REPORT

Port Macquarie Breakwater

Basis of Design (BoD)

Client: Maritime Infrastructure Delivery Office (MIDO)

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Appendices

Appendix A – Geotechnical Investigation

Appendix B – Site Survey

1 Introduction

This report documents the Basis of Design (BoD) for the southern training wall upgraded at Port Macquarie. The BoD provides an overview of the site and analysis of the site conditions. In addition, it provides a summary of the project objectives, design life and maintenance regime, and design parameters to be adopted for the design of the upgrade works and other components. Accordingly, it provides detail that defines the Purpose of the Services for the Professional Services Contract No. 21.0000138132.0257. The BoD is considered a live document, particularly during the Preliminary Design Phase with updates to be agreed between MIDO and RHDHV and the document revised, as more information comes to hand, or design risks are identified and mitigated.

1.1 Background

The Port Macquarie training wall is managed by Maritime Infrastructure Delivery Office (MIDO) within Transport for NSW (TfNSW). The training wall requires remediation and potential upgrading to address issues of toe scour, movement/displacement of rock armour and consideration of climate change impacts (including sea level rise).

The potential upgrade is also to consider the feasibility of; widening the footpath along the crest of the breakwater and training wall structures, providing access ways, and considering numerous ancillary structures such as fishing platforms, lighting and shelters as proposed in the Breakwater Master Plan prepared by Port Macquarie-Hastings City Council. Extracts from the Masterplan are provided in **Figure 1** to **Figure 3**.

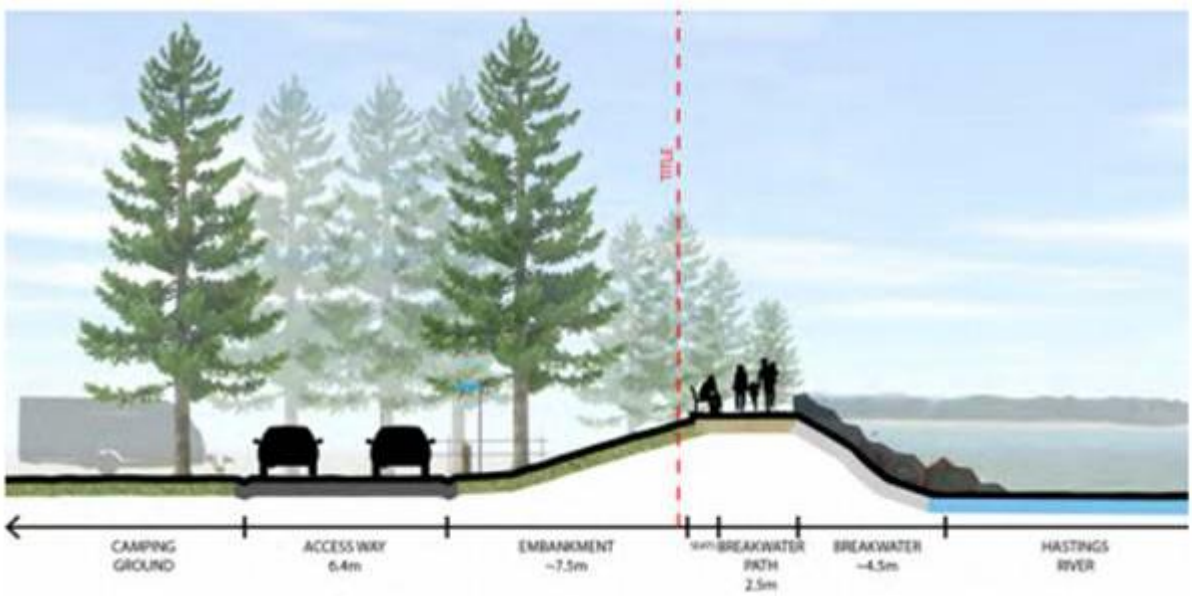

Figure 1: Cross Section of Existing Breakwater Path.

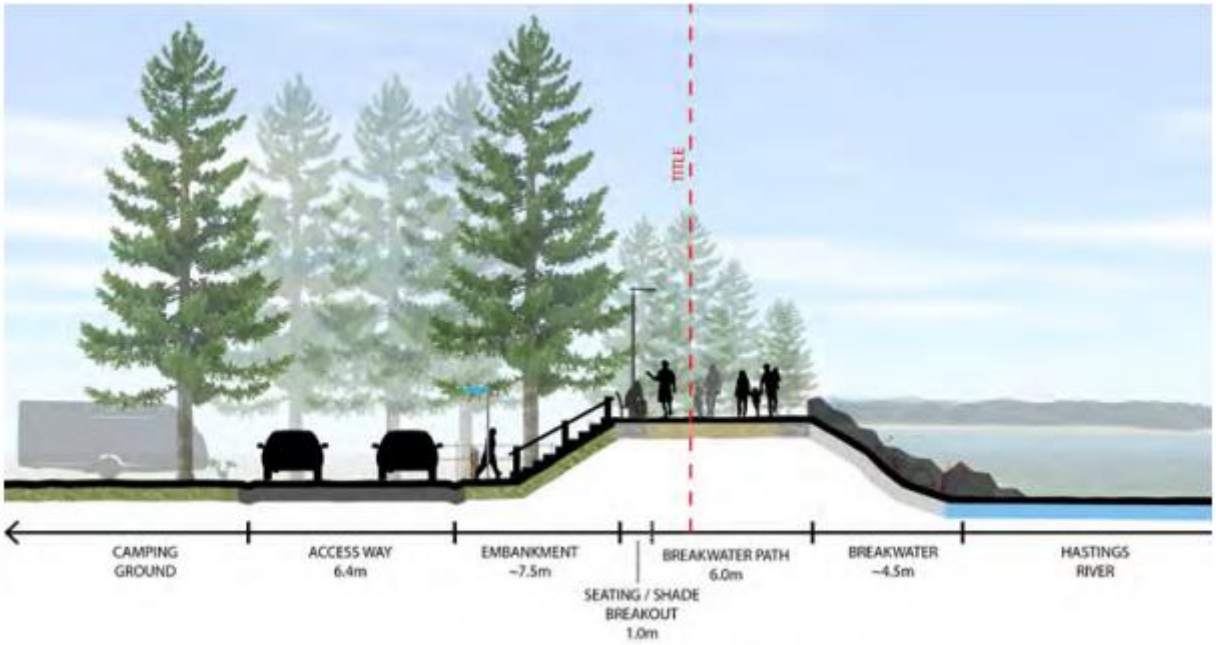

Figure 2: Cross section of proposed breakwater path.

Figure 3: Visualisation of breakwater upgrade.

1.2 Extent of Works

The proposed works include the breakwater head and the inner river revetment and crest path with a distance of approximately 700m in length as shown in Section 4 on **Figure 4**. It should be noted that the back beach revetment along Town Beach adjoining the breakwater head is not considered within the scope of the proposed works. Determining suitable access for construction will require consideration of adjoining parcels of land.

Figure 4: Extent of works (Section 4 only).

Lot boundaries are shown in **Figure 5**. The site includes:

- Lot 1 DP 1233503 (Sundowner Breakwall Tourist Park);
- Lot 2 DP 1233513 (Sundowner Breakwall Tourist Park);
- Lot 7324 DP 1165988 (adjacent skate park); and,
- Lot 7025 DP 1060950 (Town Beach Reserve).

Figure 5: Lot boundaries.

All four lots are listed as Crown Land in the Port Macquarie – Hastings Local Environment Plan (LEP), 2011. The breakwater and adjacent land are zoned Public Recreation (RE1) under the Port Macquarie –

Hastings Local Environment Plan (LEP), 2011. The Hastings River is zoned Recreation Waterways (W2).

1.3 Objectives and Functional Requirements

MIDO's objectives and functional requirements with consideration to budgetary constraints are:

- Remediate training wall to address:
  - toe scour; and,
  - movement/displacement of rock armour.

- Upgrade training wall with consideration of climate change impacts (including sea level rise).

- Improve recreational land use including:
  - widen the footpath along the crest of the breakwater and training wall structures;
  - provide access ways from the road to the footpath; and,
  - consider numerous ancillary structures such as fishing platforms, lighting and shelters as proposed in the Breakwater Master Plan prepared by Port Macquarie-Hastings City Council.

Additional objective of the works are:

- minimise impact on the Hastings River, including flooding;
- include capacity to upgrade or top up the revetment at the end of its life to further extend the structure's capacity and meet contemporary design standards;
- ensure design satisfies current design practices, including design considerations relating to appropriate toe protection level and crest height and/or ensure risk of overtopping can be managed through acceptable asset management; and,
- integrate with existing revetment/training wall (e.g. tie-ins, materials).

1.4 Chainage and Reduced Level

A chainage system is proposed running along the shoreline from the entrance extending upstream. All reference to chainage would be given in metres. Reduced Level (RL) would be provided in metres relative to Australian Height Datum (AHD). Horizontal Datum will be MGA2020 Zone 56. Any reference to chainage or RL herein, should be regarded as approximate pending completion of CAD drawings.

1.5 Inception Meeting

An inception meeting was conducted by MIDO for the project on 21 June 2021 at Port Macquarie Hastings Council (Council) Committee Room. The meeting was attended by the following:

- Andrew Mogg (TfNSW MIDO);
- Dan Champness (TfNSW);
- Dave Hopper (TfNSW MIDO);
- Sian Nivison (TfNSW);
- Rod McDonagh (TfNSW);
- Matt Dawson (TfNSW);
- Tiffany Jeffery (TfNSW);
- Dan Messiter (RHDHV);
- Craig Luff (Council);
- Melissa Watkins (Council); and,
- Jonathan Yantsch (NSW Fisheries).

Key notes from the inception meeting are provided below:

Topic	Notes
MIDO	$5M funding allocation for the Port Macquarie Southern Breakwater Upgrade Project under the Maritime Stimulus package.
Port Macquarie Town Master Plan and other Council matters	Identified issues that had been raised in relation to the southern breakwater including submissions regarding: • narrow crest walkway (conflicts between walkers/runners/strollers); • the desire for fishing platforms (including all abilities access) and a wide pathway to accommodate passive recreation (walking, photography, dolphin/whale watching); and, • the need for lighting for fishing and walking and conflicting concerns that the caravan park have around negative impacts of lighting. Register of seats and memorials located on the crest is available at Council. Council would likely be building an Amphitheatre adjacent to the seaward end of the Breakwater. The Project is expected to be completed December 2022 and there may be some overlap in forecast construction timelines.

Project Plan Overview	Overview of the project including objectives, what project success looks like, key project stages and project timeline. Key points noted were: • primary objective is to upgrade the breakwater structure to extend asset life, maintain safe boating entrance to Port Macquarie and improve pedestrian access; • secondary objective is to consider, and incorporate where possible, potential ancillary elements of the breakwater design as proposed in the Breakwater Master Plan prepared and exhibited by Council; • assessment of site conditions and extent of works will inform high level cost estimates which will result in a more clearly defined scope for construction (inclusions /exclusions within budget); and, • detailed planning and design that maps out future feasible ancillary structures that will be developed during the project to allow continued improvement in line with Council's concept plan.
Design Process	Discussed techniques and methodologies that would be used during the assessment and design process. Constraints/challenges to community expectation were tabled, particularly: • breakwater is a rubble mound structure, which is 'flexible' by design. this limits some community/masterplan expectations of extensive structures and particular structure types along the breakwater; and. • widening of the head of the breakwater is potentially very expensive and may have adverse environment/amenity impacts. Consideration of a viewing platform located further back along the structure may be a more feasible alternative.
Stakeholder/Community Engagement Discussion	The need for significant stakeholder and community engagement was acknowledged across the board given the time delay since adoption of the Council Concept Plan and the foreseen impacts to community and other stakeholders during the construction phase. Issues included painted rocks, memorial plaques, loss of access to site, disruption to businesses (particularly the Breakwater HQ café) and local events. Opportunities to improve stakeholder outcomes (e.g. addressing back flooding in the caravan park as part of works) to compensate for disruptions during construction where discussed. It was agreed the TfNSW and Council communication teams would work together on stakeholder and community engagement throughout the project.
Other	Potential need to move Aids to Navigation during project identified.

A site walkover with inception meeting attendees was undertaken to get an appreciation of the project and the key points noted above.

1.6 Site Inspection

A detailed site inspection was undertaken on August 26 and 27, 2021 to assess the condition of the structure and identify the extent of work required. A detailed walk-over visual inspection was undertaken by a coastal engineer to verify/consider the findings of the previous condition assessment/sand identify the type and extent of defects, observe possible failure mechanisms and assess the methodology and access for construction. Detailed notes and photographic record of the inspection are to be used in conjunction with detailed UAV and hydrographic survey methods and data analysis to assess the condition of the structure and to inform the development of a remediation concept design.

The data (including all site photographs) and findings of the site inspection are contained in the Condition Assessment GIS database developed for the Project as a digital tool to assist in the development of the remediation design.

Initial key general observations from the detailed site inspection are provided below:

- the southern side of the breakwater and transition to back beach revetment at Town Beach was in good condition with no significant issues observed or remediation measures required;
- recent repairs to the breakwater head appear to have been effective in the upper (above water line) profile either side of the breakwater and were in good condition. Note: Below waterline will be assessed using analysis of hydrosurvey data. However, the profile at the end (easterly facing) of the head was considered to be oversteepened and some damage at the water line was apparent. Accordingly, there appeared to be some flattening of the profile below the water line in response to the incident wave climate at the focus point at the end of the breakwater.
- Crest pavement damage at the head of the breakwater indicating ongoing movement of the head;
- Crest at the head of the breakwater is considered too narrow for lookout structure without significant widening of the breakwater crest. Ongoing movement of the head is not conducive to foundations for a rigid structure. An area was identified further west, adjacent to crest pathway where head widens significantly, which would be more suitable for a lookout (beachside and across the entrance) location;
- Trunk of breakwater west of the most recent repairs to the head of the breakwater is in generally fair condition. Localised areas of oversteepening were identified. Armour stone generally had a wide grading of sizes which creates voids and overhangs.
- Crest “containment” units along the trunk of the breakwater/training wall are generally large and embedded in the concrete pour along the crest pavement slab and form a rigid line. Many locations were observed where these units were overhanging due to the rest of the armour slope, which is a flexible and subsiding structure. Evidence of ongoing deep seated subsidence of the riverside armour slope. Further analysis of underwater hydrosurvey profile is required.
- Slope instability generally appeared to be due to insufficient filtration design and subsequent scour around the mean water level. The hydraulic stability of the armour was generally considered sufficient for the wave and current climate (estuarine dominated processes).
- Ongoing slope instability indicates that the foundation of rigid ancillary structures on the breakwater slope will not be feasible, without significant works
- Isolated areas at the upstream bend of the training wall trunk were observed, where catchment flooding flows appeared to have caused significant damage to the profile with loss of armour volume and oversteepening due to hydraulic instability. This is potentially due to focusing of the catchment flood flow at this location.

2 Management Plans, Guidelines, Standards and Reference Documents

The proposed guideline texts, standards and codes to be considered during the detailed design of the works are as follows:

- AS 4997 – Guidelines for design of maritime structures;
- AS 2758.6 – Aggregates and rock for engineering purposes, Part 6: Guidelines for the specification of armour stone;
- AS 4678 – Earth retaining structures;
- AS 1428.1 – Design for access and mobility Part 1: General requirements for access – New building work;
- AS 1657 – Fixed platforms, walkways, stairs and ladders – Design, construction and installation;
- RMS Engineering Standards and Guidelines for Maritime Structures;
- Coastline Management Manual (1990) prepared by NSW Governments (ISBN 0730575063);
- Coastal Engineering Manual prepared by the US Army Corp of Engineers (USACE);
- The Rock Manual – The use of rock in hydraulic engineering (2nd Edition) (2007) prepared by CIRIA;
- EurOtopII – Manual on wave overtopping of sea defence and related structures (2018); and,
- Environmentally Friendly Seawalls - A Guide to Improving the Environmental Value of Seawalls and Seawall-lined Foreshores in Estuaries* (NSW DECC, 2009).

Project Documents relevant for the investigation are:

- Hastings River Flood Study (Patterson Britton and Partners, 2006);
- Hastings River Flood Study Update (Advisian, 2018); and,
- Manly Hydraulics Laboratory (MHL, 1999), Hastings River Estuary Tidal Data Collection October – November 1999, Report No. MHL1030.
- Manly Hydraulics Laboratory (MHL, 2021), Macquarie River Training Walls Post March Flood Assessment – draft report dated 20 April 2021, Report no. MHL2804-D;
- Hastings River Hydrographic Survey (Astute Surveying, 2021), Date of Survey: 15-17/06/2021.

Documents to be referenced are likely to include:

- Couriel, E., Alley, K., Modra, B., (2012), OEH NSW Tidal Planes Analysis 1990-2010 Harmonic Analysis, Manly Hydraulics Laboratory Report Number MHL2053;
- Department of Environment and Climate Change and Water (2010), Coastal Risk Management Guide: Incorporating sea level rise benchmarks in coastal risk assessments, NSW Government;
- Goda (2000), Random Seas and Design of Maritime Structures – Advanced Series on Ocean Engineering – Volume 15;
- Goda (2010), Reanalysis of Regular and Random Breaking Wave Statistics, Coastal Engineering Journal, Vol. 52, No. 1, pg 71-106;
- Jacobs, R., (2016), NSW Ocean and River Entrance Tidal Levels Annual Summary 2015-2016, Manly Hydraulics Laboratory Report Number MHL2475;

- NSW and Office of Environment and Heritage (OEH, 2015), Floodplain Risk Management Guide - Modelling the Interaction of Catchment Flooding and Oceanic Inundation in Coastal Waterways;
- Shand, T.D., Wasko, C.D., Goodwin I.D., Carley J.T., You Z.J., Kulmar M., Cox R.J., (2012), Long Term Trends in NSW Coastal Wave Climate and Derivation of Extreme Design Storms;
- Floyd, C.D., and Druery, B.M. (1975) Results of *River Mouth Training on the Clarence Bar, New South Wales, Australia, International Conference on Coastal Engineering, Hawaii, 1975;*
- Vos K., Splinter K.D.,Harley M.D., Simmons J.A., Turner I.L. (2019). CoastSat: a Google Earth Engine-enabled Python toolkit to extract shorelines from publicly available satellite imagery. Environmental Modelling and Software. 122, 104528. https://doi.org/10.1016/j.envsoft.2019.104528;
- Bishop-Taylor. R., Sagar. S., Lymburner. L., Alam. I., & Sixsmith. J. (2019) Sub-Pixel Waterline Extraction: Characterising Accuracy and Sensitivity to Indices and Spectra. Remote Sensing. 11, 2984; doi:10.3390/rs11242984;
- Druery. B.M., & Nielsen. A.F. (1980) Mechanisms Operating at a Jettied River Entrance, Coastal Engineering 1980, 2907-2626;
- Glatz . M., Fithenry . M., & Kulmar. M. (2017), It's Time for an Update – Extreme Waves and Directional Distributions along the New South Wales Coastline, NSW Coastal Conference; and,
- Glamore W.C, Rahman P.F, Cox R.J, Church J.A and Monselesan D.P (June 2016), Sea Level Rise Science and Synthesis for NSW, prepared by NSW Office of Environment and Heritage's Coastal Processes and Responses Node - Technical Report.

3 Design Life and Maintenance Regime

The proposed nominal planning period for the rock works is 40 years, which is the design life of rock armour specified in AS2758.6. If required, the planning period could be extended by specifying more stringent rock (or concrete) properties for the armour units, increasing the size of the rock (or concrete) armour to allow for degradation or including additional maintenance requirements.

The training wall would be designed to be hydraulically stable.

The Rock Manual, The Use of Rock in Hydraulic Engineering (CIRIA, 2007) recommends balancing economics and operational safety relating to performance and level of risk over time to determine the loading event return period (or Average Recurrence Interval [ARI]) for a specific design life. As a starting point for the remediation design, two representative design storm events with varying acceptable damage levels, would be considered as follows:

- 50 year ARI storm event: - overtopping suitable for 'aware pedestrians;
- 50 year ARI storm event: **Serviceable** with 0-5% damage; and,
- 100 year ARI storm event: **Repairable** with 10% damage.

The probability of a design storm event occurring at least once within the 40 year design life is provided in **Table 1**. It should be noted that the joint probability of 50 or 100 year ARI wave height and water occurring simultaneously is not well understood for the study area. However, it would be less than the probabilities shown in **Table 1**. Notwithstanding, the analysis herein will consider storm wave heights and water levels occurring simultaneously.

This approach is considered to provide a cost-effective balance between construction cost (rock size) and structure maintenance. Minor work would be required to reshape the structure following a significant storm event. It is recommended that TfNSW allows for a typical maintenance regime costed at 1% per annum of the capital cost of the works.

Table 1: Probability of ARI event occurring at least one during the design life.

	ARI Storm Event						
Design Life	1	5	10	20	50	100	200
40	100.0%	100.0%	98.5%	87.1%	55.4%	33.1%	18.2%

4 Site History

The first attempts to stabilise the entrance were made in 1897 when construction started on the southern training wall and jetty. These works proved largely ineffective with the entrance varying in position over a distance of 1km along the northern beach. The pattern was for floods to cut through the northern sandspit and the new entrance to migrate slowly southwards.

A northern training wall and jetty were completed in 1940. However, the works were gradually out flanked by the prograding profiles of the adjacent beaches and expansive shoals and swash bars formed at the entrance. The entrance channel shifted seasonally in response to the prevailing wave pattern (refer **Figure 6**).

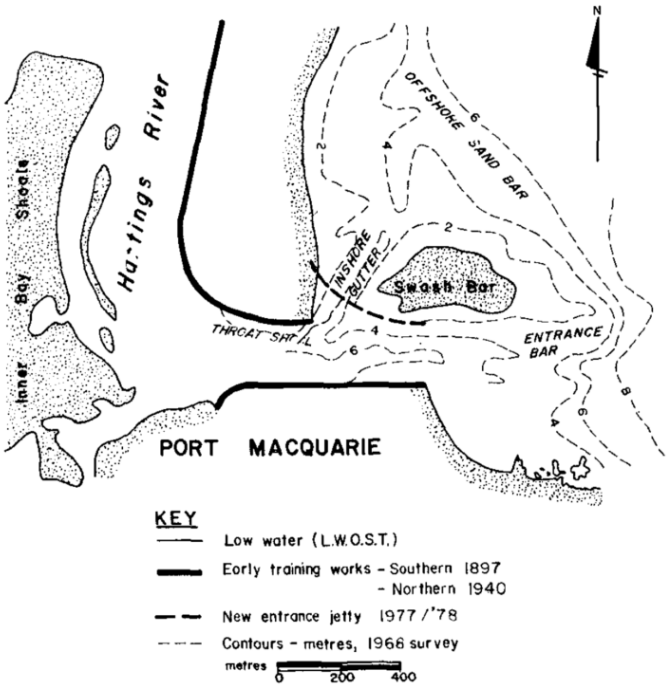

Figure 6: Sketch of Port Macquarie entrance prior to construction of northern jetty (Druery et al, 1980).

In October 1976, construction of a new northern jetty commenced with the objective of creating a more stable, safer bar for navigation. Following construction of the northern jetty, the swash bar progressively moved landward and attached to the beach (refer **Figure 7**), which caused infilling of the inshore gutter. The northern jetty has substantially reduced formation of a swash bar.

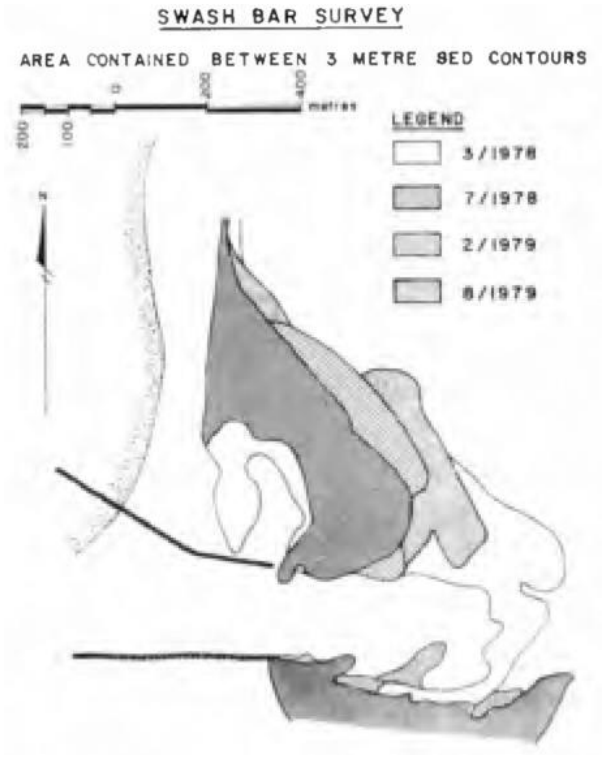

Figure 7: Swash bar surveys between 1978 and 1979 showing onshore movement of the swash bar (Druery et al, 1980).

5 Review of Project Information

5.1 Geotechnical

A geotechnical investigation was undertaken by Douglas Partners on August 26, 2021. The investigation included:

1. Geotechnical Ground Investigation

An on-site physical ground investigation to describe the substrate and representative parameters for design in the context of:

- the proposed widening of the footpath along the crest of the breakwater as per the Breakwater Master Plan prepared by Port Macquarie-Hastings City Council;
- the potential for a retaining structure to accommodate the widening;
- the potential for an engineered slope (reinforced or otherwise) to accommodate the widening; and,
- the potential for ancillary structures on the crest of the breakwater.

2. Geotechnical Risk Assessment

In recognition of the difficulty in intrusive physical testing techniques of the rubble mound that makes up the existing breakwater structure, a risk assessment has been undertaken to assess the existing breakwater condition in relation to the geotechnical stability. This risk assessment consists of the following:

- review of regional geological context and known geotechnical conditions in the immediate vicinity of the breakwater;
- site walkover (with coastal engineer from RHDHV) for visual inspection and identification of implied signs of subsurface damage or failure. e.g. including but not limited to; significant voids in the visible structure, differential settlement of the crest (cracking/rotation of crest pavement or crest top structures) or adjacent land areas, visible unmanaged surface water flow pathways through the structure;
- consideration of information obtained from the geotechnical ground investigation at the interface with the adjacent land area; and,
- reporting on the geotechnical risk in relation to the upgrade design and anticipated extended design life in the context of the Australia Geomechanics Guidelines for Landslide Risk Management.

A memo documenting the geotechnical investigation and risk assessment input to the remediation design is provided in **Appendix A**.

5.2 Survey

The following survey has been undertaken to inform the remediation design:

- Contour and detail plan by North Coast Surveys dated 29/07/2021 (horizontal datum: MGA56 [1994] and vertical datum: AHD71); and,

The survey is provided in **Appendix B**.

The elevation of the current footpath is approximately 2.4 to 4 m AHD, increasing in height towards the seaward end of the breakwater.

Furthermore, hydrographic survey of the Hastings River Channel has recently been undertaken for navigational purposes which will also be used to provide general information to be considered in the remediation design, as follows:

- Hydrographic Survey by Astute Surveying dated 15-17 June 2021 (horizontal datum: MGA56 [2020] and vertical datum: Indian Spring Low Water [ISLW], -0.74m below AHD).

5.3 Dredging Information

Information on dredging, that is relevant to the training wall design, is to be collated and reviewed.

5.4 Satellite Imagery

Satellite imagery can be used to determine shoreline position and behaviour from 30+ years of satellite imagery. Satellite platforms include Landsat 5 (1984 - 2013), 7 (1999 - present), 8 (2013 - present) and Sentinel-2 (2015 - present), with pixel resolution ranging between 10m and 15m and image capture varying from 5 to 16 days.

Two independent tools have been developed to analyse the satellite imagery, which are:

1. CoastSat a Python tool that enables extraction of shoreline position from individual satellite images at any coastline worldwide using sub-pixel extraction methods (Vos et al., 2019a, 2019b). While the horizontal errors for detected shoreline range have been reported to be between 7.3m and 12.7m Root Mean Square Error (RMSE) considering individual images, the bias of the data is much lower (Vos et al., 2019a). As such, the temporal resolution of the data means that signals can be discerned at seasonal and greater timescales.

2. Digital Earth Australia (DEA) is a tool to accurately map the boundary between land and water (the 'waterline') (Bishop-Taylor et al, 2019). The tool uses the same sub-pixel extraction method as CoastSat. However, DEA combines aerials to provide a single, annualised shoreline position and is therefore suited to determining long-term trends.

