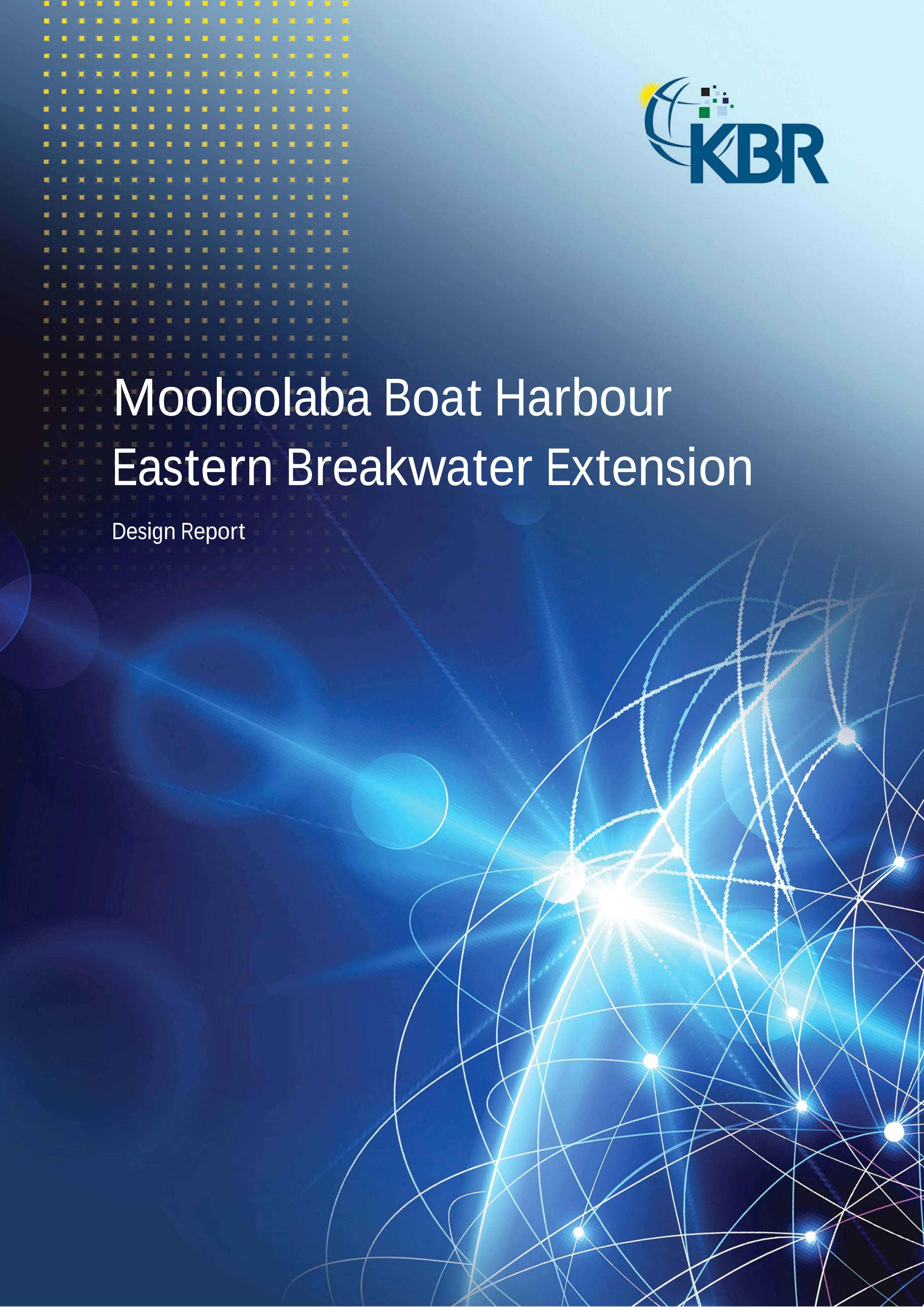

Mooloolaba Boat Harbour Eastern Breakwater Extension

Design Report

Mooloolaba Boat Harbour Eastern Breakwater Extension

Design Report

Prepared for:
DEPARTMENT OF TRANSPORT AND MAIN ROADS

Prepared by:
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BEJ952-TD-ST-REP-0002 Rev 0

Limitations Statement

The sole purpose of this report and the associated services performed by Kellogg Brown & Root Pty Ltd (KBR) is to describe the design of the Mooloolaba Breakwater Extension works in accordance with the scope of services set out in the contract between KBR and Department of Transport and Main Roads (‘the Client’). That scope of services was defined by the requests of the Client, by the time and budgetary constraints imposed by the Client, and by the availability of access to the site.

KBR derived the data in this report primarily from visual inspections, examination of records in the public domain, interviews with individuals with information about the site, supplied data and a limited amount of sub-surface explorations made on the dates indicated. The passage of time, manifestation of latent conditions or impacts of future events may require further exploration at the site and subsequent data analysis, and re-evaluation of the findings, observations and conclusions expressed in this report.

In preparing this report, KBR has relied upon and presumed accurate certain information (or absence thereof) relative to the site provided by government officials and authorities, the Client and others identified herein. Except as otherwise stated in the report, KBR has not attempted to verify the accuracy or completeness of any such information.

The findings, observations and conclusions expressed by KBR in this report are not, and should not be considered, an opinion concerning areas outside the scope of services noted including the condition and upgrade of existing breakwater components. No warranty or guarantee, whether express or implied, is made with respect to the data reported or to the findings, observations and conclusions expressed in this report. Further, such data, findings, observations and conclusions are based solely upon site conditions, information and drawings supplied by the Client in existence at the time of the investigation.

This report has been prepared on behalf of and for the exclusive use of the Client, and is subject to and issued in connection with the provisions of the agreement between KBR and the Client. KBR accepts no liability or responsibility whatsoever for or in respect of any use of or reliance upon this report by any third party..

Revision History

Revision	Date	Comment	Signatures			
			Originated by	Checked by	Technical Approval	Project Approval
A	16/03/21	Issued for Client Review	personal information			
0	16/12/21	Issued for Use				

Contents

1 Introduction

1.1 PROJECT OBJECTIVES

This project seeks to improve boating safety by enhancing the condition of the Mooloolah River Entrance. The Mooloolah River Entrance will benefit from an eastern breakwater extension to manage shoaling of the Mooloolah River Entrance caused by northwards longshore sand drift around Point Cartwright. By reducing the channel sedimentation due to increased sand trapping behind the extended breakwater, the frequency of maintenance dredging works required to maintain safe boating conditions is reduced.

1.2 COMMISSION

Kellogg Brown & Root Pty Ltd (KBR) has been commissioned by the Queensland Department of Transport and Main Roads (TMR) to design and obtain the necessary approvals for the extension of the Eastern Mooloolah River Entrance Breakwater at Mooloolaba Boat Harbour (The Project).

1.3 REPORT OBJECTIVES

The objective of this report is to document the design process and outcomes. The report should be read in conjunction with the Basis of Design report (refer BEJ952-TD-MN-DBA-0001), Safety in Design register (Appendix C), Constructability Report (BEJ952-TD-ST-REP-0002) and the Concrete Armour Unit Review technical memorandum (refer BEJ952-TD-CV-TCN-0002).

2 Scope

2.1 SCOPE OF WORK

The works comprise a 60m extension to the existing 100m long Mooloolah River entrance eastern breakwater. A 10m long demolition of the existing breakwater head is included to create the joint between the new and existing breakwaters. The purpose of these works is to provide increased protection from entrance channel shoaling caused by northwards longshore sand drift around Point Cartwright, Buddina.

2.2 SCOPE OF SERVICES

The scope of the design work is the provision of detailed design documentation in accordance with KBR's proposal (Document 6BJ722-01 Rev. 0) and BEJ952-B1-S004.

The project includes the following scope of design services:

- Review the currently available data and studies (including metocean climate, geotechnical conditions and sediment transport)
- Review armour rock and concrete armour unit availability
- Design of the eastern entrance breakwater extension, including the transition from existing to new. Repair and / or upgrade works to the existing breakwater is excluded.
- Design of heavy-duty concrete pathway along the crest of the extended breakwater
- Relocation of the existing navigation marker to the head of the breakwater extension
- Provision of documentation for tendering including: Design Basis Report, Design Report, Design Drawings, Technical Specifications and Bill of Quantities
- Preparation and lodgement of all necessary Development Approval (DA) applications including Prescribed Tidal Works and Owners Consent.
- Provision of a Concept Design stage Class 4 Capital Cost Estimate in accordance with the AACE International Cost Estimate Classification System for feasibility assessment.
- Provision of a Pre-Tender Class 2 Capital Cost Estimate in accordance with the AACE International Cost Estimate Classification System at detailed design phase.
- RPEQ certification of the design.

2.3 OUT OF SCOPE ITEMS

Project scope elements which are not currently included in KBR's commissioned Scope of Services include, but are not limited to:

- Dredging and navigational channel design
- Rock sourcing study
- Design of lighting and services
- Repair works and upgrades to the existing breakwater

- Stakeholder consultation
- Preparation of Contract Documentation and Commercial Conditions.

3 Extension Design

3.1 DESIGN BASIS

The design basis for the breakwater extension is in the KBR Design Basis Report (Ref. BEJ952-TD-ST-DBA-001). The basis for the adopted design wave heights as presented in the Design Basis Report is summarised below in Section 3.1.1. The basis for the adopted materials is also summarised in Section 3.1.2.

3.1.1 Waves and water levels

A 50-year design life to the year 2070 has been adopted as per TMR requirements which is consistent with AS 4997-2005. As advised by TMR, the breakwater shall be designed to withstand a 1 in 200 year average return interval (ARI) design event with less than 5% armour damage.

Table 3.1 Design wave properties and storm surge levels at the head of the proposed 60 m breakwater extension from BMT (2019).

ARI Event (years)	*Hs (m) (Adjusted for SLR)*	T_p (s)	Independent Tide Plus Surge Plus sea level rise (SLR) in the year 2070 (mAHD)
1	2.64	10.7	1.52
200	3.54	12.7	1.84

The design wave height (Hs) at the toe of the head of the proposed 60 m breakwater extension will be used conservatively for the design of the entire breakwater extension back to its junction with the existing breakwater.

3.2 DESIGN STUDIES

Several design studies have been carried out to inform the design and construction methodology to date. These studies are briefly discussed in the following sub-sections.

3.2.1 Physical model testing

Preliminary physical model testing was undertaken to confirm the design cross-section for the breakwater extension, including an assessment of overall stability under wave attack, together with transmitted wave energy (both as overtopping and transmission through the breakwater). Model testing was completed at the Queensland Government Hydraulics Laboratory (QGHL) with flume testing at a 1:41 non-distorted scale.

The model setup, wave calibration, and testing results for this model testing campaign are summarised in QGHL's memo "Results for Mooloolaba Breakwater Physical Modelling Project" which is provided in Appendix D.

Physical modelling of the breakwater revealed that a median 6-tonne rock weight was not suitable for the breakwater extension due to an unacceptable level of damage during the design event, sustained primarily at the transition zone between the new and existing breakwater, and on the roundhead (as demonstrated by the 'before' and 'after' shown in Error! Reference source not found.Figure 3.1). It was concluded that a larger primary armour unit was needed to meet the damage criterion of <5%.

Figure 3.1 Plan view of physical model (Before – Left, After – Right)

During the initial stages of the Mooloolaba breakwater extension design, rock sourcing investigations by TMR and KBR were unable to identify a suitable local quarry able to supply rock sizes in excess of 6 tonnes. It is for this reason that rocks in excess of 6-tonnes were not physically modelled at QGHL.

3.2.2 Constructability Review and rock sourcing

An initial constructability review investigated 5 potential methods of construction given the access constraints of the site.

Since the construction of the eastern breakwater in 1966, there have been advancements in the design standards and construction methods applied to rock armoured coastal structures. There has also been a significant land use increase, changes to site access for the construction vehicles, and increases in the marine traffic at the Mooloolah River.

There are also currently no identified construction barge loading/unloading facilities (i.e. heavy-duty boat ramps or wharves) in the broader Mooloolah River region to facilitate transfer of materials and equipment to the site.

Based on a review of the existing available information pertaining to the site, in combination with an inspection of the site, and inspection of material sources carried out on 15 March 2019, five potential construction methodologies for the Mooloolah River eastern breakwater extension were reviewed:

- Option 1 – Land-based construction via the existing Mooloolah River walking track and the Buddina road network
- Option 2 – Land-based construction via Point Cartwright Beach and the Buddina road network
- Option 3 – Marine-based construction via the Mooloolah River

- Option 4 – Land-based construction via construction of a temporary materials offloading facility (MOF) on Point Cartwright beach which is used for marine-based supply.
- Option 5 - Combined option (land-based core construction with marine-based armour construction)

For the purposes of cost estimation it is currently assumed that materials will be barged from the Port of Brisbane to the site, for stockpiling at Point Cartwright and land-based placement. Further details of the construction sequencing are provided in Section 6.

3.2.3 Armour unit options review

Precast concrete armour units can be a cost-effective option where suitable rock of the required size and quality is unavailable, or where quarry lead times may exceed the project requirement.

Five concrete armour unit types were reviewed by KBR based on their performance characteristics, availability and suitability for the project. These included Tetrapod, Xbloc, Core-loc, Antifer and Hanbar.

The Hanbar concrete unit was selected by TMR as the preferred alternative to rock armour units. It was developed by NSW Public Works and has an extensive track record in NSW (e.g. Coffs Harbour), plus laboratory testing data.

Alternative construction methods used for the procurement of concrete armour units have also been discussed within BEJ952-TD-CV-TCN-0002 Concrete Armour Unit Review. This included:

- Casting of concrete units,
- Transport to site,
- Placement,
- and construction timing considerations.

Additionally, eco-friendly additions to the extension were investigated such as modular living seawalls, reef balls and ECOoncrete additives should TMR wish to consider these further.

4 Breakwater Design

4.1 GENERAL ARRANGEMENT

Following an investigation by BMT WBM (2014), *Investigation of Capital Works Options for the Management of Shoaling at the Mooloolah River Entrance,* a breakwater extension length of 60m was selected by TMR as the most cost-effective length to reduce siltation and subsequently maintenance dredging within the Mooloolah River entrance channel.

The existing breakwater head will be partly demolished to allow a joint to be formed with the proposed breakwater extension. A 10m joint section is anticipated. Over this 10m transition, the breakwater footprint will widen from the existing to the proposed Hanbar armoured extension as described in Appendix A.

4.2 CROSS-SECTION DESIGN

The breakwater extension cross-section consists of two layers of Hanbar units as the primary armour, a double layer of secondary rock armour and a graded rock filter/underlayer (i.e. 'tertiary' filter layer). No geotextile cloth filter layer was proposed due to the difficulty of accurately placing it into the constantly active Mooloolaba wave climate. A course-grained core material is specified using with Terzaghi filter criteria to ensure no loss of fines through the filters and armour.

The proposed breakwater extension layers are listed in Table 4.1 below.

Table 4.1 Breakwater extension new materials

Layer	Description	Target D_{50} (m), M_{50} (t)
Primary Armour	Dual layer 'interlocking' precast concrete Hanbar units	1.9m, 7t
Secondary Layer	Heavy armour stone	0.8m, 1.3t
Filter Layer	Light armour stone	0.34m, 0.11t
Core	Core material & underlayer	[-], 11.8kg

The 7.0t Hanbar primary armour unit has been sized using the Hudson (1961) equation for 0-5% damage criteria during the ultimate 200 year ARI design event.

The damage coefficient adopted is consistent with values compiled by the Water Research Laboratory (WRL) of the University of New South Wales (UNSW) (2005) from physical modelling studies testing the standard upright Hanbar unit placement method (provided in in Figure 4.1). A desktop review of several Hanbar unit studies (I, Jayewardene, 2018 and G, Russell, 2013) suggest a Hudson damage coefficient (Kd) of 5-7 is a conservative estimate for the standard upright placement method with 0-5% damage.

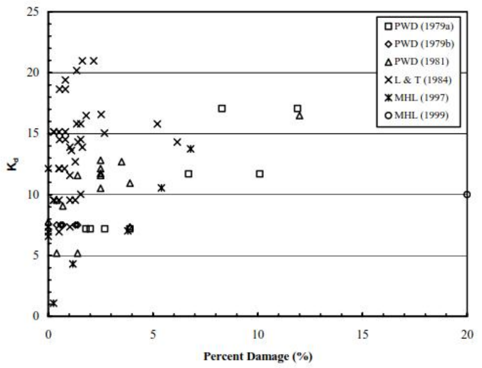


Figure 4.1 Hanbar unit damage coefficient

WLR have also reported on an improved placement method of the Hanbar unit, shown in Figure 4.2 for an ‘interlocking placement method’.

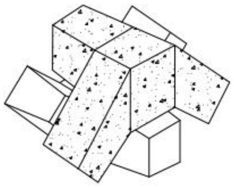


Figure 4.2 Hanbar unit interlocking placement method

The interlocking placement method uses two lifting points as opposed to the standard upright placement method that uses only one lifting point. The standard upright placement uses lifting method 1, while the new interlocking placement uses both lifting method 2 (top layer) and 3 (bottom layer). See Figure 4.3 for Hanbar unit lifting methods.

The ‘interlocking is placement method’ is applied for this design due to the improved interlocking compared with the standard upright hanbar placement.

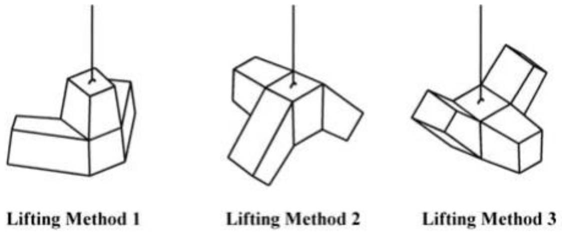

Lifting Method 1
Lifting Method 2
Lifting Method 3

Figure 4.3 Lifting methods for Hanbar unit

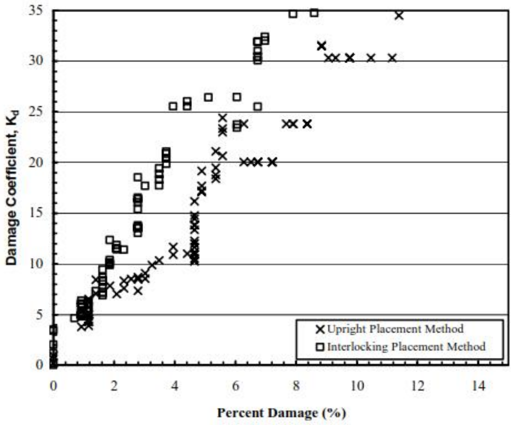


Figure 4.4 Interlocking Hanbar unit damage coefficient modelling results (WRL, 2005)

Physical modelling of Hanbar units is recommended consistent with recommendations by WRL (2005), to confirm the structure’s hydraulic performance and resistance to damage, however it is understood that the time of writing further physical model testing is outside the scope of this project. Based on the findings of limited previous Hanbar unit studies, a conservative damage coefficient of Kd=10 for interlocking Hanbar units is therefore adopted for the design of the Mooloolaba breakwater extension (Taken from Figure 4.4) assuming 0 – 5% damage. Physical modelling will provide more accurate verification of the design inputs and structure’s hydraulic response.

4.2.1 Underlayers and Core

The secondary rock armour layer has been sized based on the findings of WRL (2005) to properly support a dual layer of Hanbar units without being drawn through the gaps between the units. Coarse filter and core material are specified in accordance with Terzaghi filter criteria. A typical cross section of the breakwater extension is provided in Appendix A.

The underlayer rocks in the breakwater extension have been graded using BS EN 13383-1 as described in Table 4.2.

Table 4.2 Underlayer rock grading as per BS EN 13383-1

Material Specification	Target M_{50} (kg) **	ELL < 5%* (kg)**	NLL <10% (kg)**	NUL >70% (kg)**	EUL >97% (kg)**	M_{50min} (kg)	M_{50max} (kg)	D_{n50} (m)
Secondary layer: HMA$_{1000/1300}$ (non-standard)	1290	350	640	1840	2800	1160	1470	0.79
Tertiary/Filter layer: LMA$_{80/120}$	108	15	40	200	300	90	140	0.34
Core: CD$_{0.07/3.2}$	11.8	22	45	125	180			0.086

**Note - for light gradings (NLL < 300 kg) the ELL mass limit indicated is limited to <2% (not <5%)*

*** Note – for Core CD grading the specification is given in millimetres (not kg)*

Sand-bag Core Alternative

At the time of writing TMR has indicated that they may wish to pack dredge material into geotextile sandbags for use as the breakwater extension core in lieu of a conventional graded rock core. This option reduces the trucking of core material to site by using existing on-site sources of sand. KBR has discussed the feasibility of this concept with Australian geofabric bag suppliers .

