_{KCL}	TAA	pH_{ox}	TPA	TSA	S_{POS}	Liming Rate
				pH 6.5		pH 6.5	pH 6.5	%w/w	kg $CaCO_3$/tonne
Acid Sulfate Soil Manual (1998) -Action Criteria		Coarse Textured Soil	pH 5.0	18molH+/ tonne	pH 5.0	18molH+/ tonne	18molH+/ tonne	0.03% w/w	
Sample Reference	**Sample Depth (m)**	**Sample Description**							
BH1	1.5-1.7	Clayey Sand	4.0	30	3.8	82	52	0.03	3.9
BH1	1.5-1.7	Laboratory Duplicate	4.0	30	3.7	78	48	0.04	4.1
BH1	2.3-2.4	Sandy Clay	3.7	61	4.3	68	6	0.009	5.2
BH3	0.9-1.0	Silty Clay	6.0	<5	4.4	25	24	0.009	<0.75
BH3	1.5-1.6	Silty Clay	4.3	14	4.1	30	16	0.009	1.5
BH5	0.3-0.4	Sand	9.4	<5	7.1	<5	<5	0.04	<0.75
BH5	0.9-1.0	Clay	7.6	<5	2.7	90	90	0.27	8.7
BH6	0.1-0.2	Sandy Clay	5.6	<5	2.1	810	810	1.4	65
BH6	0.9-1.0	Clay	6.7	<5	2.6	140	140	0.31	12
Total Number of Samples			9	9	9	9	9	9	9
Minimum Value			3.7	14	2.1	25	6	0.009	1.5
Maximum Value			9.4	61	7.1	810	810	1.4	65

Values Exceeding Action Criteria **VALUE**

Appendix C: Laboratory Reports & Chain of Custody Documents

Envirolab Services Pty Ltd
ABN 37 112 535 645
12 Ashley St Chatswood NSW 2067
ph 02 9910 6200   fax 02 9910 6201
customerservice@envirolab.com.au
www.envirolab.com.au

CERTIFICATE OF ANALYSIS 212614

Client Details

Client	Environmental Investigation Services
Attention	Katrina Taylor
Address	PO Box 976, North Ryde BC, NSW, 1670

Sample Details

Your Reference	E32217BT, Blackwall
Number of Samples	13 Soil
Date samples received	04/03/2019
Date completed instructions received	04/03/2019

Analysis Details

Please refer to the following pages for results, methodology summary and quality control data.

Samples were analysed as received from the client. Results relate specifically to the samples as received.

Results are reported on a dry weight basis for solids and on an as received basis for other matrices.

Report Details

Date results requested by	11/03/2019
Date of Issue	11/03/2019

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Accredited for compliance with ISO/IEC 17025 - Testing. **Tests not covered by NATA are denoted with ***

Results Approved By
Nick Sarlamis, Inorganics Supervisor

Authorised By

Jacinta Hurst, Laboratory Manager

sPOCAS + %S w/w						
Our Reference		212614-4	212614-5	212614-7	212614-8	212614-10
Your Reference	UNITS	BH1	BH1	BH3	BH3	BH5
Depth		1.5-1.7	2.3-2.4	0.9-1.0	1.5-1.6	0.3-0.4
Date Sampled		28/02/2019	28/02/2019	28/02/2019	28/02/2019	28/02/2019
Type of sample		Soil	Soil	Soil	Soil	Soil
Date prepared	-	06/03/2019	06/03/2019	06/03/2019	06/03/2019	06/03/2019
Date analysed	-	06/03/2019	06/03/2019	06/03/2019	06/03/2019	06/03/2019
pH_{kcl}	pH units	4.0	3.7	6.0	4.3	9.4
TAA pH 6.5	moles H^+/t	30	61	<5	14	<5
s-TAA pH 6.5	%w/w S	0.05	0.1	<0.01	0.02	<0.01
pH_{Ox}	pH units	3.8	4.3	4.4	4.1	7.1
TPA pH 6.5	moles H^+/t	82	68	25	30	<5
s-TPA pH 6.5	%w/w S	0.13	0.11	0.04	0.05	<0.01
TSA pH 6.5	moles H^+/t	52	6	24	16	<5
s-TSA pH 6.5	%w/w S	0.08	0.01	0.04	0.03	<0.01
ANC_E	% $CaCO_3$	<0.05	<0.05	<0.05	<0.05	0.25
a-ANC_E	moles H^+/t	<5	<5	<5	<5	50
s-ANC_E	%w/w S	<0.05	<0.05	<0.05	<0.05	0.08
S_{KCl}	%w/w S	<0.005	<0.005	<0.005	0.007	0.02
S_P	%w/w	0.03	0.01	0.01	0.02	0.06
S_{POS}	%w/w	0.03	0.009	0.009	0.009	0.04
a-S_{POS}	moles H^+/t	20	6	6	5	26
Ca_{KCl}	%w/w	0.15	0.03	0.25	0.19	0.05
Ca_P	%w/w	0.18	0.04	0.31	0.25	0.18
Ca_A	%w/w	0.028	0.007	0.061	0.052	0.13
Mg_{KCl}	%w/w	0.029	0.034	0.017	0.021	0.019
Mg_P	%w/w	0.035	0.042	0.019	0.024	0.037
Mg_A	%w/w	0.006	0.008	<0.005	<0.005	0.019
S_{HCl}	%w/w S	0.008	0.008	<0.005	0.008	<0.005
S_{NAS}	%w/w S	0.006	0.005	<0.005	<0.005	<0.005
a-S_{NAS}	moles H^+/t	<5	<5	<5	<5	<5
s-S_{NAS}	%w/w S	<0.01	<0.01	<0.01	<0.01	<0.01
Fineness Factor	-	1.5	1.5	1.5	1.5	1.5
a-Net Acidity	moles H^+/t	52	69	7	20	<5
s-Net Acidity	%w/w S	0.08	0.11	0.01	0.03	<0.01
Liming rate	kg $CaCO_3$/t	3.9	5.2	<0.75	1.5	<0.75
s-Net Acidity without -ANCE	%w/w S	0.084	0.11	0.011	0.032	0.042
a-Net Acidity without ANCE	moles H^+/t	52	69	7.0	20	26
Liming rate without ANCE	kg $CaCO_3$/t	3.9	5.2	<0.75	1.5	2.0