Outputs from both CoastSat and DEA are available online, with key outputs for the coastline adjacent to the Hastings River Entrance provided in **Figure 8** to **Figure 12.**

Figure 11 from DEA indicates clockwise rotation of North Shore Beach between 1988 and 2020 with an annual rate of recession of approximately 1m/yr at the southern end of North Shore Beach and annual accretion of around 0.8m/yr at the northern end of the beach. However, the CoastSat timeseries in **Figure 8** and the DEA shoreline position in **Figure 12** both indicate recession at the southern end of North Shore Beach between 1988 and say 1999 with a relatively stable beach since 1999. **Figure 8** indicates the shoreline fluctuated by approximately ±15m from the mean shoreline position annually, some of which may be due to the RSME of the dataset. However, it is likely that episodic recession and progradation would occur due to annual beach rotation and storm events.

Figure 11 from DEA indicates minimal change in Town Beach (less than ±0.1m/yr). The CoastSat timeseries in **Figure 9** and **Figure 10** and the DEA shoreline position in **Figure 13** both indicate progradation of the southern end of Town Beach between approximately 1991 and 2000 with a relatively

stable beach since 2000. There has been very little change in the shoreline position at the northern end of Town Beach. **Figure 9** and **Figure 10** indicates the shoreline fluctuates by approximately ±15m from the mean shoreline position annually. As per North Shore Beach, the variability in shoreline position may be partially due to the RSME of the dataset. However, it is likely that episodic recession and progradation would occur due to annual beach rotation and storm events.

An in-depth analysis of the raw data could be undertaken to determine inter-annual each rotation and correlation of timeseries to climate indices (such as Southern Oscillation Index [SOI], Southern Annular Modulus [SAM] and Indian Ocean Dipole [IOD]). In our experience, weak correlations are observed. However, any further analysis would not provide benefit to the project herein.

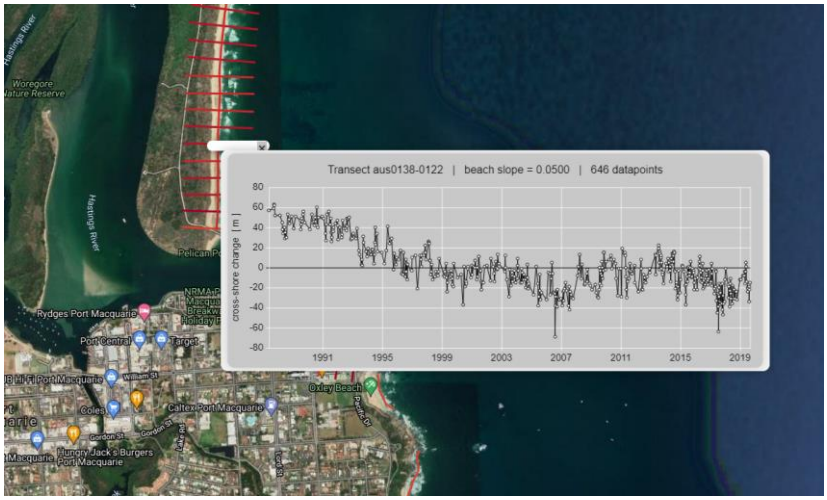

Figure 8: CoastSat shoreline position timeseries approximately 600m north of the northern training wall.

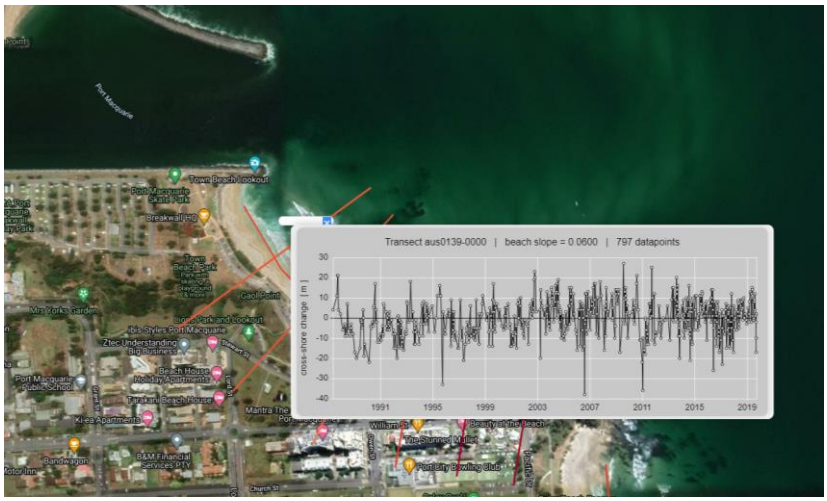

Figure 9: CoastSat shoreline position timeseries approximately 100 south of the southern training wall.

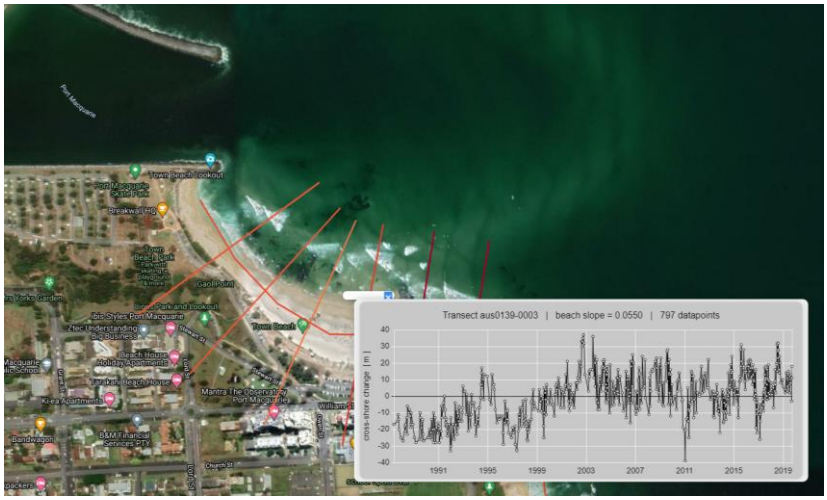

Figure 10: CoastSat shoreline position timeseries approximately 400 south of the southern training wall.

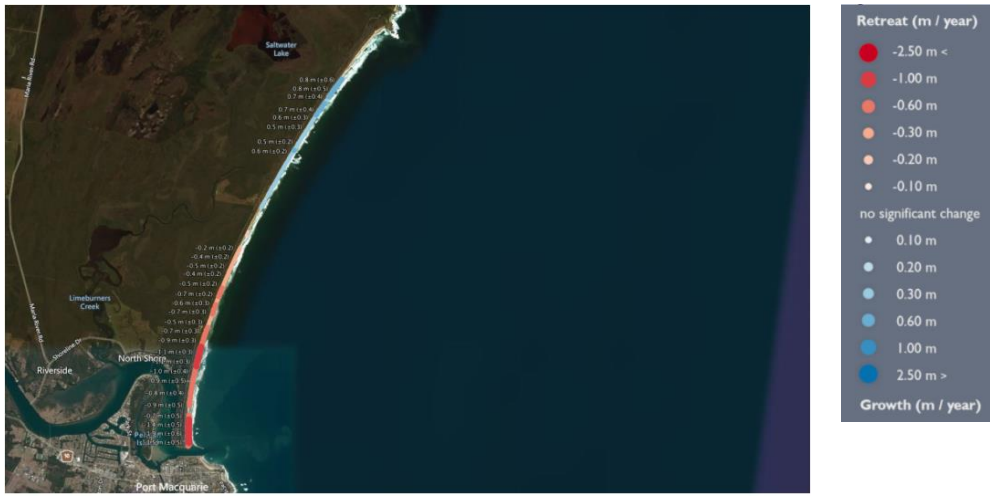

Figure 11: DEA recession analysis showing clockwise rotation of North Shore Beach.

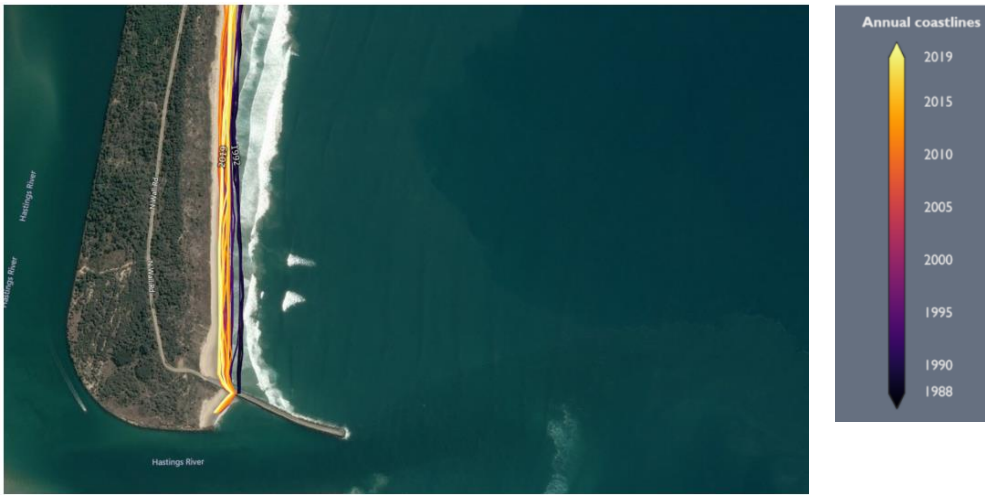

Figure 12: DEA Annual Coastlines for North Shore Beach.

Figure 13: DEA Annual Coastlines for Town Beach.

5.5 Channel Equilibrium

The relationship between bar depth, channel width and ebb flow for ideal entrance channel conditions is given by **Equation 1**.

$$D_B = \frac{Q_m}{1.5 * W}$$

Equation 1: Floyd and Druery (1975).

Where:

D_B = bar depth in metres below the peak ebb flow surface (typically 0.3m above ISLW on a spring tide).
Q_m = peak spring tide ebb flow in m^3/s.
W = channel width in m.

Peak spring tide ebb flow is approximately 1,290m^3/s (MHL, 1999) and the channel width is approximately 175m (refer **Figure 14**). The predicted bar depth is therefore 4.6m below ISLW (-5.34m AHD).

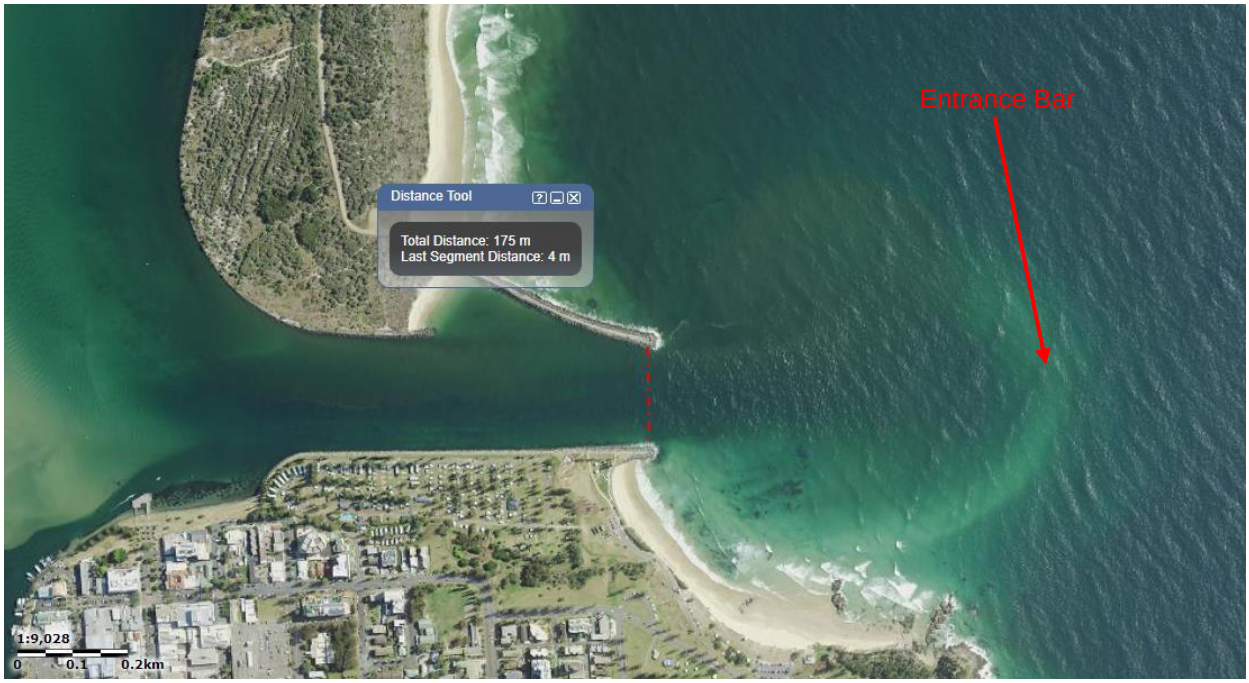

Figure 14: Channel width and entrance bar.

The present day entrance bar is 4.75-5.75m below AHD based on the hydrographic survey. The entrance bar and entrance channel are approximately in equilibrium.

5.6 Historical Aerial Photography

Historical aerial photographs may be used to determine coastal processes at the entrance, which may impact on the structure e.g. scour potential and sediment transport patterns.

5.7 Summary

The Hasting River entrance bar and the beaches to the north and south of Hastings River are approximately in an equilibrium regime meaning that they are dynamically stable. Episodic erosion and recession of the beaches and scouring of the entrance bar may occur due to storm events. However, the system would recover and revert to an equilibrium regime.

In response to sea level rise (SLR), it is assumed that the water depth over the entrance bar (relative to mean sea level over time) would remain roughly the same as present. It is expected that the relative depth to the seabed profiles at the entrance bar and adjacent foreshores would remain constant due to progradation, as a result of the sediment supply from the Hastings River and longshore drift processes, keeping pace with SLR.

6 Heritage Considerations

A search for heritage items was conducted through:

- Port Macquarie – Hastings Council Local Environmental Plan Heritage Map (2011) (refer **Figure 15**);
- Heritage NSW Heritage - State Heritage Inventory (refer **Figure 16**);
- Heritage NSW Heritage - Marine Heritage (refer **Figure 17**); and,
- Heritage NSW - Aboriginal Heritage Information Management System (AHIMS) (refer **Figure 18**).

The Clarence Valley Council Local Environmental Plan (LEP) 2011 heritage schedule and Heritage NSW registers identified the following two items of local heritage significance near the site:

- North Shore and Port Macquarie training walls and breakwalls;
- “Ballina”—paddle steamer wreck; and,
- Port Macquarie archaeology of early European settlement - parts of Town Centre and Town Beach precincts.

The Aboriginal Heritage Information Management System (AHIMS) identified a number of sites near the proposed works. However, they would not be impacted by the proposed works.

It is recommended that a Heritage Consultant is engaged to advise which parts of the training wall could be modified. The above information will be confirmed and further informed by the Review of Environmental Factors (REF) being undertaken by others.

Figure 15: Port Macquarie - Hastings LEP.

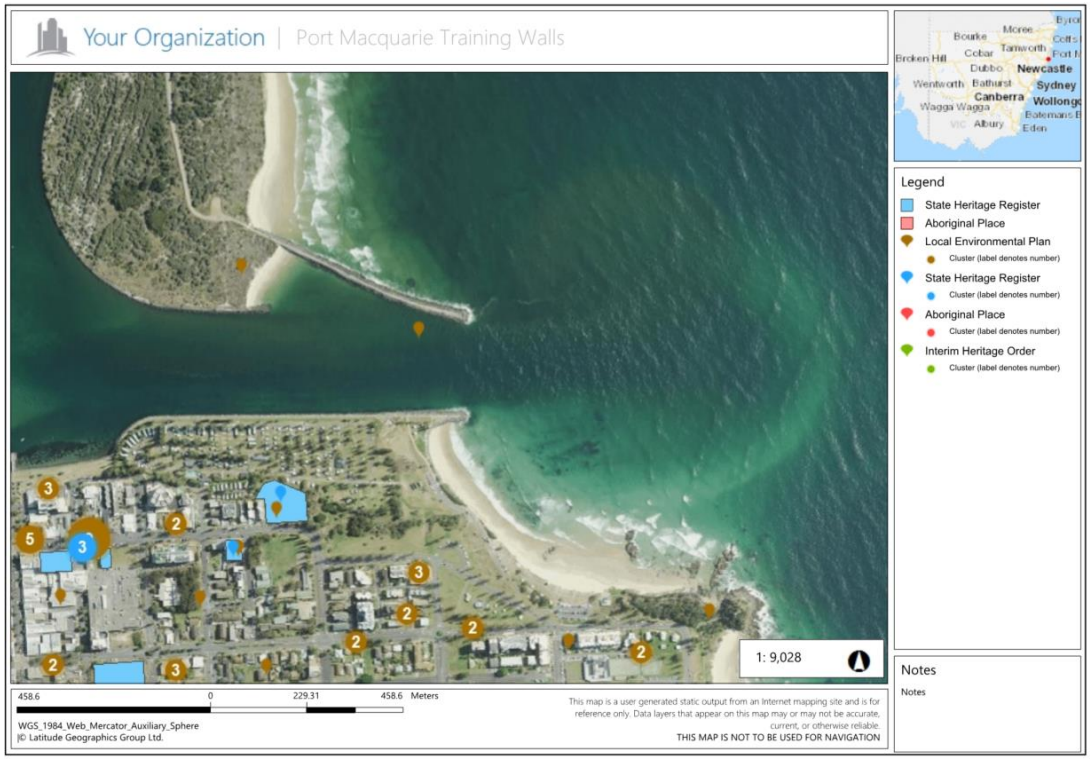

Figure 16: NSW State Heritage Inventory (Heritage NSW).

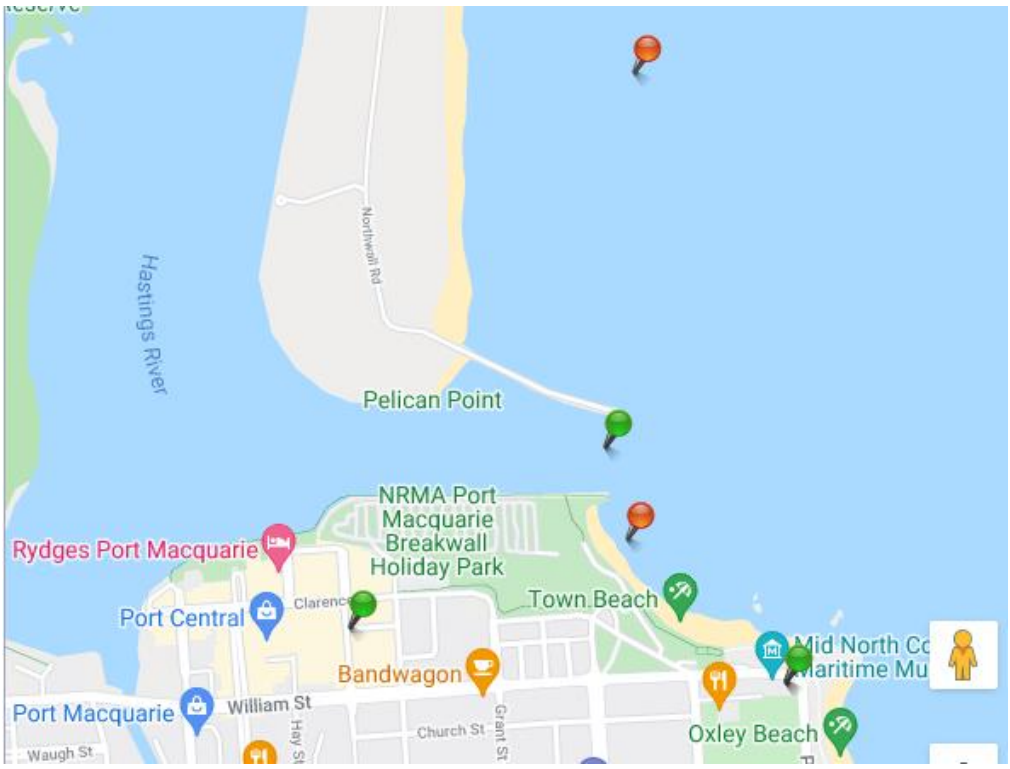

Figure 17: NSW Maritime Heritage (Heritage NSW).

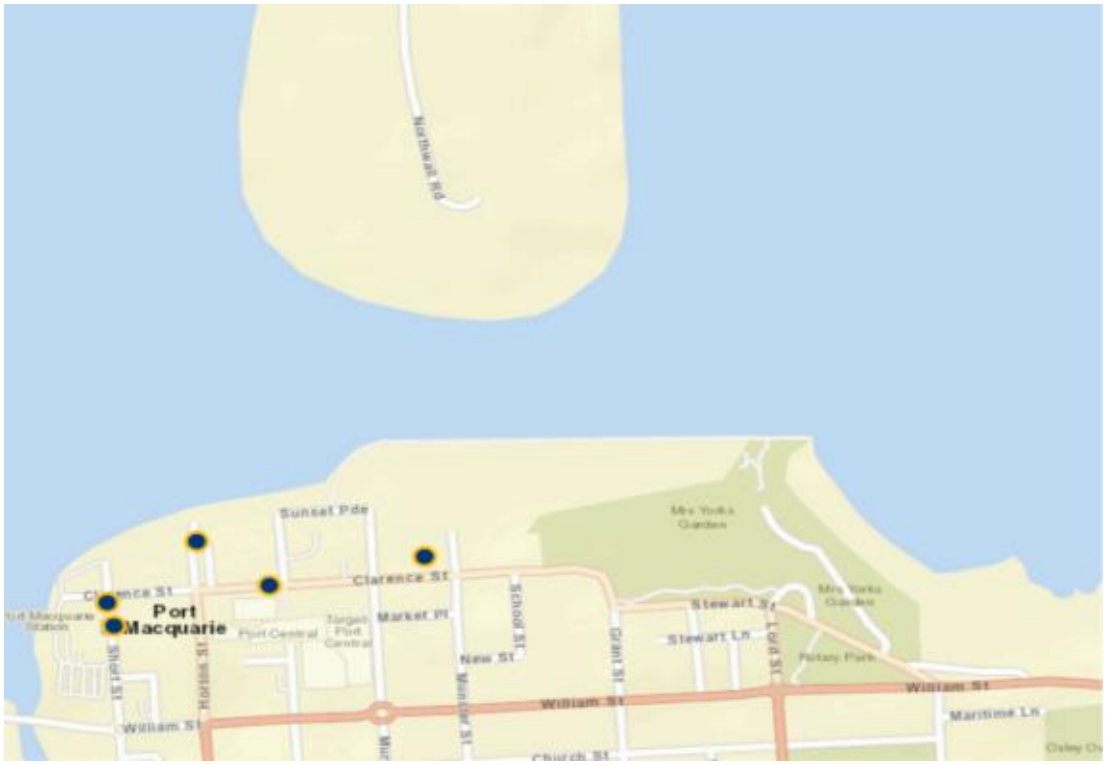

Figure 18: Aboriginal Heritage Information Management System search.

7 Design Actions

Design actions shall consider permanent actions (dead loads) and imposed actions (live loads). The dead loads include the self-weight of all structures. The minimum live loads to be considered in the structural analysis shall include:

- pedestrian and vehicle loads; and,
- hydraulic loading including waves and flood events.

Any retaining structure would be designed to be free draining. The water level differential between the rear and front of the retaining structure, due to tidal lag or ground water pressure, would therefore be zero.

7.1 Pedestrian and Vehicle Loads

Footpath and accessways would be designed for a uniformly distributed live load of 5kPa or concentrated loads of 31kN (applied over an area of 0.025m^2) in accordance with AS1170.1, which corresponds to:

1. areas without obstacles for moving people, footpaths, terraces and plazas at ground level subject to wheeled vehicles; and,
2. medium vehicle traffic areas.

7.2 Hydraulic Conditions and Loading

7.2.1 Design Scour Level

It is assumed that the entrance channel and beach near the southern training wall would have scoured to the design scour depth at least once since construction of the training wall. This would have resulted in reshaping of the toe of the training wall. Placing rock armour on top of the existing rock would ensure that the structure is founded at the design scour depth. Should this not be practical, then rock armour should be placed to form a 'falling/Dutch' toe or founded at the design scour level.

Significant flooding of the Hastings River occurred between the 19th and 24th March 2020, which corresponded to a 10-30 year ARI event. This event would have led to scouring of the entrance channel and lowering of the entrance bar.

Water depth near the toe of the southern training wall is typically -4.75 to -6.25m AHD. However, there is an area where the water depth is around -5.75 to -7.55 AHD and the outline of individual rock armour units is evident in the survey (refer **Figure 20**). Accordingly, the design scour level has been adopted as approximately -7.75m AHD.

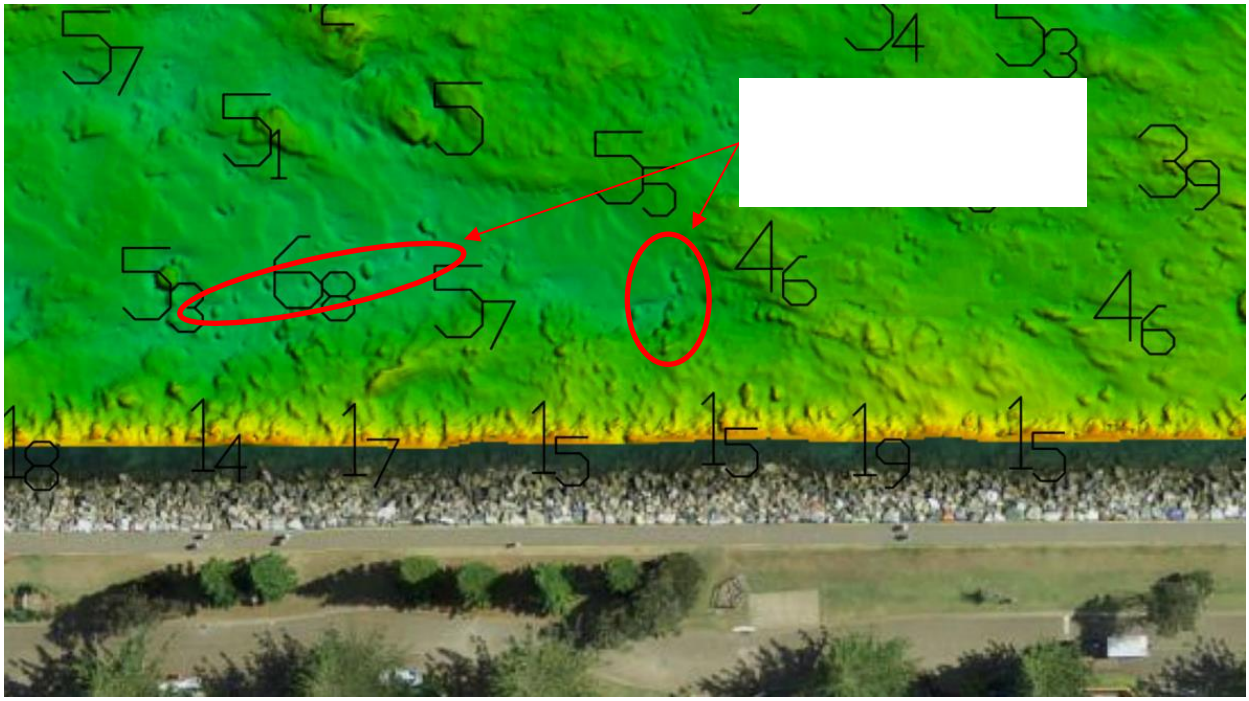

Figure 19: Extract from hydrographic survey near the training wall (annotations on plan to ISLW datum).

The design scour depth near the head and on the southern side of the training wall would be somewhat shallower due to the progradation of longshore sediment transport and lower currents. The existing bed is at approximately -3.75m AHD. The estimated design scour depth has been adopted as approximately -5.75m AHD.

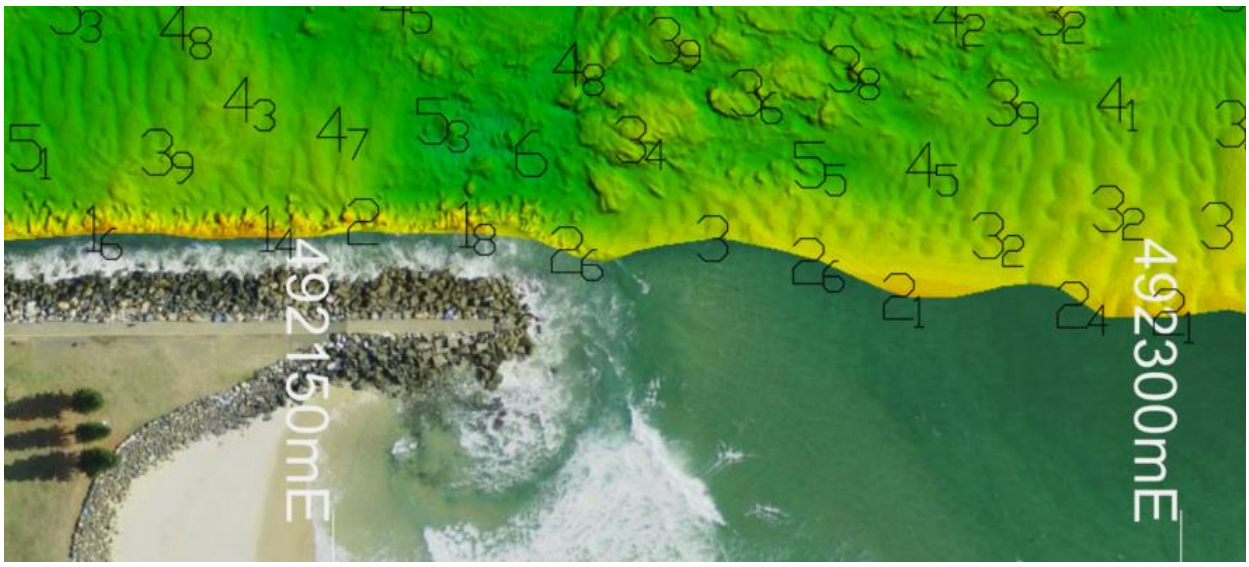

Figure 20: Extract from hydrographic survey near the head of the training wall (annotations on plan to ISLW datum).