A GB600 heavy duty marine geofabric bag with lifting straps manufactured by TenCate has been nominated by it's supplier, Geofabrics Australia. It's product data sheet can be found in Appendix F.

The GB600 geofabric bag has a capacity of 4.0m^3 and can hold 7.0 t of sand. During preliminary discussions contractors have indicated that they can fill 2-3 bags per hour at a single workstation. A bag of this weight could be placed by a long arm excavator.

Further information from the supplier confirming the constructability, onsite requirements and guidance on the pricing for this option can be found in the Mooloolaba Constructability Report (BEJ952-TD-ST-REP-0001).

4.2.2 Foundation

The footprint of the proposed breakwater extension depends on the depth to bedrock founding level.

The breakwater should be founded on the underlying bedrock material to limit settlement. Bed rock levels were investigated by FRC Environmental as documented in "*Mooloolah River Entrance Sand Depth Survey*" provided in Appendix E. The objective of the work was to estimate the bed rock level and depth of overlying sediment.

FRC Environmental reported that the bedrock varied from 4.0 to 4.7 m below LAT under the breakwater extension footprint. Therefore breakwater extension design assumes founding on bedrock at a level of RL-4.7m LAT consistent with survey Pt 5 in Appendix E.

Dredging work will be required to remove the sand prior to construction of the breakwater extension.

4.2.3 Toe Details

The breakwater extension toe detail provides protection against scour, undermining, revetment sliding, and slope slumping. The nominated toe detail is based on recommendations in CIRIA Rock Manual (2012).

Rock toe protection in front of the Hanbar concrete armour units at the base of the structure is proposed. Use of rock at the toe is generally preferred over concrete armour units to avoid structural overload of the units (i.e. the mass of the slope crushing the toe units). Additionally, the Hanbar type of concrete armour unit rely on downslope interlocking for stability in addition to their weight. Therefore they don't perform inadequately on a flat base.

The toe design is a conventional type founded on an impermeable layer (i.e. rock) as shown Figure 4.5, however no geotextile is required. The toe will be founded on an impermeable bed rock surface after removal of the overlying sand by the dredge. This design will form a berm on the toe with a 2.8m width. With these dimensions it is intended that the toe will comprise of approximately 3 rocks in height.

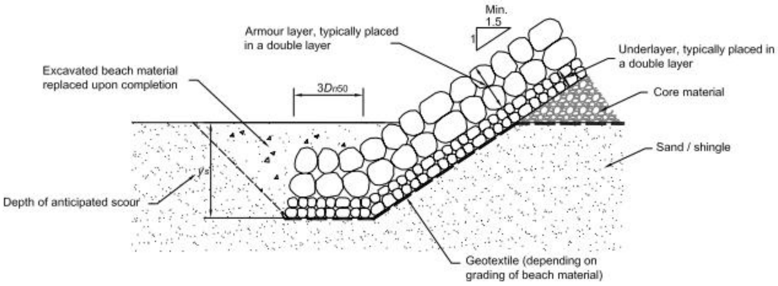


Figure 4.5 Toe design of breakwater extension (Typical toe detail per Figure 6.62 CUR Rock Manual)

It is intended that over time the toe will be buried by natural build-up of sand.

4.2.4 Crest Details

The crest design comprises of Hanbar units, a heavy-duty pavement topping slab and a cement stabilised foundation.

The pathway crown will slope upwards from RL+5.2mLAT at the existing to RL+5.9mLAT at the head, which give a slope of 1V:85H over 60m which is suitable for pedestrians and and wheelchairs. The path with have a crown plus cross falls of 0.5% each way. The edge of the path will be trimmed with a 150mm high kerbs to prevent (e.g.) prams rolling off the path, with drainage into the Hanbars through gaps in the kerbs.

Upright Hanbar units will obscure views when placed along the crest of the structure; an example of this is Coffs Harbour. However, it is proposed that the Mooloolaba extension will have the Hanbar units placed as interlocking units, offering improved interlocking, as well as improved visual amenity.

4.2.5 Overtopping

Overtopping flow rates due to wave action have been assessed using the methods described in EurOtop (2016), for the 200 year ARI design event and 1 year ARI ambient conditions based on a RL+4.9mAHD (RL+5.9mLAT) design crest elevation. The predicted overtopping rates for the 1 year and 200 year ARI events are provided in Table 4.3.

Since the submission of the design basis report, TMR have advised KBR that the design functionality of the extension will no longer include safe pedestrian access during adverse weather events. KBR advised that during adverse weather conditions public access must be restricted by (e.g.) the use of warning signs.

Table 4.3 Overtopping discharge rates

Overtopping event	Estimated mean overtopping discharge (L/s/m)	Limiting criteria (L/s/m) EurOtop 2018	Commentary based on estimated mean overtopping discharge
1 year ARI	1.1	0.5 (Safe pedestrian access)	Not safe for pedestrians. Safe for vehicles CUR Rock Manual: Very dangerous for pedestrians.
200 year ARI	29.3	30 L/s/m (Structural damage)	Not safe for pedestrians or vehicles

Overtopping event	Estimated mean overtopping discharge (L/s/m)	Limiting criteria (L/s/m) EurOtop 2018	Commentary based on estimated mean overtopping discharge
			CUR Rock Manual: Damage if back slope not protected

Note: Overtopping rates are based upon Tetrapod units’ roughness factor of 0.38 (EurOtop 2018). No guidance offered by EurOtop for Hanbar unist.

5 Safety in Design

5.1 OBJECTIVE

Safety in Design has been applied via the integration of hazard identification and risk assessments in the design process with the aim of elimination or minimising health and safety risks, so far as is reasonably practicable (SFAIRP), throughout the life of the structure.

The objective of the assessment is to identify potential hazards and associated risks in the workplace to provide a safe workplace for Contractors, employees, visitors and the public.

The *Australian Work Health and Safety Act 2011* and Regulations require that persons who have a duty to ensure health and safety to ‘manage risks’ by eliminating health and safety risks SFAIRP, and if it is not practical to do so, to minimise those risks as low as reasonably practicable (ALARP).

5.2 HAZARD IDENTIFICATION AND RISK ASSESSMENT

The Hazard and Risk assessment has been performed to identify the hazards, assess the risk, and define the controls necessary to eliminate or mitigate the risk for the proposed work activities. These hazards have been included in an OH&S (Operational Health & Safety) Hazard and Risk Register in Appendix C.

An initial review of the hazards associated with the design, construction and operation of the facility has been provided to TMR on 12 July 2019 as an attachment to the Design Basis Report (BEJ955-TD-CV-DBA-0001) for review and discussion. The identified hazards and risks formed the basis for key project decisions in order to address hazards associated with access (particularly prevention of public access) and construction and operational machinery access constraints.

The attached OH&S Hazard and Risk Register has been reviewed following further development of the preliminary design and identifies design measures that have either:

- been implemented through the design process (‘closed’)
- identifies controls that are proposed to be included in the Contract Drawings and Technical Specification by detailed designers in future design phase (‘active’)
- identifies key recommendations identified relating to the requirements of TMR, the construction Contractor and the maintenance/emptying Contractor to consider during construction and through the operational life of the structure (‘active’).

A full description of the identified risks and currently identified treatment strategies are provided in the register in Appendix C.

It is assumed that further hazard and risk assessments will be carried out by designers, contractors and the Client throughout the life of the project to identify any additional risks and to refine mitigation measures.

5.3 KEY RESIDUAL SAFETY RISKS

Two key identified residual safety risks which shall be further considered by TMR are summarised as follows:

- Overtopping rates are considered to exceed the pedestrian safe criteria using EurOTop 2018. Consistent with TMR functional requirements outlined in BEJ952-TD-ST-0001 Design Basis, provisions for safe access for pedestrians is excluded. Public access should therefore be restricted to the breakwater during adverse weather conditions by (e.g) the use of warning signs.

- The breakwater extension will have a new paved pathway whilst the existing breakwater pathway will not be upgraded. It is outside of KBR’s scope to assess the condition of the existing pathway. KBR advises TMR that the existing breakwater pathway condition should be assessed and if needed, upgraded to suit.

The reader is referred to Appendix C for a complete summary of the identified risks and treatments.

6 Constructability

6.1.1 Contractor Correspondence

KBR and Marine Safety Queensland (MSQ) have sought constructability advice from contractors who are familiar with maritime construction works on the Sunshine Coast. Three contractors were contacted;

1. Hall Construction
2. Haslin
3. Marine Civil

These contractors provided advice on both land and marine based construction methodology. All contractors have advised that the best methodology for a balance between reducing cost and reducing adverse impacts on the local community will involve a combination of transporting materials via sea and construction from the land. Details of the constructability assessment and advice are documented within BEJ952-TD-MN-REP-0001 Constructability Report.

7 Cost Estimate Basis

A cost estimate has been prepared by KBR based on the typical cross-section design and quantities developed via AutoCAD.

This section of the report outlines the methodology used in estimating probable costs for the 60m extension, plus 10m demolition and rebuild of the existing breakwater at tie-in. Materials are assumed to be supplied from Brisbane and barged to site for stock piling on land as discussed in Section 6.

7.1 ESTIMATE BASIS

7.1.1 Estimate Purpose

The purpose of the estimate is to establish likely Capex construction costs for budget planning purposes. In support of this purpose, KBR has developed a preliminary Class 4 cost estimate to AACE International Estimate Guideline with a nominal accuracy range of -15% to +30% which is suitable for Study or Feasibility in accordance with AACE International Recommended Practice.

The estimate has been based on available market pricing from similar, non-identical projects, which provides benchmark pricing for key construction activities. Some rates have been factored to account for specific conditions at the site including access and material delivery.

Where costing information is unavailable, allowances have been included based on the best estimate of experienced employees.

7.1.2 Estimate Criteria

The basis for this estimate is as follows:

- Prepared to a non-binding, nominal accuracy of -15% to +30%
- Expressed in Australian dollars
- Developed excluding Goods and Services Tax (GST)
- This estimate is presented as of market conditions at January 2021 and excludes lifecycle costs.
- Works are escalated to 2022 (1 year allowance)

The limitations and assumptions associated with estimates of costs are detailed in Section 7.5.

7.2 SOURCE DOCUMENTS AND DRAWINGS

The estimates have been developed based on the scope of work documented in KBR's Design Basis document (BEJ952-TD-CV-DBA-0001 Rev 0) and BEJ952-B1-S004. Quantities and estimates have been developed using typical cross-section details from the sketch provided in Appendix A as well details on constructability presented in BEJ952-TD-MN-REP-0001 Constructability Report.

7.3 COSTED ITEMS

The following sections provide a summary of the items and quantities used to develop the overall cost estimate. The build-up of costs is provided in Appendix B.

7.3.1 Primary Concrete Armour and Underlayer Rock

Volumes have been estimated based on material take-offs from the typical section provided in Appendix A and validated from AutoCAD measurements. An allowance for removal and replacement of a 10m section at the interface of the extension is included, in addition to the 60m proposed breakwater upgrade section.

Costs include supply and installation of the primary concrete armour 7t hanbar units, with a secondary rock armour (under) layer to be 1.3t and 0.11t rock filter (tertiary) layer. Sizes and grades have been selected based on the ARI 200-year event under the design conditions (Refer Section 3.1 for design criteria).

It is proposed that the majority of the 10m section of rock for demolishing will be removed and graded for reuse in the extension.

Rock quantities have been estimated and are provided below in Table 7.1.

Table 7.1 Rock quantities

Description	Cubic meters	Total quantity, t*
Secondary Armour / Underlayer (Double Layer)	5,600	14,840
Tertiary Armour / Filter (Double Layer)	1,800	4,770
Imported Core material, granular	7,070	18,736
Secondary and recycled primary Armour to toe	3,700	9,805

*nominal 20% of the total underlayer qty. To be confirmed on site.

A bulk density of 1.96t/m3 is applied for primary and secondary armour in accordance with guidance from the *CUR Rock Manual (CIRIA C683), 2007* for double-layered densely placed armourstone.

Concrete armour unit quantities for the primary armour layer have been estimated and are provided below in Table 6.2.

Table 7.2 Concrete unit quantities

Description	Unit quantity	Volume, m3	Total quantity, t
Hanbar concrete unit, 7t	1,300	3,900	9,360

7.3.2 Dredging

Cost allowances for dredging works include, mobilisation/demonization and removal of 9,450m^3 unsuitable material from under the breakwater via a cutter suction dredge.

Dredge quantities are based on the latest available survey information to the bed rock level (approximately 1.5m depth). No allowance for additional siltation beyond the survey is included.

A component of the rate for dredging cost estimation assumes the dredged material is clean sand and is placed on Mooloolaba Beach.

Initially the client indicated that they may wish to pack dredge material into geotextile sandbags and use in the extensions core. Geofabrics Australia have advised on a unit cost of AUD$525 each which translates to AUD$131.25/m^3 of contained volume. This rate is only inclusive of supply and transport. KBR has been advised by the client that geofabric bags should no longer be investigated. Hence, no allowances for the geofabric bags have been made in the estimate at this stage.

7.3.3 Preliminaries and site allowances

The estimate includes the establishment, disestablishment (including making good) and removal of all non-permanent facilities which may be required by the Contractor to support the construction effort. This includes the provision of traffic management for temporary access to the designated site compound and construction laydown areas, temporary barge landing site at the east side of Mooloola River security fencing, preparation and management of safety and environmental plans; ongoing site facility hire, site set out and associated work.

7.3.4 Contingency

An amount of contingency has been provided in all the estimates options to cover the anticipated variances between the specific values given in the base estimate and the final actual project cost in order for the total estimated value to represent the most likely outcome.

It is expected that, should the project proceed, all contingency monies will be spent in the execution of the project. It is noted that contingency is not intended to cover changes from design performance, nor is it intended to cover the qualifications and exclusions listed.

A calculated contingency amount has been included at the rate of 25%.

7.3.5 Additional allowances

In addition to the design items described in the section above, the following has also been allowed:

- Removal and reinstatement of the existing navigation aid
- Site preparation allowances covering a 100 m segment of existing pavement on the breakwater crest, plus additional allowances for making good (i.e. reinstatement) of a 300 m segment of pavement used for ‘haulage’ on site.
- Installation and management of temporary navigation aids during construction
- Head Contractor Margin (Profit and overheads) of 15%
- Construction support services allowance of 6%
- Escalation to completion based on a 5% increase p.a. to 2022 (1 year)
- Project insurance at a rate of 1.5%
- Qleave fees at a rate of 0.575% included in direct costs

7.4 VALUE OF ESTIMATE

The cost estimate is summarised as follows:

Item	Estimated cost (Excl. GST)
Direct construction costs (WBS 1000) (i.e. Breakwater construction and associated works)	$16,067,617
Escalation (1 year)	$803,381
SUBTOTAL Directs	$16,870,997
Indirect costs:	
Construction support	$1,012,260
Project insurances	$268,249
Contingency (23%)	$4,515,514
SUBTOTAL Indirects	$5,796,023
TOTAL (including contingencies)	$22,667,020

- Cost predominantly relates to the breakwater works and material costs with 41% of the costs associated with the breakwater construction works
- 11% of the costs are associated with external site preparation allowances associated with demolition and reinstatement of the existing bitumen pavement and existing breakwater crest surface due to the use of heavy machinery at the breakwater crest.

7.5 QUALIFICATIONS, ASSUMPTIONS AND EXCLUSIONS

7.5.1 Qualifications and Assumptions

The estimate has been based on the following underlying assumptions:

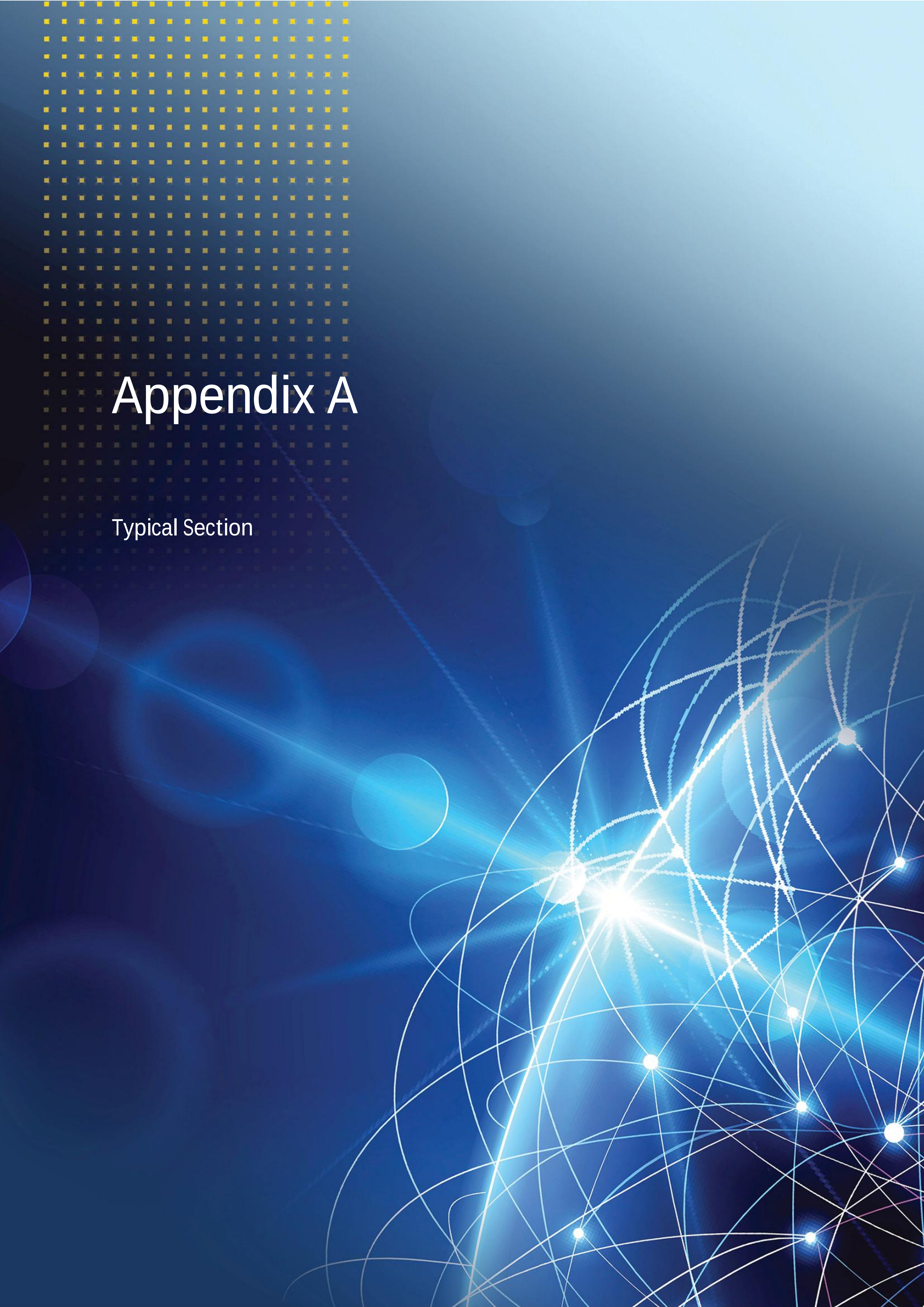
- Quantities are based on volumes estimated from typical sections prepared for preliminary design development and are subject to change through the development of more detailed plans.
- It is assumed that rock from the 10m demolished section of existing breakwater will be stockpiled for grading to be re-used in the underlayer material of the 60m extension or for repairs to existing breakwater sections (‘Secondary and recycled primary Armour to toe’)
- Estimates are non-binding to the accuracies stated above
- The estimate has been based on available market pricing from similar, non-identical projects, with factoring to account for specific conditions at the site.
- Wastage allowance has been excluded. It is the Contractor’s responsibility to make suitable allowance for material wastage and shall be addressed in the technical specification.