sPOCAS + %S w/w				
Our Reference		212614-11	212614-12	212614-13
Your Reference	UNITS	BH5	BH6	BH6
Depth		0.9-1.0	0.1-0.2	0.9-1.0
Date Sampled		28/02/2019	28/02/2019	28/02/2019
Type of sample		Soil	Soil	Soil
Date prepared	-	06/03/2019	06/03/2019	06/03/2019
Date analysed	-	06/03/2019	06/03/2019	06/03/2019
pH_{kcl}	pH units	7.6	5.6	6.7
TAA pH 6.5	moles H^+ /t	<5	<5	<5
s-TAA pH 6.5	%w/w S	<0.01	<0.01	<0.01
pH_{Ox}	pH units	2.7	2.1	2.6
TPA pH 6.5	moles H^+ /t	90	810	140
s-TPA pH 6.5	%w/w S	0.14	1.3	0.22
TSA pH 6.5	moles H^+ /t	90	810	140
s-TSA pH 6.5	%w/w S	0.14	1.3	0.22
ANC_E	% $CaCO_3$	<0.05	<0.05	<0.05
a-ANC_E	moles H^+ /t	<5	<5	<5
s-ANC_E	%w/w S	<0.05	<0.05	<0.05
S_{KCl}	%w/w S	0.04	0.11	0.03
S_P	%w/w	0.31	1.5	0.34
S_{POS}	%w/w	0.27	1.4	0.31
a-S_{POS}	moles H^+ /t	170	860	200
Ca_{KCl}	%w/w	0.05	0.06	0.05
Ca_P	%w/w	0.09	0.09	0.10
Ca_A	%w/w	0.041	0.030	0.052
Mg_{KCl}	%w/w	0.046	0.073	0.056
Mg_P	%w/w	0.058	0.087	0.069
Mg_A	%w/w	0.011	0.014	0.013
S_{HCl}	%w/w S	<0.005	<0.005	<0.005
S_{NAS}	%w/w S	<0.005	<0.005	<0.005
a-S_{NAS}	moles H^+ /t	<5	<5	<5
s-S_{NAS}	%w/w S	<0.01	<0.01	<0.01
Fineness Factor	-	1.5	1.5	1.5
a-Net Acidity	moles H^+ /t	120	860	160
s-Net Acidity	%w/w S	0.19	1.4	0.25
Liming rate	kg $CaCO_3$ /t	8.7	65	12
s-Net Acidity without -ANCE	%w/w S	0.19	1.4	0.25
a-Net Acidity without ANCE	moles H^+ /t	120	860	160
Liming rate without ANCE	kg $CaCO_3$ /t	8.7	65	12

Method ID	Methodology Summary
Inorg-064	sPOCAS determined using titrimetric and ICP-AES techniques. Based on Acid Sulfate Soils Laboratory Methods Guidelines, Version 2.1 - June 2004.