7.2.2 Nearshore Water Depth

Nearshore bathymetry, obtained from Navionics SonarCharts™, is presented in **Figure 21**. The 'base' for the SonarChart™ are nautical charts and other regional or local sources offering additional data, as well as Navionics surveys. Sonar logs recorded by end users are then validated, adjusted for tide correction and filtering to remove "bad" data. The sonar logs are then integrated with the 'base' to produce a new SonarChart™ on a daily basis. The data has not been collected by a registered hydrographic surveyor. However, in the absence of additional information, the SonarChart™ provides a useful reference.

Due to headland, islands, reefs and sand shoals, ocean swell from the south and southeast would have minimal impact on the southern training wall without significant diffraction and refraction of the wave energy. Ocean waves from the east and north east are more likely to have a significant impact on the training wall due to the direct exposure to these directions. Refraction and diffraction would not be considered for waves approaching from the east and north east in analytical methods of design wave height determination described in sections below.

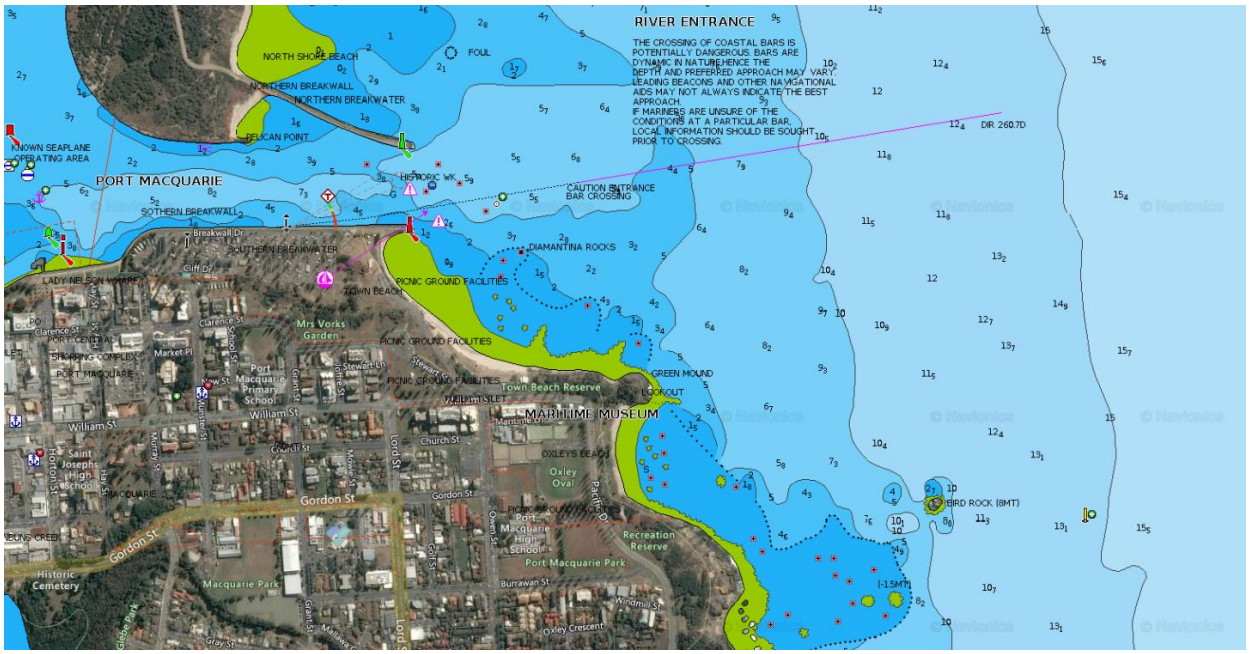

Figure 21: Bathymetry obtained from Navionics SonarChartTM (Chart Datum).

7.2.3 Wave Climate

The wave climate at the site is primarily contributed to by a combination of ocean sea and swell. Ocean sea and swell would shoal and break offshore of the site. Local wind waves (on their own) and boat-generated waves would have a negligible impact on the wave climate at the site. However, it is noted that local wind effects during significant ocean storm events can influence water levels in the transition to the nearshore zone and therefore influence the shoaling of deepwater waves as they propagate shoreward.

Directional wave roses from the Crowdy Head waverider buoy are provided in **Figure 22** and an extreme wave analysis undertaken by Glatz et al (2017) is provided in **Figure 23**.

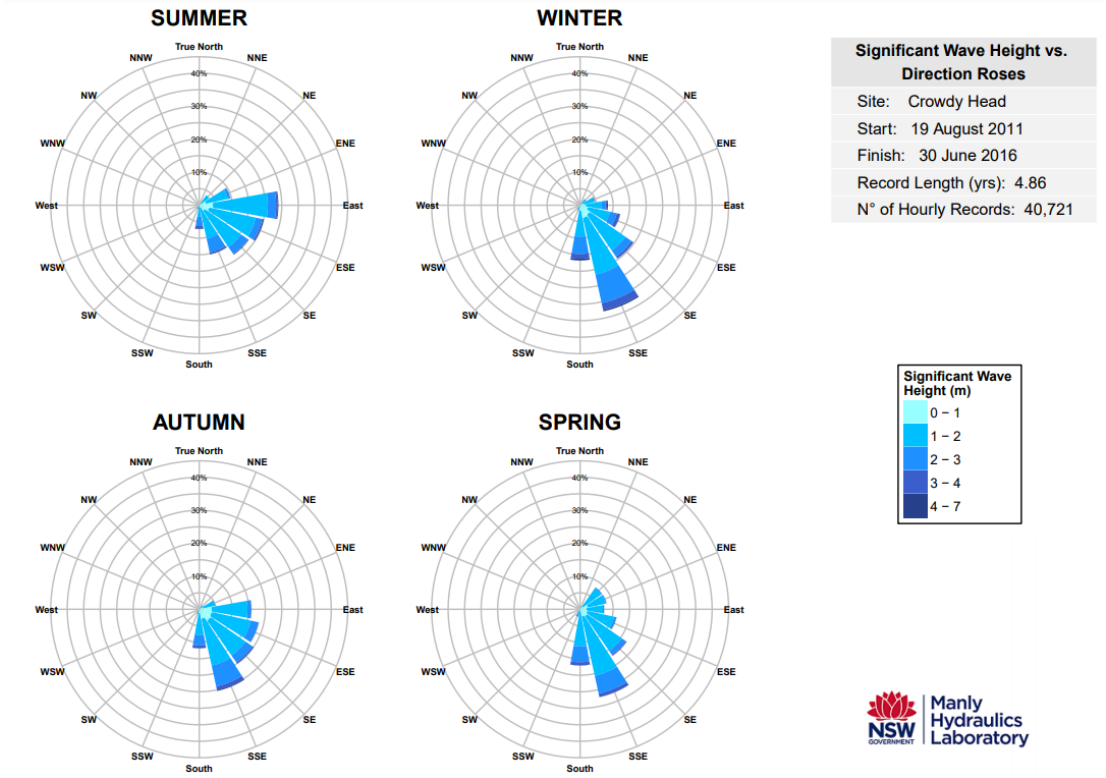

Figure 22: Seasonal directional wave rose (MHL, 2021).

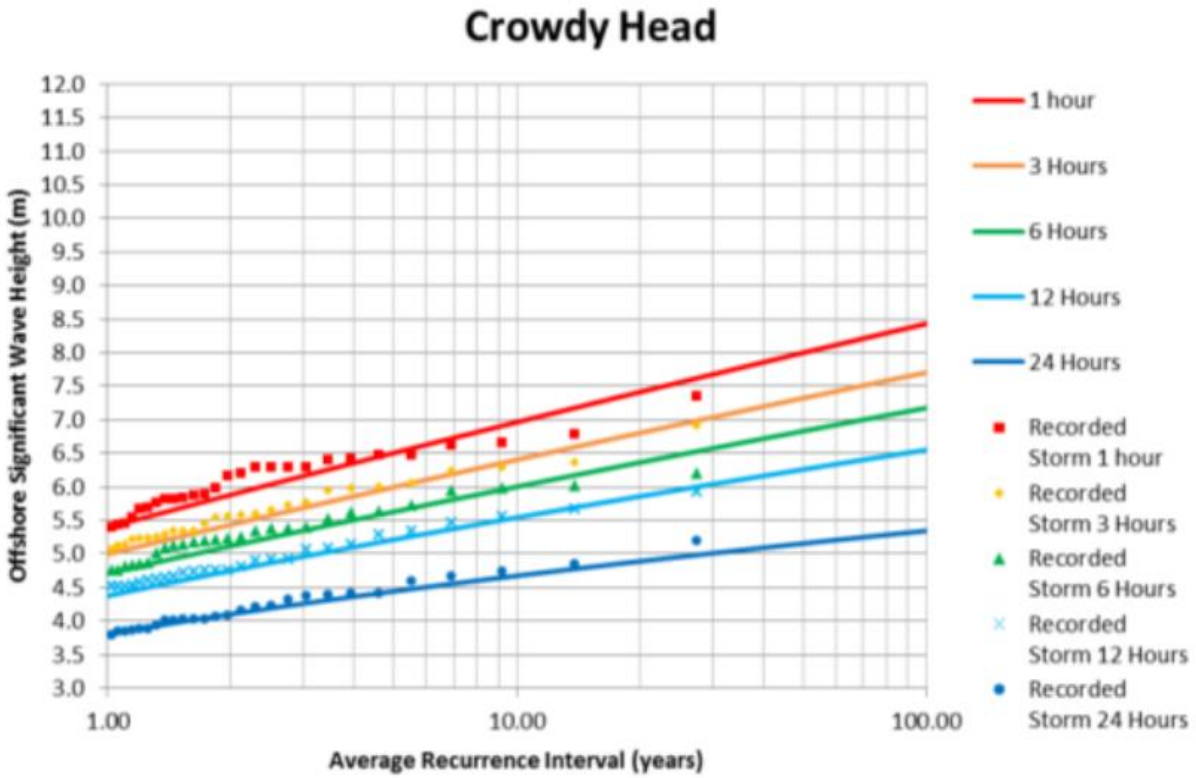

Figure 23: Extreme wave analysis for Crowdy Head (Glatz et al, 2017).

Glatz et al (2017) also prepared directional 1 hour wave height exceedances for Sydney and Byron Bay (refer **Figure 24**). However, the period of observation is insufficient for the corresponding extreme value Analysis (EVA) to be considered suitable for use in design.

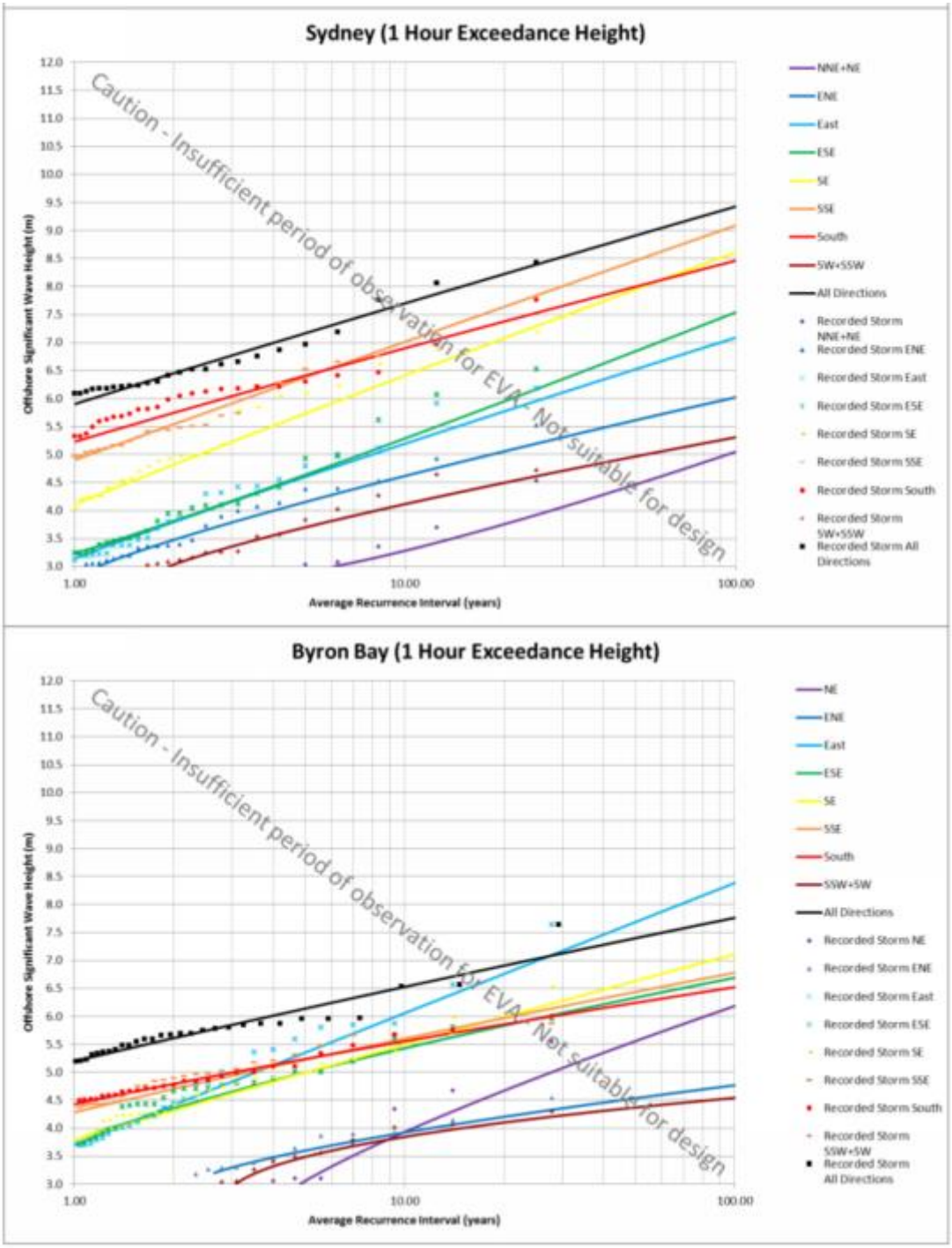

Figure 24: Directional wave analysis for Byron Bay and Sydney (Glatz et al, 2017).

Shand et al (2012) provides a directional significant wave height analysis for Sydney and Byron Bay for a 10 year ARI event. Baird (2017) applied the relationship developed by Shand et al (2012) for the 10 year ARI event to 5, 20 and 100 year ARI events. The one-hour significant wave height from Baird is provided in **Table 2** and **Table 3**.

Table 2: One Hour Exceedance Hs for Byron Bay (Baird, 2017).

Average Recurrence Interval	Parameter	Direction		
		NE	ESE	SSE
5 year	H_s (m)	4.10	6.1	5.81
	T_p (s)	11.2	12.4	12.2
20 year	H_s (m)	4.57	6.8	6.48
	T_p (s)	11.5	12.8	12.6
100 year	H_s (m)	5.11	7.6	7.24
	T_p (s)	11.8	13.3	13.0

Table 3: One Hour Exceedance Hs for Sydney (Baird, 2017).

Average Recurrence Interval	Parameter	Direction		
		NE	ESE	SSE
5 year	H_s (m)	4.20	5.79	7.00
	T_p (s)	11.0	12.0	12.8
20 year	H_s (m)	4.80	6.61	8.00
	T_p (s)	11.4	12.6	13.5
100 year	H_s (m)	5.40	7.44	9.00
	T_p (s)	11.8	13.1	14.1

Wave period is not strictly correlated to wave height although loose empirical associations have been described. Shand et al (2012) reports that the mean peak wave period (T_p) statistics for Sydney, Crowdy Head and Byron Bay are near identical, as outlined in **Table 4**. Accordingly, the directional wave period in **Table 2** or **Table 3** is to be adopted for application offshore of the Port Macquarie site.

Table 4: Descriptive wave period statistics (Shand et al, 2012).

Descriptive Statistic	Sydney	Crowdy Head	Byron Bay
Mean T_p (sec)	9.72	9.71	9.59
Median T_p (sec)	9.77	9.50	9.50
10% Exceed (sec)	12.50	12.20	12.20
1% Exceed (sec)	15.10	15.10	15.10
Maximum (sec)	20.00	19.79	19.70

Due to the uncertainty in **directional** wave heights along the NSW Coast, and also considering the likelihood of the coincidence of design wave heights with design water levels (which is tidal dependant) and the nature of rubble mound structures which suffer damage related to number of waves of a period of time, the 6 hour exceedance significant offshore wave height for all directions at Crowdy Head (**Figure**

23) would be adopted. The period would be approximated from **Table 2** and **Table 3**. The 1 year ARI, 50 year ARI and 100 year ARI design wave height and period would be:

- 1 year ARI event – 4.7m (period 12 seconds);
- 50 year ARI event – 6.8m (period 13 seconds); and,
- 100 year ARI event – 7.2m (period 13.5 seconds).

Sensitivity of the breakwater design would be undertaken to the 1 hour exceedance significant offshore wave height to assess failure (>25% damage) potential.

It should be noted that the offshore wave height will be attenuated in the nearshore area and will likely be depth limited at the structure (refer **Section 7.2.6**). Accordingly, consideration of water level at the time of the design wave event is of particular importance in defining the design wave height at the structure.

7.2.4 Design Water Level

The water level at the site would be determined based on a combination of:

- tidal fluctuation;
- storm surge; and,
- sea level rise.

Tidal planes for Port Macquarie are provided in **Table 5**.

Table 5: Tidal Planes for Hastings River, Port Macquarie (MHL, 2016, MHL, 2012).

Tidal Plane	2010 Water Level (m AHD)	2061 Water Level (m AHD)*
Highest Astronomical Tide, HAT*	1.04	1.34
Mean High Water Springs, MHWS	0.54	0.84
Mean High Water, MHW	0.43	0.73
Mean Sea Level, MSL	0.00	0.30
Mean Low Water, MLW	-0.43	-0.13
Mean Low Water Springs, MLWS	-0.54	-0.24
Lowest Astronomical Tide, LAT*	-0.74	-0.44

*Note: * includes 0.3m SLR.*

Design ocean still water levels incorporating storm surge are reported in the Floodplain Risk Management Guide (OEH, 2015). Design ocean still water levels are defined for waterways north and south of Crowdy Head. Five groups of waterway entrances are defined. Port Macquarie is best defined as Group 1, estuary with trained entrance. Design ocean still water levels are outlined in **Table 6.**

Table 6: Elevated still water levels for coastal locations North of Crowdy Head (Floodplain Risk Management Guide, 2015).

Average Recurrence Interval (ARI) ARI	Peak Design Ocean Water Level Excluding Wave Setup and Run up (m AHD) [1]	2061 Peak Design Still Water Level Excluding Wave Setup and Run up (m AHD) [2]
20	1.50	1.8
100	1.55	1.85

Note:

1. *Assume peak ocean design water level is reported for 2010.*
2. *Based on sea level rise of 0.3m relative to 2010.*

The NSW Office of Environment and Heritage's Coastal Processes and Responses Node - Technical Report (2016) discusses various scenarios for sea level rise. Sea level rise predictions for Port Macquarie would be based on the scenario described as, '*a stabilisation scenario in which total radiative forcing is stabilised shortly after 2100, without any overshoot, by the application of a range of technologies and strategies for reducing greenhouse gas emissions (RCP2.6).*' See level rise predictions for this scenario are outlined in **Table 7**.

Sea level rise by 2061 (40 year design life) would be adopted as 0.3m, which can be added to the tidal planes and elevated still water levels as shown in **Table 5** and **Table 6**. It should be noted that the projected rise in sea level in the NSW Office of Environment and Heritage's Coastal Processes and Responses Node - Technical Report (2016) exceeds the allowances in AS4997, which specifies 0.2 m for a 50 year design life and 0.4 for a 100 year design life.

Table 7: Projection of Sea Level Rise averaged along the New South Wales coast relative to 1986-2005 mean.

Year	RCP2.6
2050	0.22 [0.15-0.30]
2070	0.34 [0.23-0.46]

Note: The central values are given for each greenhouse gas scenario with the likely range (66% confidence limits) in brackets.

The design still water level, including SLR would be adopted as:

- 1 year ARI event – ~1.35, AHD;
- 50 year ARI event – 1.83m AHD; and
- 100 year ARI event – 1.85m AHD.

7.2.5 Wave Setup

Wave setup (η_{max}) is the variation in water level as a result of breaking waves. Goda (2000) provides a method for estimating wave setup at the shoreline of a uniformly sloped beach as outlined in **Figure 25**. For a beach slope of 1V:100H and adopting the wave height in **Section 7.2.3**, wave setup is provided in **Table 8**.

Table 8: Wave setup (m) for various directional extreme wave heights.

Average Recurrence Interval	Return Period		
	1 year	50 year	100 year
Wave Setup (m)	0.16	0.12	0.12
Water Level in 2061 including wave setup (m AHD)	1.51	1.95	1.97

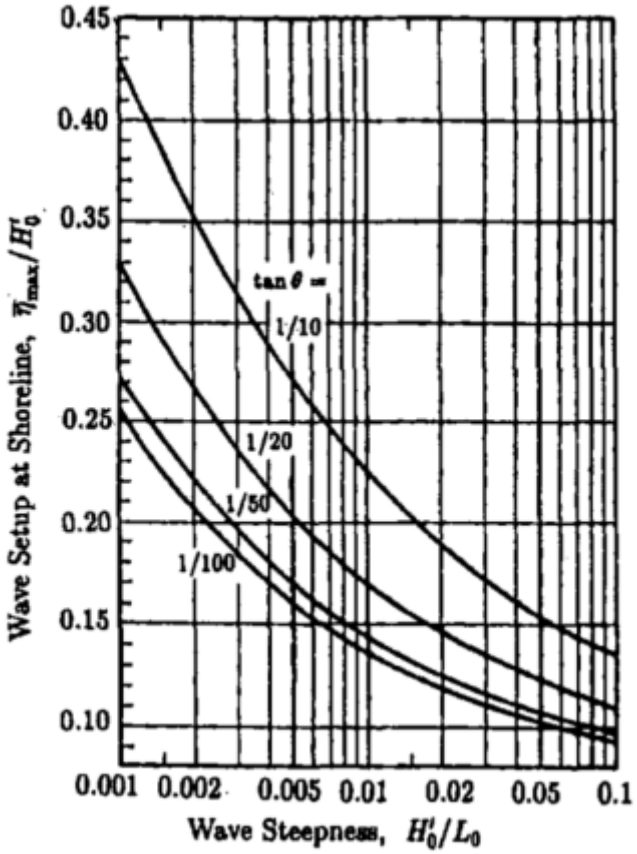

Figure 25: Wave setup at the shoreline of a uniformly sloped beach (Figure 3.35, Goda 2000).

7.2.6 Nearshore Wave Height

The nearshore wave height near the training walls and breakwater would be altered due to main processes:

1. wave diffraction – spreading of the wave crest and decrease in wave height in the lee of headland, reefs and breakwaters;
2. wave refraction – bending of the wave crests as a wave moves from deepwater to shallow water, which results in the focussing of wave energy on shoals; and,
3. friction losses and turbulence – wave heights could be expected to decrease as waves loose energy due to friction losses and turbulence, particularly during the process of wave breaking.

A SWAN model has been completed for the study site to determine wave heights near the toe of the training wall and breakwaters. The SWAN model considered waves from three directions as follows:

- East northeast (ENE);
- East (E); and,
- East southeast (ESE).

The water level included in the analysis is 1.9m AHD (1:100 year ARI water level in 2060). Sea level rise is not included in the water depth herein as it is assumed that the entrance bar will prograde at the same

rate as sea level rise to remain in equilibrium. The analysis herein does not include any assessment of freshwater flows from the river, or ebb tidal flows, which would effectively attenuate waves.

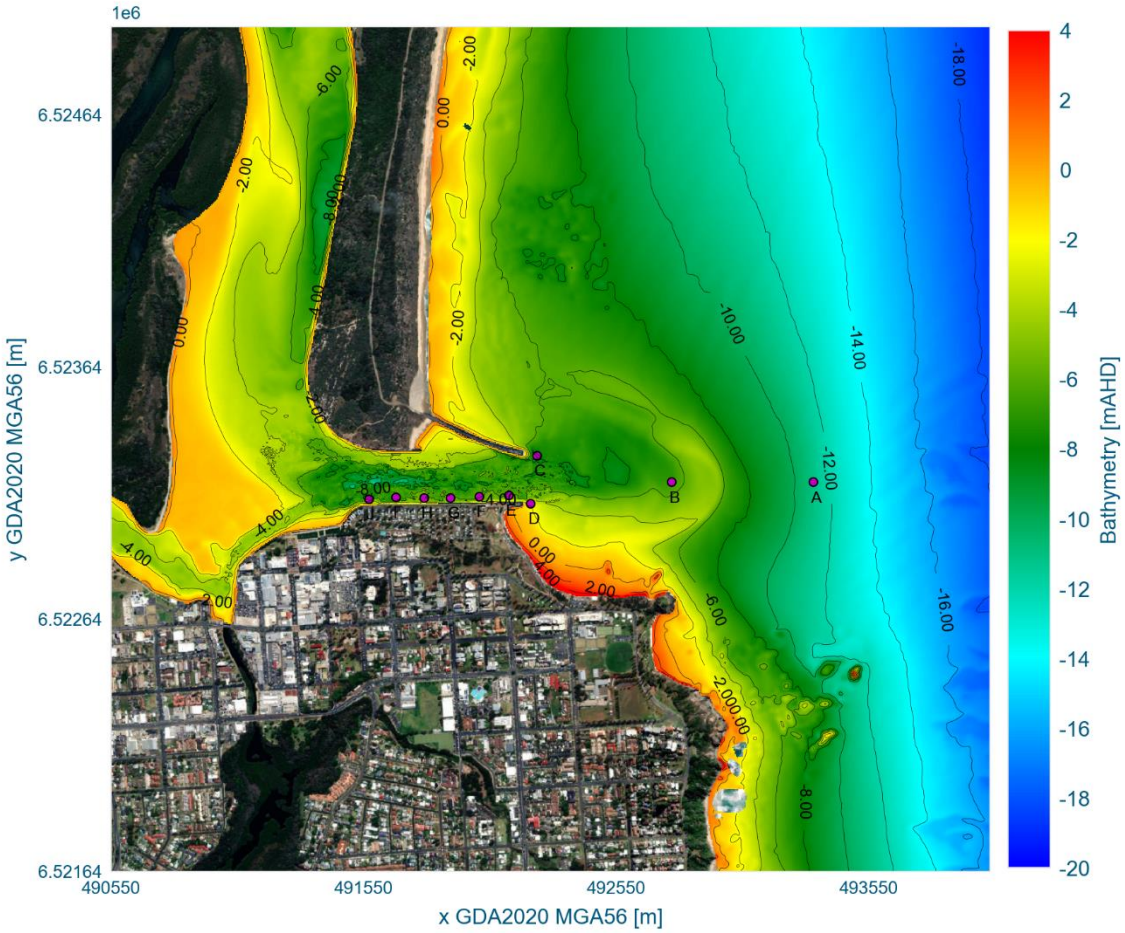

Figure 26: Reporting locations from SWAN Model.

Table 9: SWAN model results.