7.5.2 Exclusions

The following exclusions apply:

- Principals costs
- Statutory charges

- Design fees
- Council management costs
- Environmental, cost and schedule related issues
- Financing costs
- Other fees and charges
- Internal or external legal costs
- Goods and Services Tax (GST)
- Drastic changes in the price of diesel fuel not linked to the CPI
- Cultural Heritage Monitors for the duration during construction.
- Vegetation offsets.
- Referral and commonwealth approval of vegetation clearing.
- Application fees and managing of conditions of approvals.
- Management and removal of contaminated materials or acid sulphate soils management as a result of dredging.
- Disposal of rock (assumed existing rock will be reused at the breakwater transition or can be utilised for repair of existing breakwater sections)
- Provisions for additional costs or schedule delays due to extended periods of inclement weather or industrial unrest.
- Potential variations in costs due to currency rise and fall.
- Potential additional costs or schedule delays due to restrictions associated with COVID-19

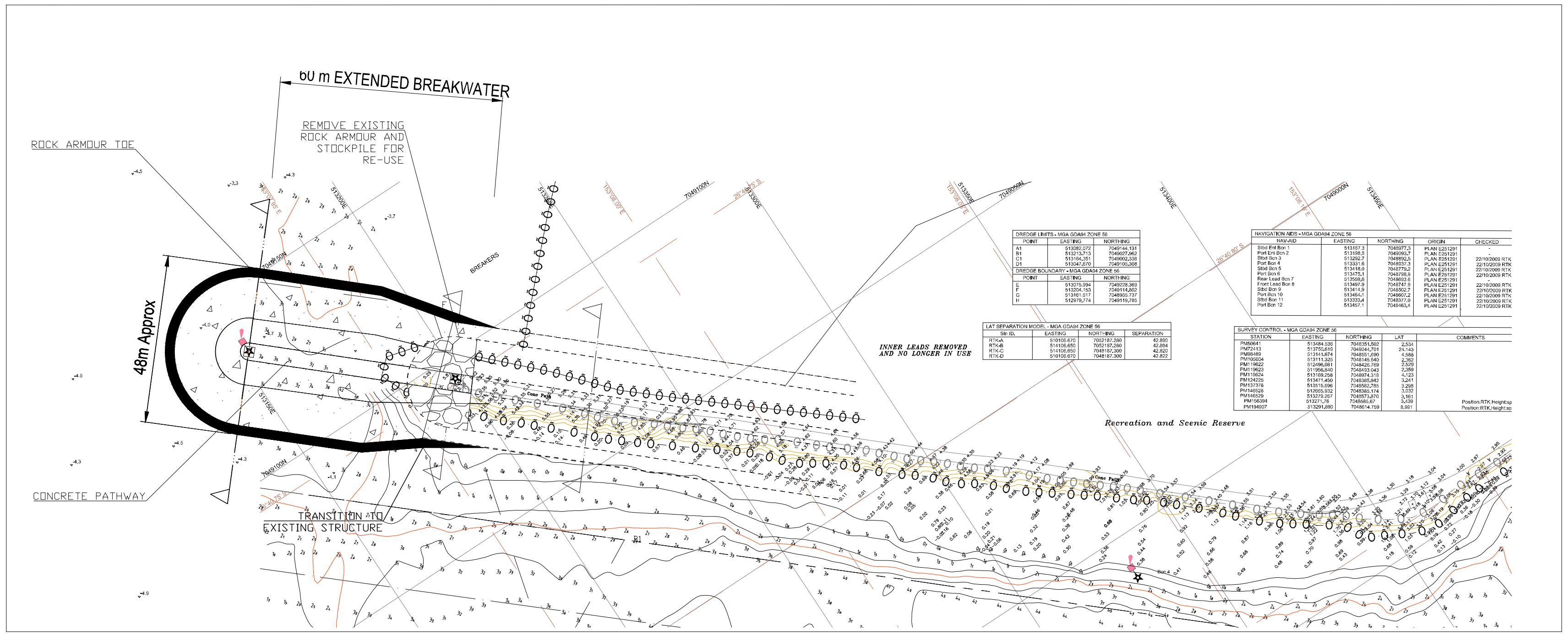


Appendix A

Typical Section

INFORMATION ONLY
30 November 2020

NOTES:
1. LEVELS IN m LAT (LOWEST ASTRONOMICAL TIDE) U. N. O.
2. SURVEY LEVELS FROM TMR PLAN E251-455
3. BEDROCK LEVELS FROM FRC BEDROCK SURVEY



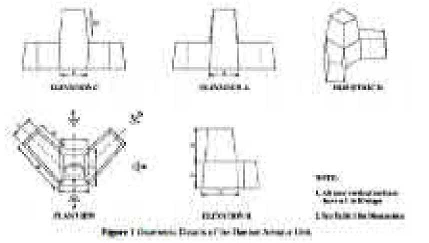
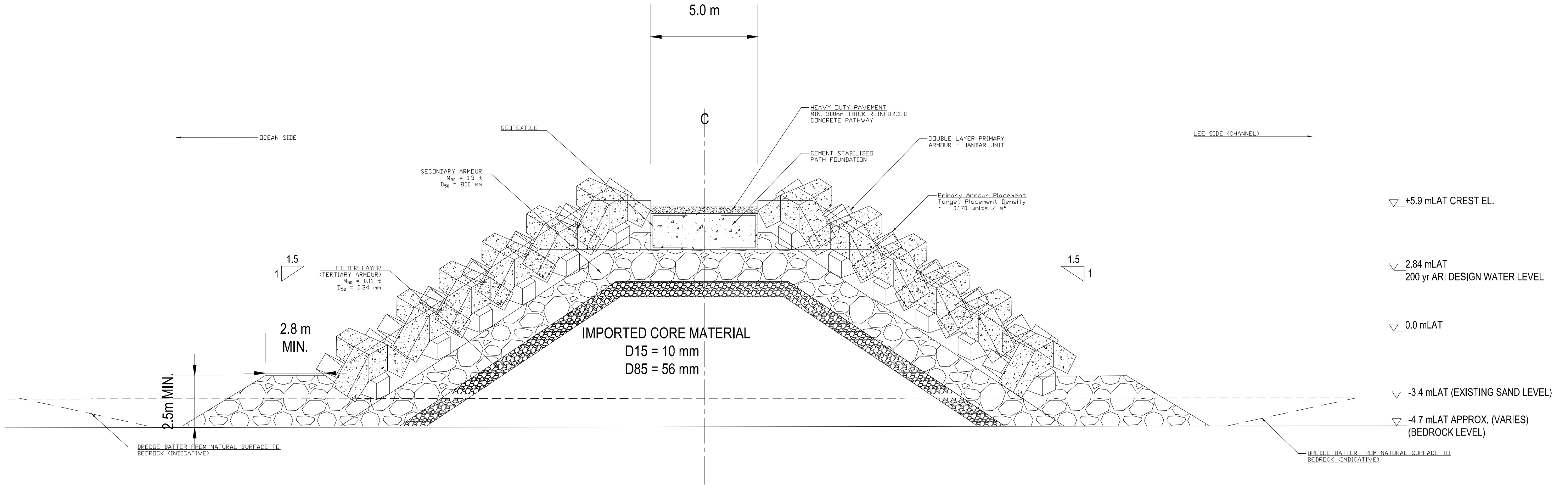
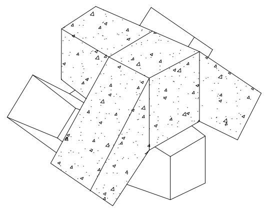


TABLE 1: HANBAR UNIT GEOMETRIC DETAILS

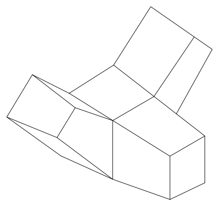
Mass of Unit – 7 t

Dimensions (mm)	
A	900
B	1750
C	670
D	970
E	1140
F	340
G	840
H	1060

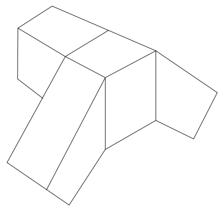




'INTERLOCKING' PLACEMENT PATTERN



LAYER 1 (BOTTOM)



LAYER 2 (TOP)

FILEPATH

FILE

Kellogg Brown & Root Pty Ltd
ABN 91 007 660 317

Monday, 30 November 2020 10:56:20 AM

0 100 200 300 400
SCALE 1: 5000 SCALE IN m

MOOLOOLABAH BREAKWATER
CONCEPT – HANBAR PRIMARY UNITS
SHEET 1

Appendix B

Cost Estimate Breakdown

umber: BEJ952
tle: Mooloolaba Breakwater Extension
on Number: BEJ952-TD-CV-CST-0001
on Title: Class 4 AACE Cost Estimate

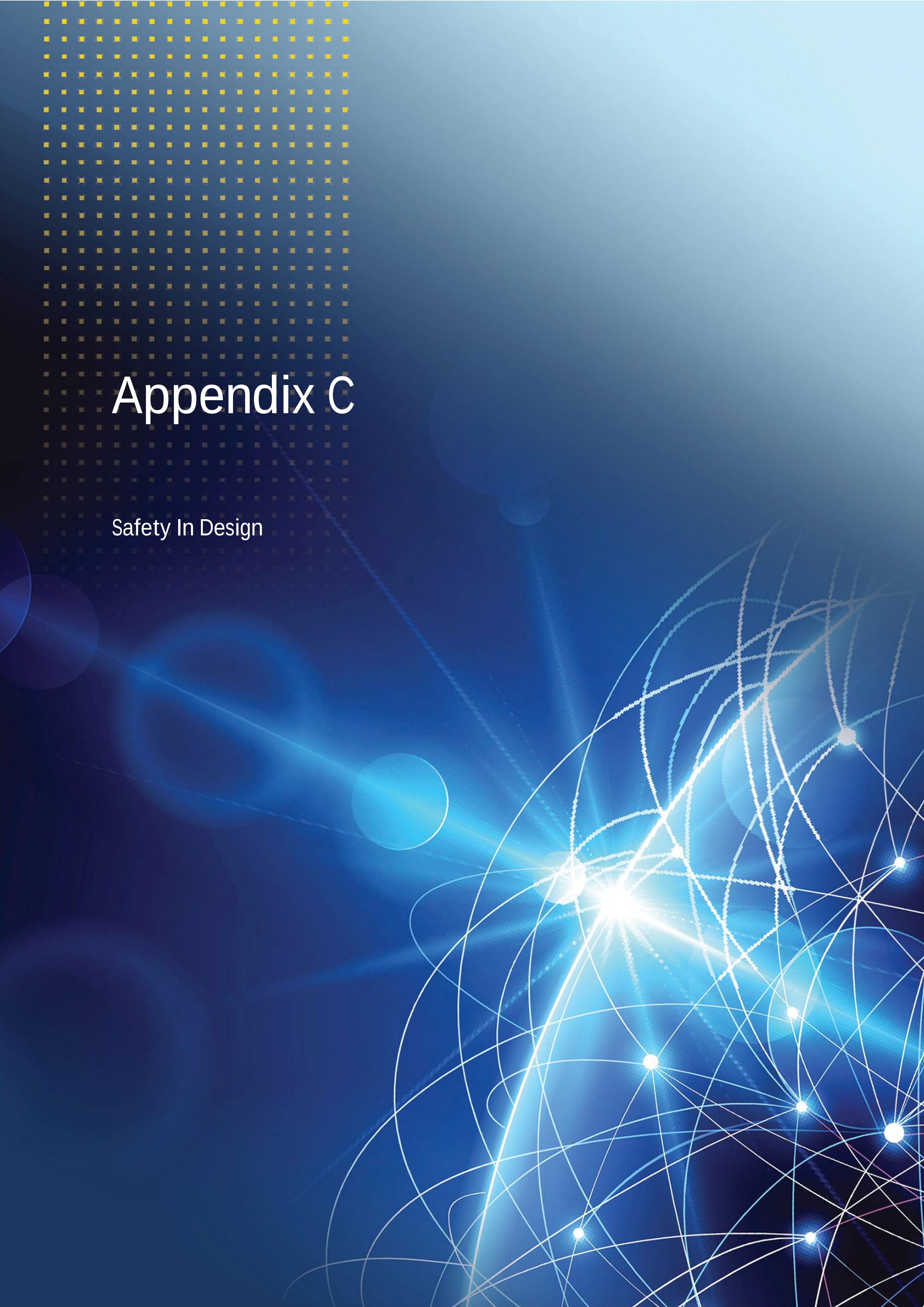
Prepared By: K Stemm
Checked By: B. Middleton
Date: 15/02/2021
Revision: A

1.1 Hanbar Concrete Armour Unit Breakwater

Area/Facility Code	Work Category	Scope Item	Unit	Bulk Material/Equipment Quantity					Principal Contractor Costs					Cost	Estimate Basis		Contingency Analysis								Monte Carlo Contingency Analysis			
																									%Rate		%Quantity	
				Quantity	$\text{Factor}_{\text{take-off}}$	$\text{Factor}_{\text{growth}}$	$\text{Factor}_{\text{construct}}$	$\text{Quantity}_{\text{total}}$	Unit Rate	Site Costs	Office Costs	Margin	$\text{Unit Rate}_{\text{total}}$		Basis of Quantity	Rate Source	$\text{Quality}_{\text{design}}$	$\text{Quality}_{\text{pricing}}$	$\%_{\text{quantity}}$	$\%_{\text{pricing}}$	$\%_{\text{RMS}}$	Contingency	Total Cost	$\%_{\text{cont.total}}$	Optimistic (-)	Pessimistic	Optimistic (-)	Pessimistic
						0%				9%	4%	15%																
DIRECT COSTS																												
1000																												
1100		PRELIMINARIES AND SITE ALLOWANCES																										
1101	SITE	Preparation of management and safety plans and permits in accordance with the specification	Item	1		0%		1	$ 440,000	9%	4%	15%	$ 573,602	$ 573,602		Historical	CONCEPT	OTHER PROJ.	18%	15%	23%	$ 134,399	$ 708,001	3.36%	-10%	18%	-10%	15%
1102	SITE	Mobilisation on site including, but not limited to, establishment of the site, contractor's site facilities, security fencing, temporary landing on the east side of the Mooloolah River, site preparation and associated work; excluding dredging	Item	1		0%		1	$ 1,116,000	9%	4%	15%	$ 1,454,862	$ 1,454,862		Allowance	CONCEPT	OTHER PROJ.	18%	15%	23%	$ 340,885	$ 1,795,747	8.52%	-10%	18%	-10%	15%
1103	SITE	Provision for traffic management	Days	110		0%		110	$ 1,500	9%	4%	15%	$ 1,955	$ 215,101		Historical	CONCEPT	OTHER PROJ.	18%	15%	23%	$ 50,400	$ 265,500	1.26%	-10%	18%	-10%	15%
1104	SITE	Development and implementation of Environmental Management in accordance with an approved Environmental Management Plan	Item	1		0%		1	$ 213,500	9%	4%	15%	$ 278,327	$ 278,327		Historical	CONCEPT	OTHER PROJ.	18%	15%	23%	$ 65,214	$ 343,541	1.63%	-10%	18%	-10%	15%
1105	SITE	Installation, maintenance of temporary navigation aids	No	2		0%		2	$ 103,200	9%	4%	15%	$ 134,536	$ 269,071		Allowance	CONCEPT	ALLOWANCE	18%	25%	31%	$ 82,890	$ 351,961	1.67%	-10%	18%	-30%	30%
1106	SITE	Ongoing site facility hire (Assume site office, crib facility & toilets at a minimum)	Weeks	22		0%		22	$ 300	9%	4%	15%	$ 391	$ 8,604		Allowance	CONCEPT	ALLOWANCE	18%	25%	31%	$ 2,651	$ 11,255	0.05%	-10%	18%	-30%	30%
1107	SITE	Demobilise from site	No	1		0%		1	$ 103,200	9%	4%	15%	$ 134,536	$ 134,536		Allowance	CONCEPT	ALLOWANCE	18%	25%	31%	$ 41,445	$ 175,980	0.84%	-10%	18%	-30%	30%
													SUBTOTAL FOR AREA 1100	$2,934,103								$717,883	$3,651,986					
1200		DREDGING (EARTHWORKS)																										
1201	EARTH	Allowance for mobilisation of Dredge	item	1		0%		1.0	$ 300,000.00	9%	4%	15%	$ 391,092	$ 391,092		Historical	CONCEPT	OTHER PROJ.	18%	15%	23%	$ 91,636	$ 482,728	2.29%	-10%	18%	-10%	15%
1202	EARTH	Removal of unsuitable material from under breakwater via Cutter Suction Dredging (including direct disposal to Mooloolaba Beach Nourishment) (Clean sand assumed) (Dredge area = 90m x 70m x 1.5m depth)	CM	9450		0%		9450.0	$ 30.00	9%	4%	15%	$ 39	$ 369,582		Historical	CONCEPT	OTHER PROJ.	18%	15%	23%	$ 86,596	$ 456,178	2.17%	-10%	18%	-10%	15%
1203	EARTH	Demobilisation and disestablishment of dredge from site	item	1		0%		1.0	$ 100,000.00	9%	4%	15%	$ 130,364	$ 130,364		Historical	CONCEPT	OTHER PROJ.	18%	15%	23%	$ 30,545	$ 160,909	0.76%	-10%	18%	-10%	15%
													SUBTOTAL FOR AREA 1200	$891,038								$208,777	$1,099,815					
1300		BREAKWATER																										
1301	EARTH	Remove existing breakwater roundhead (10m section assumed) and stockpile suitable material:				0%				9%	4%	15%				Historical	CONCEPT	OTHER PROJ.	18%	15%	23%				-10%	18%	-10%	15%
1302	EARTH	Primary armour rock (approx. 6t)	CM	1000		5%		1050.0	$ 31.20	9%	4%	15%	$ 41	$ 42,707		Historical	CONCEPT	OTHER PROJ.	18%	15%	23%	$ 10,007	$ 52,714	0.25%	-10%	18%	-10%	15%
1303	EARTH	Secondary armour rock	CM	500		5%		525.0	$ 31.20	9%	4%	15%	$ 41	$ 21,354		Historical	CONCEPT	OTHER PROJ.	18%	15%	23%	$ 5,003	$ 26,357	0.13%	-10%	18%	-10%	15%
1304	EARTH	Core	CM	500		5%		525.0	$ 31.20	9%	4%	15%	$ 41	$ 21,354		Historical	CONCEPT	OTHER PROJ.	18%	15%	23%	$ 5,003	$ 26,357	0.13%	-10%	18%	-10%	15%
1305	EARTH	Subgrade preparation (trim seabed and exposed face of existing breakwater)	SM	300		5%		315.0	$ 44.14	9%	4%	15%	$ 58	$ 18,125		Historical	CONCEPT	OTHER PROJ.	18%	15%	23%	$ 4,247	$ 22,372	0.11%	-10%	18%	-10%	15%
1306	EARTH																											
1307	EARTH	Supply Double layer 7t Hanbar Concrete armour units (V = 3m3/unit)	No.	1300		0%		1300.0	$ 2,000.00	9%	4%	15%	$ 2,607	$ 3,389,464		Historical	CONCEPT	OTHER PROJ.	18%	15%	23%	$ 794,177	$ 4,183,641	19.86%	-10%	18%	-10%	15%
1308	EARTH	Secondary armour rock	CM	5600		0%		5600.0	$ 252.00	9%	4%	15%	$ 329	$ 1,839,697		Historical	CONCEPT	OTHER PROJ.	18%	15%	23%	$ 431,055	$ 2,270,752	10.78%	-10%	18%	-10%	15%
1309	EARTH	Tertiary Armour / Filter (Double Layer)	CM	1800		0%		1800.0	$ 252.00	9%	4%	15%	$ 329	$ 591,331		Historical	CONCEPT	OTHER PROJ.	18%	15%	23%	$ 138,553	$ 729,884	3.46%	-10%	18%	-10%	15%
1310	EARTH	Imported Core material, granular	CM	7070		0%		7070.0	$ 145.50	9%	4%	15%	$ 190	$ 1,341,035		Historical	CONCEPT	OTHER PROJ.	18%	15%	23%	$ 314,215	$ 1,655,249	7.86%	-10%	18%	-10%	15%
1311	EARTH																											
1312	EARTH																											
1313	EARTH	Supply Double layer 7t Hanbar Concrete armour units (V = 3m3/unit)	No.	1300		0%		1300.0	$ 65.00	9%	4%	15%	$ 85	$ 110,158		Historical	CONCEPT	OTHER PROJ.	18%	15%	23%	$ 25,811	$ 135,968	0.65%	-10%	18%	-10%	15%
1314	EARTH	Secondary armour rock	CM	5600		0%		5600.0	$ 65.00	9%	4%	15%	$ 85	$ 474,525		Historical	CONCEPT	OTHER PROJ.	18%	15%	23%	$ 111,185	$ 585,710	2.78%	-10%	18%	-10%	15%
1315	EARTH	Tertiary Armour / Filter (Double Layer)	CM	1800		0%		1800.0	$ 91.00	9%	4%	15%	$ 119	$ 213,536		Historical	CONCEPT	OTHER PROJ.	18%	15%	23%	$ 50,033	$ 263,569	1.25%	-10%	18%	-10%	15%
1316	EARTH	Imported Core material, granular	CM	7070		0%		7070.0	$ 50.00	9%	4%	15%	$ 65	$ 460,837		Historical	CONCEPT	OTHER PROJ.	18%	15%	23%	$ 107,977	$ 568,814	2.70%	-10%	18%	-10%	15%
1317	EARTH	Sort and retrieve Secondary and recycled primary armour to toe	CM	3700		0%		3700.0	$ 166.00	9%	4%	15%	$ 216	$ 800,696		Historical	CONCEPT	OTHER PROJ.	18%	15%	23%	$ 187,609	$ 988,305	4.69%	-10%	18%	-10%	15%
													SUBTOTAL FOR AREA 1300	$9,324,817								$2,184,875	$11,509,692					
1400		CREST																										
1401	CON	Pavement:				0%				9%	4%	15%				Historical	CONCEPT	Historical	18%	18%	25%				-10%	18%	-13%	18%
1402	CON	N40 Concrete to heavy duty pavement, 300mm thick	SQM	350		2%		357	$ 350	9%	4%	15%	$ 456	$ 162,890		Historical	CONCEPT	OTHER PROJ.	18%	15%	23%	$ 38,166	$ 201,056	0.95%	-10%	18%	-10%	15%
1403	CON	Mesh reinforcement to heavy duty pavement (Top and Bottom Reinf. Assumed)	SQM	350		2%		357	$ 25	9%	4%	15%	$ 33	$ 11,635		Historical	CONCEPT	OTHER PROJ.	18%	15%	23%	$ 2,726	$ 14,361	0.07%	-10%	18%	-10%	15%
1404	CON	Formwork to edge of heavy duty pavement, 300mm high	LM	80		2%		81.6	$ 35	9%	4%	15%	$ 46	$ 3,723		Historical	CONCEPT	OTHER PROJ.	18%	15%	23%	$ 872	$ 4,596	0.02%	-10%	18%	-10%	15%
1405	CON	Sawcut existing to new joint and install jointing material incl dowels	Item	1		2%		1.02	$ 500	9%	4%	15%	$ 652	$ 665		Historical	CONCEPT	OTHER PROJ.	18%	15%	23%	$ 156	$ 821	0.00%	-10%	18%	-10%	15%
1406	CON	Cement Stabilised Sand as bedding layer for (and backing for primary rock armour, 5m wide x 1.6m thickness approx).	CM	720		5%		756	$ 154	9%	4%	15%	$ 201	$ 151,775		Historical	CONCEPT	OTHER PROJ.	18%	15%	23%	$ 35,562	$ 187,337	0.89%	-10%	18%	-10%	15%
1407	CON	Geotextile for pavement	SQM	910		2%		928.2	$ 8	9%	4%	15%	$ 10	$ 9,680		Historical	CONCEPT	OTHER PROJ.	18%	15%	23%	$ 2,268	$ 11,948	0.06%	-10%	18%	-10%	15%
1408	CON	Navigation Aid:				0%				9%	4%	15%				Historical	CONCEPT	Historical	18%	18%	25%				-10%	18%	-13%	18%
1409	CON	Allowance for the removal and reinstatement of navigation aid including footing and connection to services	Item	1		0%		1	$ 30,000	9%	4%	15%	$ 39,109	$ 39,109		Allowance	CONCEPT	Allowance	18%	25%	31%	$ 12,048	$ 51,157	0.24%	-10%	18%	-30%	30%
													SUBTOTAL FOR AREA 1400	$379,477								$91,799	$471,276					
1500		EXTERNAL ALLOWANCE - SITE PREPARATION																										
1501	SITE	Allowance to take-up and dispose off site existing pavement (100m section of existing breakwater)	m	100		5%		105	$ 80	9%	4%	15%	$ 104	$ 10,951		Historical	CONCEPT	OTHER PROJ.	18%	15%	23%	$ 2,566	$ 13,516	0.06%	-10%	18%	-10%	15%
1502	SITE	Allowance to reinstate existing pavements (100m of existing breakwater), 5m wide x 300mm thick concrete pathway assumed.	m	100		5%		105	$ 500	9%	4%	15%	$ 652	$ 68,441		Historical	CONCEPT	OTHER PROJ.	18%	15%	23%	$ 16,036	$ 84,477	0.40%	-10%	18%	-10%	15%
1503	SITE	Allowance to reinstate existing pavements (300m of existing breakwater), 5m wide x 100mm thick bitumen pathway assumed.	m	300		10%		330	$ 500	9%	4%	15%	$ 652	$ 215,101		Historical	CONCEPT	OTHER PROJ.	18%	15%	23%	$ 50,400	$ 265,500	1.26%	-10%	18%	-10%	15%
1504	SITE	Allowance for general site clearing including remove and dispose offsite vegetation and topsoil	SQM	2000		5%		2100	$ 20	9%	4%	15%	$ 26	$ 54,753		Historical	CONCEPT	OTHER PROJ.	18%	15%	23%	$ 12,829	$ 67,582	0.32%	-10%	18%	-10%	15%
1505	SITE	Remove and dispose off site crushed rock sub base 100mm thick including 5% cement stabiliser over existing access tracks and footpaths	SQM	4000		2%		4080	$ 10	9%	4%	15%	$ 13	$ 53,189		Historical	CONCEPT	OTHER PROJ.	18%	15%	23%	$ 12,462	$ 65,651	0.31%	-10%	18%	-10%	15%
1506	SITE	Survey set-out, conformance, and As Constructed survey including embankment slope, underlayer, and primary armour profiles	Item	1		0%		1	$ 10,400	9%	4%	15%	$ 13,558	$ 13,558		Historical	CONCEPT	OTHER PROJ.	18%	15%	23%	$ 3,177	$ 16,735	0.08%	-10%	18%	-10%	15%
1507	SITE	Make good the site (including landscaping)	SQM	5000		5%		5250	$ 300	9%	4%	15%	$ 391	$ 2,053,233		Allowance	CONCEPT	ALLOWANCE	18%	25%	31%	$ 632,516	$ 2,685,749	12.75%	-10%	18%	-30%	30%
													SUBTOTAL FOR AREA 1500	$2,469,225								$729,986	$3,199,210					
1600		QLEAVE																										
1601	SITE	Qleave - Contractor costs	%	0.575		0%		0.575	$ 91,992	9%	4%	15%	$ 119,925	$ 68,957		Historical	CONCEPT	OTHER PROJ.	18%	15%	23%	$ 16,157	$ 85,114	0.40%	-10%	18%	-10%	15%
													SUBTOTAL FOR AREA 1600	$68,957								$16,157	$85,114					
													SUBTOTAL FOR AREA 1000	$16,067,617														
																						Contingency	Total					
2000		Escalation																										
2100																												
2101		Calculated escalation to completion - Allowance for project escalation to construction (1 year at 5%assumed)	Item	1		0%		1	$ 803,381	0%	0%	0%	$ 803,381	$ 803,381		Allowance	CONCEPT	Allowance	18%	25%	31%	$ 247,488	$ 1,050,869	4.99%	-10%	18%	-30%	30%