QUALITY CONTROL: sPOCAS + %S w/w					Duplicate				Spike Recovery %	
Test Description	Units	PQL	Method	Blank	#	Base	Dup.	RPD	LCS-1	[NT]
Date prepared	-			06/03/2019	4	06/03/2019	06/03/2019		06/03/2019	[NT]
Date analysed	-			06/03/2019	4	06/03/2019	06/03/2019		06/03/2019	[NT]
pH $_{kcl}$	pH units		Inorg-064	[NT]	4	4.0	4.0	0	91	[NT]
TAA pH 6.5	moles H^+/t	5	Inorg-064	<5	4	30	30	0	95	[NT]
s-TAA pH 6.5	%w/w S	0.01	Inorg-064	<0.01	4	0.05	0.05	0	[NT]	[NT]
pH $_{Ox}$	pH units		Inorg-064	[NT]	4	3.8	3.7	3	95	[NT]
TPA pH 6.5	moles H^+/t	5	Inorg-064	<5	4	82	78	5	117	[NT]
s-TPA pH 6.5	%w/w S	0.01	Inorg-064	<0.01	4	0.13	0.12	8	[NT]	[NT]
TSA pH 6.5	moles H^+/t	5	Inorg-064	<5	4	52	48	8	[NT]	[NT]
s-TSA pH 6.5	%w/w S	0.01	Inorg-064	<0.01	4	0.08	0.08	0	[NT]	[NT]
ANC_E	% $CaCO_3$	0.05	Inorg-064	<0.05	4	<0.05	<0.05	0	[NT]	[NT]
a-ANC_E	moles H^+/t	5	Inorg-064	<5	4	<5	<5	0	[NT]	[NT]
s-ANC_E	%w/w S	0.05	Inorg-064	<0.05	4	<0.05	<0.05	0	[NT]	[NT]
S_{KCl}	%w/w S	0.005	Inorg-064	<0.005	4	<0.005	<0.005	0	[NT]	[NT]
S_P	%w/w	0.005	Inorg-064	<0.005	4	0.03	0.04	29	[NT]	[NT]
S_{POS}	%w/w	0.005	Inorg-064	<0.005	4	0.03	0.04	29	[NT]	[NT]
a-S_{POS}	moles H^+/t	5	Inorg-064	<5	4	20	22	10	[NT]	[NT]
Ca_{KCl}	%w/w	0.005	Inorg-064	<0.005	4	0.15	0.15	0	[NT]	[NT]
Ca_P	%w/w	0.005	Inorg-064	<0.005	4	0.18	0.19	5	[NT]	[NT]
Ca_A	%w/w	0.005	Inorg-064	<0.005	4	0.028	0.041	38	[NT]	[NT]
Mg_{KCl}	%w/w	0.005	Inorg-064	<0.005	4	0.029	0.028	4	[NT]	[NT]
Mg_P	%w/w	0.005	Inorg-064	<0.005	4	0.035	0.036	3	[NT]	[NT]
Mg_A	%w/w	0.005	Inorg-064	<0.005	4	0.006	0.008	29	[NT]	[NT]
S_{HCl}	%w/w S	0.005	Inorg-064	<0.005	4	0.008	0.008	0	[NT]	[NT]
S_{NAS}	%w/w S	0.005	Inorg-064	<0.005	4	0.006	0.006	0	[NT]	[NT]
a-S_{NAS}	moles H^+/t	5	Inorg-064	<5	4	<5	<5	0	[NT]	[NT]
s-S_{NAS}	%w/w S	0.01	Inorg-064	<0.01	4	<0.01	<0.01	0	[NT]	[NT]
Fineness Factor	-	1.5	Inorg-064	<1.5	4	1.5	1.5	0	[NT]	[NT]
a-Net Acidity	moles H^+/t	5	Inorg-064	<5	4	52	55	6	[NT]	[NT]
s-Net Acidity	%w/w S	0.01	Inorg-064	<0.01	4	0.08	0.09	12	[NT]	[NT]
Liming rate	kg $CaCO_3$/t	0.75	Inorg-064	<0.75	4	3.9	4.1	5	[NT]	[NT]
s-Net Acidity without -ANCE	%w/w S	0.01	Inorg-064	<0.01	4	0.084	0.087	4	[NT]	[NT]
a-Net Acidity without ANCE	moles H^+/t	5	Inorg-064	<5	4	52	55	6	[NT]	[NT]

QUALITY CONTROL: sPOCAS + %S w/w					Duplicate				Spike Recovery %	
Test Description	Units	PQL	Method	Blank	#	Base	Dup.	RPD	LCS-1	[NT]
Liming rate without ANCE	kg $CaCO_3$/t	0.75	Inorg-064	<0.75	4	3.9	4.1	5	[NT]	[NT]

Result Definitions

NT	Not tested
NA	Test not required
INS	Insufficient sample for this test
PQL	Practical Quantitation Limit
<	Less than
>	Greater than
RPD	Relative Percent Difference
LCS	Laboratory Control Sample
NS	Not specified
NEPM	National Environmental Protection Measure
NR	Not Reported