ARI (year)	Wave parameters	Location									
		A	B	C	D	E	F	G	H	I	J
1	Hm0	4.5	3.2	3.1	2.0	2.5	2.0	1.2	1.0	0.8	0.7
	Tp	11.8	11.9	11.9	11.9	11.9	11.9	11.9	11.9	11.9	11.8
	Tm-1,0	10.6	9.5	9.2	9.1	9.2	10.6	8.7	8.5	7.8	7.6
50	Hm0	6.1	3.5	3.4	2.1	2.8	2.2	1.3	1.1	0.9	0.7
	Tp	12.9	12.9	12.9	12.9	12.9	12.9	12.9	12.9	12.9	12.9
	Tm-1,0	11.0	10.3	10.1	9.9	10.0	9.8	9.4	9.3	8.6	8.3
100	Hm0	6.3	3.5	3.4	2.1	2.8	2.2	1.3	1.2	0.9	0.7
	Tp	13.6	13.7	13.7	13.7	13.7	13.7	13.7	13.7	13.6	13.6
	Tm-1,0	11.3	10.7	10.4	10.2	10.3	10.1	9.8	9.6	9.0	8.7

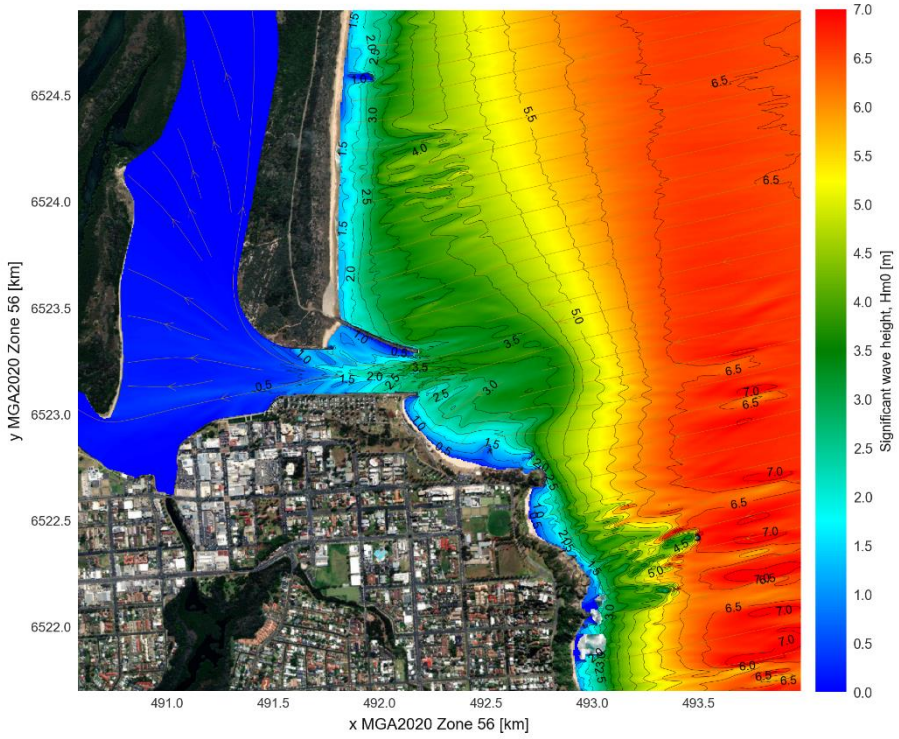

Figure 27: 100 year ARI SWAN model for ENE swell.

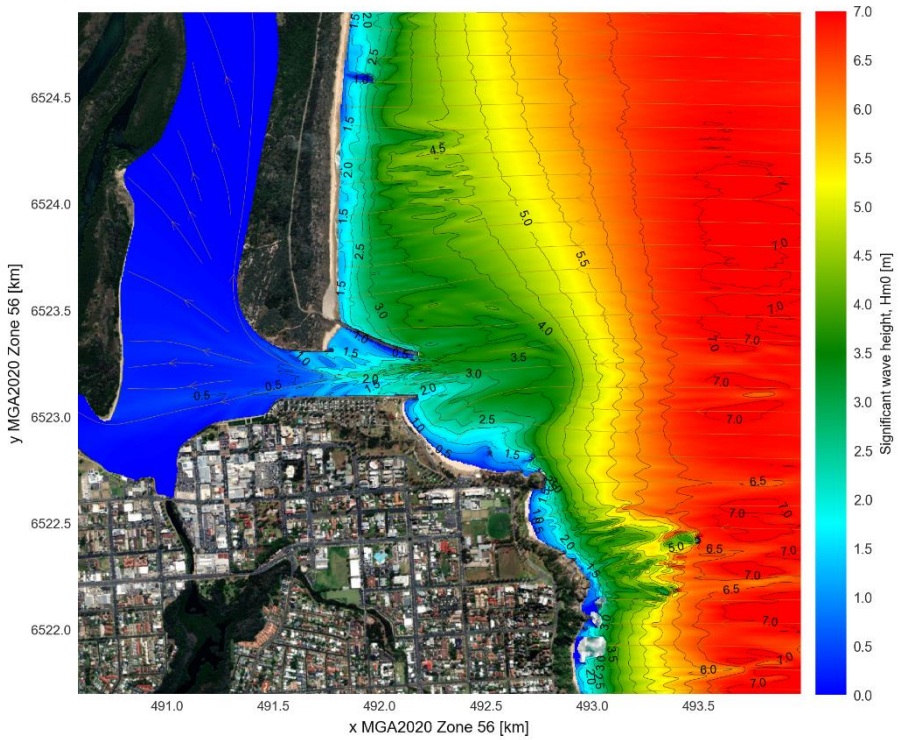

Figure 28: 100 year ARI SWAN model for E swell.

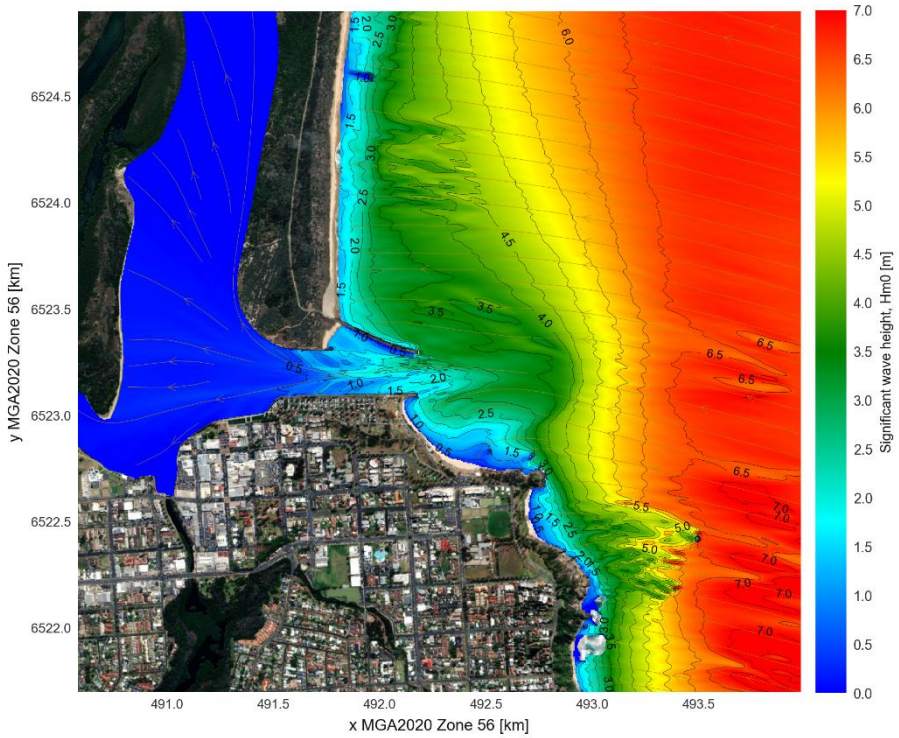

Figure 29: 100 year ARI SWAN model for ESE swell.

7.2.7 Flood Velocity and Flood Extent

Currents at the site would be primarily due to freshwater flooding and to a lesser extent tidal currents.

7.2.7.1 Tidal Currents

Spring ebb tide and flood tide currents are reported to be 1.65m/s approximately 0.55km upstream from the ocean end of the southern breakwater (MHL, 1999).

7.2.7.2 Flood Currents

A flood study was completed by Patterson Britton and Partners (PBP, 2006), which was updated by Advisian (2018).

PBP (2006) provided flood mapping for a 20 year and 100 year ARI event. The reported flood velocities are approximately 2.8m/s and 4.3m/s, respectively (refer **Figure 30** and **Figure 31**).

The update by Advisian included five scenarios as follows:

- Scenario 1 - 100 year ARI catchment event with 900 mm Sea Level Rise + 10% increase in rainfall intensity and volume (refer **Figure 32**, reported flood velocity approximately 4m/s).
- Scenario 2 - 100 year ARI catchment event with 900 mm SLR
- Scenario 3 - 100 year ARI catchment event with 400mm SLR + 10% increase in rainfall intensity and volume
- Scenario 4 - 100 year ARI catchment event with 400 mm SLR
- Scenario 5 - PMF event with 900 mm SLR (900mm SLR applied to the adopted 100yr Tide_2.2 m AHD)

The resolution of the flood velocities is relatively course and accurate velocities near the southern training wall are difficult to ascertain from the flood studies.

The studies by PBP and Advisian were based on an ocean water level of 2.2m AHD for a 100 year ARI event, which is higher than the design ocean still water level of 1.5m AHD recommended in the Floodplain Risk Management Guide (OEH, 2015). The design water level within 500m of the entrance (i.e. in the vicinity of the training walls) is approximately equal to the ocean water level. Due to the variation in the ocean water level, the flood level and extent in both flood studies are excluded from further consideration. A lower ocean water level may result in a marginally higher flood velocity due to the increase in hydraulic gradient. However, any variation is expected to be minimal.

The flood velocity at the site would be adopted as:

- 50 year ARI event – 3.5m/s; and
- 100 year ARI event – 4.3m/s.

Flood levels would be as per the design water level in **Section 7.2.4**.

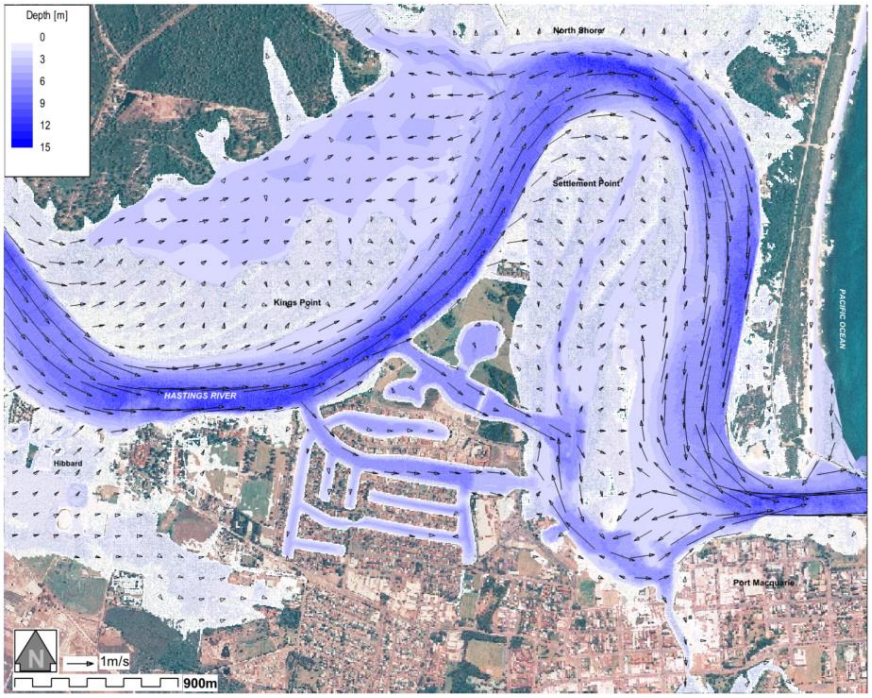

Figure 30: Peak floodwater depths and velocity vectors in the vicinity of Port Macquarie for the 20 year recurrence flood (PBP, 2006). Peak velocity near southern training wall ~2.8m/s.

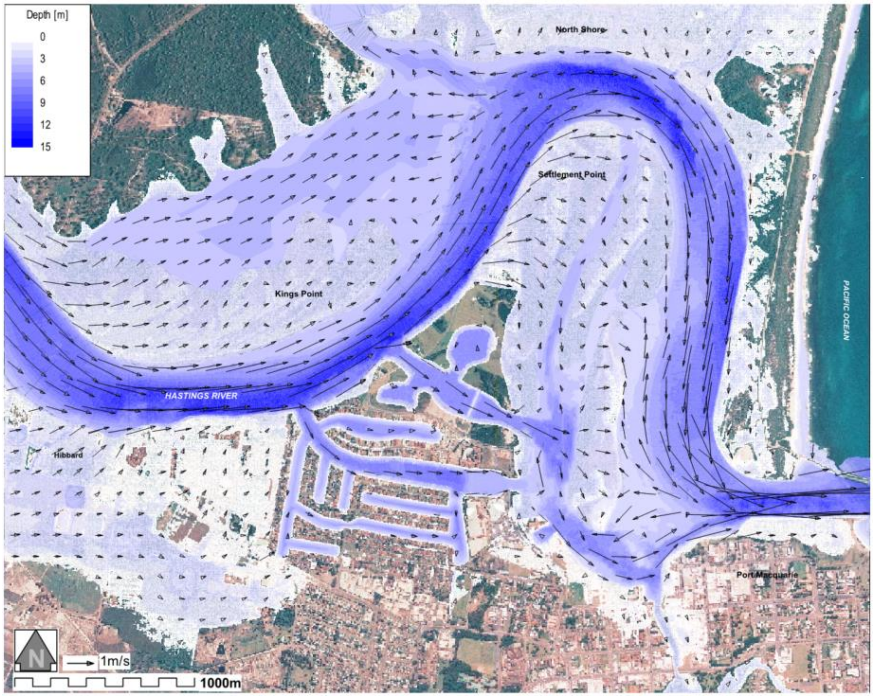

Figure 31: Peak floodwater depths and velocity vectors in the vicinity of Port Macquarie for the 100 year recurrence flood (PBP, 2006). Peak velocity near southern training wall ~4.3m/s.

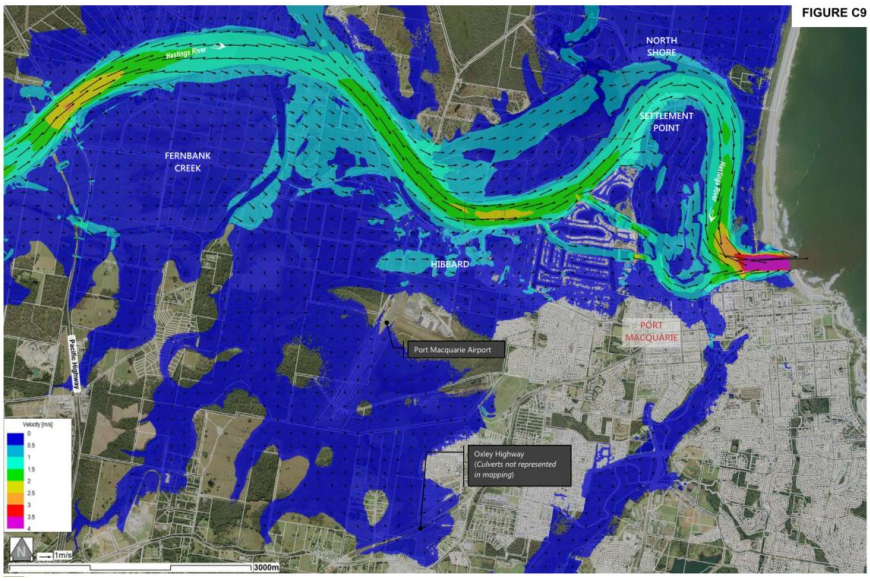

Figure 32: Peak floodwater velocity vectors in the vicinity of Port Macquarie for the 100 year recurrence flood for Scenario 1 (Advisian, 2018). Peak velocity near southern training wall ~4m/s.

7.2.8 Spectral Wave Parameters

Spectral shallow water wave parameters, required for overtopping and design, are calculated herein. The methodology is outlined in CIRIA (2008) and EuroTop (2018) based on work by Battjes and Groenendijk (2000).

Wave parameters for $H_{\frac{1}{3}}$, $H_{\frac{1}{10}}$ and $H_{2\%}$ corresponding to 33%, 10% and 2% exceedance of the wave height are determined in accordance with **Table 10** where:

$H_{tr} = (0.35 + 5.8\, tan\alpha)h$

$H_{rms} = \left(0.6725 + 0.2025(\frac{H_{m0}}{h})\right) H_{m0}$

$tan\alpha$ is the bed slope.

Table 10: Spectral shallow water wave height.

		Characteristic height		
		$H_{1/3}/H_{rms}$	$H_{1/10}/H_{rms}$	$H_{2\%}/H_{rms}$
Non dimensional transitional wave H_{tr}/H_{rms}	0.05	1.279	1.466	
	0.5	1.28	1.467	
	1	1.324	1.518	
	1.2	1.371	1.573	
	1.35	1.395	1.626	
	1.5	1.406	1.683	
	1.75	1.413	1.759	
	2	1.415	1.786	
	2.5	1.416	1.799	
	3	1.416	1.8	1.978

Spectral shallow water wave heights and periods are provided in **Table 11**.

Table 11: Spectral wave parameters.

ARI (year)	Water Depth (m)	Period (seconds)	Wave parameters	Location		
				D	E	F
1	5.8	12.0	Hm0	2.0	2.5	
			$H_{1/10}$	2.5	3.0	
			$H_{2\%}$	2.7	3.2	
			Tp	11.9	11.9	
			Tm-1,0	9.1	9.2	
50	6.3	13.0	Hm0	2.1	2.8	
			$H_{1/10}$	2.7	3.4	
			$H_{2\%}$	2.9	3.5	

			Tp	12.9	12.9	12.9
			Tm-1,0	9.9	10.0	9.8
100	6.3	13.5	Hm0	2.1	2.8	2.2
			$H_{1/10}$	2.7	3.4	2.8
			$H_{2\%}$	2.9	3.6	3.0
			Tp	13.7	13.7	13.7
			Tm-1,0	10.2	10.3	10.1

7.2.9 Wave Overtopping

The wave run-up height is defined as the vertical difference between the highest point of wave run-up and the still water level (SWL) as shown in **Figure 33**. Wave setup is not included in the still water level calculations when determining overtopping and wave runup, as it is implicit in the empirical equations calculating wave runup level.

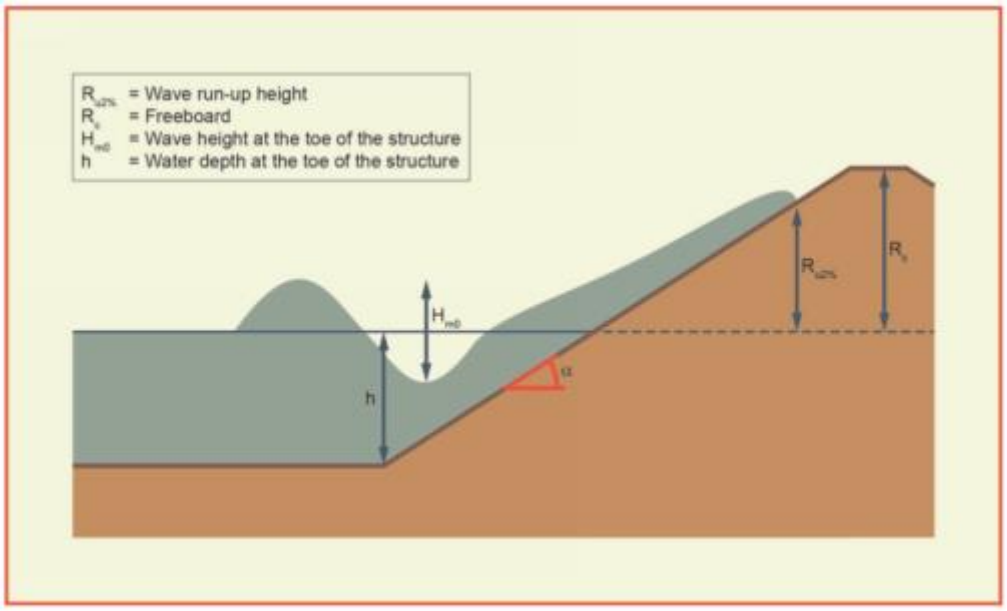

Figure 33: Depth-limited significant wave heights for uniform foreshore slopes (Figure 5.4, EurOtop 2018)

Wave overtopping of a rock structure along a shallow foreshore can be determined using the following equation:

$$\frac{q}{\sqrt{g \times H_{m0}^3}} = 0.1035 \times \exp\left(-(1.35 \frac{R_c}{\gamma_f \times \gamma_\beta \times H_{m0}})^{1.3}\right)$$ Equation 6.6, EurOtop 2018

Where:

q = mean overtopping discharge (m³/s/m)
γ_f = influence factor for the permeability and roughness of or on the slope (0.66)
γ_β = oblique wave attack influence factor.

Overtopping volumes are shown in **Table 12**.

Table 12: Wave overtopping.

Storm Event ARI	Training Wall Slope (cotα)	Water Level inc. SLR (m AHD)	Crest Height (m AHD)	Location	Wave Overtopping ($m^3/m/s$)
1 year	1.5	1.65	4	D	0.01
				E	0.01
				F	0.00
50 year	1.5	1.83	4	D	0.05
				E	0.06
				F	0.03
100 year	1.5	1.85	4	D	0.05
				E	0.07
				F	0.03

Note that the peak water level and peak wave condition may not coincide and any reduction in one or the other would reduce overtopping. In accordance with EuroTopII (2018), unsafe mean overtopping is defined as:

Return Period	Tolerable Discharge	Comment
1 in 1 year	q<1.0 l/s per m, V_{max} < 600 l/m.	For people and vehicles - People at seawall / dike crest. Clear view of the sea. H_{m0} = 2 m.
1 in 1 year	q<1.0 l/s per m V_{max} < 500 l/m.	For pedestrians – Trained staff, well shod and protected, expecting to get wet, overtopping flows at lower levels only, no falling jet, low danger of fall from walkway.
1 in 200 year	q<5.0 l/s per m, V_{max} < 2,0003,000 l/m	For structural design - Grass covered crest and landward slope; maintained and closed grass cover; H_{m0} = 1 – 3 m
1 in 200 year	q<10 l/s per m	For damage to defence – No damage to crest and rear face of embankment of clay.

Overtopping at the head of the breakwater is far more than a level that would typically be considered safe. Accordingly, other measures would need to be implemented at the site to ensure the risk to life and property is managed appropriately. Measures may include; overtopping event forecasting and early warning, closure of access to the revetment crest (or a portion of the crest) during overtopping events, revetment structure maintenance program and funding allocation.

However, it is noted that there is some uncertainty around the nearshore wave climate, which would influence overtopping volumes. Detailed wave modelling may be effective in providing more confidence

in nearshore wave climate. Furthermore, the overtopping calculations would apply to the exposed head of the training wall. Overtopping volumes would reduce further upstream.

8 Proposed Materials and Dimensions

8.1 Training Wall

Development of the upgrade of the Port Macquarie training wall is described below. Concept elements may be modified during concept development and detailed design phases.

8.1.2 Rock Armoured Training Wall

Upgrade of the training wall is proposed comprising relatively uniformly rock armour. The revetment would comprise rock armour with a narrow grading of $0.8*D_{50}$ to $1.2*D_{50}$. The rock type proposed for the revetment would be investigated during concept development and detailed design phases. The rock armour would be required to achieve material acceptance requirements for use in a marine environment.

Design parameters are provided in **Table 13**. Underlayers would satisfy filter rule requirements, particularly; internal stability, permeability and retention criterion. The use of existing rock armour sizes and gradings in the upgrade design will need careful consideration to optimise reuse of material (rather than waste) in the context of the defined design parameters.

Table 13: Rock revetment design parameters for waves.

Location	D		E		F		G	H	Comments
Design Parameters	50 year ARI	100 year ARI	50 year ARI	100 year ARI	50 year ARI	100 year ARI	50 year ARI	50 year ARI	
Significant Wave Height	2.10	2.10	2.80	2.80	2.20	2.20	1.3	1.1	
$H_{1/10}$	2.70	2.70	3.40	3.40	2.80	2.80	1.65	1.4	
$H_{2\%}$	2.90	2.90	3.50	3.60	3.00	3.00	2.3	1.96	
Tm	9.9	10.2	10	10.3	9.8	10.1	9.4	9.3	
Damage	0-5% Sd=2	10% Sd = 5	0-5% Sd=2	10% Sd = 5	0-5% Sd=2	10% Sd = 5	0-5% Sd=2	0-5% Sd=2	
Slope (V:H)	1:1.5								
Minimum Design Rock Dry Density	2.6 t/m^3								
Water Depth								6.28m	
Storm Duration	6 hr (considering water level variation over that time)								
Van der Meer Shallow Water								1t (0.7m)	

	Carver and Heimbaugh (1989)	1.9t (0.9m)	1.8t (0.9m)							for damage

Table 14: Rock revetment design parameters for fresh water flow events..

Design Parameters			50 year ARI	100 year ARI	50 year ARI	100 year ARI	
			Straight Reach		Transition/Outer Bend		
Flood Velocity				4.3 m/s		4.3 m/s	
		M50	60	210	500	1700	No reduction for damage
		Dn50	290	430	570	870	
	Escarameia and May	M50	330	1140	1980	6800	No reduction for damage
		Dn50	500	760	910	1380	

9 Opportunities and Constraints

The following Opportunities have been identified:

1. The nearby beaches and entrance bar are approximately in equilibrium.
2. It is assumed that the entrance channel and beach near the southern training wall would have scoured to the design scour depth at least once since construction of the training wall. This would have resulted in reshaping of the toe of the training wall. Placing rock armour on top of the existing rock would ensure that the structure is founded at the design scour depth.
3. The design wave height at the structure has been refined and reduced by undertaking a detailed SWAN model that considers freshwater or ebb tidal flows and nearshore bathymetry. The model assists in design optimisation including spatially armour requirements and overtopping risk.
4. Available space landward of the head where the breakwater head widens to locate a viewing outlook area for Town Beach and across the entrance which does not require crest widening or conflict with the crest accessway.

The following Constraints have been identified:

1. The southern training wall is identified as a heritage item. Additional consultation is recommended.
2. Spatial constraints and ability to widen footpath while maintaining the existing access roads (including at the breakwater head where a viewing lookout has been proposed in Council Masterplans).
3. Ongoing geotechnical instability limiting the feasibility of founding ancillary structures on the breakwater rock slope.
4. Risk of overtopping, particularly near the head of the training wall.

10 Safety in Design Considerations for Construction and Accessways

In accordance with Clause 295 of the *Work Health and Safety Regulations 2017*, a designer of a structure or part of a structure must provide a written report to a person conducting a business or undertaking (PCBU) who commissioned the design. The report must specify the hazards relating to the design of the structure that, so far as the designer is reasonably aware:

- create a health and safety risk to persons who are to carry out the construction work; and,
- are associated only with the particular design and not with other designs of the same type of structure.

The Safety Report captures designs of structures that have unusual or atypical features which present hazards and risks during the construction phase that are unique to the particular design.

The safety report should include information about:

- any hazardous materials or structural features and the designer's assessment of the risk of injury or illness to construction workers arising from those hazards; and,
- the action the designer has taken to control those risks, for example changes to the design.

Safety in Design considerations are included in the Safety Report. They should consider the various stages of the lifecycle of a structure including:

- construction;
- use;
- maintenance;
- modification; and,
- demolition and dismantling.

10.1 Description of Works

The works at Port Macquarie comprises:

- upgrade of the training wall; and,
- foreshore improvements.

It is expected that the works would be constructed with a combination of floating and land based plant and equipment. Typical hazards for construction of this type of structure include:

- exposure to the elements (heat, cold, UV radiation);
- aquatic and/or terrestrial fauna bites/stings;
- slips, trips and falls (e.g. unstable, wet, slippery surfaces, excavations, exposed reinforcement bars and posts/pegs);
- manual handling and heavy lifts;
- risk of unsecure loads falling during handling and placement;
- geotechnical instability and/or unstable batters, particularly where high loads are positioned at the crest of an embankment;
- use, handling and storage of hazardous chemicals/dangerous goods;

- excessive noise for long periods of time;
- presence of hazardous airborne contaminants (e.g. gasses and vapours, silica etc.);
- underground or overhead utilities;
- work near or over water and risk of drowning;
- diving activities; and,
- activities involving floating plant and equipment including the risk of overturning or submersion.

10.2 Risk Assessment

The following risk assessment is provided to ensure that all stages of the project life cycle are as safe as possible through good engineering design and to satisfy requirements of the *Work Health and Safety Regulations 2011*. Having regard to the *Code of Practice Safe Design of Structures* (SafeWork NSW, 2019), the risk assessment involves the following steps:

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

The risk associated with a hazard is a combination of the likelihood of the hazard occurring and its consequence. The description of likelihood and consequence is presented in **Table 15** and **Table 16,** respectively.

Table 15: Likelihood table.