1.1 Hanbar Concrete Armour Unit Breakwater

Area/Facility Code	Work Category	Scope Item	Unit	Bulk Material/Equipment Quantity					Principal Contractor Costs					Cost	Estimate Basis		Contingency Analysis								Monte Carlo Contingency Analysis			
				Quantity	Factor$_{take-off}$	Factor$_{growth}$	Factor$_{construct}$	Quantity$_{total}$	Unit Rate	Site Costs	Office Costs	Margin	Unit Rate$_{total}$		Basis of Quantity	Rate Source	Quality$_{design}$	Quality$_{pricing}$	%$_{quantity}$	%$_{pricing}$	%$_{RMS}$	Contingency	Total Cost	%$_{cont.total}$	%Rate		%Quantity	
																									Optimistic (-)	Pessimistic	Optimistic (-)	Pessimistic
						0%				9%	4%	15%																
INDIRECT COSTS																												
5000																												
5100		Construction Support																										
	5101	Construction support services							6%					$1,012,260														
													SUBTOTAL FOR AREA 5000	$1,012,260														
6000																												
6100		Design, tendering, construction management and contract - EXCLUDED																										
									0%																			
													SUBTOTAL FOR AREA 6000	$0														
7000																												
7100		Owner's costs and insurances																										
	7101	PLSL & State Training Levy - (Included in Direct Costs for Qleave)							0.000%					$0														
	7102	Owner's internal costs - EXCLUDED							0%					$0														
	7103	Statutory Chargers - EXCLUDED							0.00%					$0														
	7103	Project insurances							1.50%					$268,249														
													SUBTOTAL FOR AREA 7000	$268,249														
													ESTIMATE TOTAL (EXCLUDING CONTINGENCY)	$18,151,506														
8000																												
8100		Contingency																										
	8101	Contingency Level							25%					$4,515,514														
													SUBTOTAL FOR AREA 8000	$4,515,514														
													ESTIMATE TOTAL (INCLUDING CONTINGENCY)	$22,667,020														



Appendix C

Safety In Design

Assessment Activity: Safety in Design Assessment	Location: Mooloolaba Breakwater, Point Cartwright
Date of Assessment: 30/04/19	Assessment Team: personal information (Reviewer) (30/4/19
Date of Re-Assessment: 16/03/21, 16/12/21	Prepared By: personal information
General Comments Review and update register as required throughout the design process. It is assumed that further assessments will be carried out throughout the life of the project to identify any additional risks and to refine mitigation measures.	Reference No. 003

Activity	Hazard	Maximum credible hazard impact	Consequence	Likelihood	Risk	Risk treatment strategies	Residual Risk	Comments
Activity	Hazard	Maximum credible hazard impact	Consequence	Likelihood	Risk	Risk treatment strategies	Residual Risk	Comments
Construction	Land and sea-based construction site access – pedestrians/public risk. Public at risk of injury by machinery or construction activities.	Extensive injuries or fatality.	5	2	M	Secure the area to minimise public interactions as far as reasonable during construction.	M	STATUS: Active Risk treatments to be addressed by Contractor in preparing Technical Specification. Treatments will depend on the Contract and Contractor's work

Activity	Hazard	Maximum credible hazard impact	Consequence	Likelihood	Risk	Risk treatment strategies	Residual Risk	Comments
								method.
Construction	Personnel and visitors not being aware of hazards on site (both land and sea-based).	Significant injury or fatality.	5	3	H	No public access will be available during construction. Site access restrictions are to be implemented. Tenderer/Contractor is required to prepare a Work Health and Safety Management Plan for inclusion in the tender documentation or technical specification during detailed design to address hazards specific to the Works. All personnel and visitors to the site required to undergo a site-specific safety induction and wear appropriate Personal Protective Equipment (PPE).	M	STATUS: Active Risk treatments to be addressed by Contractor in preparing Technical Specification. Treatments will depend on the Contract and Contractor's intended work method.
Construction	Site access for construction poses some difficulties for large and heavy machines.	Significant injury or fatality.	5	3	H	Contractor shall be required to prepare a Work Health and Safety Management Plan to address hazards specific to the works including risk of over-water construction. For land-based construction, contractor shall be required to install temporary water filled barriers lining pathways adjacent to waterways subject to truck haulage. Adequate area for vehicle manoeuvring shall be provided within project constraints. Recommended that the following is included in technical specifications: • Preparation and implementation of a JHA/SWMS required for all works. • Machine operator must be suitably qualified and experienced in the activity (to be assessed during tender phase).	M	STATUS: Active Risk treatments to be addressed by Contractor in preparing Technical Specification. Treatment measures will depend on the Contract and Contractor's work method.
Construction	Handling of heavy construction materials (e.g., rock armour)	Significant injury; risk of drowning or	5	3	H	Contractor shall be required to prepare a Work Health and Safety Management Plan to address hazards specific to the works including	M	STATUS: Active

Activity	Hazard	Maximum credible hazard impact	Consequence	Likelihood	Risk	Risk treatment strategies	Residual Risk	Comments
	from floating barge or land-based cranes/excavators – drowning or crushing risk.	fatality.				risk of over-water construction. Works shall be undertaken by competent and qualified operators. Recommended that the following is included in technical specifications: Preparation and implementation of a JHA/SWMS required for all works. Machine operator must be suitably qualified and experienced in the activity.		Risk treatments to be addressed by Contractor in preparing Technical Specification. Treatment measures will depend on the Contract and Contractor's intended work method. Contractor staff to wear appropriate PPE (self-inflating vests) where identified in SWMS.
Construction / maintenance	Construction near or over water – drowning risk.	Personnel falling into water and drowning.	5	3	H	Contractor to develop and implement SWMS. Contractor staff to wear appropriate PPE (self-inflating vests) where identified in SWMS. Contractor personnel must be suitably qualified and experienced in the activity.	M	STATUS: Active Risk treatments to be addressed by Contractor in preparing Technical Specification. Contractor to implement site specific worksite inductions, develop and implement SWMS and wear appropriate PPE.
Construction	Increased traffic movements between sites with heavy vehicles increases risk of collision.	Significant injury or fatality.	5	3	H	TMR to notify residents of the increased traffic movements and associated hazards through consultation during the design process. Roads or public access may need to be restricted or closed. Depending on chosen construction method, car-parks and/or boat ramps may also require closure.	M	STATUS: Active Consultation with residents required to notify of traffic changes. Risk treatments to be

Activity	Hazard	Maximum credible hazard impact	Consequence	Likelihood	Risk	Risk treatment strategies	Residual Risk	Comments
						The requirement for a Traffic Management Plan is to be included in tender documentation or technical specification during detailed design. Contractor to identify Traffic hazards and Traffic control measures to be implemented. Contractor to implement timing limitations on construction activities (e.g., restrict heavy vehicle movement during school holidays). Contractor is required to consider and limit impacts to nearby community infrastructure and stakeholders (e.g., sporting grounds, shopping centres, schools, boat ramps, parks).		addressed by Contractor in preparing Technical Specification. Treatments will depend on the Contract and Contractor's intended work method to be considered at tender stage.
Construction	Pre-dredging sand material – use of suction dredge in potentially dangerous surf conditions.	Significant injury; risk of drowning or fatality.	5	5	C	Avoid conducting works over November – April to reduce likelihood of subjection to cyclone conditions and storms. Contractor to develop and implement SWMS. Contractor staff to wear appropriate PPE (self-inflating vests) where identified in SWMS. Contractor personnel must be suitably qualified and experienced in the activity.	M	STATUS: Active Risk treatments to be addressed by Contractor in preparing Technical Specification. Contractor to implement site specific worksite inductions, develop and implement SWMS and wear appropriate PPE.
Construction	Unstable slopes – rock stockpiles – crushing risk.	Significant injury or fatality.	5	3	H	Secure the rock pile away to minimise public interactions as far as reasonable during construction. Contractors and sub-contractors to follow CEMP and SWMS. Public consultation and notice to mariners.	M	STATUS: Active Stockpile to be in an exclusion zone and not near public areas. Contractors and sub-contractors to follow CEMP and SWMS. Public consultation

Activity	Hazard	Maximum credible hazard impact	Consequence	Likelihood	Risk	Risk treatment strategies	Residual Risk	Comments
								and notice to mariners.
Construction	Adverse weather, including storms, high wind events and exceptionally high tides.	Significant injury; risk of drowning or fatality.	5	4	C	Suitable procedures to be put in place during construction to monitor storms and secure the partially structures against damage in the event a storm is anticipated to affect the works. Avoid conducting works over November – April to reduce likelihood of subjection to cyclone conditions and storms. The following treatments to be incorporated in project technical specification by detailed designers and is to be assessed at tender stage: Tenderer/Contractor to nominate intended work method for construction at the site. Construction to be completed in stages where practical to prevent exposure of partially constructed structure/revetment during construction. Tenderers to be familiar with site conditions and have prior experience in similar construction projects.	M	STATUS: Active Construction treatments will depend on the Contract and Contractor's intended work method. Consultation with stakeholders may be required to notify of construction hazards associated with carrying out work during high storm event risk period.
Construction / maintenance	Altered marine navigation approaches – vessel collision risk (partially built submerged breakwater, barges and cranes).	Significant injury; risk of drowning or fatality.	5	3	M	Contractor to issue a notice to mariners to declare obstacles during construction/maintenance. Contractor to use temporary navigation markers/buoys during construction/maintenance. Low likelihood as mariners are familiar with mooring of dredge equipment at Mooloolah River entrance, with sufficient width for marine traffic.	2	STATUS: Active Contractor to issue notice to mariners for navigational hazards. Contractor to install navigational markers/buoys to delineate possible navigational hazards. Contractor to consult Harbour Master to seek advice for temporary

Activity	Hazard	Maximum credible hazard impact	Consequence	Likelihood	Risk	Risk treatment strategies	Residual Risk	Comments
								construction works.
Construction / maintenance	UV exposure.	Minor injury – sunburn, heatstroke	2	5	H	Contractor staff to wear appropriate PPE (hats, long sleeve shirts, plants, sunscreen, and eyewear) where identified in SWMS.	L	STATUS: Active Contractors and sub-contractors to follow CEMP and SWMS.
Construction	Inferior products/materials utilised in construction which may deteriorate over life of the project.	Significant injury; crushing risk, risk of drowning or fatality.	5	3	H	Inspections and hold points to be incorporated in project technical specification. Construction supervision required to ensure material compliance with relevant standards and project specifications.	M	STATUS: Active Risk treatments to be addressed by Contractor in preparing Technical Specification. TMR to ensure appropriate construction supervision is implemented.
Construction	Location of services, including any underground and overhead power cables is unknown.	Risk of electrocution causing significant injury or fatality.	5	2	M	Services location survey to be carried out during detailed design process. The following treatments to be incorporated in project technical specification and is to be assessed at tender stage: The Contractor shall ascertain from the appropriate Authorities the position and the depth/height of all public utility or other services which may be affected during the works.	M	STATUS: Active TMR to consider services location survey in future design stage. Requirement for Contractors to identify location of services to be addressed by Contractor in preparing Technical Specification.
Construction / decommissioning	Hot works – UV burns including to eyes. Burns due to heat. Gas cylinders containing explosive gases. Explosion of	Serious injury/lost time injury or fatality	5	2	M	Contractor/sub-contractor to develop and implement SWMS concerning welding work and cutting, including the use of qualified technicians and appropriate PPE.	M	STATUS: Active Contractors and sub-contractors to follow CEMP and SWMS.