Quality Control Definitions

Blank	This is the component of the analytical signal which is not derived from the sample but from reagents, glassware etc, can be determined by processing solvents and reagents in exactly the same manner as for samples.
Duplicate	This is the complete duplicate analysis of a sample from the process batch. If possible, the sample selected should be one where the analyte concentration is easily measurable.
Matrix Spike	A portion of the sample is spiked with a known concentration of target analyte. The purpose of the matrix spike is to monitor the performance of the analytical method used and to determine whether matrix interferences exist.
LCS (Laboratory Control Sample)	This comprises either a standard reference material or a control matrix (such as a blank sand or water) fortified with analytes representative of the analyte class. It is simply a check sample.
Surrogate Spike	Surrogates are known additions to each sample, blank, matrix spike and LCS in a batch, of compounds which are similar to the analyte of interest, however are not expected to be found in real samples.

Australian Drinking Water Guidelines recommend that Thermotolerant Coliform, Faecal Enterococci, & E.Coli levels are less than 1cfu/100mL. The recommended maximums are taken from "Australian Drinking Water Guidelines", published by NHMRC & ARMC 2011.

Laboratory Acceptance Criteria

Duplicate sample and matrix spike recoveries may not be reported on smaller jobs, however, were analysed at a frequency to meet or exceed NEPM requirements. All samples are tested in batches of 20. The duplicate sample RPD and matrix spike recoveries for the batch were within the laboratory acceptance criteria.

Filters, swabs, wipes, tubes and badges will not have duplicate data as the whole sample is generally extracted during sample extraction.

Spikes for Physical and Aggregate Tests are not applicable.

For VOCs in water samples, three vials are required for duplicate or spike analysis.

Duplicates: >10xPQL - RPD acceptance criteria will vary depending on the analytes and the analytical techniques but is typically in the range 20%-50% – see ELN-P05 QA/QC tables for details; <10xPQL - RPD are higher as the results approach PQL and the estimated measurement uncertainty will statistically increase.

Matrix Spikes, LCS and Surrogate recoveries: Generally 70-130% for inorganics/metals; 60-140% for organics (+/-50% surrogates) and 10-140% for labile SVOCs (including labile surrogates), ultra trace organics and speciated phenols is acceptable.

In circumstances where no duplicate and/or sample spike has been reported at 1 in 10 and/or 1 in 20 samples respectively, the sample volume submitted was insufficient in order to satisfy laboratory QA/QC protocols.

When samples are received where certain analytes are outside of recommended technical holding times (THTs), the analysis has proceeded. Where analytes are on the verge of breaching THTs, every effort will be made to analyse within the THT or as soon as practicable.

Where sampling dates are not provided, Envirolab are not in a position to comment on the validity of the analysis where recommended technical holding times may have been breached.

Measurement Uncertainty estimates are available for most tests upon request.

Envirolab Services Pty Ltd
ABN 37 112 535 645
12 Ashley St Chatswood NSW 2067
ph 02 9910 6200 fax 02 9910 6201
customerservice@envirolab.com.au
www.envirolab.com.au

SAMPLE RECEIPT ADVICE

Client Details	
Client	Environmental Investigation Services
Attention	Katrina Taylor

Sample Login Details	
Your reference	E32217BT, Blackwall
Envirolab Reference	212614
Date Sample Received	04/03/2019
Date Instructions Received	04/03/2019
Date Results Expected to be Reported	11/03/2019

Sample Condition	
Samples received in appropriate condition for analysis	YES
No. of Samples Provided	13 Soil
Turnaround Time Requested	Standard
Temperature on Receipt (°C)	7.8
Cooling Method	Ice Pack
Sampling Date Provided	YES

Comments
Nil

Please direct any queries to:

Aileen Hie	Jacinta Hurst
Phone: 02 9910 6200	**Phone: 02 9910 6200**
Fax: 02 9910 6201	**Fax: 02 9910 6201**
Email: ahie@envirolab.com.au	**Email: jhurst@envirolab.com.au**

Analysis Underway, details on the following page:

Envirolab Services Pty Ltd
ABN 37 112 535 645
12 Ashley St Chatswood NSW 2067
ph 02 9910 6200 fax 02 9910 6201
customerservice@envirolab.com.au
www.envirolab.com.au

Sample ID	sPOCAS + %S w/w	On Hold
BH1-0.2-0.4		✓
BH1-0.6-0.7		✓
BH1-1.0-1.1		✓
BH1-1.5-1.7	✓	
BH1-2.3-2.4	✓	
BH3-0.1-0.2		✓
BH3-0.9-1.0	✓	
BH3-1.5-1.6	✓	
BH5-0-0.1		✓
BH5-0.3-0.4	✓	
BH5-0.9-1.0	✓	
BH6-0.1-0.2	✓	
BH6-0.9-1.0	✓	

The '✓' indicates the testing you have requested. **THIS IS NOT A REPORT OF THE RESULTS.**

Additional Info
Sample storage - Waters are routinely disposed of approximately 1 month and soils approximately 2 months from receipt.
Requests for longer term sample storage must be received in writing.