Likelihood	Description
5 Almost Certain	
4 Likely	
3 Possible	
2 Unlikely	
1 Rare	

Table 16: Consequence 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 17**. An interpretation of risk assessment scores in regard to “tolerable” and “unacceptable” risk is provided in the table.

Table 17: 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)	
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 *Code of Practice Safe Design of Structures* (SafeWork NSW, 2019). The Code of Practice is made under the *Work Health and Safety Act 2011*. 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 works would be undertaken during the detailed design phase of the project. 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.

11 References

Refer **Section 2**.

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CPL COMMS PILLAR
EPIL ELECTRICAL PILLAR
EPIT ELECTRICAL PIT
FFL FINISHED FLOOR LEVEL
FH HYDRANT
GLY STORM WATER GRATE
LP LIGHT POLE
MH MAN HOLE
PP POWER POLE
RDG RIDGE HEIGHT
SGN SIGN
SMH SEWER MANHOLE
SIO SEWER INSPECTION OPENING
SV STOP VALVE
SVR STOP VALVE (RECYCLED)
TAP TAP
WM WATER METER
BIN BIN

TITLE BOUNDARY
BOUNDARY ADJACENT
EASEMENT
KERB
KERB INVERT
ROAD CENTRELINE
ROAD EDGE
BOTTOM BANK
TOP BANK
FENCE
EL ELECTRICAL (U/G)
DR DRAINAGE (U/G)
SE SEWER (U/G)
EDGE CONCRETE
EDGE ROCK

CONTOUR MAJOR (Interval 1.0m)
CONTOUR MINOR (Interval 0.2m)
CONTOUR MAJOR (Interval 1.0m)
CONTOUR MINOR (Interval 0.2m)

N

TIDAL INFORAMTION :

SITE	HAT	LAT	HAT (AHD)	LAT (AHD)	RANGE (HAT-LAT)	MSL	MSL (AHD)
PORT MACQUARIE	1.04	-0.74	1.04	-0.74	1.78	0.02	0.02
(MHL2574 - Ocean Tides Annual Report 2016-17)							

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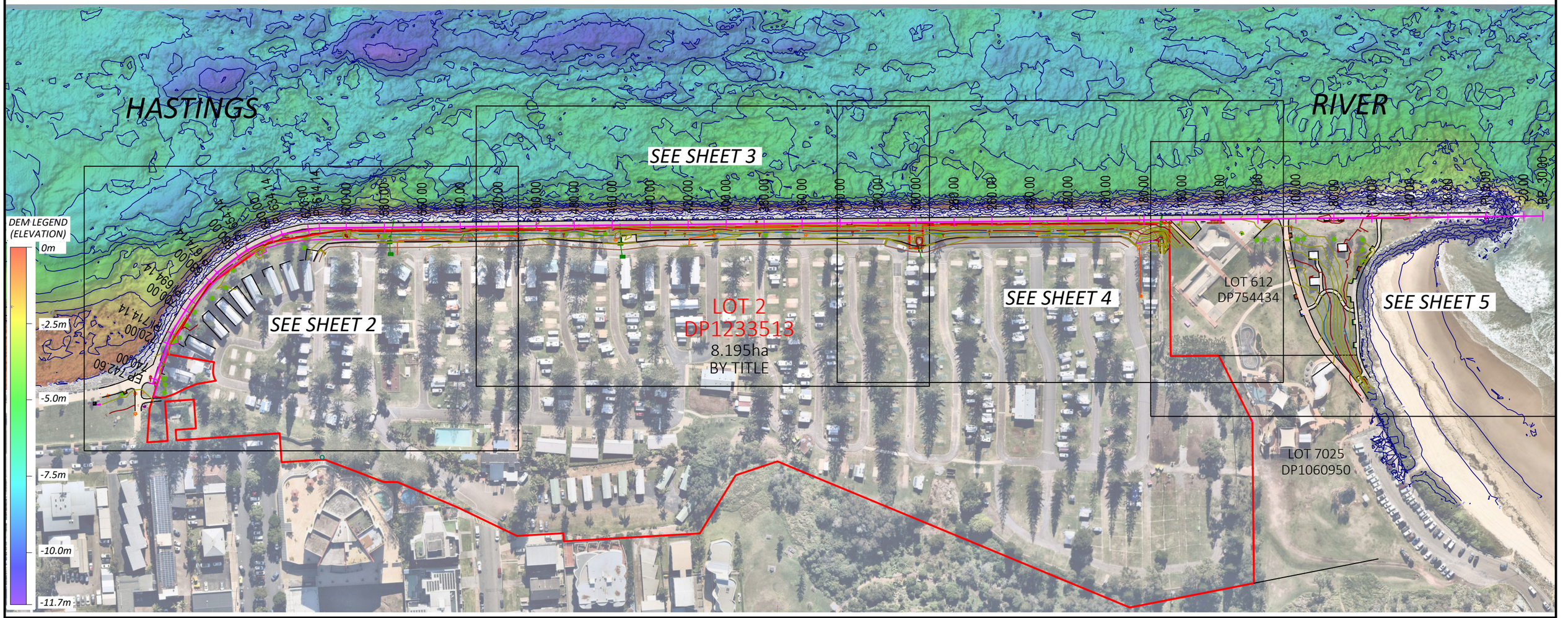

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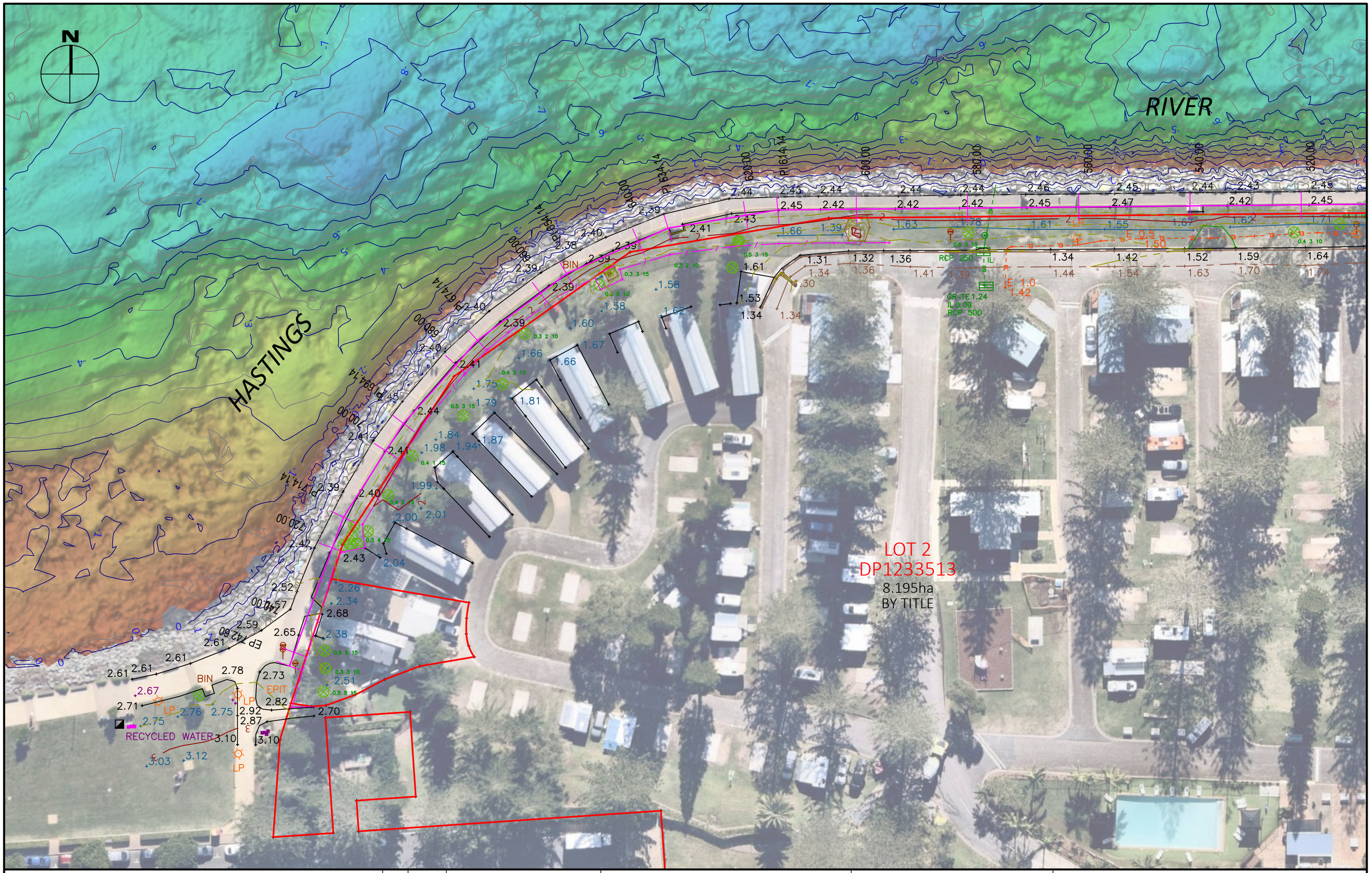

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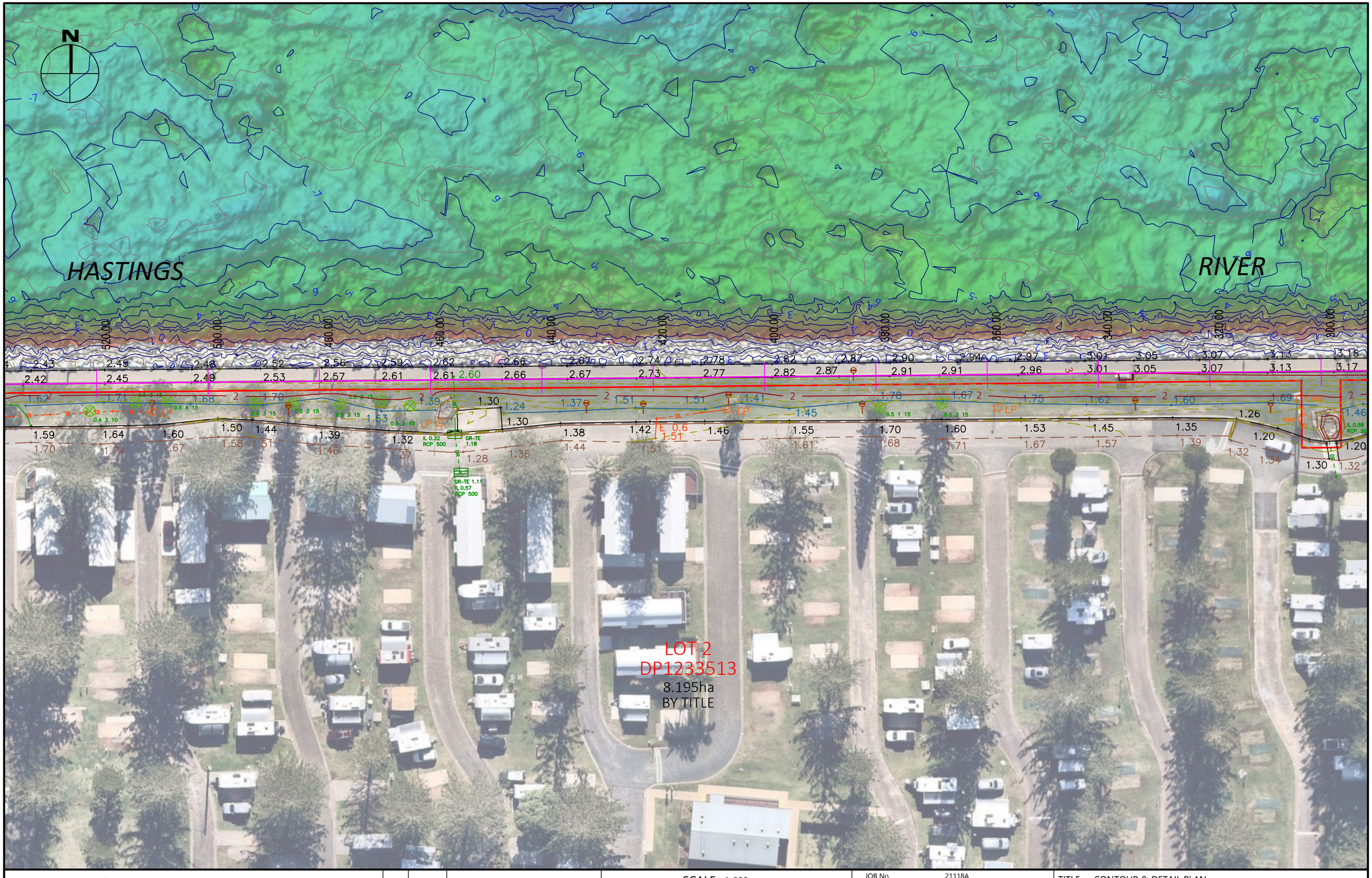

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CAUTION: THE ORIGINAL SCALE OF THIS DRAWING MAY HAVE BEEN ALTERED BY REDUCTION, ENLARGEMENT AND/OR PRINTING. THE SCALE SHOULD BE VERIFIED PRIOR TO TO USING THE DRAWING.

JOB No.	21118A
SURVEYED BY	SB
CHECKED BY	TS
DATE OF SURVEY	29/07/2021
Vz DATUM	AHD71
ORIGIN	SSM10544
Hz DATUM	MGA56 (1994)

ORIGINAL SHEET SIZE A3

TITLE: CONTOUR & DETAIL PLAN

SITE: PORT MACQUARIE BREAK WALL, PORT MACQUARIE, NSW 2444

CLIENT: TRANSPORT NSW

DWG No: 21118A.DWG SHEET 4 OF 16 REV 0

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REV	DATE	DESCRIPTION
0	29/07/2021	ORIGINAL ISSUE

SCALE 1:600

6 0 6 12 18 24 30

CAUTION: THE ORIGINAL SCALE OF THIS DRAWING MAY HAVE BEEN ALTERED BY REDUCTION, ENLARGEMENT AND/OR PRINTING. THE SCALE SHOULD BE VERIFIED PRIOR TO TO USING THE DRAWING.

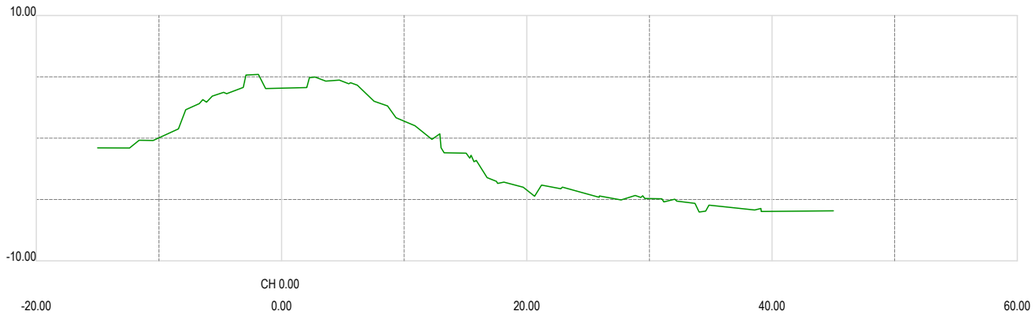
JOB No.	21118A
SURVEYED BY	SB
CHECKED BY	TS
DATE OF SURVEY	29/07/2021
Vz DATUM	AHD71
ORIGIN	SSM10544
Hz DATUM	MGA56 (1994)
ORIGINAL SHEET SIZE	A3

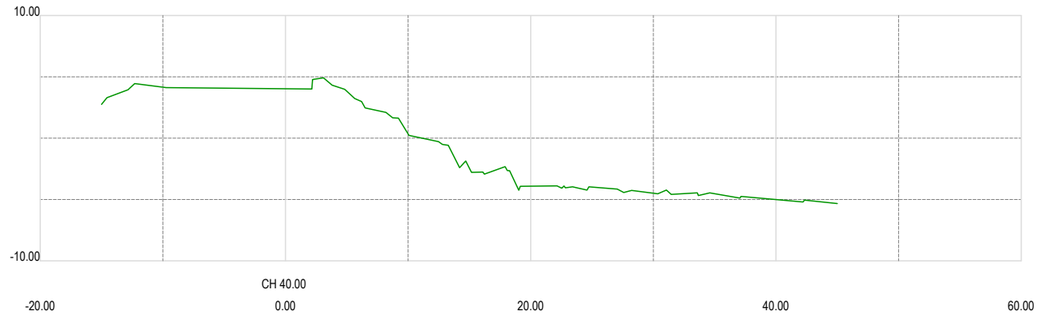
TITLE: CONTOUR & DETAIL PLAN

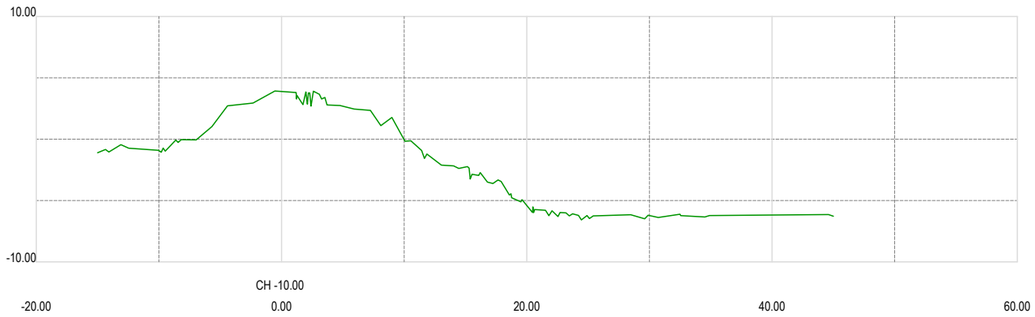
SITE: PORT MACQUARIE BREAK WALL, PORT MACQUARIE, NSW 2444

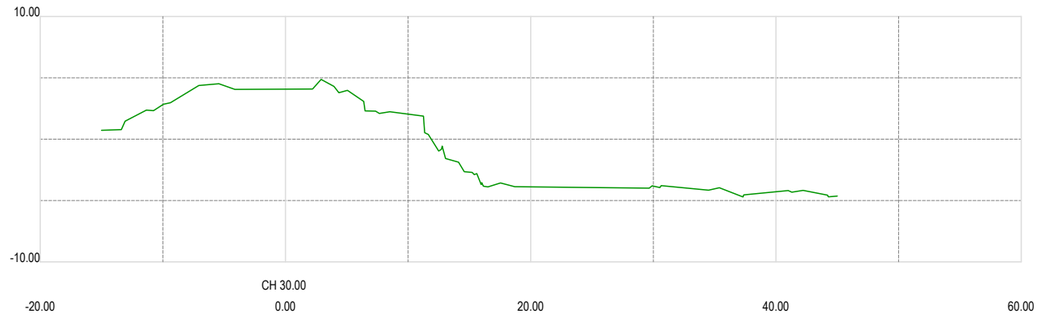
CLIENT: TRANSPORT NSW

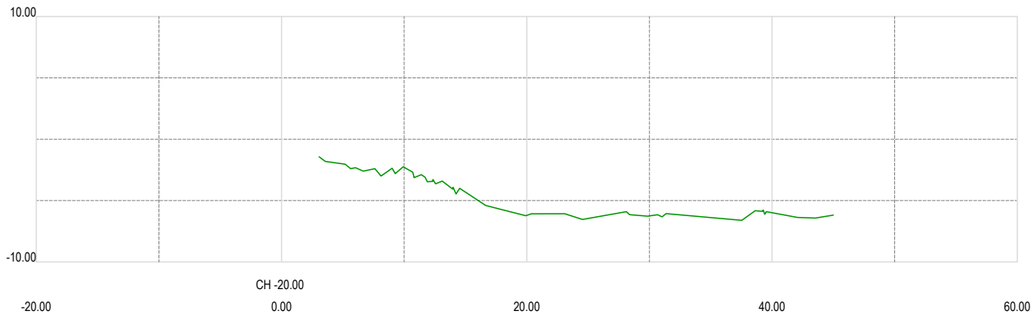
DWG No: 21118A.DWG SHEET 5 OF 16 REV0

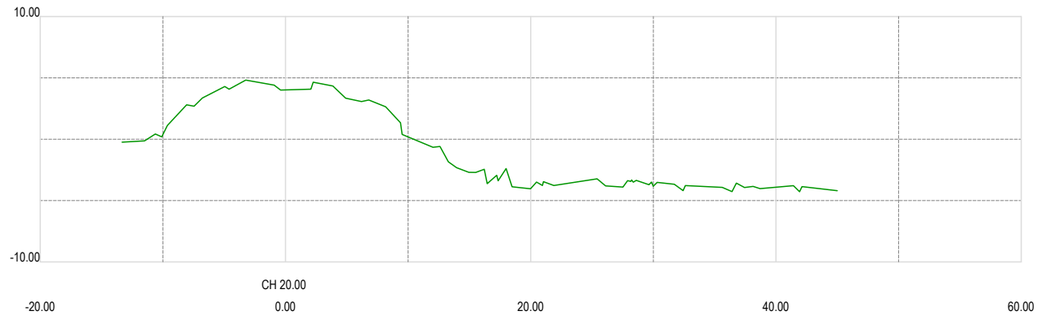


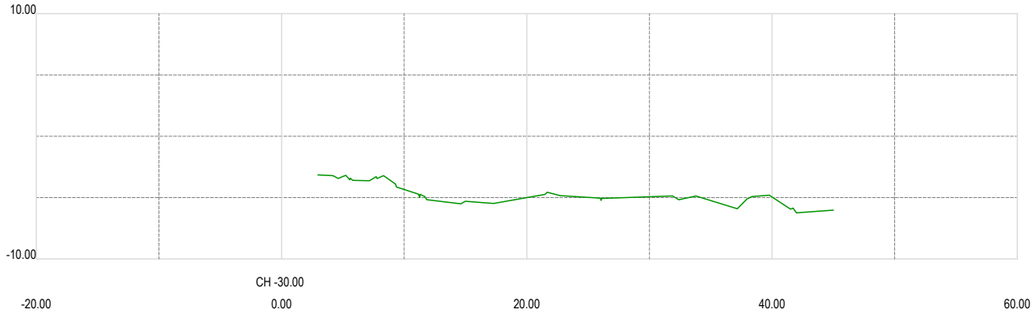


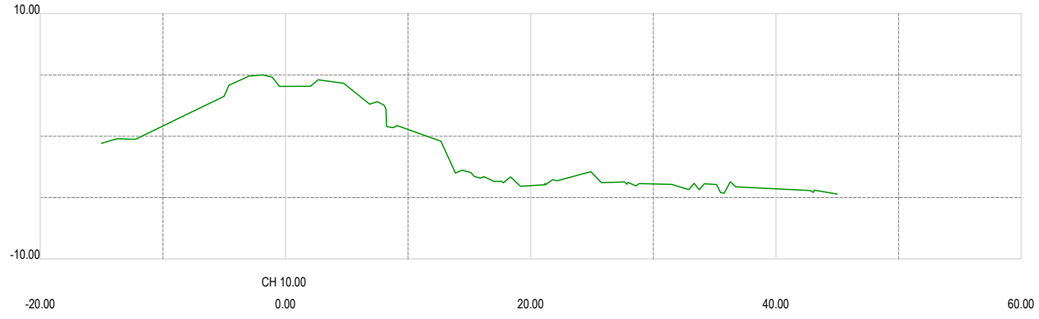












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REV	DATE	DESCRIPTION
0	29/07/2021	ORIGINAL ISSUE

SCALE

Scale Horizontal 1:500 Vertical 1:500

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JOB No.	21118A
SURVEYED BY	SB
CHECKED BY	TS
DATE OF SURVEY	29/07/2021
Vz DATUM	AHD71
ORIGIN	SSM10544
Hz DATUM	MGA56 (1994)

ORIGINAL SHEET SIZE A3

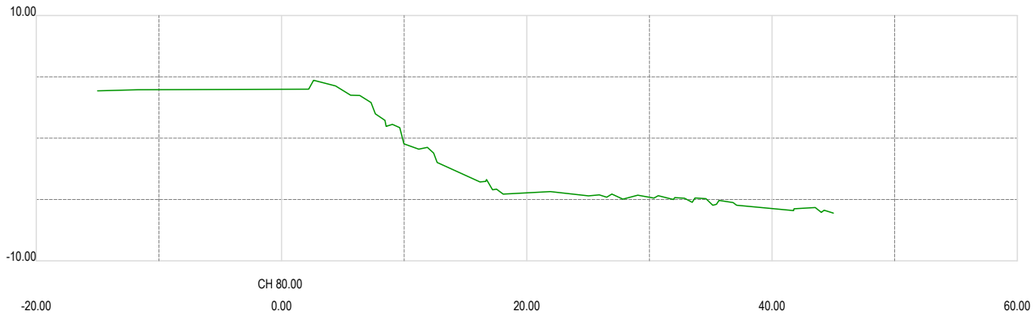
TITLE: CROSS SECTIONS CH-10-40

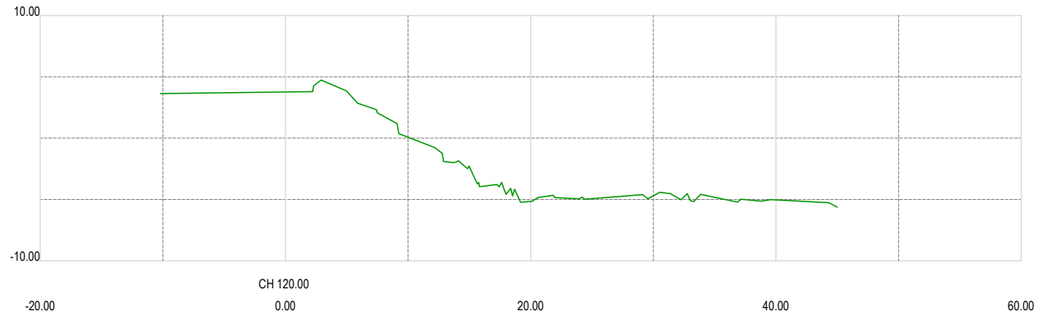
SITE: PORT MACQUARIE BREAK WALL, PORT MACQUARIE, NSW 2444

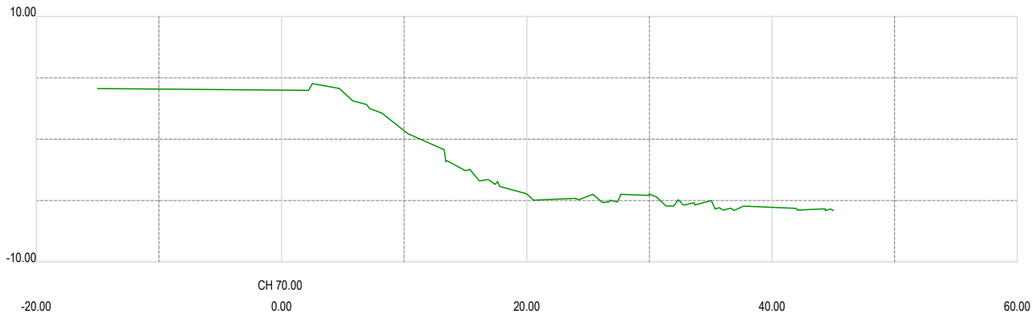
CLIENT: TRANSPORT NSW

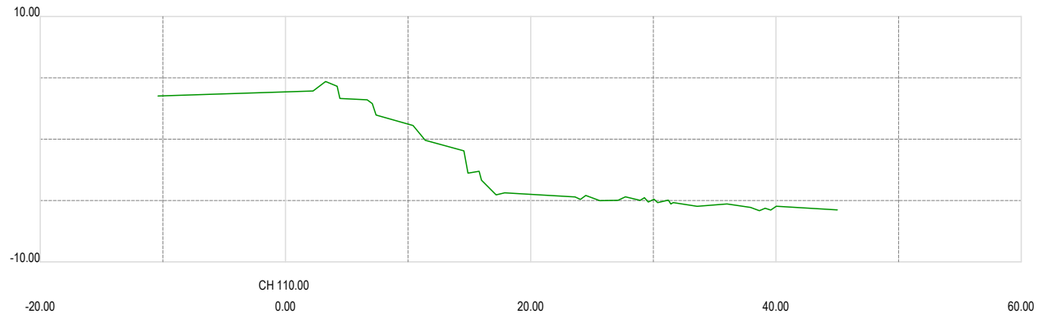
DWG No: 21118A.DWG

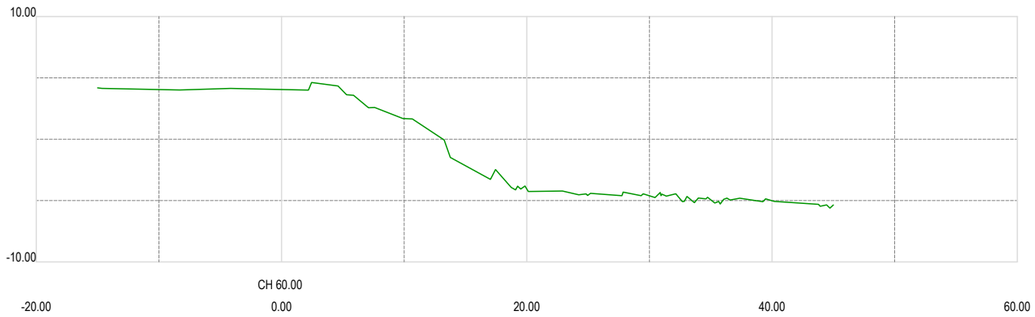
REV0

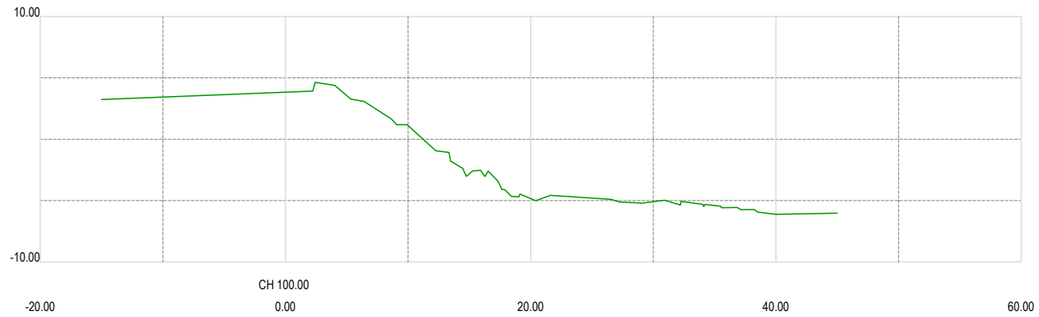


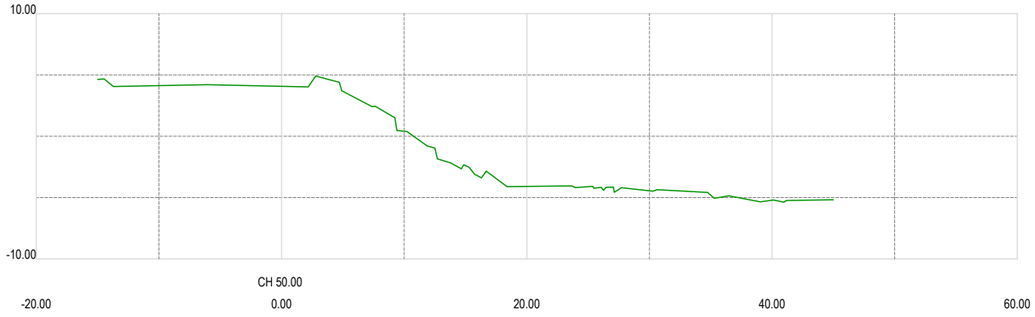


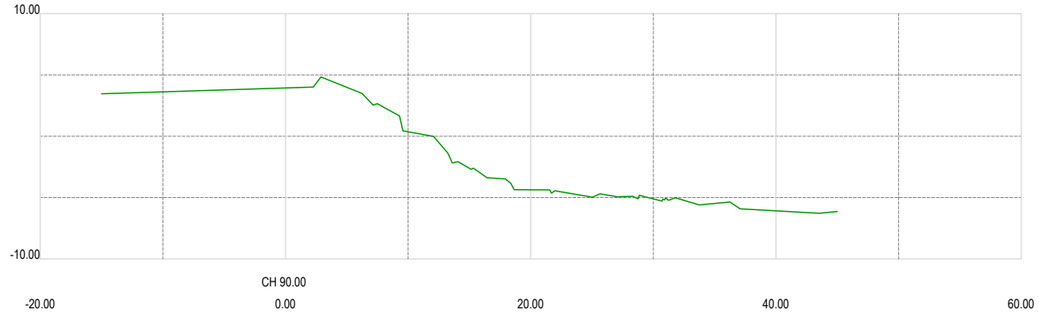












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REV	DATE	DESCRIPTION
0	29/07/2021	ORIGINAL ISSUE