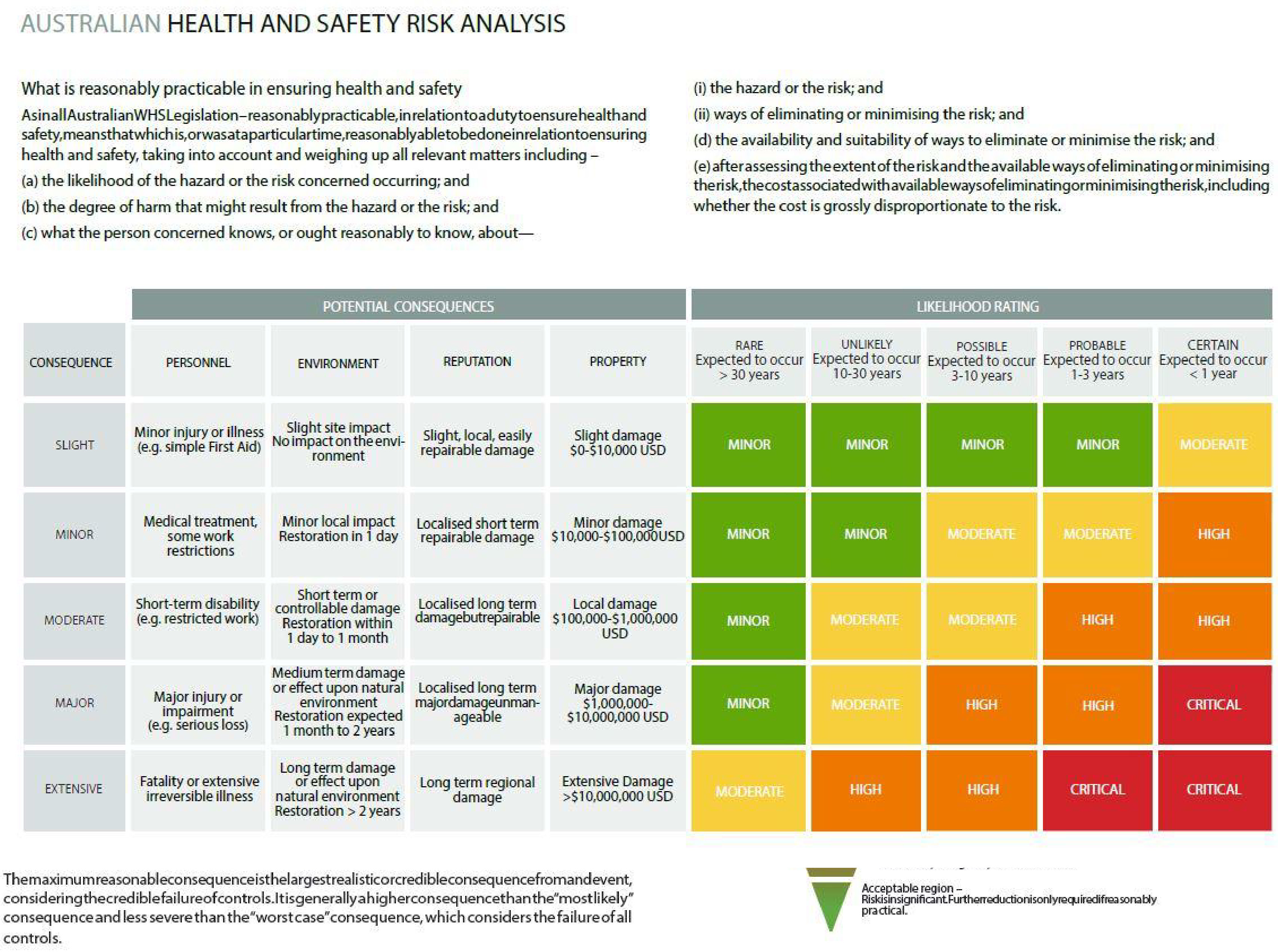
Activity	Hazard	Maximum credible hazard impact	Consequence	Likelihood	Risk	Risk treatment strategies	Residual Risk	Comments
	leaked gases.							
Construction / maintenance	Environmental contamination / spillage causing harmful impacts to marine flora.	Fauna death/injury, environmental damage or incident, pollution.	4	5	C	Contractor to actively implement pollution controls for the existing environment (e.g., pollution control booms, construction waste management, localised spill kits). Designer to consider measures to reduce environmental impact through space planning.	M	STATUS: Active Risk treatments to be addressed by Contractor in preparing Technical Specification.
Construction	Fall from height resulting in injury and death. For example, falling from top of breakwater onto underlying rocks.	Significant injury or fatality.	5	4	C	Safety barriers, contractors to exercise minimum of three-points of contact at all times, fall restraint harness, safety inductions. Contractor / sub-contractor to implement training and use competent and qualified personnel.	M	STATUS: Active Contractors and sub-contractors to follow CEMP and SWMS. Risk treatments to be addressed by Contractor in preparing Technical Specification.
Construction / maintenance	Contact or exposure to hazardous materials; working with flammable / combustible materials (e.g., fuel).	Serious injury/lost time injury	5	2	M	Restrict access to hazardous materials to qualified personnel and provide PPE. Maintain various piping connections (fuel) and ensure safe methods are practiced.	M	STATUS: Active Contractors and sub-contractors to follow CEMP and SWMS.
Construction / maintenance	Over-night collision of water vehicle with breakwater and/or equipment during construction or maintenance.	Significant injury; Risk of drowning or fatality.	5	4	C	Proper lighting during construction and life of structure, notice to mariners of works and potential hazards, harbour master approval of breakwater and construction site boundaries, frequent inspections to ensure lighting is working.	M	STATUS: Active Contractors and sub-contractors to follow CEMP and SWMS. Risk treatments to be addressed by Contractor in preparing Technical Specification.
Construction / maintenance	Impact from drifting boats during severe weather events.	Significant injury; Risk of drowning	5	4	C	Contractor shall have a spotter during severe weather events and a	M	STATUS: Active Risk treatments to be

Activity	Hazard	Maximum credible hazard impact	Consequence	Likelihood	Risk	Risk treatment strategies	Residual Risk	Comments
		or fatality.				service boat equipped for towing/pushing if required.		addressed by Contractor in preparing Technical Specification.
Maintenance	Solar lighting maintenance – working at heights risk.	Permanent disability or fatality.	5	2	M	Contractor to minimise frequency of maintenance through appropriate design (e.g., long-life bulbs/LED). Contractor to use collapsible lighting posts wherever appropriate. Contractor to ensure fittings have appropriate safeguards against electrocution by meeting appropriate Australian Standards. Contractor to develop implement appropriate SWMS for maintenance activity prior to conducting maintenance. Use only appropriately trained/experienced staff to conduct maintenance.	L	STATUS: Active Risk treatments to be addressed by Contractor in preparing Technical Specification.
Maintenance	Condition inspections of breakwater including underwater areas risks injury to inspectors.	Serious injury/lost time injury	3	2	M	Designer to consider condition inspection and assessment during design phase. Consider remote methods for inspections (i.e., laser scan/drones + multibeam survey). Develop implement appropriate SWMS for maintenance inspection activity prior to conducting maintenance	L	STATUS: Active Contractor/TMR to consider remote inspection methods and develop and implement SWMS for maintenance/condition inspections.
In service	Wave overtopping – pedestrians swept off their feet and/or swept off breakwater into water.	Significant injury; Risk of drowning or fatality.	5	5	C	Allow visibility of the ocean where practicable. Design crest elevation to account for pedestrian access during 1-year ARI wave event and subsequent overtopping. Signs to warn pedestrians that breakwater can be overtopped and don't enter during storms/large wave events.	M	STATUS: Active Design does not consider safe overtopping during a 1 year ARI event in accordance with revised Basis of Design (Rev 1) Signage or pedestrian exclusion

Activity	Hazard	Maximum credible hazard impact	Consequence	Likelihood	Risk	Risk treatment strategies	Residual Risk	Comments
								required to warn of risks for more severe events.
In service	Risk of people climbing breakwater / jumping from breakwater.	Significant injury; risk of drowning or fatality.	5	5	C	Signage to be erected to notify of shallow water hazards and deter people from climbing/jumping. A life ring mounted on a stand shall be provided on the breakwater head.	H	STATUS: Active Contractor to ensure life ring and appropriate signage requirements are captured in technical specification.
In service	Risk of fishermen falling from breakwater/swept away by waves.	Significant injury; risk of drowning or fatality.	5	5	C	Signage to be erected to notify of hazards and discourage users to access breakwater rocks. Bolted harness locations or fishing rod holders to encourage fishermen to fish from locations where it is safer to do so. A life ring mounted on a stand shall be provided on the breakwater head.	H	STATUS: Active Contractor to ensure life ring and appropriate signage requirements are captured in technical specification.
In service	Pedestrians using breakwater during an earthquake may be unsafe. Prior warning to evacuate is not possible.	Significant injury	2	1	L	As per AS 1170.4-2007 the likelihood and earthquake intensity in Sunshine Coast Region is low and wave loads govern the design. Combined probability that earthquakes and waves occur at the same time as maximum wave loading extremely low. Design for maximum wave loads to ensure breakwater stability.	L	STATUS: Design prepared on the basis of wave conditions in the Basis of Design (Rev 1).
In service	Vessel berthing – potential catastrophic damage to vessel.	Significant injury; Risk of drowning or fatality.	5	4	C	Deter vessels from docking at breakwater with the use of appropriate signage and absence of mooring berths/ladders. Installation of speed limit signage. Installation of appropriate lighting to delineate a navigation obstruction.	H	STATUS: Active Appropriate signage to deter vessel berthing and impose speed limits in proximity of jetty to be addressed by TMR. Public consultation and notice to mariners.

Activity	Hazard	Maximum credible hazard impact	Consequence	Likelihood	Risk	Risk treatment strategies	Residual Risk	Comments
In service	Navigational Hazard	Significant injury; Risk of drowning or fatality.	5	4	C	Design toe to extend further than existing breakwater (no toe was designed on existing breakwater segments). Due to extended toe, additional navigational aids and notices required to advise public of changes to navigation. Declared channel to be assessed with consideration of underwater toe detail.	H	STATUS: Active Appropriate signage, navigation aids and designated navigation channel widths to be publicised to to deter vessels from navigating close to breakwater. Public consultation and notice to mariners.
In service	Solar lighting failure – Marine navigation lights – vessel collision risk.	Extensive injuries.	5	4	C	Design with safety and redundancy and provide adequate reflective surfaces on the breakwater. Maintain lighting to minimise reliability issues. Installation of bird-deterrent measures around lighting solar panels as per MRTS98.	L	STATUS: Active Contractor to incorporate sufficient redundancy/factor of safety to ensure sufficient reliability. Contractor to seek to incorporate reflective (night-time) treatments on breakwater. Contractor to install anti-roosting measures around lighting solar panels. TMR to maintain lighting to minimise reliability issues. TMR to update marine navigation charts to be issued together with associated notices to mariners.

Activity	Hazard	Maximum credible hazard impact	Consequence	Likelihood	Risk	Risk treatment strategies	Residual Risk	Comments
								TMR to implement navigational speed limits to reduce vessel speeds.



AUSTRALIAN HEALTH AND SAFETY RISK ANALYSIS

What is reasonably practicable in ensuring health and safety

AsinallAustralianWHSLegislation–reasonablypracticable,inrelationtoadutytoensurehealthand safety,meansthatwhichis,orwasataparticulartime,reasonablyabletobedoneinrelationtoensuring health and safety, taking into account and weighing up all relevant matters including –

(a) the likelihood of the hazard or the risk concerned occurring; and

(b) the degree of harm that might result from the hazard or the risk; and

(c) what the person concerned knows, or ought reasonably to know, about—

(i) the hazard or the risk; and

(ii) ways of eliminating or minimising the risk; and

(d) the availability and suitability of ways to eliminate or minimise the risk; and

(e) afterassessing the extent of the risk and the available ways of eliminating or minimising therisk,thecostassociatedwithavailablewaysofeliminatingorminimisingtherisk,including whether the cost is grossly disproportionate to the risk.

	POTENTIAL CONSEQUENCES				LIKELIHOOD RATING				
CONSEQUENCE	PERSONNEL	ENVIRONMENT	REPUTATION	PROPERTY	RARE Expected to occur > 30 years	UNLIKELY Expected to occur 10-30 years	POSSIBLE Expected to occur 3-10 years	PROBABLE Expected to occur 1-3 years	CERTAIN Expected to occur < 1 year
SLIGHT	Minor injury or illness (e.g. simple First Aid)	Slight site impact No impact on the environment	Slight, local, easily repairable damage	Slight damage $0-$10,000 USD	MINOR	MINOR	MINOR	MINOR	MODERATE
MINOR	Medical treatment, some work restrictions	Minor local impact Restoration in 1 day	Localised short term repairable damage	Minor damage $10,000-$100,000USD	MINOR	MINOR	MODERATE	MODERATE	HIGH
MODERATE	Short-term disability (e.g. restricted work)	Short term or controllable damage Restoration within 1 day to 1 month	Localised long term damagebutrepairable	Local damage $100,000-$1,000,000 USD	MINOR	MODERATE	MODERATE	HIGH	HIGH
MAJOR	Major injury or impairment (e.g. serious loss)	Medium term damage or effect upon natural environment Restoration expected 1 month to 2 years	Localised long term majordamageunmanageable	Major damage $1,000,000-$10,000,000 USD	MINOR	MODERATE	HIGH	HIGH	CRITICAL
EXTENSIVE	Fatality or extensive irreversible illness	Long term damage or effect upon natural environment Restoration > 2 years	Long term regional damage	Extensive Damage >$10,000,000 USD	MODERATE	HIGH	HIGH	CRITICAL	CRITICAL

Themaximumreasonableconsequenceisthelargestrealisticorcredibleconsequencefromandevent, consideringthecrediblefailureofcontrols.Itisgenerallyahigherconsequencethanthe"mostlikely" consequenceandless severe than the "worstcase" consequence, which considers the failure of all controls.

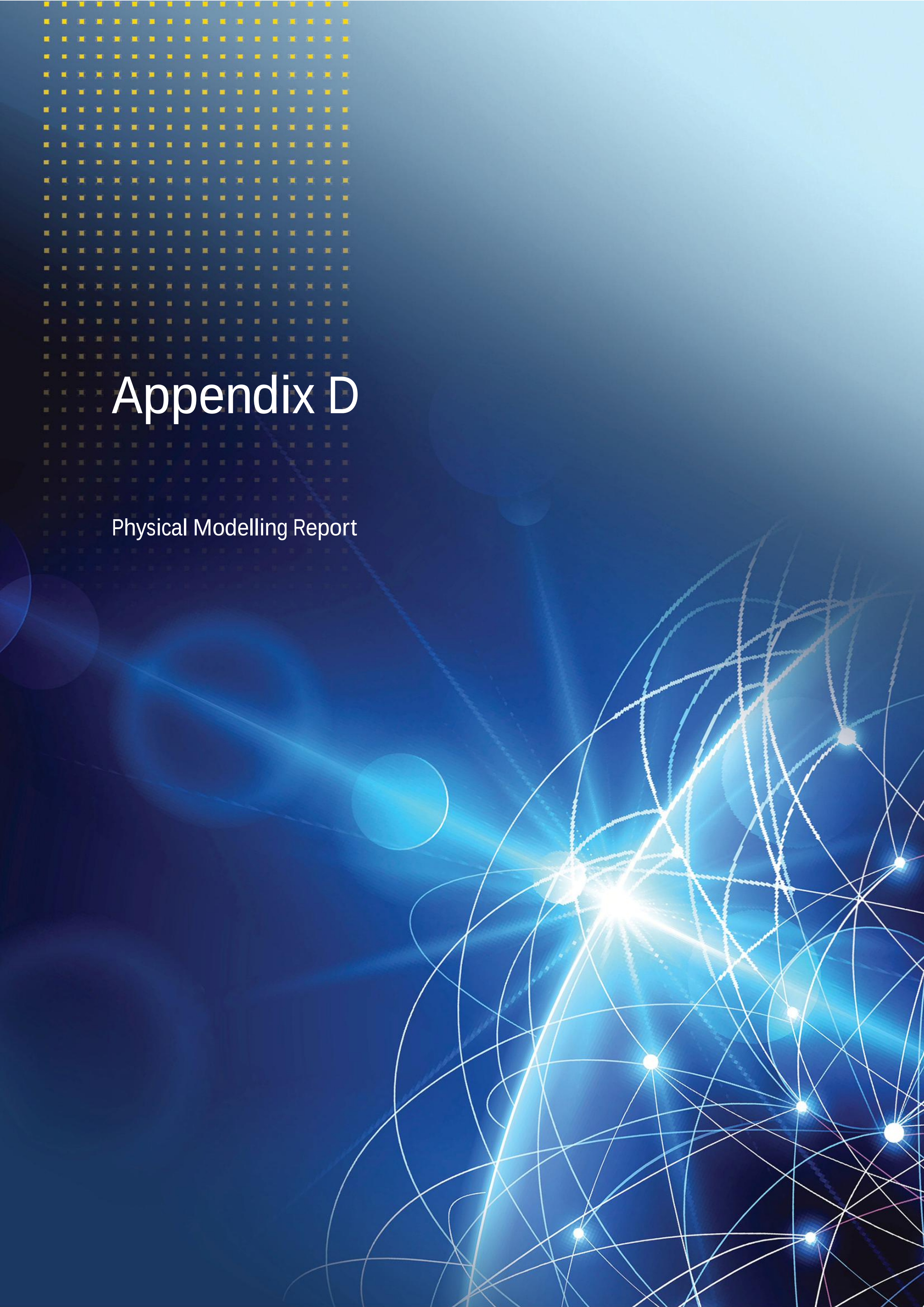
Acceptable region –
Riskisinsignificant.Furtherreductionisonlyrequiredifreasonably practical.

Consequence	Likelihood Rating				
	1 Rare	2 Unlikely	3 Possible	4 Probable	5 Certain
1 - Slight	LOW	LOW	LOW	LOW	MODERATE
2 - Minor	LOW	LOW	MODERATE	MODERATE	HIGH
3 - Moderate	LOW	MODERATE	MODERATE	MODERATE	HIGH
4 - Major	LOW	MODERATE	HIGH	HIGH	CRITICAL
5 - Extensive	MODERATE	MODERATE	HIGH	CRITICAL	CRITICAL

NOTE: Items highlighted above that are deemed to have a SIGNIFICANT RESIDUAL RISK should be communicated on the drawings and in asset owners maintenance manual as appropriate.

An example SHE (Safety, Health and Environment) Box that can be included on drawings is shown below;

SAFETY HEALTH AND ENVIRONMENTAL INFORMATION
In addition to the hazards normally associated with the types of work detailed on this drawing, note the following risks and information:
It is assumed works will be carried out by a competent contractor working, where appropriate to an approved method statement



Appendix D

Physical Modelling Report

Mooloolaba Breakwater
Physical Modelling Project

5m Flume Investigations

Queensland Government Hydraulics Laboratory, Physical Modelling Facility

March 2020 – Version 0

Security Classification: Confidential

Version: 0

	Name	Signature	Date
Written by			
Reviewed by	personal information		
Approved by			

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March 2020

Table of Contents

1 Introduction

The Queensland Government Hydraulics Laboratory (QGHL) was contracted by the Client (KBR) to conduct physical model wave flume (5m wide) testing to investigate the structural stability of a proposed design of a new rock breakwater extension (60m) for the Mooloolaba breakwater (Figure 1). This document details the methodology in Section 2, and test results and data associated with the physical modelling in Section 3.

Figure 1: Proposed location of the breakwater extension (Source: SOW).

2 Methodology

This section presents the methodology for the Design, Setup, Testing and Data Analysis of the physical modelling project.

A section of the existing breakwater together with the 60m extension part (M50 = 6tonne) was tested first in the flume. Structural stability was assessed for a single design wave and water level condition, as specified by the Client.

2.1 Quality Assurance

Quality Assurance for the project was maintained in accordance with the QGHL Quality Management System (QMS) that is developed in alignment with the ISO9001:2015 quality management standard. The QGHL operates in accordance with its own QMS as well as the physical modelling best-practices described in Frostick et al. (2011) and Hughes (1993). By following these guides, the project quality is assured for model design (e.g. appropriate scaling), implementation (e.g. model construction with regular surveys, model setup with drive signal generation), data collection (e.g. regular instrument calibration and test record forms) and data storage (e.g. appropriate file naming convention).

2.2 Wave conditions and water levels

The wave paddle is an electronically actuated piston-type paddle and the wave generation is driven by HR Merlin (HR Wallingford software). More detail on wave generation is provided in Section 2.5.

The Client has specified two sets of conditions (Water levels and waves) to test the breakwater under. Condition set one will be applied for Test 1 and Test 2, as necessary with a water level corresponding to +1.84 m AHD and waves with spectral significant wave height H_{m0} = 3.54 m, and peak period T_p = 12.7 s. The second condition set will be applied for Test 3, which is a test to fail scenario where the wave conditions will be adjusted to increase chance of structural failure (i.e. H_s > 3.54 m and T_p > 12.7 s, to be finalised, but within the limits of the model and wave paddle). Table 1 provides the test schedule summary which includes a possible three separate tests. To date (31/03/2020), Test 1 has been completed, the project is currently on hold while the client finalises the details of the next test.

Table 1: Test series description and wave conditions for test series 1 to 3. Model scale values for wave height, period and duration are provided in brackets.