SAMPLE AND CHAIN OF CUSTODY FORM

TO:		FROM:
ENVIROLAB SERVICES PTY LTD 12 ASHLEY STREET CHATSWOOD NSW 2067 P: (02) 99106200 F: (02) 99106201 Attention: Aileen	EIS Job Number: E32217BT Date Results Required: STANDARD Page: ~~1-Jan~~ 1/1	JKEnvironments REAR OF 115 WICKS ROAD MACQUARIE PARK, NSW 2113 P: 02-9888 5000 F: 02-9888 5001 Attention: Katrina Taylor

Location: Blackwall | Sample Preserved in Esky on Ice

Sampler: JH | Tests Required

Date Sampled	Lab Ref:	Sample Number	Depth (m)	Sample Container	Sample Description	sPOCAS	pH (1:5 water)										
28/02/2019	1	BH1	0.2-0.4	P	FILL												
↓	2	↓	0.6-0.7	↓	↓												
↓	3	↓	1.0-1.1	↓	↓												
↓	4	↓	1.5-1.7	↓	↓	X											
↓	5	↓	2.3-2.4	↓	↓	X											
↓	6	BH3	0.1-0.2	↓	FILL												
↓	7	↓	0.9-1.0	↓	↓	X											
↓	8	↓	1.5-1.6	↓	↓	X											
↓	9	BH5	0-0.1	↓	NATURAL												
↓	10	↓	0.3-0.4	↓	↓	X											
↓		↓		↓	↓												
↓	11	↓	0.9-1.0	↓	↓	X											
↓	12	BH6	0.1-0.2	↓	↓	X											
↓	13	↓	0.9-1.0	↓	↓	X											

Envirolab Services
12 Ashley St
Chatswood NSW 2067
Ph: (02) 9910 6200

ENVIROLAB

Job No: 212614

Date Received: 4/3/19

Time Received: 10:45

Received by: LN

Temp: Cool/Ambient

Cooling: Ice/Icepack

Security: Intact/Broken/None

Remarks (comments/detection limits required):		Sample Containers: G - 250mg Glass Jar A - Ziplock Asbestos Bag P - Plastic Bag	
Relinquished By: JH	Date: 1/3/19	Time:	Received By: ELS LILY NOTT	Date: 4/3/19 10:45

Appendix F

Safety in Design Risk Register

Risk Assessment

The following risk assessment is provided to facilitate in ensuring that all stages of the project life cycle are as safe as possible through good engineering design. Having regard to the *Safety in Design Code of Practice (Safe Work Australia, 2014)*, the risk assessment involves the following steps:

- identify reasonably foreseeable hazards associated with the design of the structure. Hazards include those that affect people, infrastructure and the environment;
- if necessary, assess the risks arising from the hazards;
- eliminate or minimise the risk by designing control measures, and,
- review the control measures.

The risk associated with a particular hazard is a combination of the likelihood of the hazard occurring and its consequence. The description of likelihood and consequence is presented in **Table 1** and **Table** *2* respectively.

Table 1: Likelihood table.

Likelihood	Description
5 Almost Certain	Could happen at any time under normal circumstances. Is expected to occur at regular intervals.
4 Likely	Probably will occur under normal circumstances. Has occurred several times in the past on similar projects.
3 Possible	Possibility is will occur under normal circumstances. Has occurred a few times in the past on similar projects.
2 Unlikely	Could happen but unlikely under normal circumstances. Has occurred once in the past on a similar project.
1 Rare	Will probably never occur. May happen in exceptional circumstances.

Table 2: Severity table.

Severity	Description
5 Catastrophic	People – potential death. Infrastructure – catastrophic failure incurring significant financial loss. Environment – significant and irreversible environmental impact with regional or national effect.
4 Major	People – potential permanent or long term disability or illness requiring urgent medical attention and hospital admission. Infrastructure – significant damage incurring high financial loss. Environment – significant environmental impact with regional long term effects.
3 Moderate	People – potential temporary disability or illness requiring medical attention. Infrastructure – moderate damage incurring moderate financial loss. Environment – significant environmental impact with localised short term effect or moderate environmental impact with regional long term effect.
2 Minor	People – minor injury requiring first aid. Infrastructure – minor damage incurring low financial loss. Environment – minor environmental impact with localised short term effect.
1 Insignificant	People – negligible injury or discomfort. Nor medical treatment or measurable physical effects. Infrastructure – negligible damage possibly requiring minor repairs and negligible financial loss. Environment – negligible environmental impact with localised short term effect.