SCALE

Scale Horizontal 1:500 Vertical 1:500

CAUTION: THE ORIGINAL SCALE OF THIS DRAWING MAY HAVE BEEN ALTERED BY REDUCTION, ENLARGEMENT AND/OR PRINTING. THE SCALE SHOULD BE VERIFIED PRIOR TO TO USING THE DRAWING.

JOB No.	21118A
SURVEYED BY	SB
CHECKED BY	TS
DATE OF SURVEY	29/07/2021
Vz DATUM	AHD71
ORIGIN	SSM10544
Hz DATUM	MGA56 (1994)

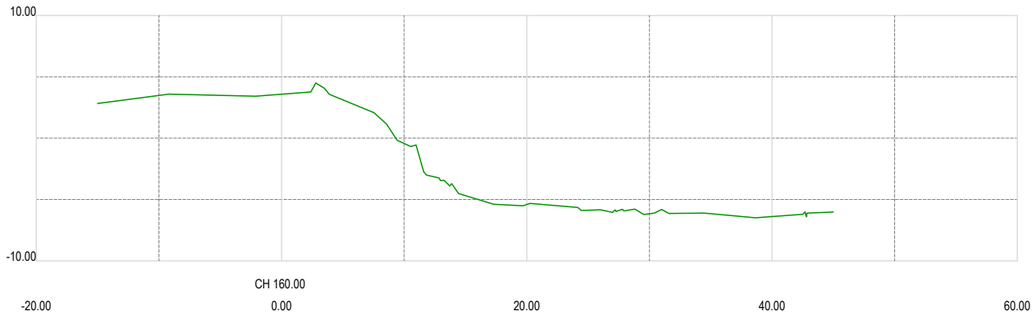
ORIGINAL SHEET SIZE A3

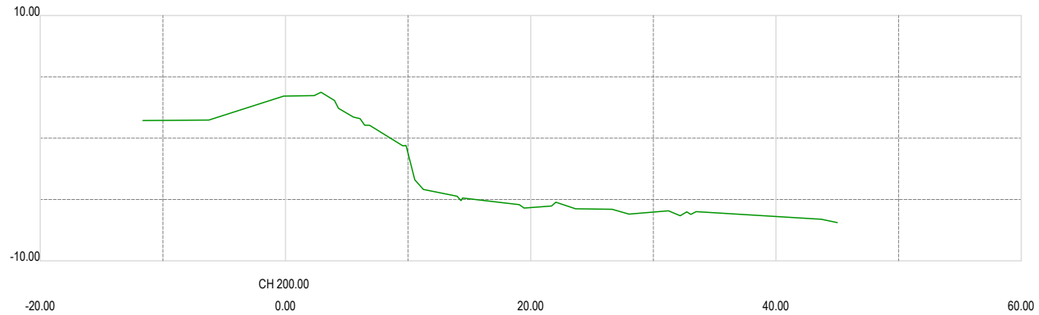
TITLE: CROSS SECTIONS CH50-120

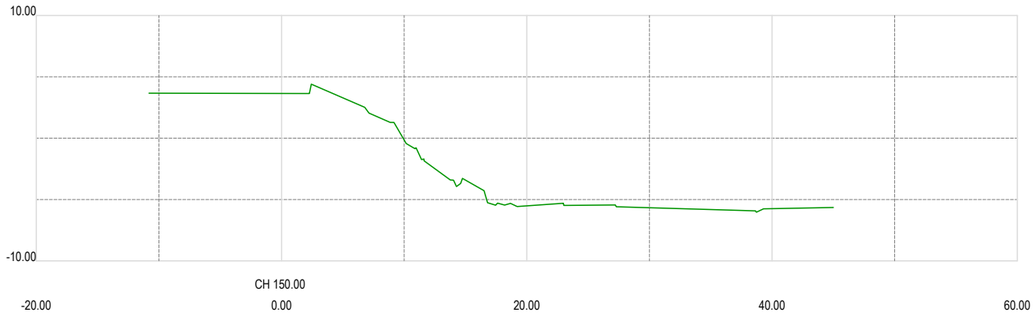
SITE: PORT MACQUARIE BREAK WALL, PORT MACQUARIE, NSW 2444

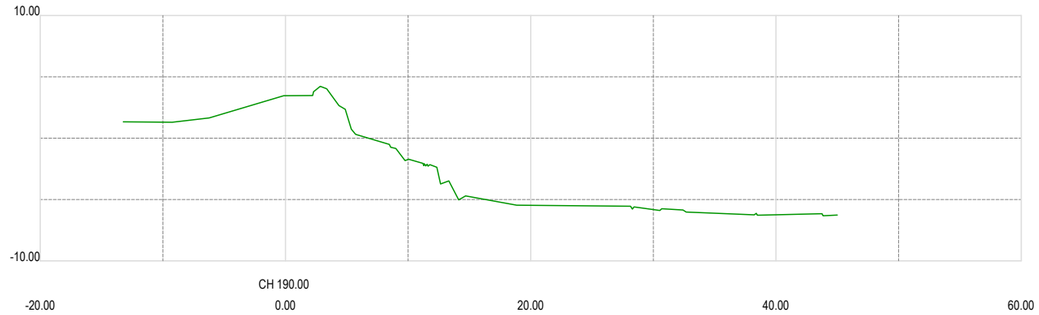
CLIENT: TRANSPORT NSW

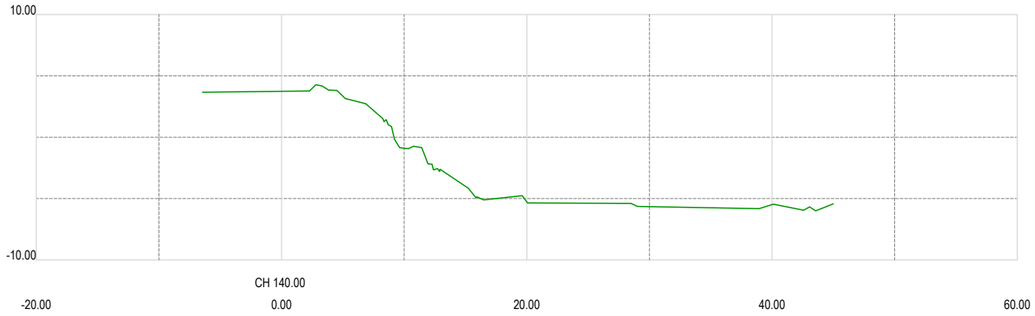
DWG No: 21118A.DWG SHEET 7 OF 16 REV0

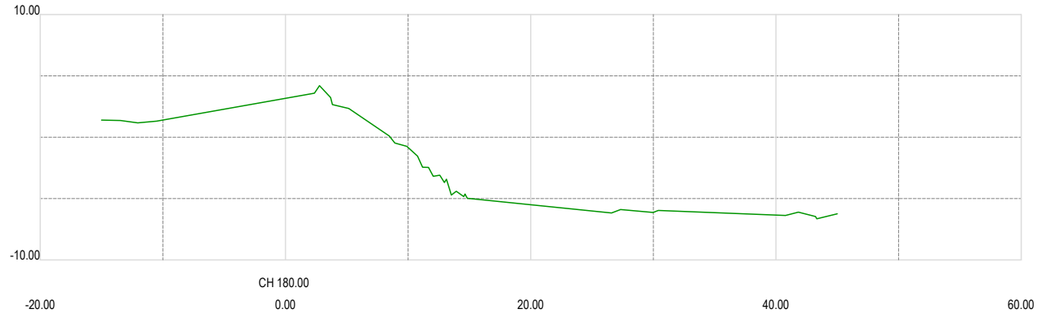


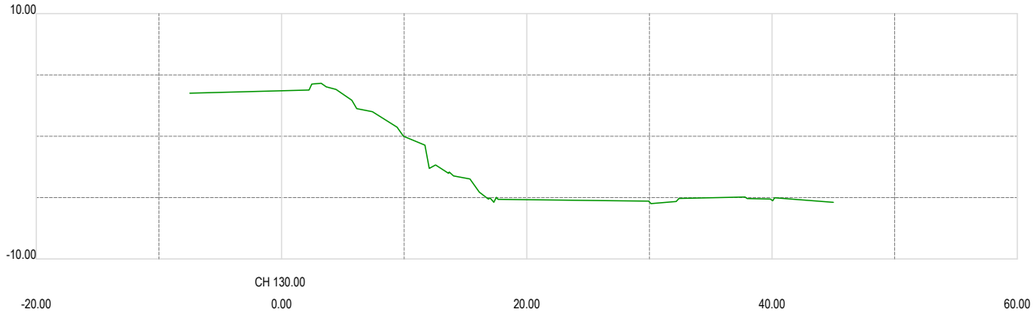


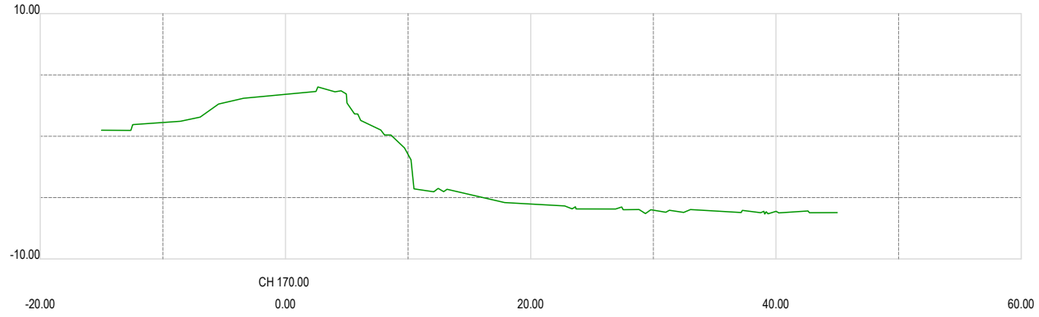












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REV	DATE	DESCRIPTION
0	29/07/2021	ORIGINAL ISSUE

SCALE

Scale Horizontal 1:500 Vertical 1:500

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JOB No.	21118A
SURVEYED BY	SB
CHECKED BY	TS
DATE OF SURVEY	29/07/2021
Vz DATUM	AHD71
ORIGIN	SSM10544
Hz DATUM	MGA56 (1994)

ORIGINAL SHEET SIZE A3

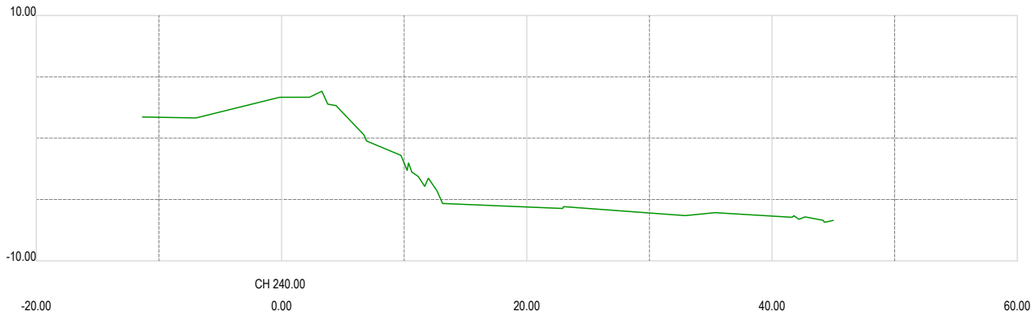
TITLE: CROSS SECTIONS CH130-200

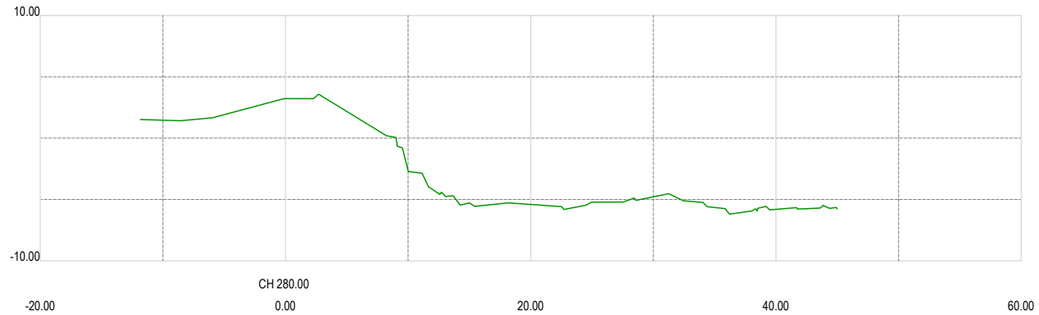
SITE: PORT MACQUARIE BREAK WALL, PORT MACQUARIE, NSW 2444

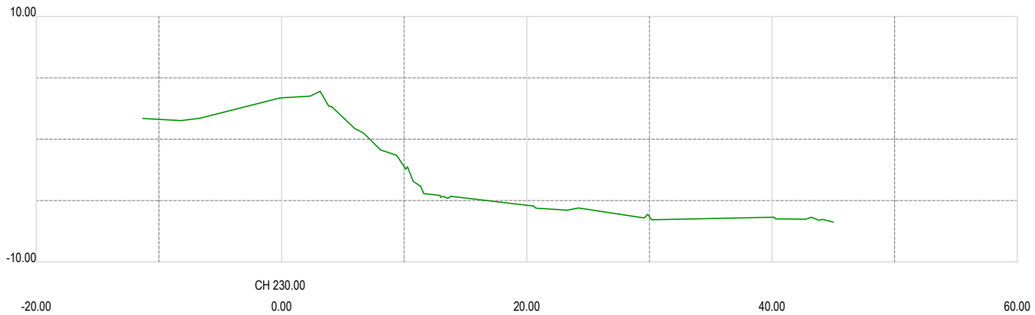
CLIENT: TRANSPORT NSW

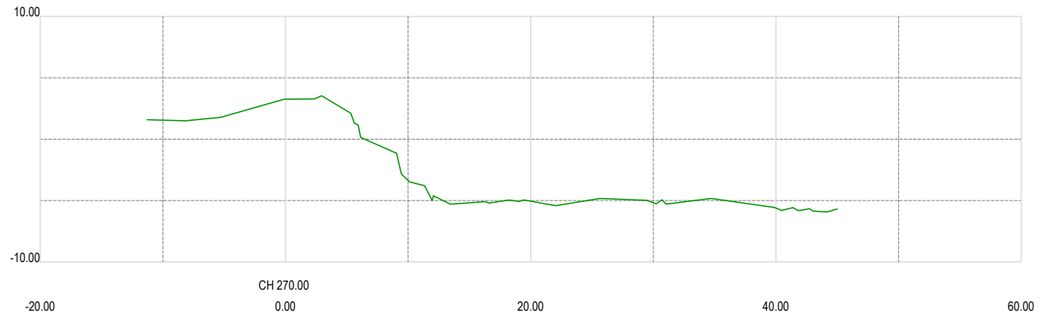
DWG No: 21118A.DWG

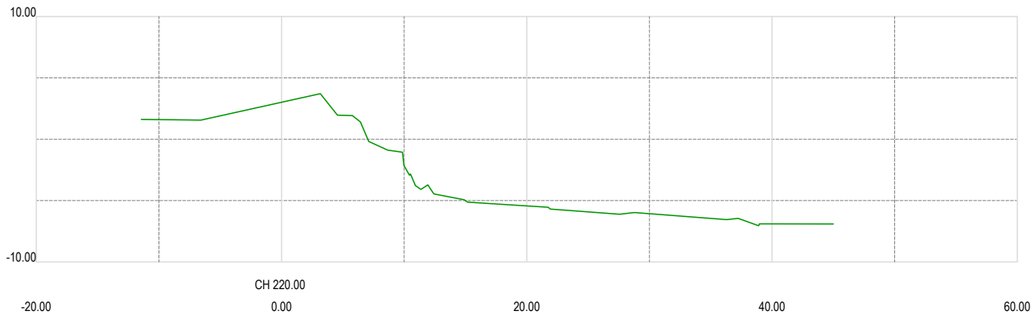
REV0

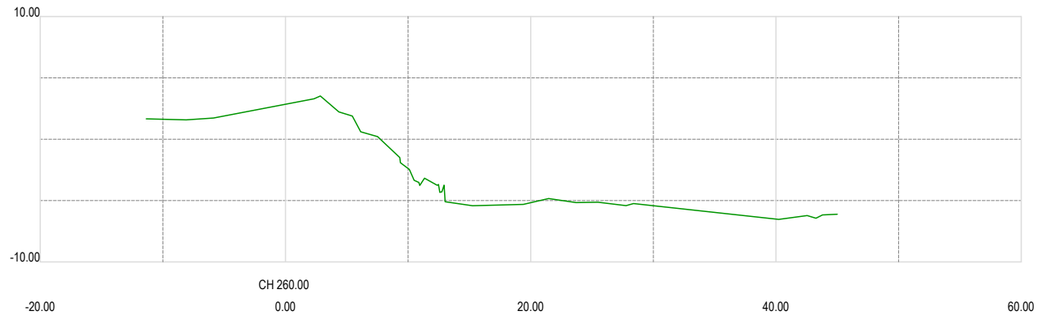


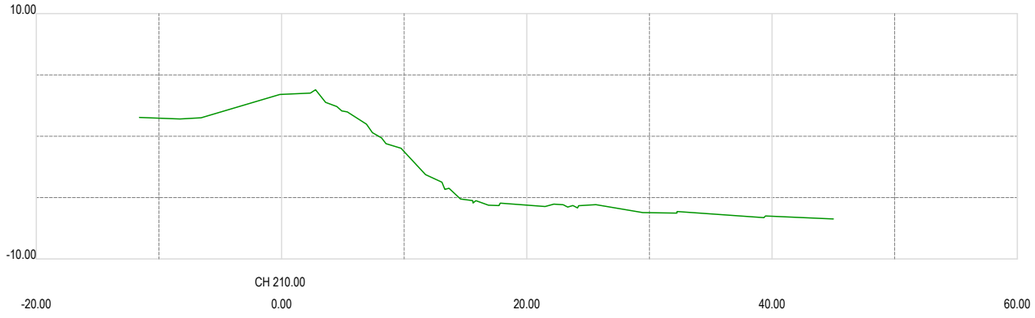


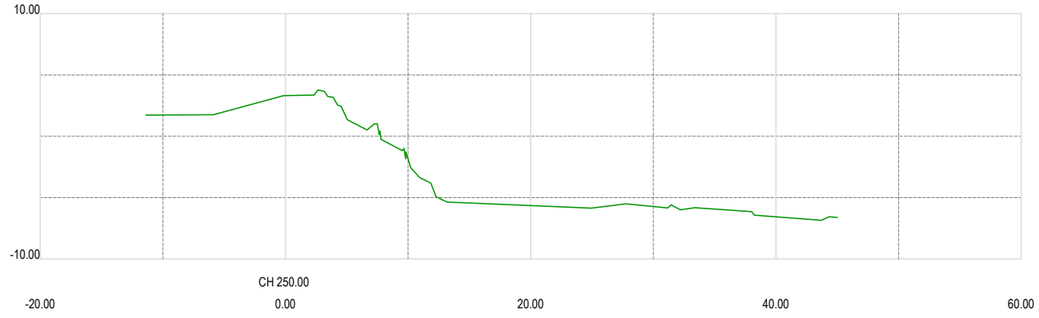












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REV	DATE	DESCRIPTION
0	29/07/2021	ORIGINAL ISSUE

SCALE

Scale Horizontal 1:500 Vertical 1:500

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JOB No.	21118A
SURVEYED BY	SB
CHECKED BY	TS
DATE OF SURVEY	29/07/2021
Vz DATUM	AHD71
ORIGIN	SSM10544
Hz DATUM	MGA56 (1994)

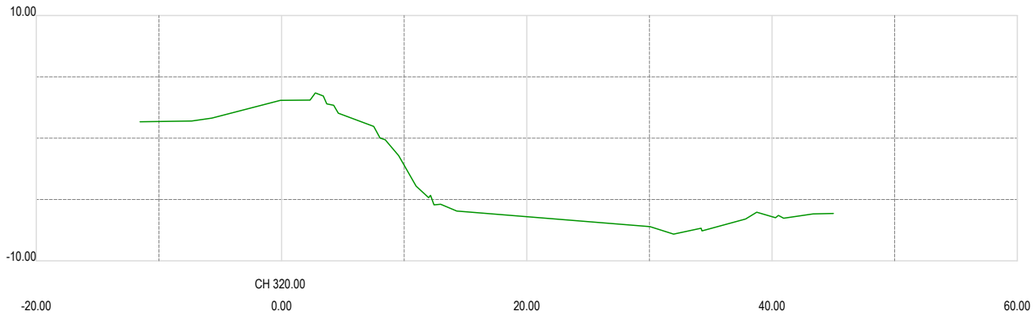
ORIGINAL SHEET SIZE A3

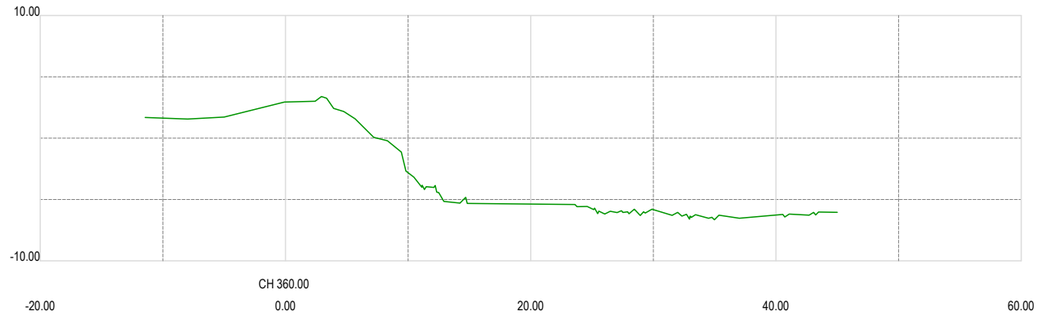
TITLE: CROSS SECTIONS CH210-280

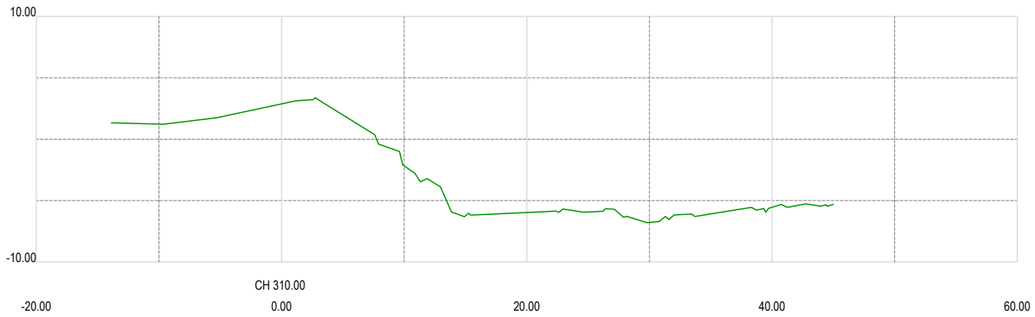
SITE: PORT MACQUARIE BREAK WALL, PORT MACQUARIE, NSW 2444

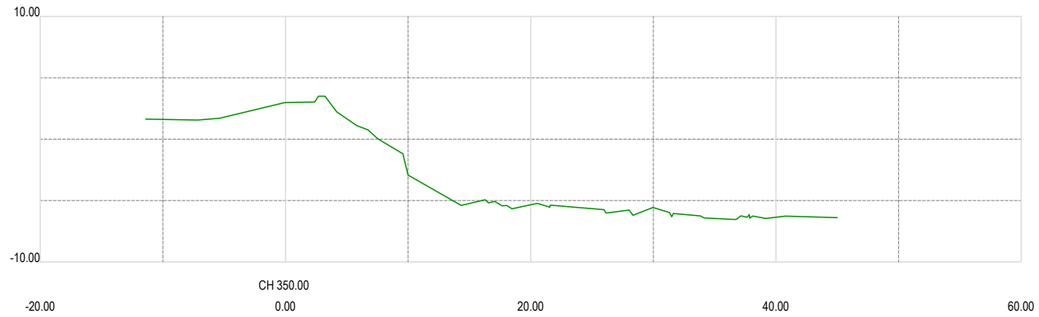
CLIENT: TRANSPORT NSW

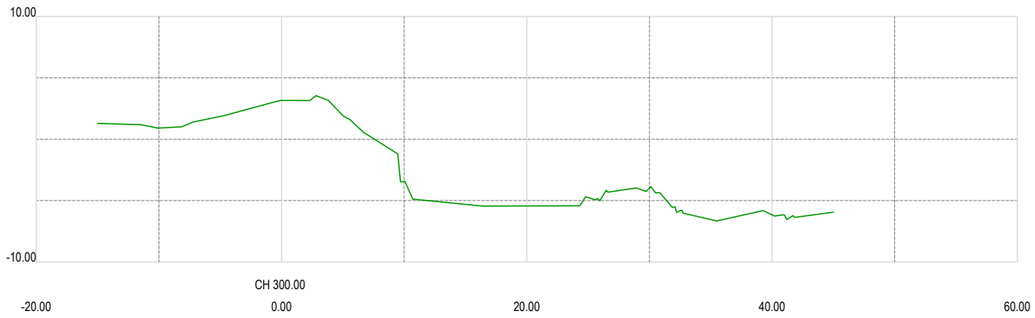
DWG No: 21118A.DWG SHEET 9 OF 16 REV0

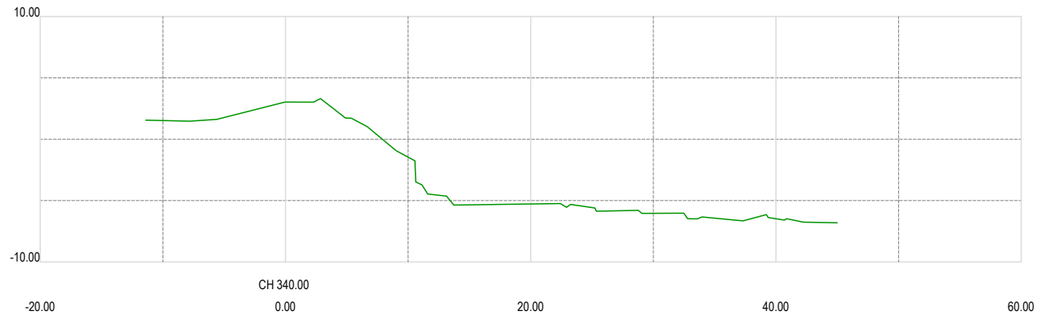


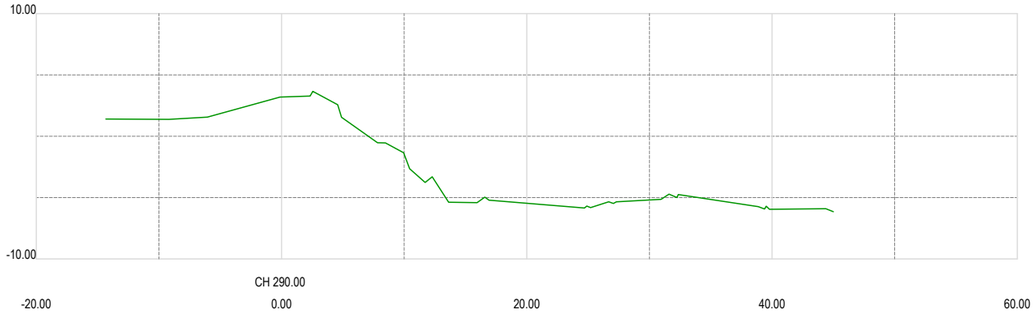


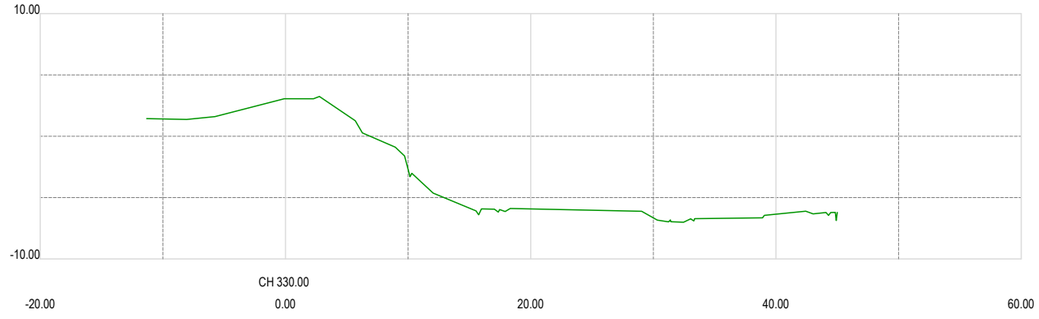












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REV	DATE	DESCRIPTION
0	29/07/2021	ORIGINAL ISSUE