Test	Test Requirement	Description	Water level	Significant Wave Height, Hm0	Peak Period, Tp	Duration
(-)	(-)	(-)	(m AHD)	(m)	(s)	(minutes)
Test 1	To be tested	Scenario A - M_{50} 6 tonne rock	1.84	3.54 (0.088)	12.7 (1.98)	240 (37.5)
Test 2	If M_{50} 6t fails	Scenario B – Xbloc/AccoropodeII? (TBC)	1.84	3.54 (0.088)	12.7 (1.98)	4 (37.5)
Test 3	Test to fail final structure	Scenario C - Test to failure by increasing wave height	1.84	>3.54 (>0.088)	>12.7 (>1.98)	>4 (>37.5)

2.3 Scaling

Froude scaling was applied to scale the model from the prototype design. Geometric scaling was applied for rock scaling for both the armour and the filter layers, with the Hudson formula (Hudson, 1959) used to calculate the appropriate model rock grading (Section 2.2.2).

2.3.1 Model scaling

Following Hughes (1993), a prototype-to-model length scale ratio of

$$N_L = \frac{L_{Prototype}}{L_{Model}} = 41$$

was applied to simulate the wave conditions and water levels that are required for the test series within the limits of the testing facility. Froude scaling requires the following prototype-to-model time scale ratio:

$$N_T = \sqrt{N_L} = 6.4$$

2.3.2 Primary armour scaling

Rock mass scaling N_{Wa}, for the model was conducted using the Hudson formula (Hudson et al., 1979);

$$N_{W_a} = \frac{(W_a)_p}{(W_a)_m} = \frac{\frac{(\gamma_a)_p}{(\gamma_a)_m}\left(\frac{L_p}{L_m}\right)^3}{\left(\frac{(S_a)_p - 1}{(S_a)_m - 1}\right)^3}$$

where subscripts $_p$ and $_m$ denote prototype and model values respectively, W_a is the weight of the armour unit (kg) (i.e. *ROCK*$_{prototype}$, Figure 1), Υ_a is the specific weight of the armour unit (kg m^{-3}), L is the characteristic length (m), and S_a is the specific gravity relative to water ($^{\gamma_a}/_{\gamma_w} - 1$, where Υ_w is the specific weight of water).

Using a model rock density (2,712 kg m^{-3}) indicates that the model rock dimensions and are above the acceptable size limit (d_{50} > 25 mm, Frostick et al., 2011) in the flume model scale for the weight range specified by the Client, see Figure 2. The Reynolds stability number indicated for a 3.54 m prototype wave height is a little lower than the recommended value of R_n > 30,000 (Hudson et al., 1979, Figure 3). However, the Reynolds numbers are still considered turbulent and the scatter associated with the original Hudson et al. (1979) figure indicates a fair degree of variability in the potential stability numbers.

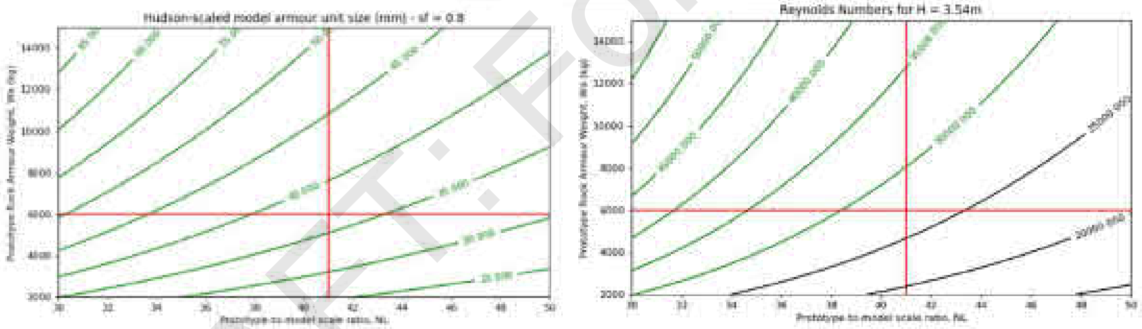


Figure 2: Left: Scaled nominal model rock sizes. Right: Reynolds stability number for the rocks over range of scales for a 3.54 m prototype wave height.

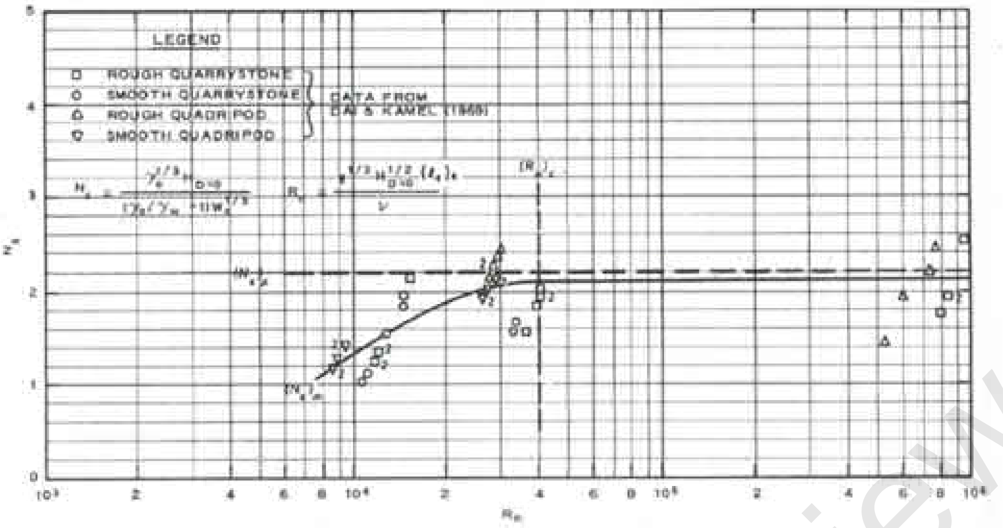


Figure 3: Scale effects of rubble-mound stability models (after Dai and Kamel, 1969), taken from Hudson et al. (1979, p. 344). N_s = stability number, R_n = Reynolds number. Demonstrating relationship between model Reynolds number and stability number. For example, by the trend lines in plot, for R_n ~ 10,000 in the model, we may expect N_s = 2.2/1.3 = 1.69 (approx.), meaning the model would be more likely to deform.

Rock sorting was conducted at the QGHL rock sorting facility (Figure 4), where raw rock was sorted by size and weight to ensure the grading of the primary armour conformed to the design grading, supplied by the Client. The final armour layer grading curves for the 6 tonne primary armour rock used in the model are provided in Figure 5.Good agreement was achieved between the target and measured weight grade. Figure 6 shows an image of a sample of the sorted rock used for the primary armour.

Figure 4: Example photos of rock sorting. Raw stock rock piles (left); sieve fractionation of model rock (right).

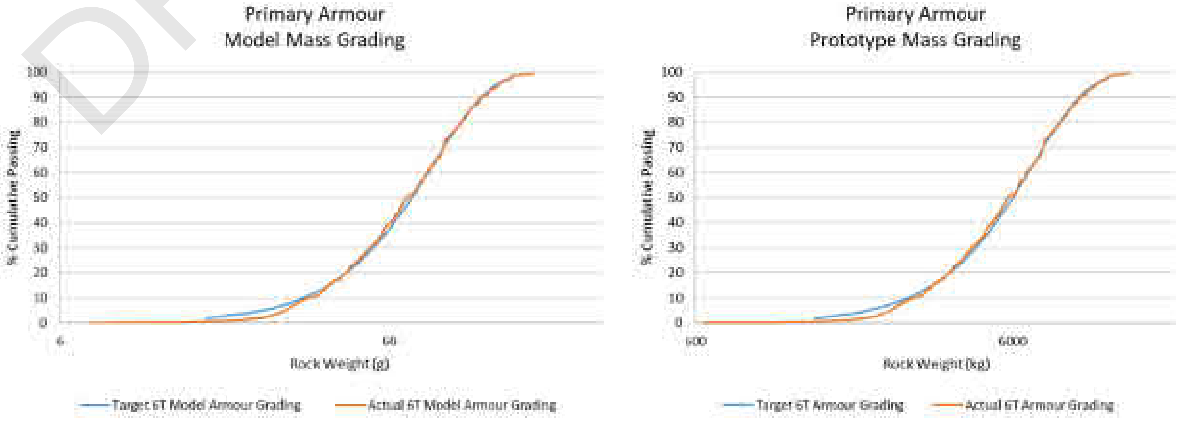


Figure 5: Armour rock grading curves.

Figure 6: Example of Armour rocks used.

2.3.3 Filter layer and core grading and scaling

A geometric scaling of the filter layer would result in hydraulic porosity lower for the modelled filter than the prototype filter rocks. A reduced porosity affects the wave transmission and hydraulic head build up within the structure. This can result in early failure of the structure due to the hydraulic gradient between the inside and outside of the structure. Considering the preference to operate in favour of a conservative model (rather than attempting to scale to an unknown ultimate porosity and blockiness in the final prototype structure), the filter layer was also scaled geometrically based on the Client's estimate of the core composition and rock grading.

The specification and model-scaled values are provided in Table 2. The prototype filter rock is modelled to have a density ρ_{rock} = 2,700 kg m^{-3}. No secondary armour layer is present for the existing breakwater section, so the primary armour was placed directly on the core. The secondary armour layer rock sizes for the breakwater extension was built with two layers of ~ 900 kg rocks (converted to prototype values), corresponding to model rock sizes in the range 20 mm < D < 25 mm. The filter layer grading curve is provided in Figure 7, and Figure 8 shows an example of the model secondary armour rock, painted red for easy identification and differentiation from the core (yellow) and primary armour rock (grey, white, and blue).

The prototype core was indicated to be constructed of quarry run rock in the size range 150 mm < D < 820 mm (approx. 2/3rd of the armour unit size). To obtain these for the model, a mix of gravel and rocks was used, size range: 4mm < D < 20mm.

Table 2: Core and filter layer specifications

	Density	Prototype weight M50	Nominal Prototype Diameter	Realistic Prototype Diameter (nom/sf, sf = 0.8)	Realistic Model Diameter
Unit	[kg/m3]	[kg]	[m]	[m]	[mm]
Core Min	2650	-	-	0.15	4
Core Max	2650	-	-	0.82	20
Secondary Armour	2700	900	0.69	0.87	21

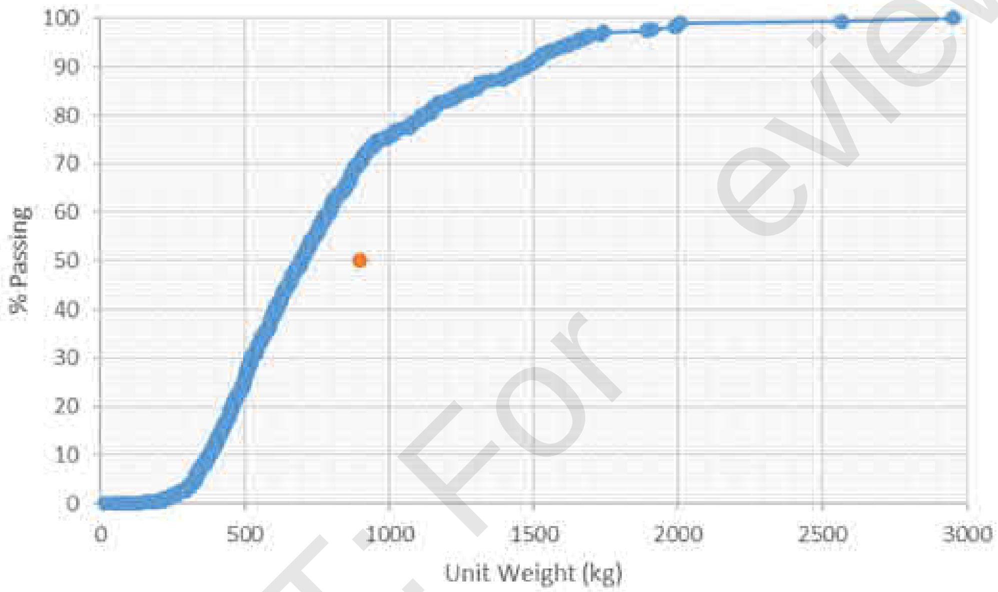


Figure 7: Sorted secondary armour rock, converted to prototype values (blue). Target point (orange).

Figure 8: Example of core (yellow) and filter rocks (red).

2.4 Construction of the bathymetry and breakwater cross sections

2.4.1 Representative bathymetry

As the primary objective for this physical modelling was to investigate the structural stability of the breakwater extension, it was important to measure the offshore waves over the flume bed to ensure comparable incident wave conditions. After discussion with the Client, it was agreed that the bathymetry may be reasonably represented by a simplified bathymetry featuring a planar slope that would result in representative wave transformation leading up to the structure (Figure 9). A 1/10 gradient transition slope connected the flume bed (bed elevation at −19.8 m LAT prototype elevation) to the model domain bathymetry. The main model domain bathymetry was constructed with a 1/60 gradient slope between the end of the transition slope and the structure.

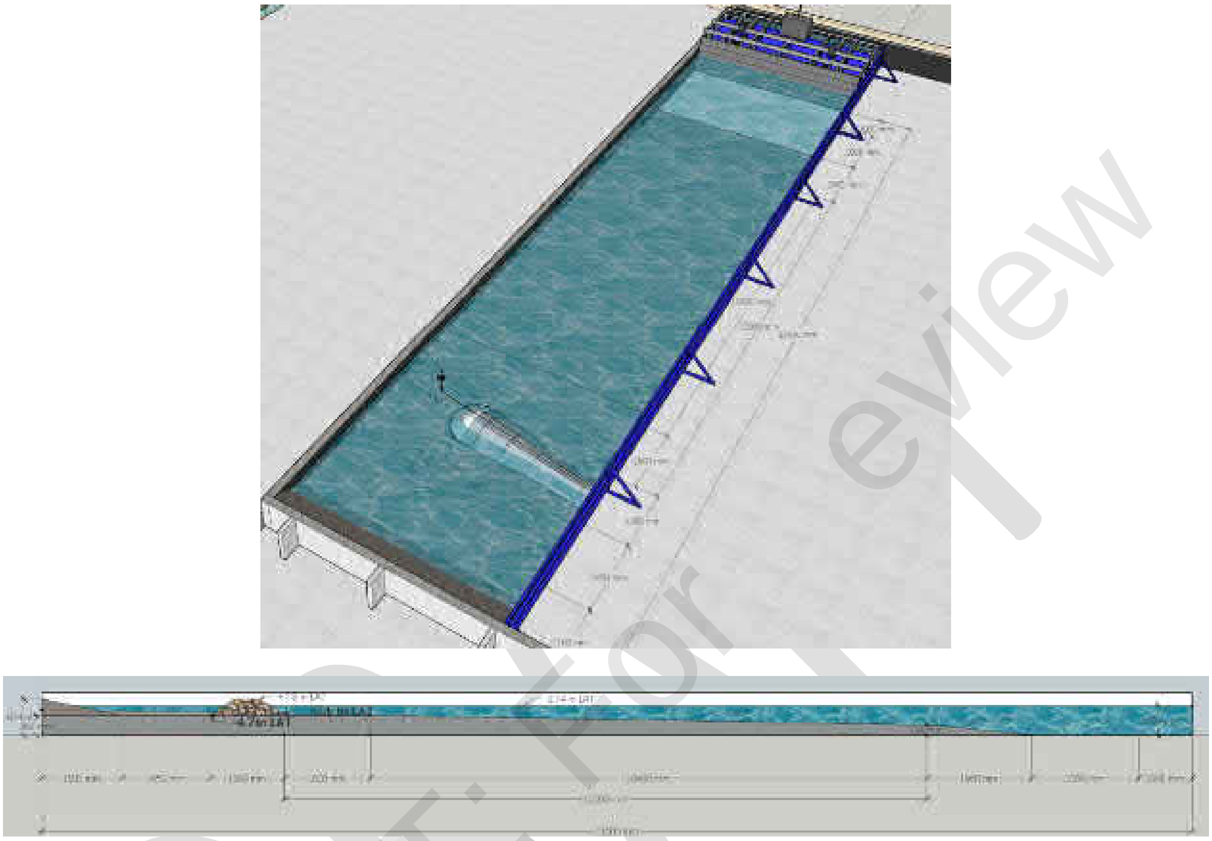


Figure 9: 3D and 2D view of Model domain.

The construction of the basin model was a multi-stage process. A supporting wall was installed first to contain the model bathymetry and support the breakwater. After the wall was installed, road base fill was introduced and compacted to ensure a stable base for the 50 mm concrete cap (Figure 10 to Figure 12). Prior to the concrete pour and screeding, wooden templates were installed on the side walls relative to a benchmark located in the centre of the flume on the flume floor. This benchmark (prototype elevation = -19.8 m LAT) defined the offshore water depth h_0, and was used to ensure consistency in design profile elevations through the templates and model construction.

Figure 10: Flume bathymetry construction - supporting walls installation, road base fill and compaction progressing.

Figure 11: Concrete pour progressing.

Figure 12: Completed concrete bathymetry.

Figure 13 shows the surveyed model bathymetry, including the 1/10 transition slope (measured as 1/12). The 2D profile plot has 8 profiles plotted at 0.5m increments over the middle 4 m of the basin to give an indication of the uniformity in the longshore. The profile slope over the model domain is between 1/58 and 1/61, which is in good agreement with the design (i.e. 1/60) and the horizontal section towards the right is approximately at the bedrock elevation and where the breakwater will be constructed before filling around the structure with a weak grout mix of sand and cement (sand-to-cement ratio ≈ 9:1) to continue the representative profile (Figure 9 and Figure 14).

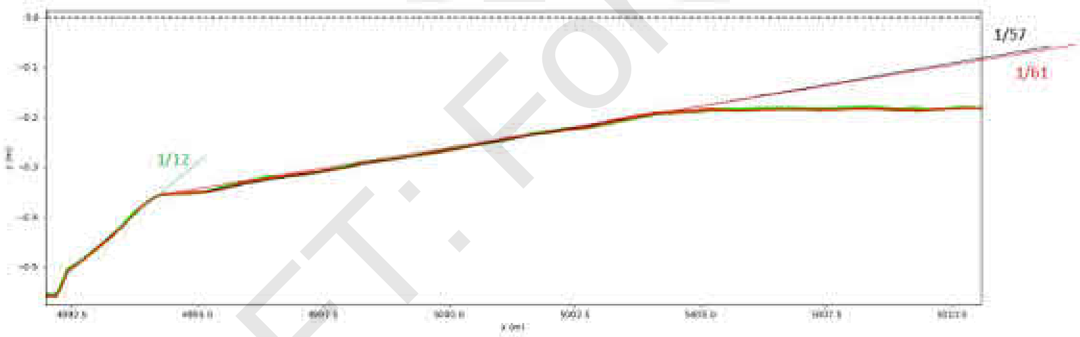


Figure 13: Surveyed flume bathymetry at 0.5m increments over the middle 4m of the basin.

Figure 14: Installation of the removable bathymetry (weak grout) (left). Completed bathymetry (right).

2.4.2 Breakwater cross sections

Table 3 details the three breakwater cross sections that were tested in the flume. Cross sections were built using templates that provided the outline for each of the layers in the cross sections. The core material was installed first, followed by the filter rock and finally the armour rock layer (Figure 15). The rocks were placed so that no rock units protruded above the respective design elevations of each layer.

Table 3: Three breakwater cross-section designs including description.