The risk assessment scores consider likelihood and consequence, which are presented in the Risk Matrix in **Table 3**. An interpretation of risk assessment scores in regards to “tolerable” and “unacceptable” risk is provided in the table.

Table 3: Risk Matrix.

Likelihood	**Consequence**				
	5 Catastrophic	4 Major	3 Moderate	2 Minor	1 Insignificant
5 Almost Certain	E25	E20	E15	H10	M5
4 Likely	E20	E16	H12	M8	L4
3 Possible	E15	H12	M9	M6	L3
2 Unlikely	H10	M8	M6	L4	L2
1 Rare	M5	L4	L3	L2	L1

Risk Assessment Scores	**Action Required**
Extreme (20-25) High (10-20)	Unacceptable risk requires immediate attention to eliminate or reduce risk.
Medium (5-9)	Control the risks and hazards. If residual risk exists, which are not possible to control, work may proceed provided stakeholders understand the residual risk.
Low (1-4)	Acceptable to tolerable risk, work can proceed.

Managing “unacceptable” risks is achieved by implementing control measures. A hierarchy of control measures is established in the *Safety in Design Code of Practice (Safe Work Australia, 2014)*. The most effective control measure should be implemented where practical. The hierarchy of control measures are as follows in order of most effective to least effective:

- Eliminate – ‘Design out’ the hazard when new materials, equipment and work systems are being purchased for the workplace;

If it is not reasonably practicable to eliminate a hazard the following control measures should be considered:

- Substitute - Substitute less hazardous materials, equipment or substances and use smaller sized containers;
- Isolate – separate the workers from hazards using barriers, enclosing noisy equipment and providing exhaust or ventilation systems;
- Engineering – use engineering controls to reduce the risks such as guards on equipment, hoists or other lifting and moving equipment;
- Administrative – Minimise the risk by adopting safe working practices or providing appropriate training, instruction or information.
- Personal Protective Equipment – Make sure that appropriate PPE is available and used correctly.

A detailed risk assessment for the Watsons Bay Seawall is provided in the **Table** attached. The risk assessment cross references design standards and appoints a “Risk Manager” who is responsible for ensuring controls and mitigation measures are implemented correctly. When a new or unforeseen hazard is identified or the controls and mitigation measures are deemed insufficient to manage the risk of a hazard, the site and/or works should be made safe and work should cease until the risk is adequately addressed.

PROJECT TITLE:	Project Number:	Office Address:
Rip Road Reserve Foreshore Rehabilitation	PA1952	Haskoning Australia, Level 14, 56 Berry Street North Sydney NSW 2060
BRIEF DESCRIPTION OF DESIGN ELEMENT:	**PHASE OF STRUCTURAL LIFECYCLE:**	
Design of Rock Revetment and Block Sandstone Seawall	**Design Stage - Risk Assessment**	

Date:	Responsible Officer:	Project Role:	
12-12-19	Gary Blumberg	Project Director	