SCALE

Scale Horizontal 1:500 Vertical 1:500

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JOB No.	21118A
SURVEYED BY	SB
CHECKED BY	TS
DATE OF SURVEY	29/07/2021
Vz DATUM	AHD71
ORIGIN	SSM10544
Hz DATUM	MGA56 (1994)

ORIGINAL SHEET SIZE A3

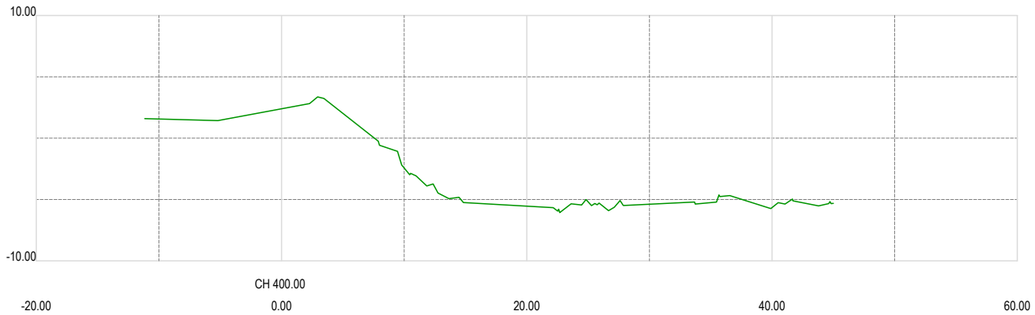
TITLE: CROSS SECTIONS CH290-360

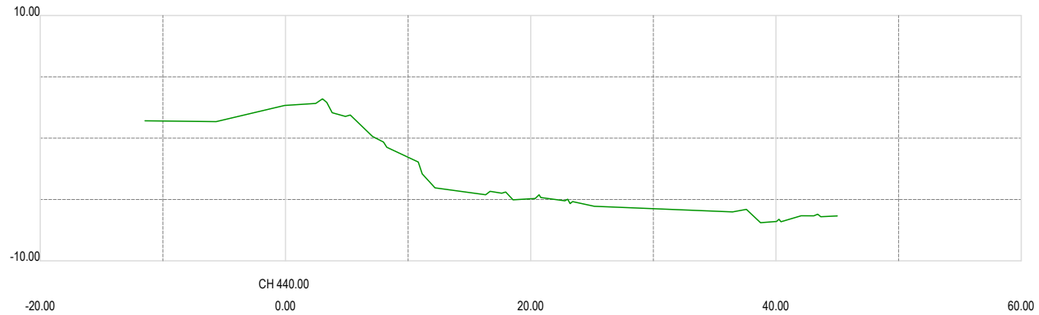
SITE: PORT MACQUARIE BREAK WALL, PORT MACQUARIE, NSW 2444

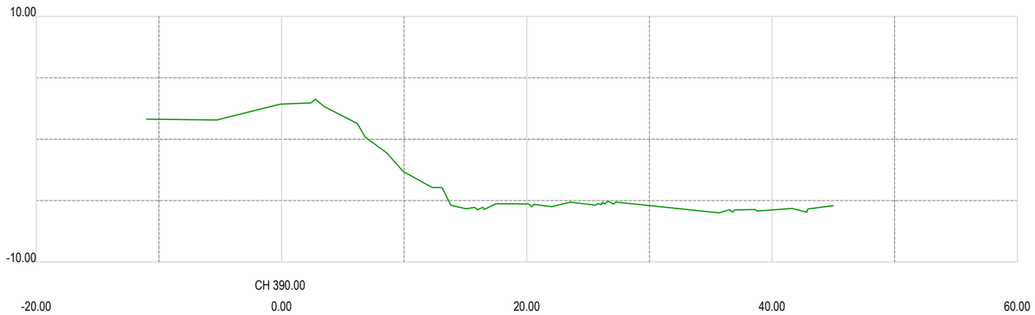
CLIENT: TRANSPORT NSW

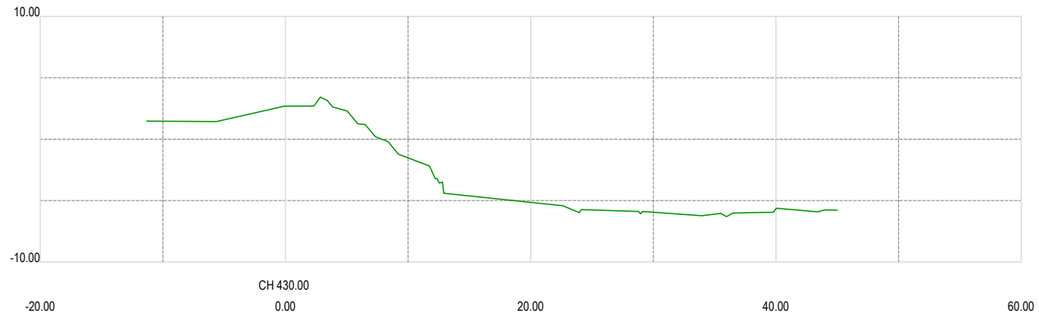
DWG No: 21118A.DWG

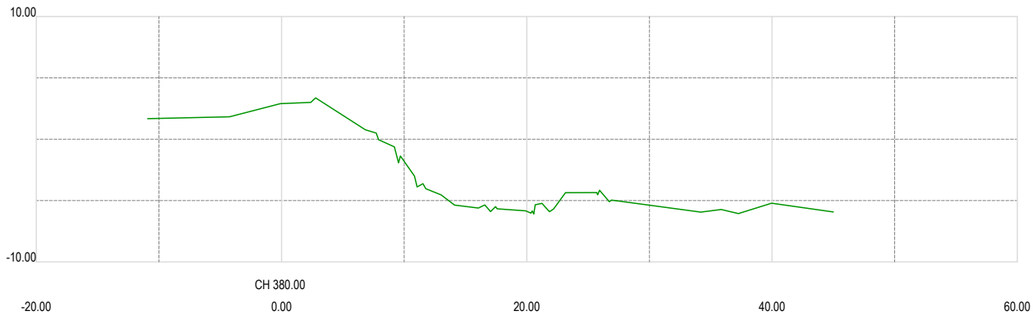
REV0

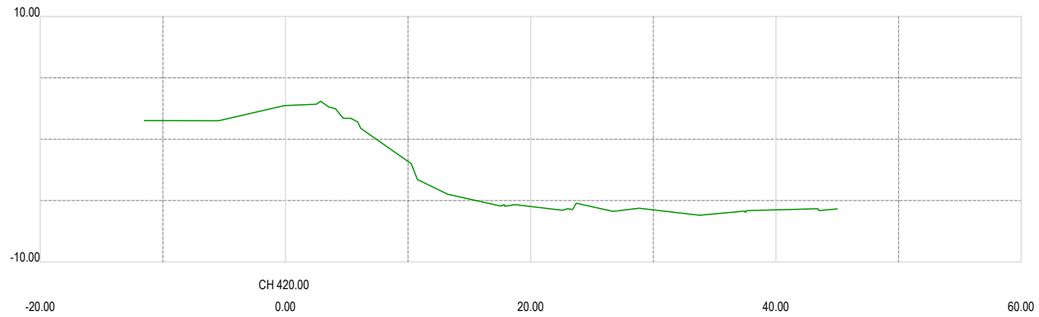


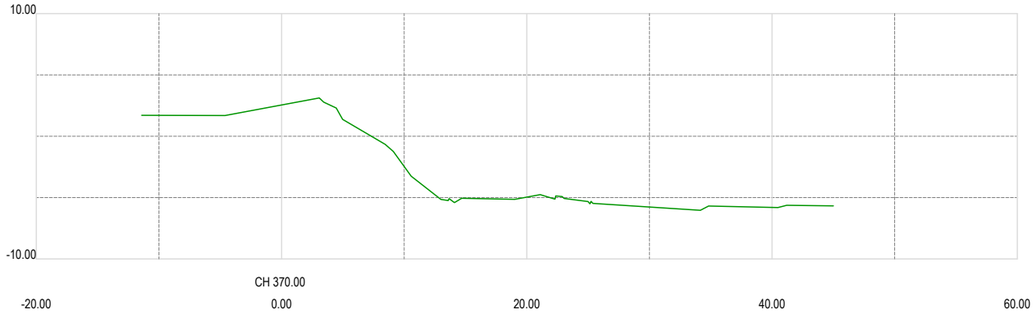


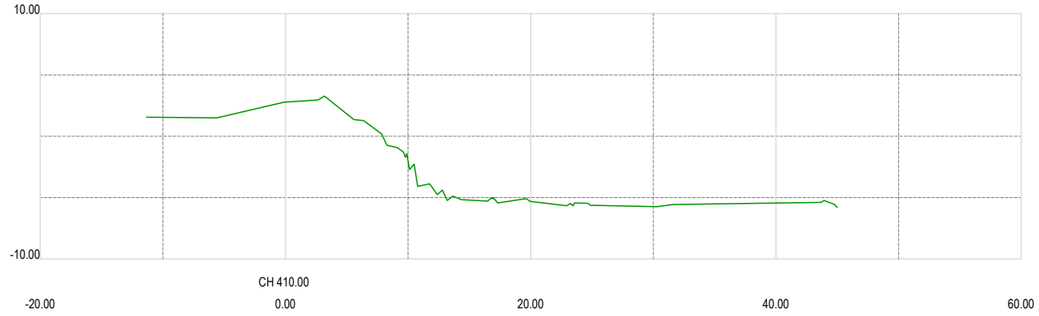












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REV	DATE	DESCRIPTION
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SCALE

Scale Horizontal 1:500 Vertical 1:500

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JOB No.	21118A
SURVEYED BY	SB
CHECKED BY	TS
DATE OF SURVEY	29/07/2021
Vz DATUM	AHD71
ORIGIN	SSM10544
Hz DATUM	MGA56 (1994)

ORIGINAL SHEET SIZE A3

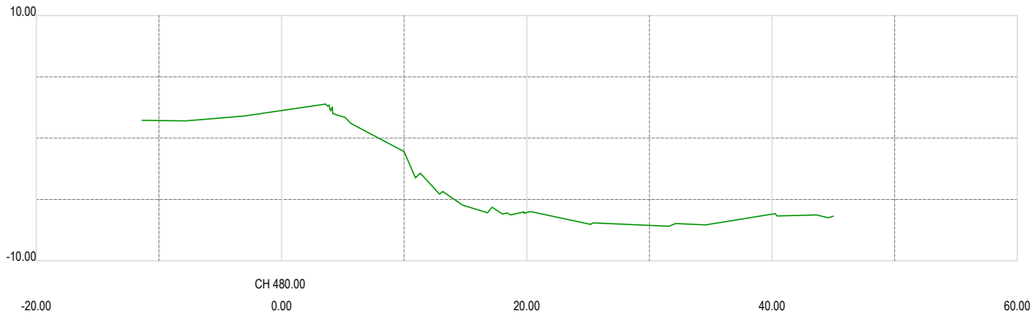
TITLE: CROSS SECTIONS CH370-440

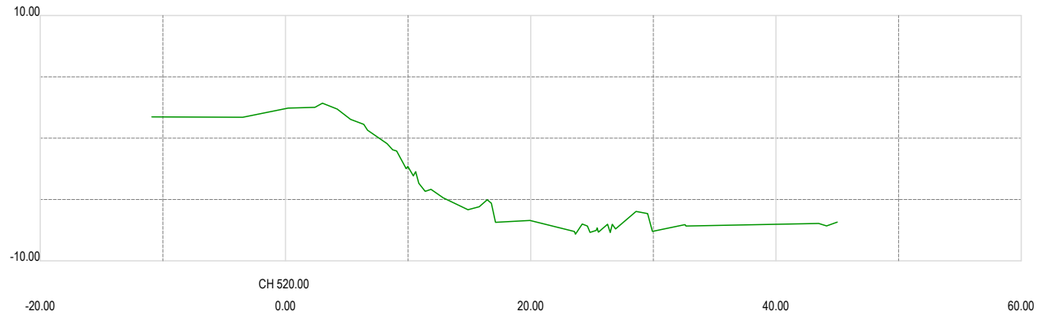
SITE: PORT MACQUARIE BREAK WALL, PORT MACQUARIE, NSW 2444

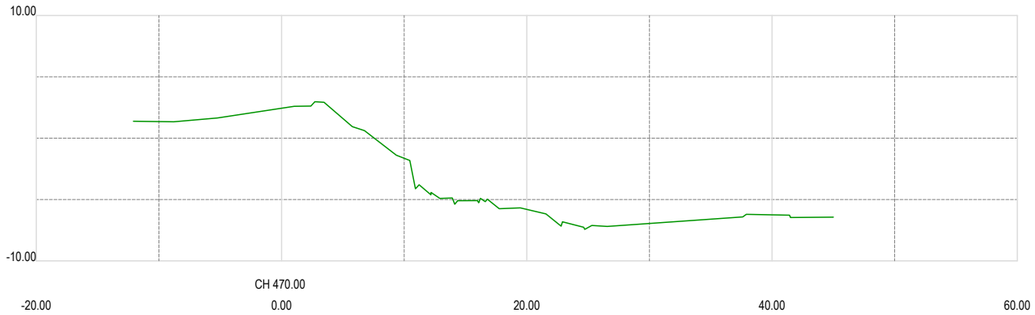
CLIENT: TRANSPORT NSW

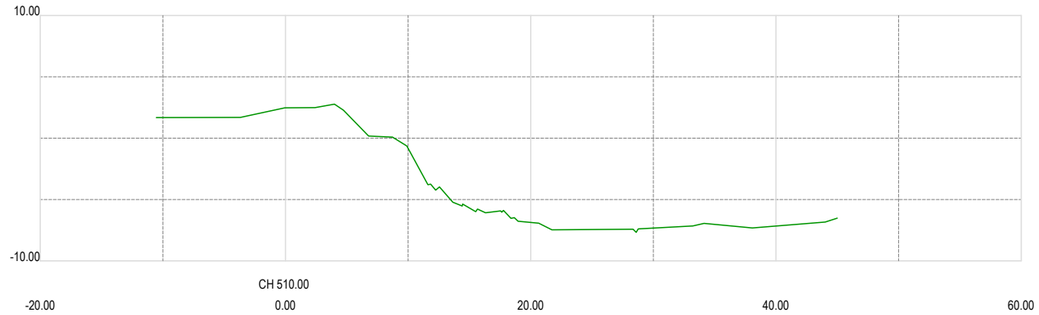
DWG No: 21118A.DWG

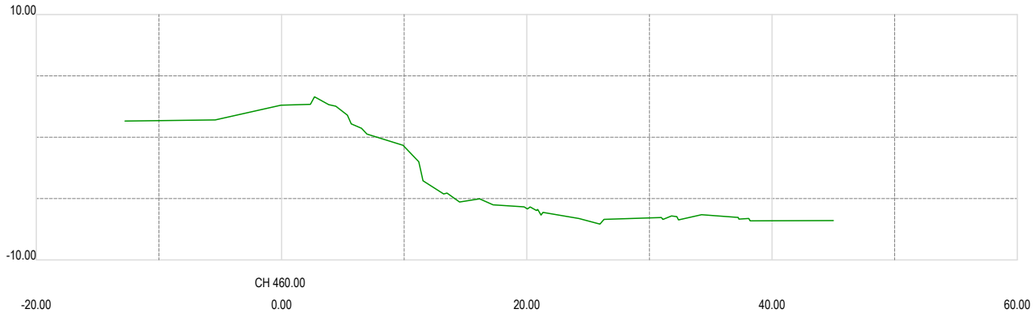
REV0

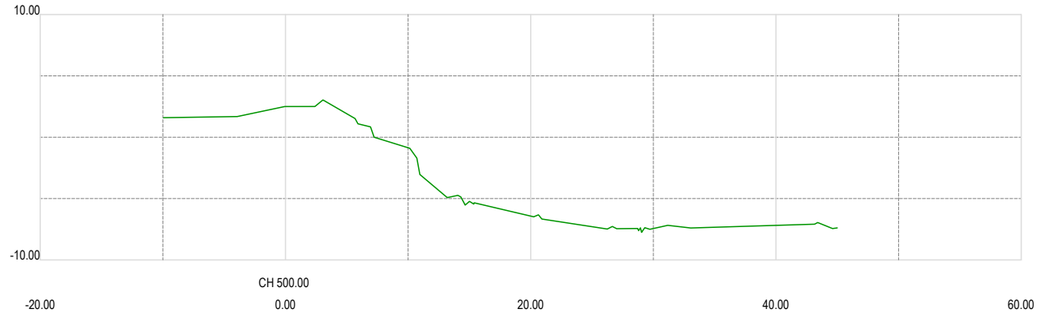


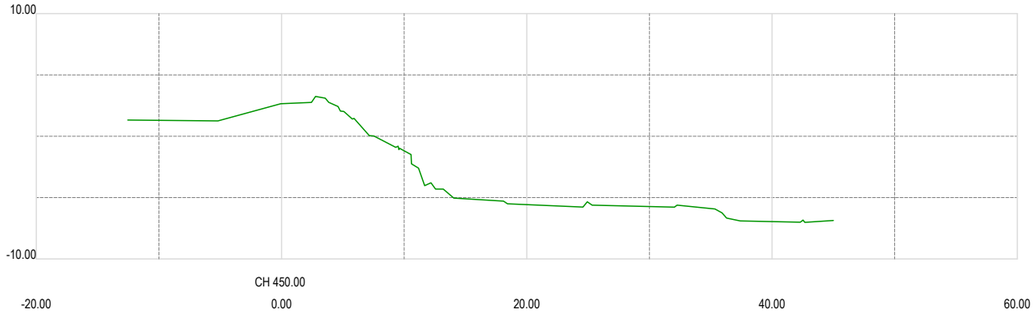


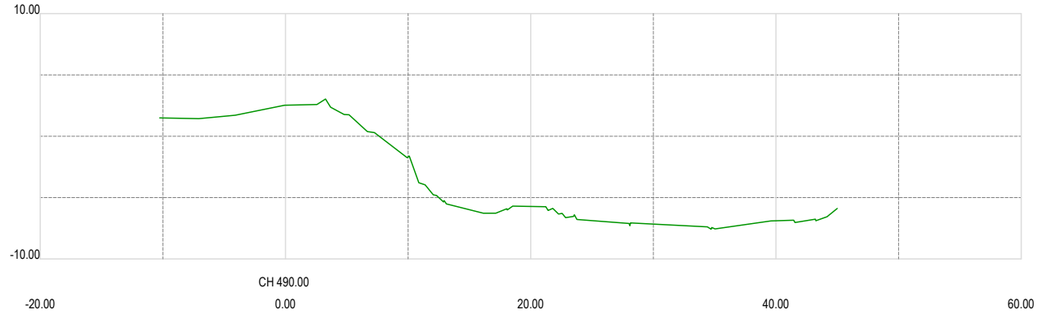












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REV	DATE	DESCRIPTION
0	29/07/2021	ORIGINAL ISSUE

SCALE

Scale Horizontal 1:500 Vertical 1:500

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JOB No.	21118A
SURVEYED BY	SB
CHECKED BY	TS
DATE OF SURVEY	29/07/2021
Vz DATUM	AHD71
ORIGIN	SSM10544
Hz DATUM	MGA56 (1994)

ORIGINAL SHEET SIZE A3

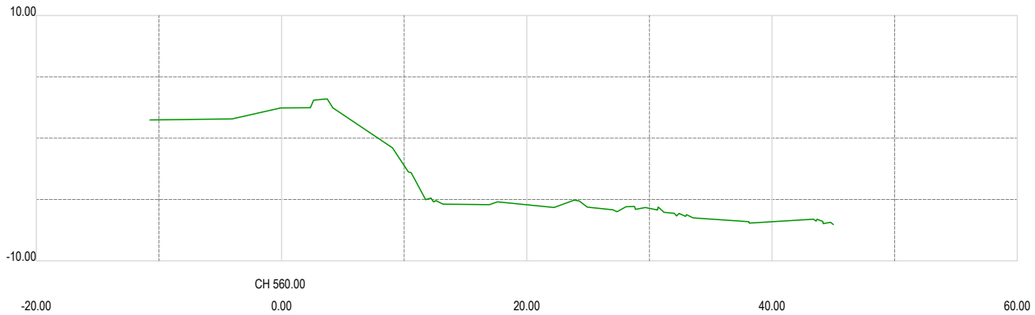
TITLE: CROSS-SECTIONS CH450-520

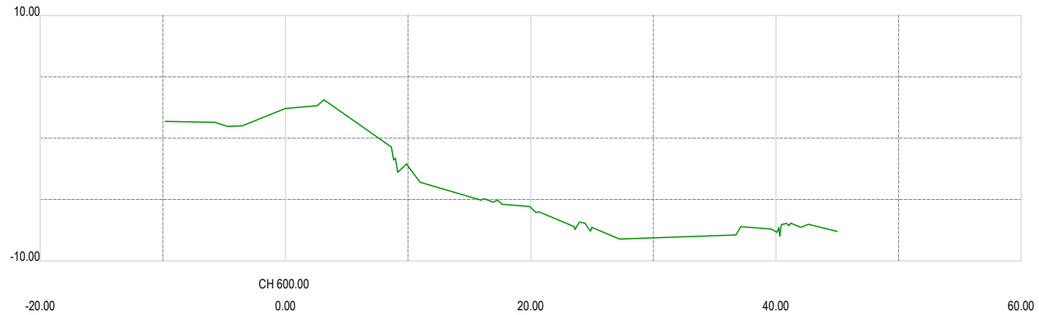
SITE: PORT MACQUARIE BREAK WALL, PORT MACQUARIE, NSW 2444

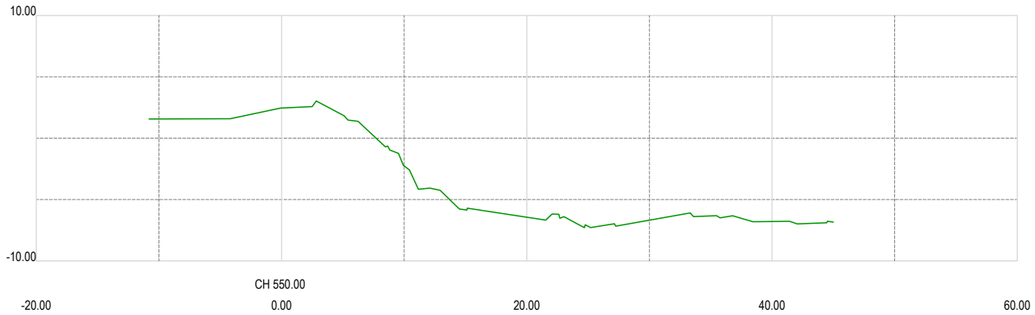
CLIENT: TRANSPORT NSW

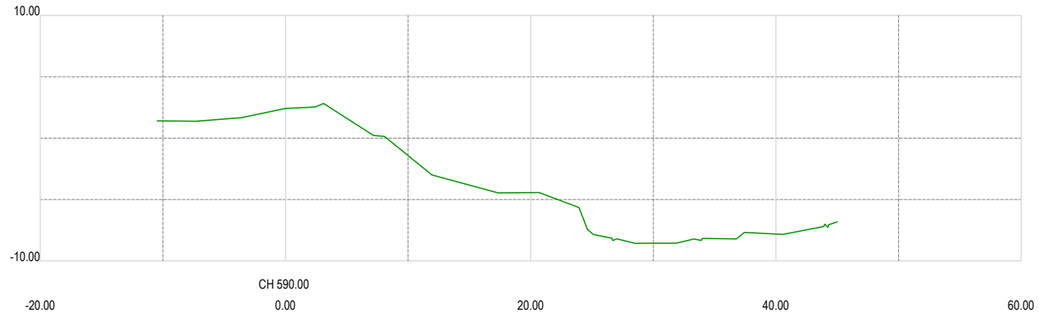
DWG No: 21118A.DWG

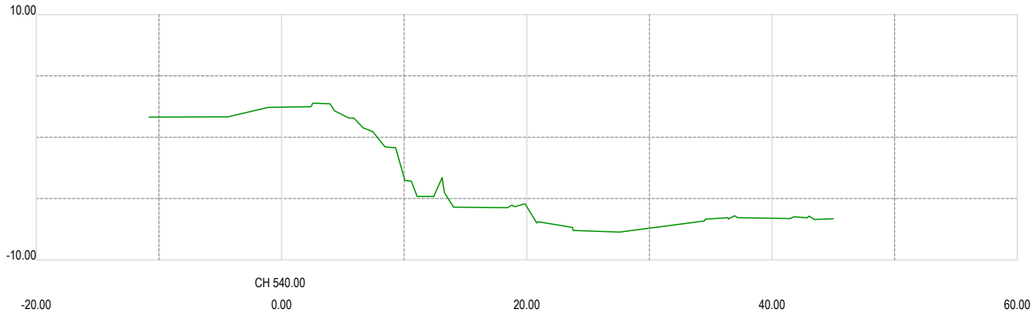
REV0

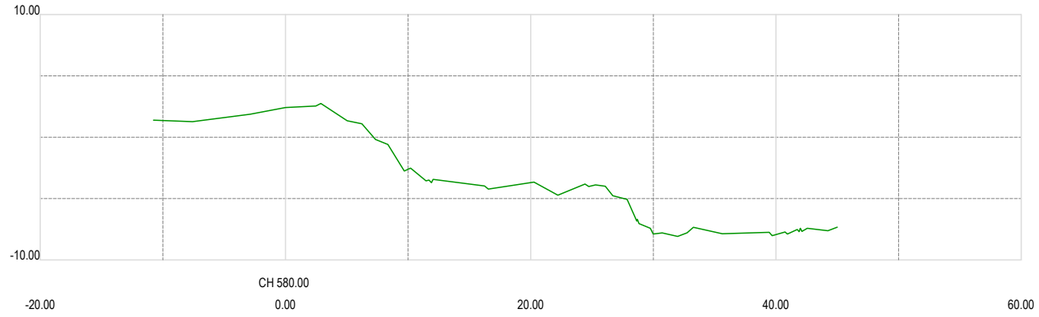


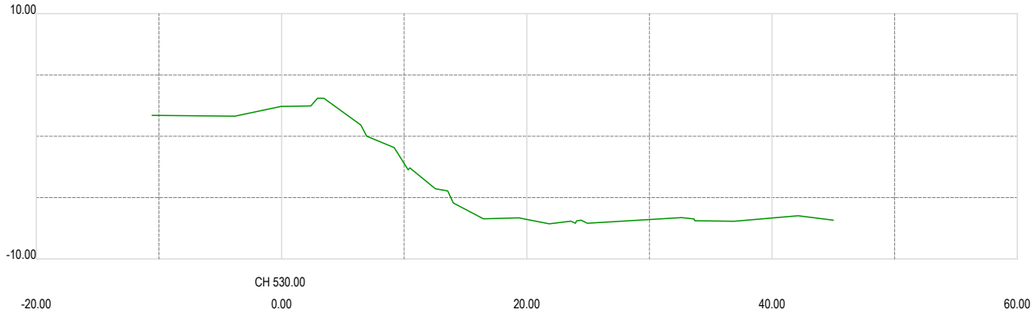


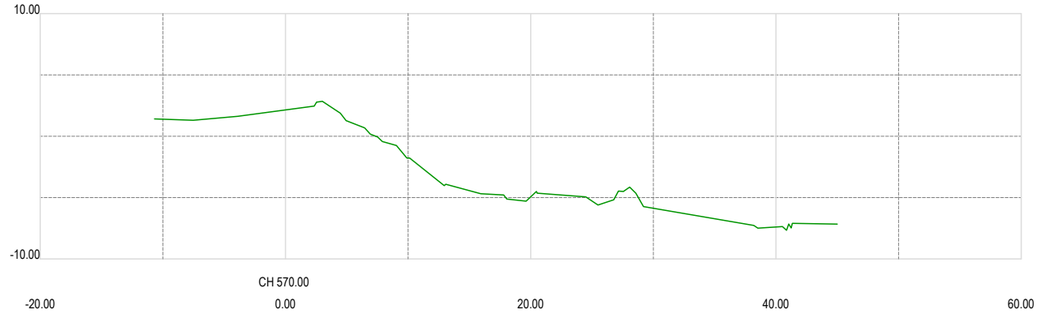












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REV	DATE	DESCRIPTION
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SCALE