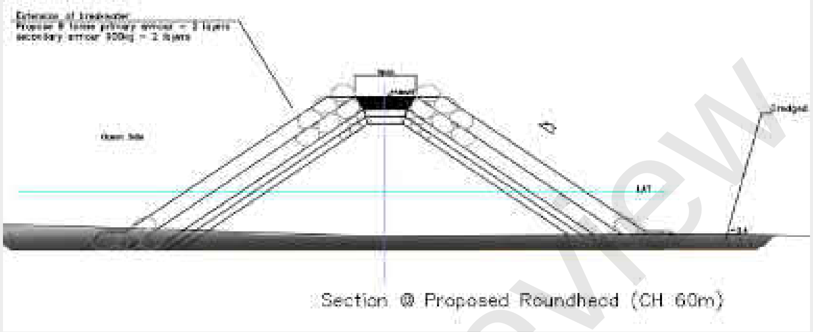
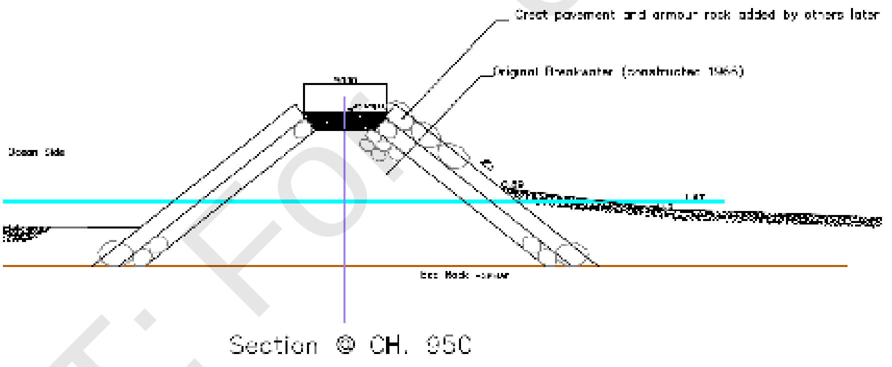
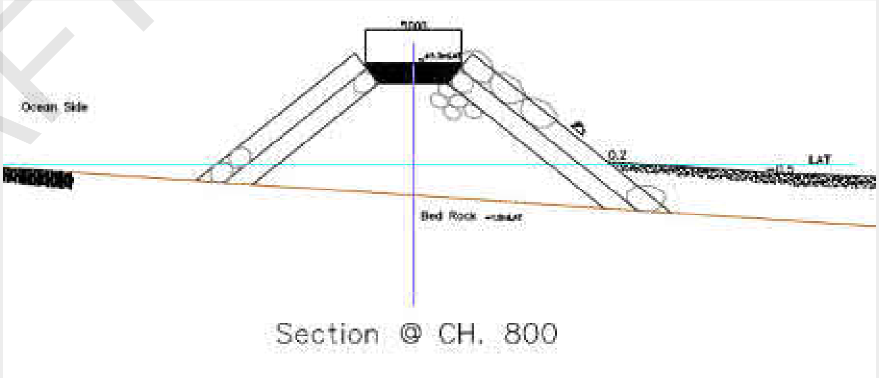
Cross-section	Design Description	Photo
CH 60	Breakwater: crest elevation +7.8m AHD, crest width 5 m, slope 1:1.5	
CH 950	Breakwater: crest elevation +5.4m AHD, crest width 5 m, slope 1:1.25	
CH 800	Rock revetment: crest elevation +5.3 m AHD, crest width 5 m, slope 1:1.25	

Figure 15: Photos depicting the installation of the core (top) and armour layers (bottom left) using templates. The footprint of the breakwater layers was drawn on the concrete floor as shown in the bottom right picture.

Following the installation of the armour layer, a sand and cement grout was placed along the length of the crest of the breakwater in the model to represent the 5 m-wide walkway in the prototype. To provide a similar build-approach to the prototype, geofabric was placed to contain the pour of the concrete path (Figure 16).

Figure 16: installation of the 5m wide walkway.

Following the installation of the breakwater, approximately 1,000 bedding-in waves were run to allow settlement of the structure prior to official testing. The drive signals produced for the 200-year ARI event (Source SOW) were used as bedding-in waves with a span (gain factor) set to 0.6 (i.e. 60% of the official test energy), followed by 0.8, for approximately 10 minutes each. Rocks were observed and confirmed to have settled during these waves prior to commencing of the official tests.

2.5 Instrumentation

Testing was performed in the QGHL 5 m wide wave flume. Instrumentation included cameras (still and video), wave paddles, resistance probes and conductivity meter.

2.5.1 Wave measurements

Offshore and depth-limited wave conditions were measured using six resistance wave probes. During testing, the wave probes were calibrated within 2 hours prior to testing. Figure 17 indicates the positions of the probes for Test 01. Figure 18 shows a photo of the model setup with probes deployed. The two offshore probes (S001 and S002) are visible in this photo, nearest the paddle

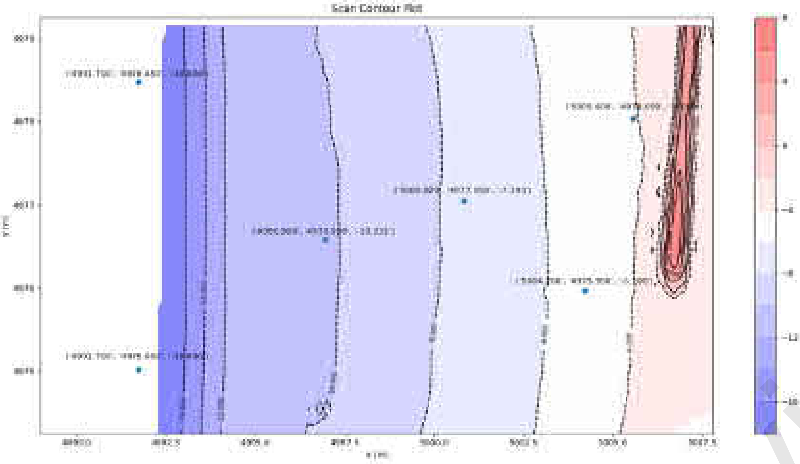


Figure 17. Contour lines of flume bathymetry and six wave probes locations (last number for each wave probes indicates floor elevation (m LAT)).

Figure 18: Flume model setup overview showing
the location of the six deployed wave probes (green circles).

The wave paddle was driven by the HR Merlin software to generate random JONSWAP spectra (Gamma = 3.3, i.e., default) waves. Wave drive signals were generated to match the H_{m0} specified for the offshore wave conditions (Table 1) through adjustment of the gain factor, if necessary.

Drive signals were generated to match offshore JONSWAP wave spectra (Figure 19). resistance probes were used to monitor the incident waves and wave propagation over the model bathymetry. The wave drive signal was generated during the setup stage of the project, with the breakwater in place. The breakwater was then rebuilt before commencing official testing (Test 1).

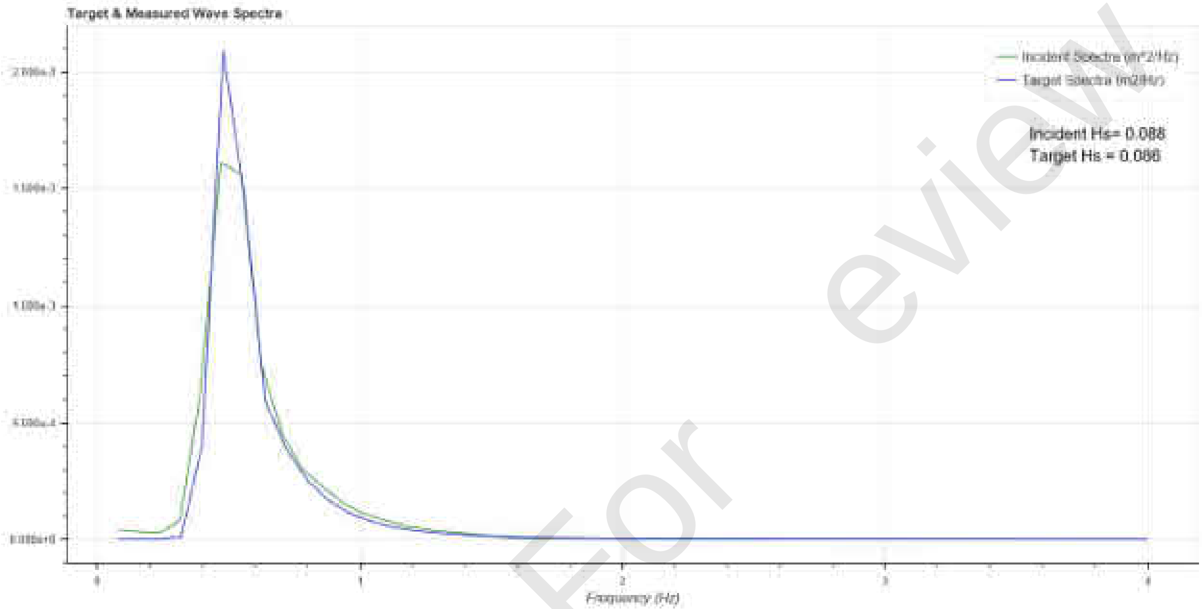


Figure 19: Example of measured and target JONSWAP spectrum for Test 001.

2.5.2 Photography and videography

Side placement and overhead view video cameras (in total three) documented each test. Photos were also taken opportunistically through testing. The overhead video camera was used to take before and after images of the structures to determine movement during testing. All before and after photos are provided in Appendix B.

Figure 20: Side view video cameras (left). A frame from the overhead video camera is also shown (right).

2.6 Data processing and analysis

This section describes the data processing and analysis which were used to obtain the wave and overtopping discharge data.

2.6.1 Wave data

Wave data was analysed using the DHI WS Online data acquisition package and MIKE Zero ocean wave analysis toolbox. The spectra and spectral parameters (spectral significant wave height H_{m0} and peak period T_p) were calculated for each wave probe for each test.

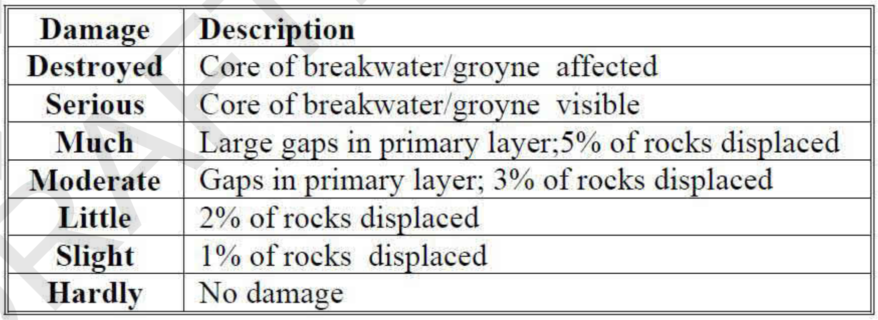
2.6.2 Stability analysis

Structural stability was assessed by visual observations of any rock movement during testing and the model runs were documented with video and before/after photography. The number of rocks rocking and displaced were determined by the before and after overhead photos.

Movement is described as either rocking or displaced. Rocking is defined as a single rock moving forward, backward or sideways, but remains in its approximate starting location (i.e. within half the rock's approximate diameter) at the end of the test. Displaced rocks are defined when a single rock moves further than half its approximate diameter from its initial location by the end of the test.

Following Hudson (1959), damage was quantified by the percentage of rocks (relative to the number of rocks on the top layer). The total number of rocks on the top layer of the structure was estimated by the exposed area divided by D_{50}^2. Damage was also classified following HR Wallingford (Report EX 6361, 2010), a description of the classification system is reproduced in Table 4.

Table 4: Damage classification in model breakwaters, source HR Wallingford Report EX 6361 (2010).



Damage	Description
Destroyed	Core of breakwater/groyne affected
Serious	Core of breakwater/groyne visible
Much	Large gaps in primary layer;5% of rocks displaced
Moderate	Gaps in primary layer; 3% of rocks displaced
Little	2% of rocks displaced
Slight	1% of rocks displaced
Hardly	No damage

3 Results

This section presents the results of the Mooloolaba physical model testing. The results of each test series are summarised in the following sections.

3.1 Wave analysis

The wave analysis results are provided in Table 5, along with some example plots of the surface elevation time series in Figure 21: Surface elevation time series sample for the offshore probe S1 (top), trunk probe S4 (middle), and roundhead probe S5 (bottom)., and the corresponding wave spectra in Figure 22: Wave Spectrum for offshore probe S1 (top), trunk probe S4 (middle), and roundhead probe S5 (bottom).. All other plots are available from the Data Analysis directory (MB08_DataAnalysis/WaveAnal/AnalysisFigures/) on the data drive. It is important to note that the results of the wave analysis are taken from single points in the basin and as such contain the incident and reflected wave components. This has resulted in the small increases in Hm0 as the waves propagate towards the structure. There is also a higher energy low-frequency component associated with probe S4, located in front of the roundhead (Figure 18). This feature is expected to be related to the probe location, which would likely have contained significant reflected wave components.

Table 5: Wave analysis outputs for each probe for 6T Rock Armour Test

Probe no.	S0	S1	S2	S3	S4	S5
Hm0	0.085	0.086	0.085	0.089	0.091	0.096
Tp	1.932	1.969	1.932	1.896	2.008	1.969
Hs	0.082	0.082	0.085	0.092	0.096	0.106
Have	0.053	0.052	0.053	0.057	0.059	0.065
Hmax	0.153	0.160	0.158	0.214	0.178	0.182
Tz	1.585	1.553	1.627	1.691	1.466	1.677

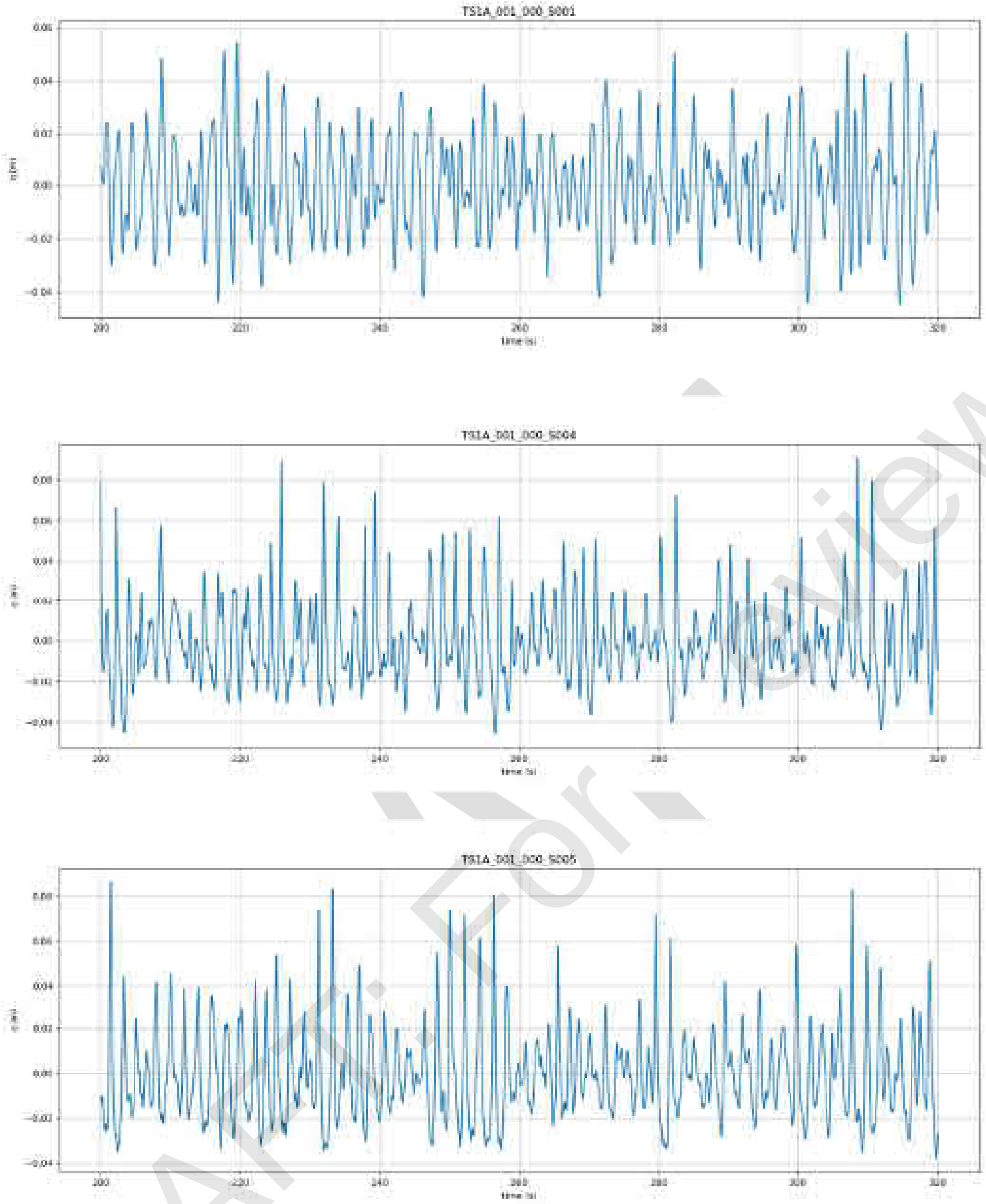


Figure 21: Surface elevation time series sample for the offshore probe S1 (top), trunk probe S4 (middle), and roundhead probe S5 (bottom).

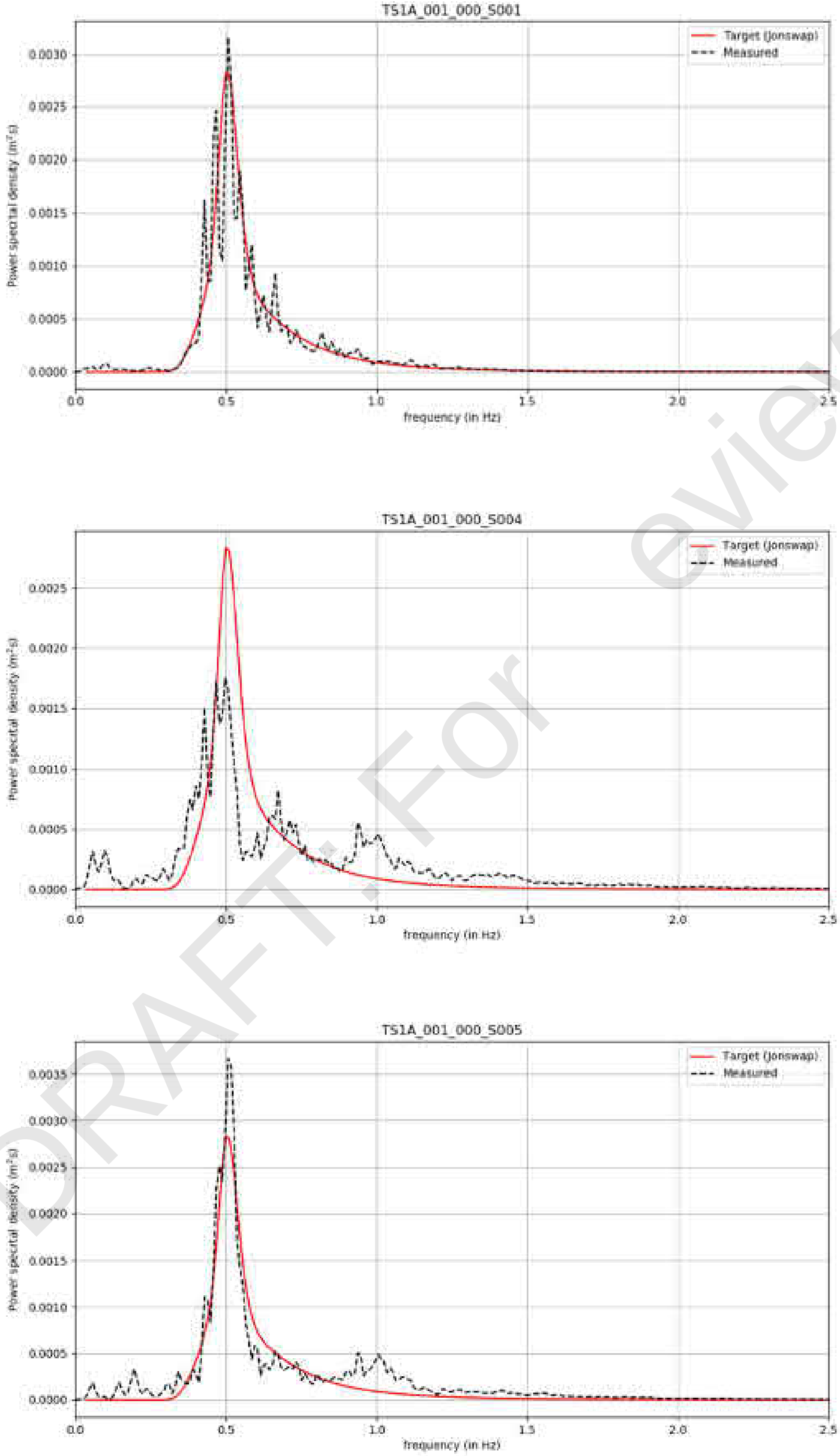


Figure 22: Wave Spectrum for offshore probe S1 (top), trunk probe S4 (middle), and roundhead probe S5 (bottom).