Risk ID	Hazard	Reference	Initial risk or risk at previous design stage			Strategy/Mitigation Measure	Residual Risk			Risk Manager	Additional Comments
			Consequence 1-5	Likelihood 1-5	Rating		Consequence 1-5	Likelihood 1-5	Rating		
1 SAFETY IN DESIGN											
1.1	Design objectives not achieved		4	2	8	Ongoing engagement with Council and relevant stakeholders to ensure the design objectives are achieved.	3	1	3	RHDHV / Council	
1.2	Damage during design life caused by incorrect design parameters (water level and wave height)	5, 7	4	2	8	Design parameters, in particular water level and waves assessed by during development of the concept design. Design parameters to be reviewed for detaield design using industry recognised best practice.	4	1	4	RHDHV	
1.3	Damage during design life caused by inferior materials	2, 5, 20, 22, 23	3	2	6	Materials requirements to be outlined in the Technical Specification in accordance with industry recognised standards and guidelines.	3	1	3	RHDHV	
1.4	Damage during design life caused by inadequate Factor of Safety	4, 5, 20	4	2	8	Design to be in accordance with Australian Standards and an appropriate Factor of Safety.	4	1	4	RHDHV	
1.5	Inadequate geotechnical information and/or geotechnical instability	1	4	2	8	Geotechnical information and review of geotexchncial stability provided JK Geotechnics (2019)	4	1	4	RHDHV / Contractor	
1.6	Inadequate survey information and design located outside of Council owned land		3	2	6	Survey carried out by Stephen Thorne and Associates (2019). DBYD request submitted to assist in location of services.	2	1	2	RHDHV / Council	
1.7	Design not approved by agencies/Council		4	2	8	Regular communication maintained with Council. Council to consult with relevant stakeholders to ensure their expectations are incorporated into the design.	4	1	4	RHDHV / Council	
1.8	Design not complying with relevant legislation or standards	2, 3, 4, 5, 7	4	2	8	Legislation and standards to be reviewed and relevant requirements to be brought to the attention of Council and Agencies where required.	4	1	4	RHDHV	
1.9	Design exceeds Councils budget		3	2	6	Regular communication with Council to understand expectations.	4	1	4	RHDHV / Council	
1.10	Adverse impact on natural environment including riparian vegetation and Aboriginal Heritage	15	3	2	6	Council to assess impact on the environment and complete a Review of Environmental Factors or similar. Council and RHDHV engaged with Guringai Tribal Link Aboriginal Corporation during inception site walkover	2	1	2	RHDHV / Council	
1.11	Confusion between water and land based vertical datum's		4	3	12	Adopt Australian Height Datum (AHD) as project datum for all design documentation and clearly state datum on drawings.	4	1	4	RHDHV	
2 RISK DURING CONSTRUCTION											
2.1	Structure cannot be constructed with available resources and equipment		4	2	8	Experienced coastal engineers involved in completing the detailed design and thorough tendering processes to be carried out. Tender to incluе a Method Statement Schedule.	4	1	4	RHDHV / Council / Contractor	
2.2	Unsuitable contract		3	3	9	Council is to prepare contract documentation and a Request for Tender. This process shall include selection of contract terms (e.g. lump sum or rates based contract) and inclusion of the appropriate Australian Standard or other suitable documentation.	2	1	2	Council	

Risk ID	Hazard	Reference	Initial risk or risk at previous design stage			Strategy/Mitigation Measure	Residual Risk			Risk Manager	Additional Comments
			Consequence 1-5	Likelihood 1-5	Rating		Consequence 1-5	Likelihood 1-5	Rating		
2.3	Inadequate communication of information		3	2	6	Council is to liaise with the Contractor and provide all documentation to complete the works in a timely manner including Drawings and any revisions to the drawings and the Technical Specification.	2	1	2	Council	
2.4	Works damaged during construction		2	3	6	The Contractor would be responsible for protecting the Works and selecting suitable methodology to construct the Works. The Contractor is required to make its own assessment of wave action, shoreline erosion, rainfall/stormwater and high water levels to ensure the Works are not adversely impacted.	2	2	4	Contractor	
2.5	Impacts on the environment, in particular the marine and foreshore environment.	8, 9, 10, 11, 13, 14, 17	4	3	12	Technical Specification to include a template for preparation of a Works Environmental Management Plan (WEMP) by Contractor. WEMP to address legislative, administrative and implementation requirements for all risks during construction. Contractor's WEMP to be approved by Council prior to Contractor commencing work.	4	2	8	Council / Contractor	
2.6	Injury to workers or the public	11, 16	4	3	12	All Contractors and Sub Contractors to implement Safe Work Method Statements. Construction site to be fenced to prevent public access at all times.	4	2	8	Council / Contractor	
2.7	Noise generation during construction	14	2	5	10	Contractor to address noise in Works Environmental Management Plan (WEMP) that shall be approved by Council prior to Contractor starting work.	2	2	4	Council / Contractor	
2.8	Working outside standard construction hours and noise generation	14	3	3	9	Contractor to address working hours in Works Environmental Management Plan (WEMP) that shall be approved by Council prior to Contractor starting work.	3	1	3	Council / Contractor	
2.9	Impact on riparian vegetation during construction	15	3	2	6	Contractor to address riparian vegetation in Works Environmental Management Plan (WEMP) that shall be approved by Council prior to Contractor starting work.	3	1	3	Council / Contractor	
2.10	Impact on Aboriginal heritage during construction	15	3	2	6	Contractor to address Aboriginal heritage in Works Environmental Management Plan (WEMP) that shall be approved by Council prior to Contractor starting work.	3	1	3	Council / Contractor	
2.10	Pollution due to fuel spill	8, 9, 10	2	4	8	To be addressed in WEMP including storage location for oils, paints and fuels, permissible locations for refuelling and procedure to follow in the event of a spill.	2	2	4	Council / Contractor	
2.11	Pollution due to dust and exhaust emission	9, 10	1	4	4	To be addressed in WEMP including fitting plant with appropriate exhaust control measures and dust control spray systems for stockpiles and exposed surfaces.	1	2	2	Council / Contractor	
2.12	Soil contamination (including ASS) causing harm to workers and the environment	10, 18	3	2	6	To be addressed in WEMP including contingency plan with measures to manage, treat and/or dispose of contaminated material if encountered onsite. Contaminated material is not to be used as fill on site, unless the material only contains ASS soils and the material can be placed permanently below the water table.	3	1	3	Council / Contractor	
2.13	Inadequate sediment & erosion control during construction	9, 10, 17, 21	2	4	8	To be addressed in WEMP including control of stormwater runoff, bunds around stockpiles or working areas, and filtering devices to treat water discharged from the site.	2	2	4	Council / Contractor	
2.14	Damage to existing services during construction	12, 13, 21	4	3	12	Service locator to be engaged by Contractor and services to be confirmed prior to construction.	4	2	8	Contractor	
2.15	Vehicle congestion during construction		2	4	8	To be addressed in WEMP including working hours and traffic control measures.	2	2	4	Council / Contractor	
2.16	Rubbish not disposed from site		2	4	8	To be addressed in WEMP including disposal locations of rubbish and measures to be implemented to keep the site tidy.	2	2	4	Contractor	
2.17	Site compound not restored or rehabilitated		2	4	8	To be addressed in WEMP including procedure to restore the site.	2	2	4	Council / Contractor	
2.18	Safety of pedestrians during construction	13	4	3	12	To be addressed in WEMP including traffic control measures and the location and requirements for man-proof temporary fencing around the works area.	4	2	8	Council / Contractor	