Scale Horizontal 1:500 Vertical 1:500

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JOB No.	21118A
SURVEYED BY	SB
CHECKED BY	TS
DATE OF SURVEY	29/07/2021
Vz DATUM	AHD71
ORIGIN	SSM10544
Hz DATUM	MGA56 (1994)

ORIGINAL SHEET SIZE A3

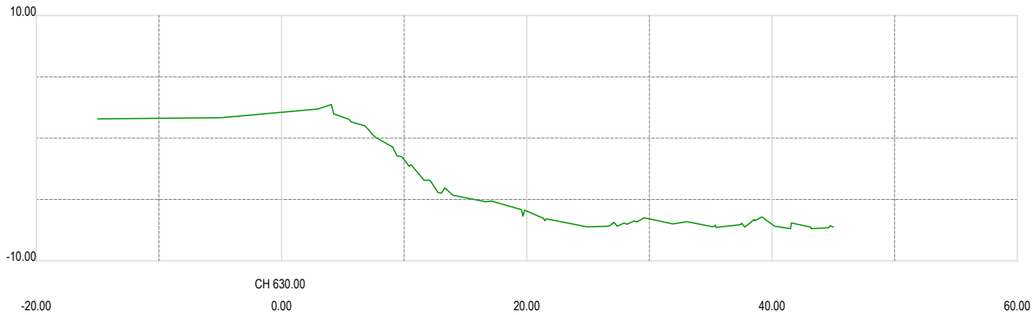
TITLE: CROSS SECTIONS CH530-600

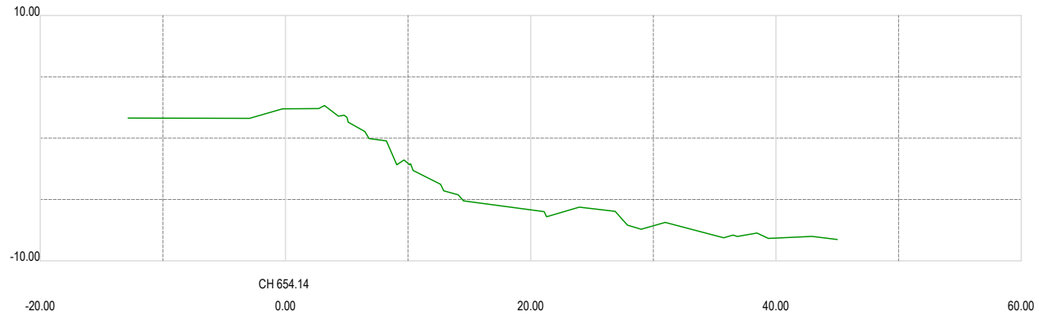
SITE: PORT MACQUARIE BREAK WALL, PORT MACQUARIE, NSW 2444

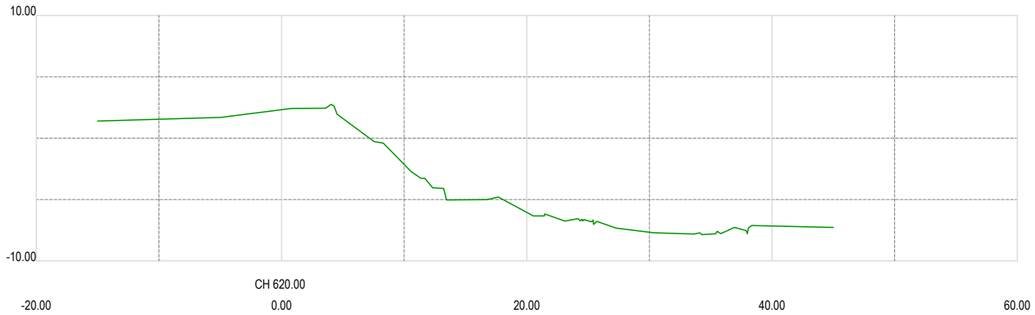
CLIENT: TRANSPORT NSW

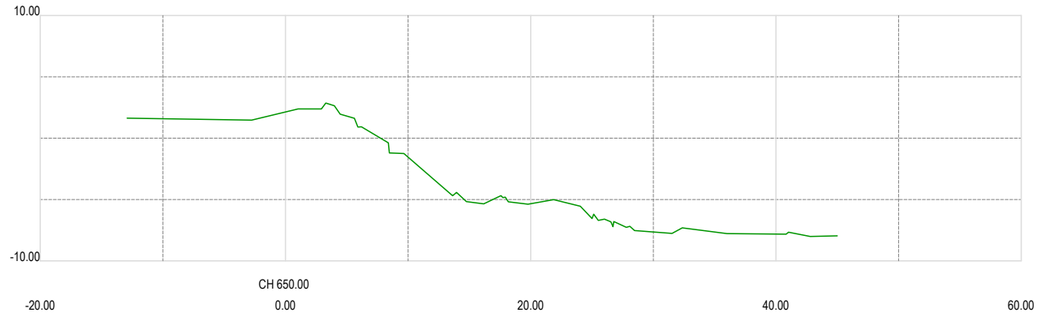
DWG No: 21118A.DWG

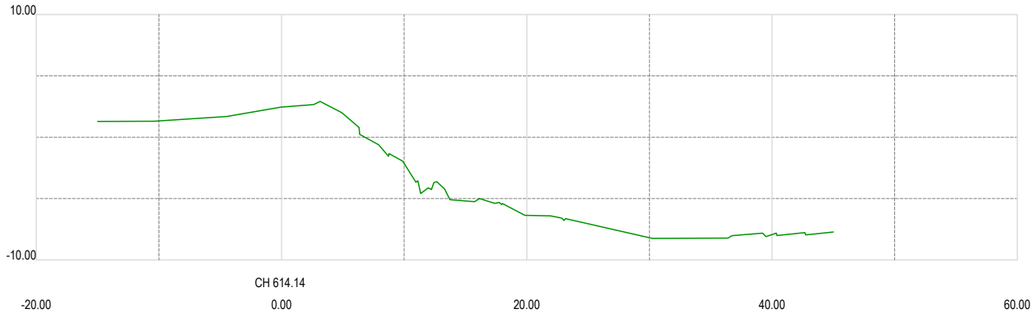
REV0

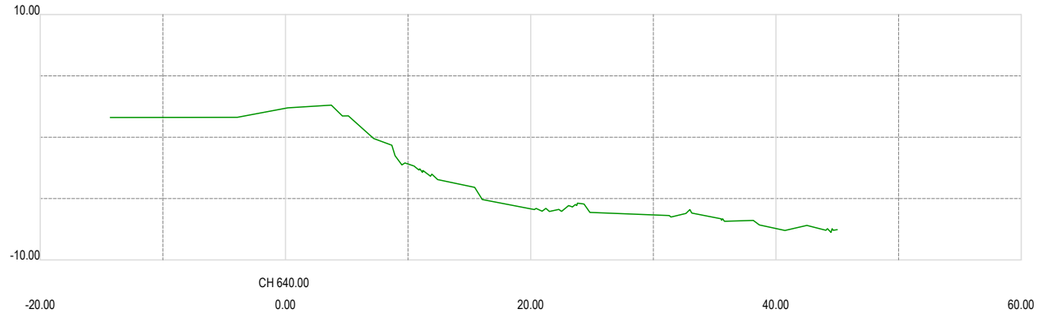


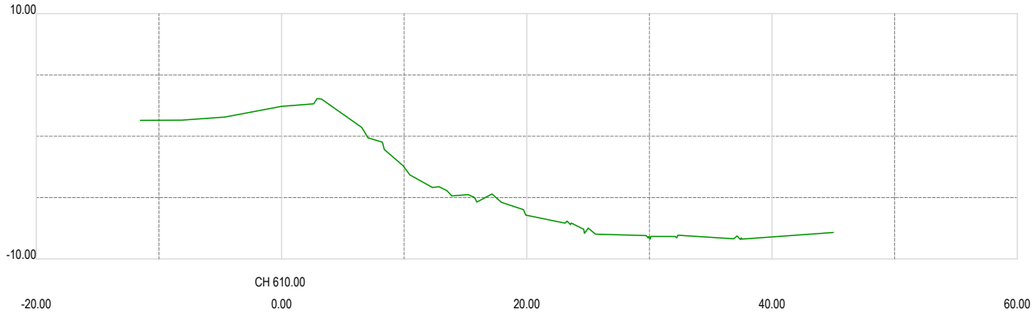


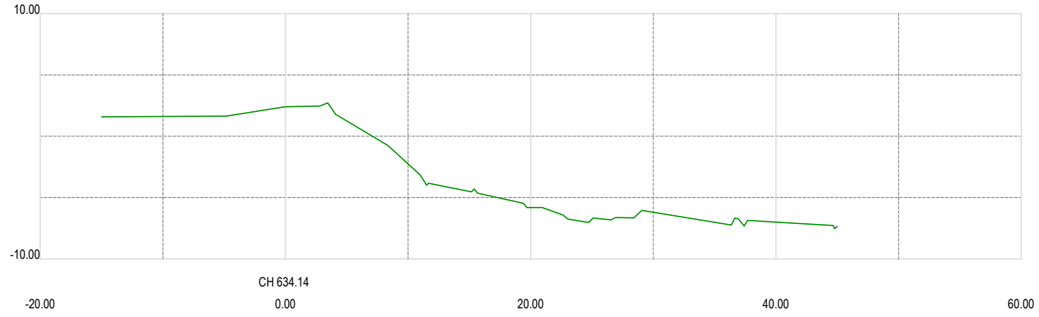












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SCALE

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JOB No.	21118A
SURVEYED BY	SB
CHECKED BY	TS
DATE OF SURVEY	29/07/2021
Vz DATUM	AHD71
ORIGIN	SSM10544
Hz DATUM	MGA56 (1994)

ORIGINAL SHEET SIZE A3

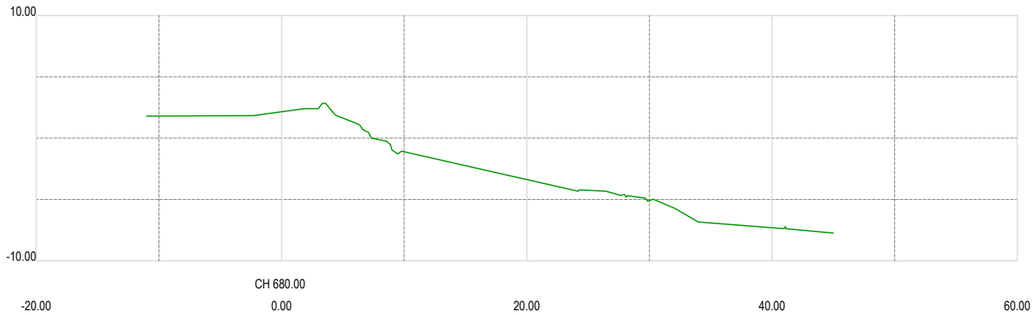
TITLE: CROSS SECTIONS CH610-654

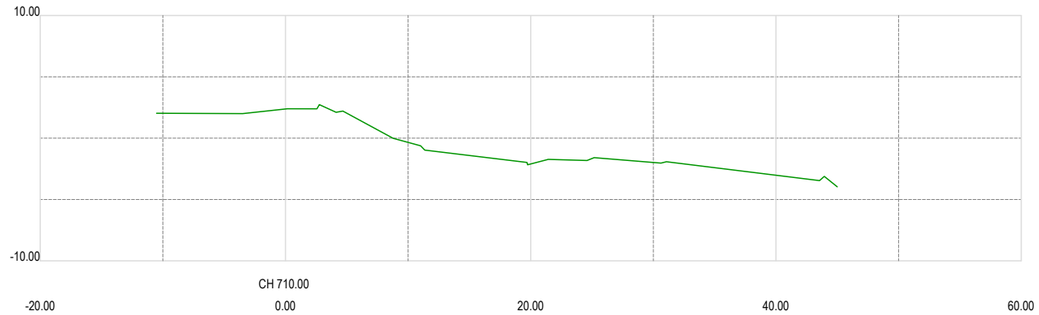
SITE: PORT MACQUARIE BREAK WALL, PORT MACQUARIE, NSW 2444

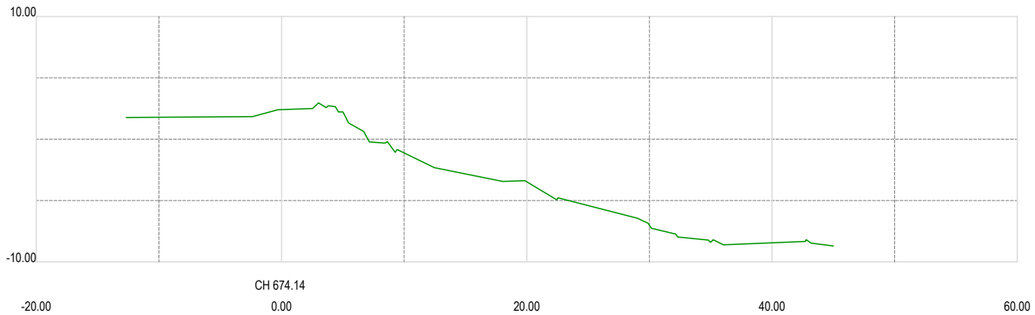
CLIENT: TRANSPORT NSW

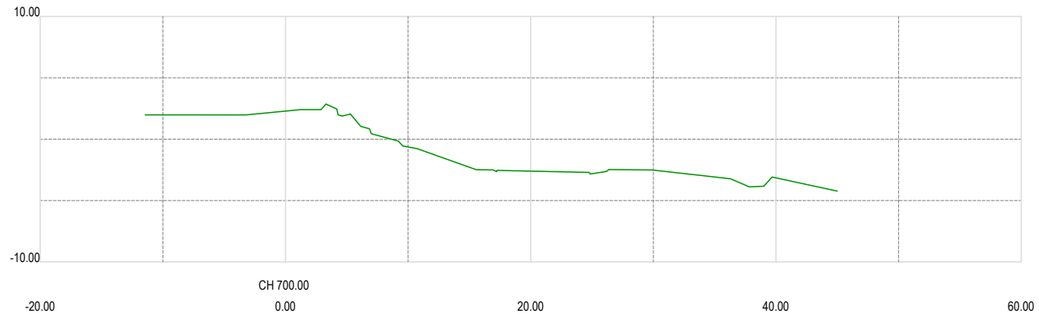
DWG No: 21118A.DWG

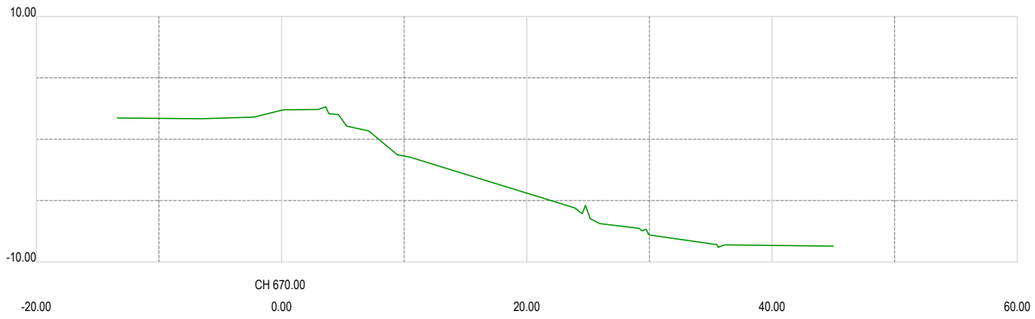
REV0

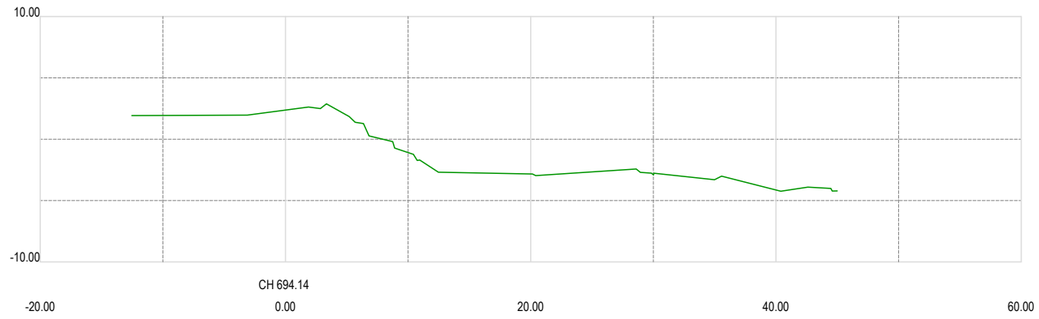


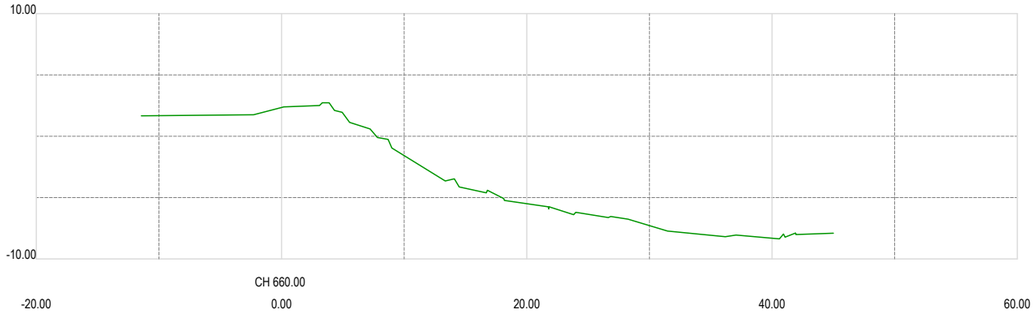


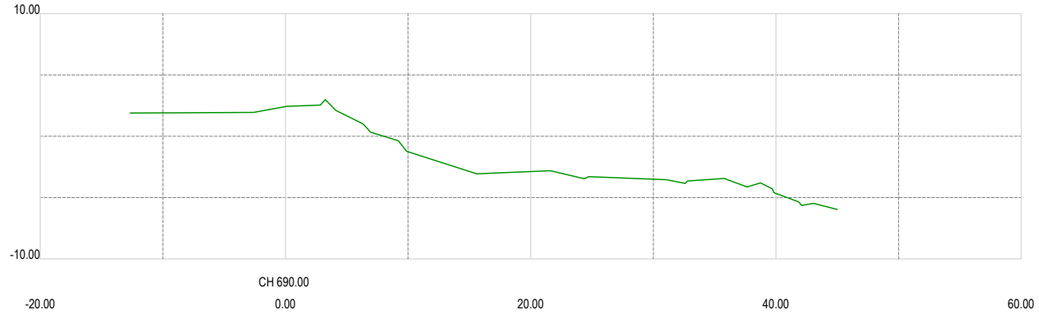












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REV	DATE	DESCRIPTION
0	29/07/2021	ORIGINAL ISSUE

SCALE

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JOB No.	21118A
SURVEYED BY	SB
CHECKED BY	TS
DATE OF SURVEY	29/07/2021
Vz DATUM	AHD71
ORIGIN	SSM10544
Hz DATUM	MGA56 (1994)

ORIGINAL SHEET SIZE A3

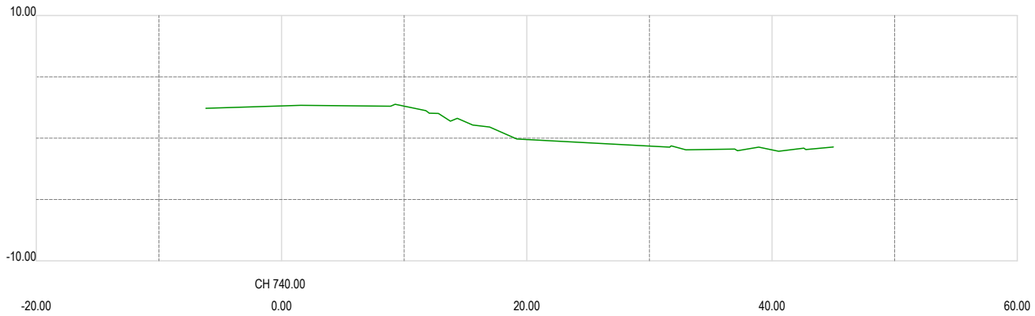
TITLE: CROSS-SECTIONS CH660-710

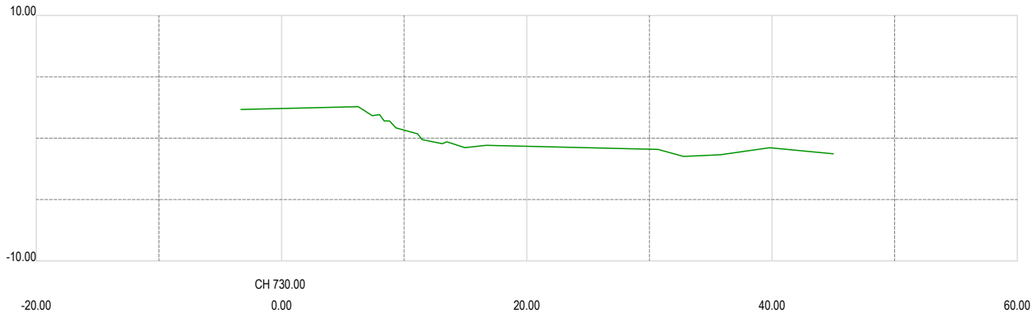
SITE: PORT MACQUARIE BREAK WALL, PORT MACQUARIE, NSW 2444

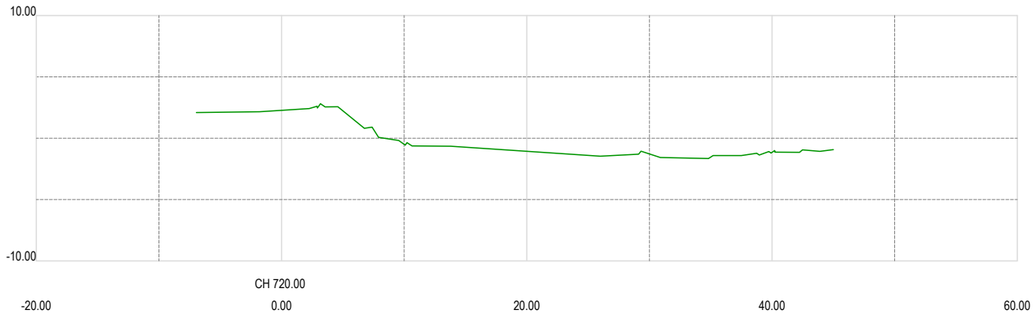
CLIENT: TRANSPORT NSW

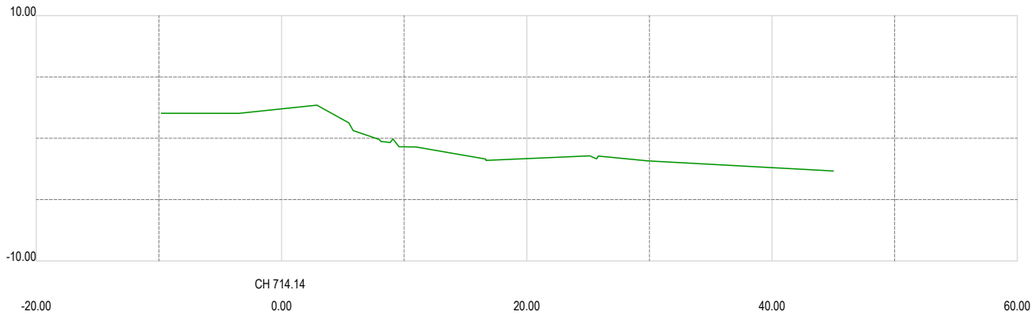
DWG No: 21118A.DWG

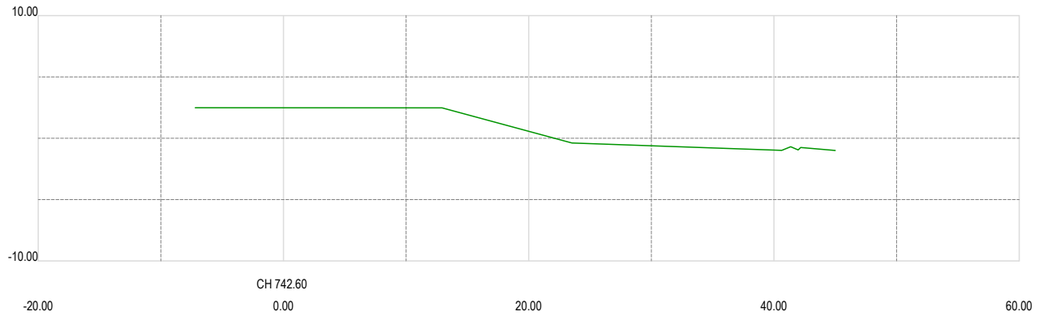
REV0











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REV	DATE	DESCRIPTION
0	29/07/2021	ORIGINAL ISSUE

SCALE

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JOB No.	21118A
SURVEYED BY	SB
CHECKED BY	TS
DATE OF SURVEY	29/07/2021
Vz DATUM	AHD71
ORIGIN	SSM10544
Hz DATUM	MGA56 (1994)

ORIGINAL SHEET SIZE A3

TITLE: CROSS-SECTIONS CH714-742

SITE: PORT MACQUARIE BREAK WALL, PORT MACQUARIE, NSW 2444

CLIENT: TRANSPORT NSW

DWG No: 21118A.DWG SHEET 16 OF 16 REV0