Risk ID	Hazard	Reference	Initial risk or risk at previous design stage			Strategy/Mitigation Measure	Residual Risk			Risk Manager	Additional Comments
			Consequence 1-5	Likelihood 1-5	Rating		Consequence 1-5	Likelihood 1-5	Rating		
2.19	Navigation risk on the water during construction		1	1	1	To be addressed in WEMP including navigational markers.	1	1	1	Council / Contractor	
2.20	Inadequate parking during construction		2	5	10	To be addressed in WEMP including provisions for contractors plant, equipment and vehicles.	2	3	6	Contractor	
2.21	Project runs over budget or over time		4	3	12	Council to open tender to numerous contractors to obtain competitive quote within allowable time frame. Council to specify Liquidated Damages.	4	2	8	Council / Contractor	
2.22	Contractor defaults on project		5	2	10	Council to select reputable Contractor with history of financial security.	5	1	5	Council	
2.23	Impact on existing businesses		1	1	1	No businesses in the vicinity of the works area.	1	1	1	Council / RHDHV	
3 RISK DURING OPERATION											
3.1	Safety to the public as a consequence of fall hazard	5, 19, 20	4	3	12	Guardrail/handrail to be installed next to stair accessways and around areas that present a fall hazard.	3	1	3	Council	
3.2	Works failing within design life due to inadequate materials or poor workmanship	2, 3, 6, 22, 23	4	2	8	Inspection and testing procedure for materials to be determined during design of the seawall and specified in the Technical Specification. Superintendent to confirm workmanship during construction.	4	1	4	Council / Contractor	
3.3	Inadequate drainage from behind the seawall	4, 5, 20, 22, 23	4	3	12	Drainage to be incorporated into the design of the seawall. It is assumed in-situ material behind the seawall is free draining (sand and minimal fines). If this assumption is found to be incorrect during construction, the design may need to be altered.	4	1	4	RHDHV / Contractor	
3.4	Excessive loading due to unexpected environmental factors such as higher than expected sea level rise	5, 7, 20, 22, 23	4	2	8	Wall to be designed to Australian Standards and allow for modification to raise the crest in the future.	4	1	4	Council	
3.5	Overtopping of the seawall.	5, 7, 20, 22, 23	3	4	12	The rate of overtopping was assessed to be acceptable. The crest of the structures could be raised in the future to address overtopping resulting from unexpected sea level rise.	4	1	4	Council	

References	
Reference Number	**Standard, Regulation, Code or Guideline**
1	Australians Standard AS 1726 (1993), *Geotechnical site investigations*
2	Australians Standard AS 2758.6 (2008), *Aggregate for rock engineering purposes Part 6: Guidelines for the specification of armourstone*
3	Australians Standard 3600 (2008), *Concrete Structures*
4	Australians Standard AS 4678 (2002), *Earth-retaining structures*
5	Australians Standard AS 4997 (2005), *Guidelines for the design of maritime structures*
6	British Standard BS 6349-1-4 (2013), *Maritime works - Part 1-4: General - Code of practice for materials*
7	Australian/New Zealand Standard AS/NZS 1170, *Structural design actions*
8	NSW WorkCover (2014), *Construction work code of practice*
9	NSW WorkCover (2014), *Demolition work code of practice*
10	NSW WorkCover (2014), *Excavation work code of practice*
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