3.2 Structural damage _ Test 001

General observations made during testing included:

- Smaller rocks located on the face tended to move down-slope.
- Some rocks at the crest for existing breakwater section tended to move towards the lee side.
- Larger rocks tended to rock when unstable and would often settle into a more stable location after some time.
- The greatest damage was observed in these two areas:
  - The start of the transition section (end of section B, i.e. where the extension joins the existing).
  - On the lee-side of the roundhead (section H). This is a common and well-known point of weakness on breakwaters with a round head, located typically at an angle β of 90° to 135° from the wave direction (Hofland et al., 2014). The waves that travel over the side of the roundhead can form water jets that give large loads on the armour units.
- The existing section of the breakwater (crest at +5.4m LAT), experienced more wave overtopping than the extension section (crest at +7.8m LAT). This higher overtopping rate for the existing section may be the main contributor to the noticeable damage on the lee side. This is a common response of breakwaters that encounter regular wave overtopping, particularly of green water (Argente et al., 2018).
- Wave diffraction and refraction did not significantly contribute to the amount of damage in the lee side of the breakwater trunk.

Before and after each test series a 3D laser scan was obtained. Figure 23 presents a difference plot of the total change in the breakwater after Test 01. There was a fairly uniform reduction in elevations near the crest and increase in elevations towards the toe, indicating a down-slope displacement of rocks. During testing, rocks were also occasionally observed to be displaced from the face, washing leeward onto the walkway and further, to the lee side of the breakwater.

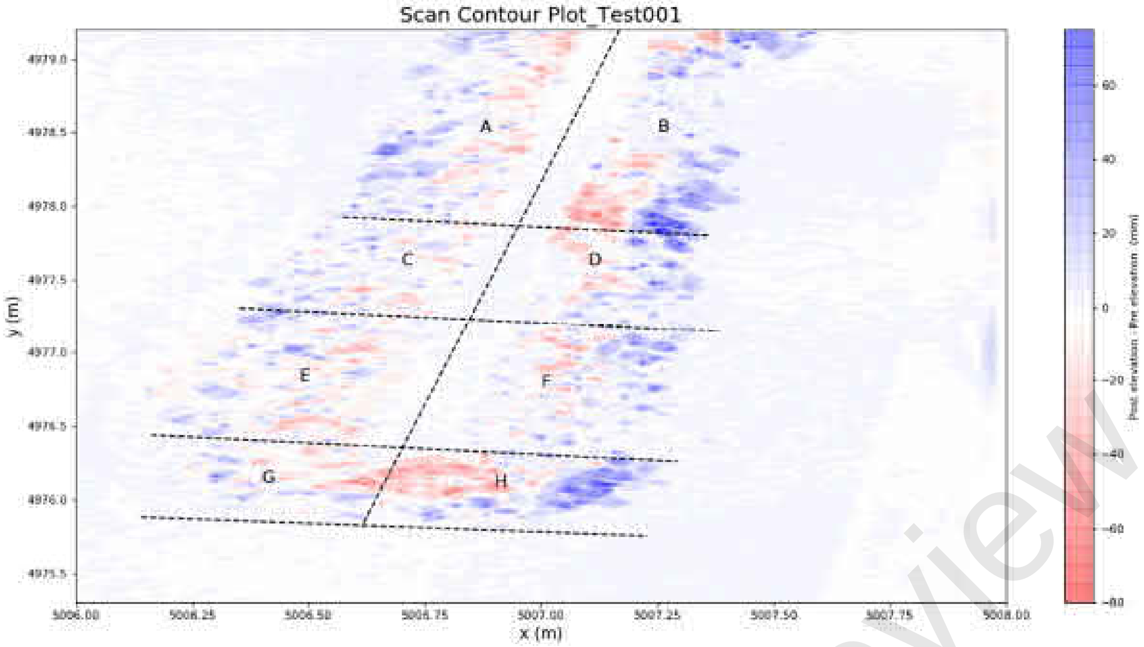


Figure 23: Difference plot of total accumulated structural changes before and after Test01, showing eroded and accumulated areas. Noticeable damage at section H and section B. (Description of the labels is provided in Table 6).

The overhead photo stability analysis was performed with the camera located above the structure (Figure 24) with a phot captured before and after each test. The number of rocks displaced and rocking were determined on the basis of those pictures, Table 6.

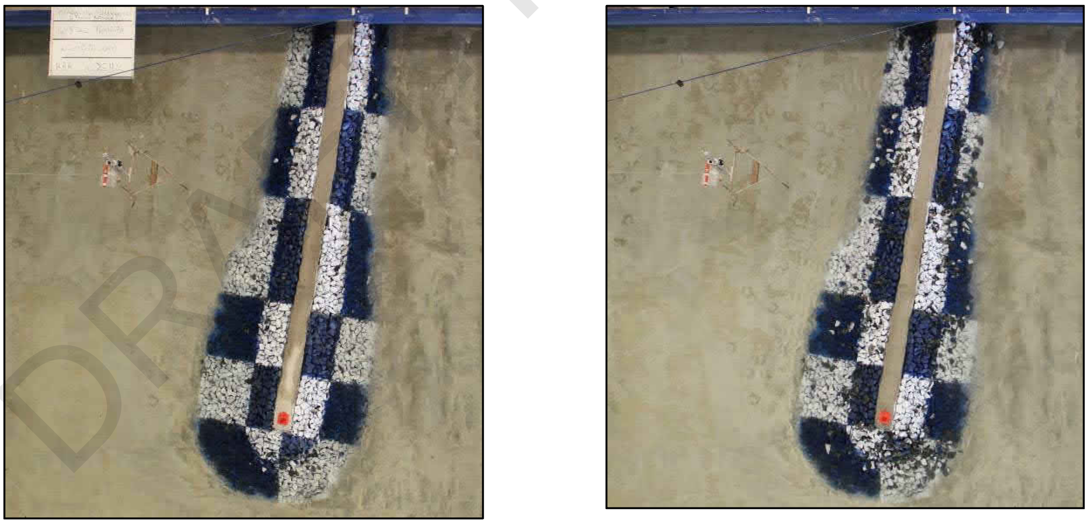


Figure 24: Overhead photo before Test01 (left) and after Test01 (right).

Table 6: Structural Damage Assessment Test Series 1.

Test001_000_20200221						
Section	Side	Label	Area (mm2)	Rocking	Displacement	Damage(%)
Section1_CH950&800	Front	A	516971	44	22	10
	Back	B	516971	41	32	15
Section2_Transition	Front	C	288035	36	14	12
	Back	D	288035	30	17	14
Section3_CH60	Front	E	462499	64	15	8
	Back	F	462499	58	14	7
Section4_Roundhead	Front	G	295260	30	9	7
	Back	H	295260	43	21	17

The greatest damage was observed for the roundhead (section H), with 17% of rocks displaced and a relative damage classification of “Much”. The filter layer was exposed in some spots of this section.

3.3 Structural damage _ ReTest 001

After conducting the Test 01, an issue was recognised with the resistance wave probes. The probe calibrations likely drifted during the test, which is thought to be due to temperature changes in the water affecting the conductivity. This may have resulted in slightly larger offshore H_{m0} = 0.091 m (target was 0.086 m) being recorded. The wave probes returned a +20% error (reading ~120mm when offset by 100mm) after approximately three hours following the calibration. Although this issue with the wave probes did not affect the wave paddle performance and hence the actual wave height have met the target wave height, it was decided to perform a repeat test to increase confidence in results.

By overfilling the basin the day before and circulating through the filtration system overnight and running the test first thing in the morning, the occurrence of the same conductivity issue was avoided. This ensured the water is well mixed and minimised potential effects of daytime temperature fluctuations.

The results of the repeat test can be seen in Appendix A. overall the damage magnitude was lower for the repeat test which is thought to be due to the settlement of the core and underlayer after Test01. This settlement provided a stronger base for the armour layer to settle in. The greatest damage areas remained almost the same between these two tests as follows:

- On the lee side of the roundhead (section H).

- At the end of the existing section where the transition section starts (section B).

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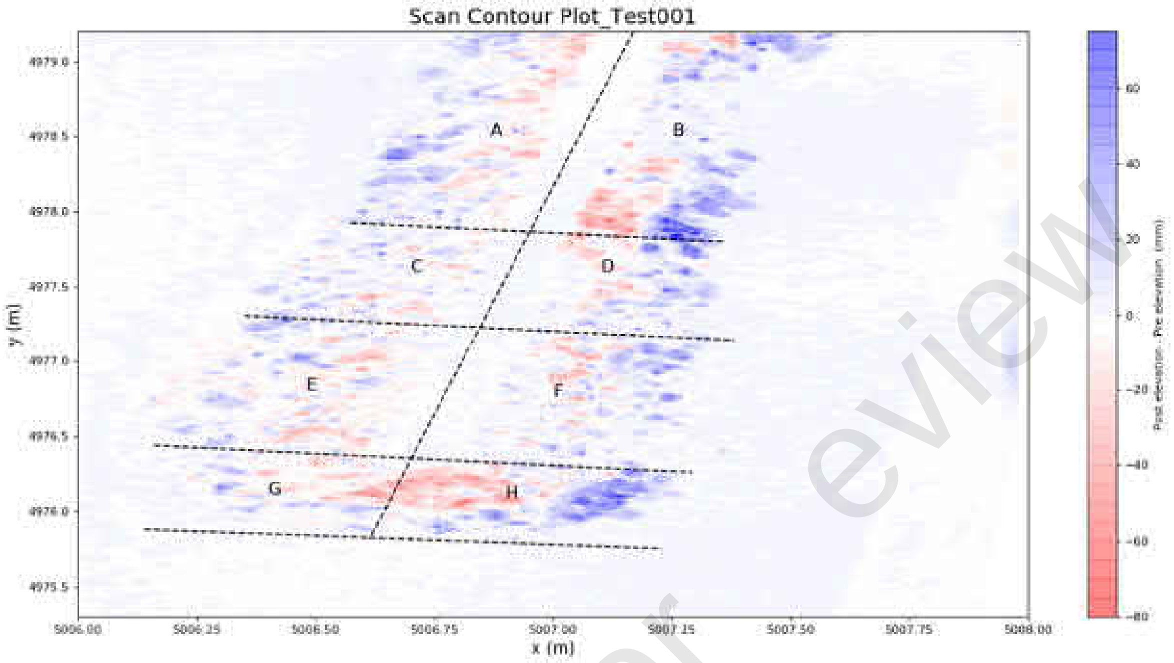
Appendix A: All Testing Results Overview

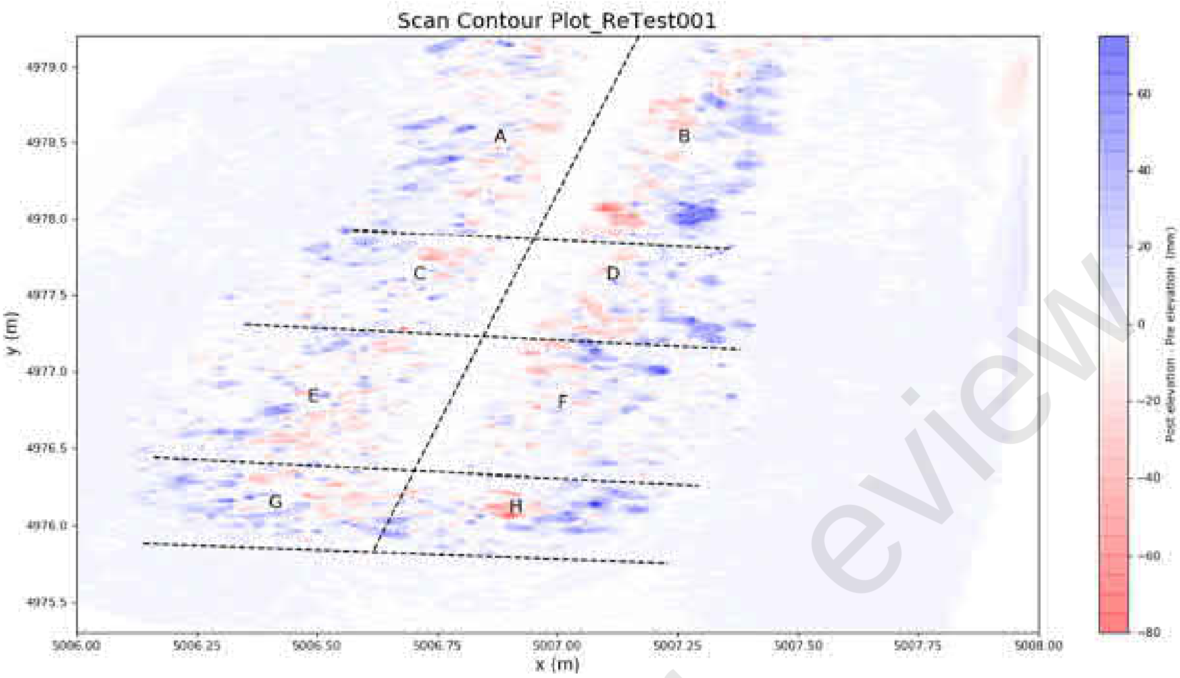
Test0_ Difference plot of structural changes



Test0_ Damage analysis

Test001_000_20200221						
Section	Side	Label	Area (mm2)	Rocking	Displacement	Damage(%)
Section1_CH950&800	Front	A	516971	44	22	10
	Back	B	516971	41	32	15
Section2_Transition	Front	C	288035	36	14	12
	Back	D	288035	30	17	14
Section3_CH60	Front	E	462499	64	15	8
	Back	F	462499	58	14	7
Section4_Roundhead	Front	G	295260	30	9	7
	Back	H	295260	43	21	17

ReTest0_ Difference plot of structural changes



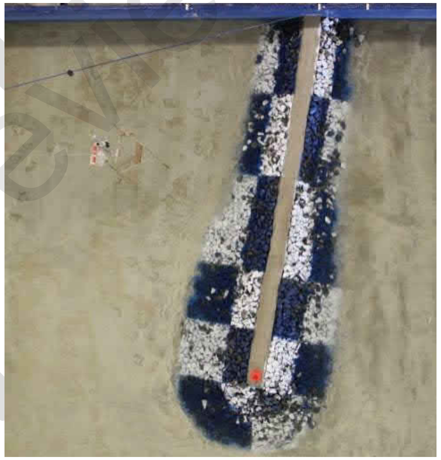
ReTest0_ Damage analysis

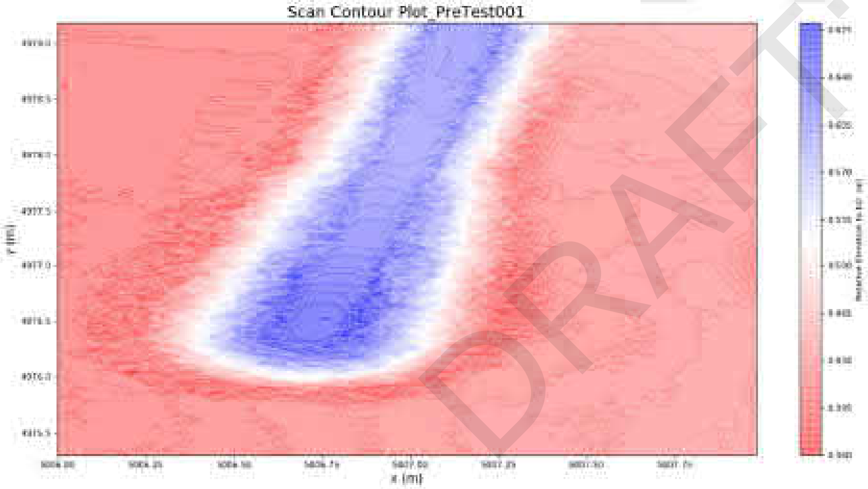
ReTest001_001_20200227						
Section	Side	Label	Area (mm2)	Rocking	Displacement	Damage(%)
Section1_CH950&800	Front	A	516971	74	17	8.0
	Back	B	516971	60	21	9.9
Section2_Transition	Front	C	288035	42	4	3.4
	Back	D	288035	44	11	9.3
Section3_CH60	Front	E	462499	78	8	4.2
	Back	F	462499	65	9	4.7
Section4_Roundhead	Front	G	295260	37	5	4.1
	Back	H	295260	50	13	10.7

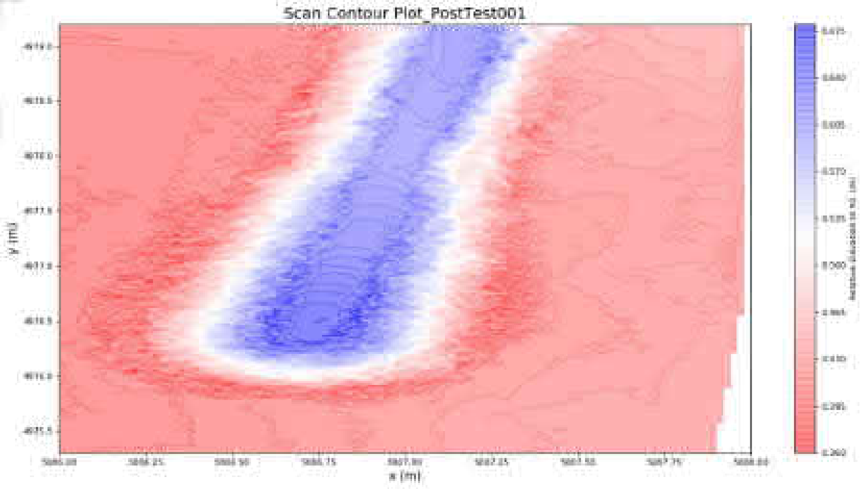
Appendix B: Before and after photography and scan

Test01_Pre

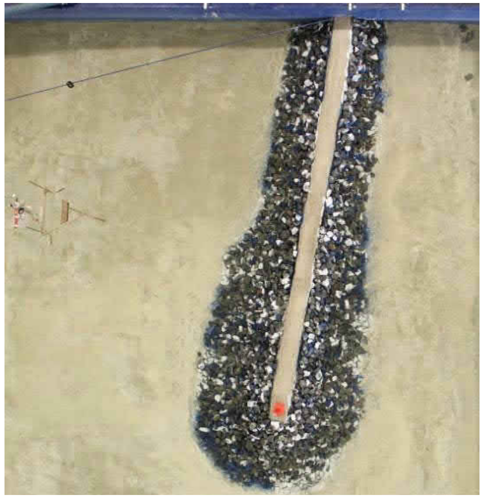
Test01_Post



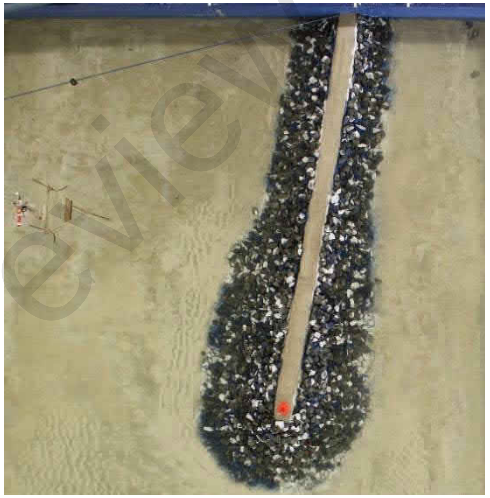


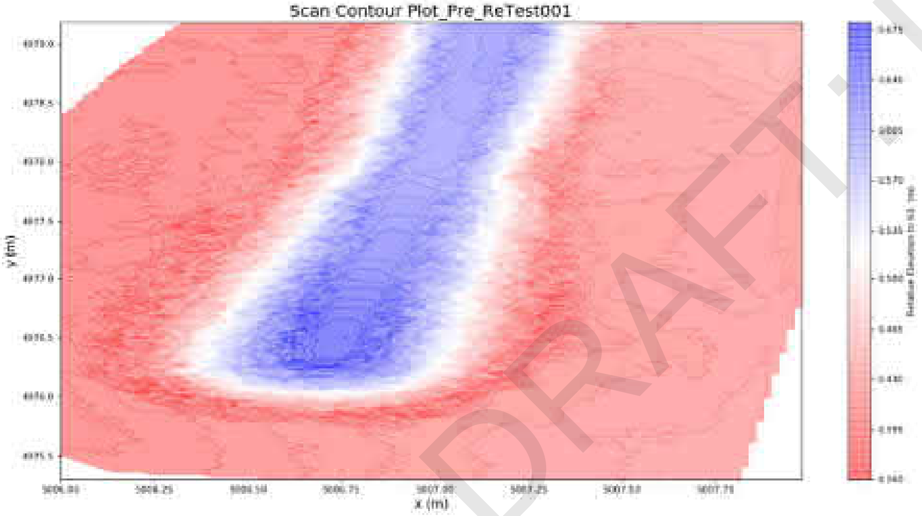


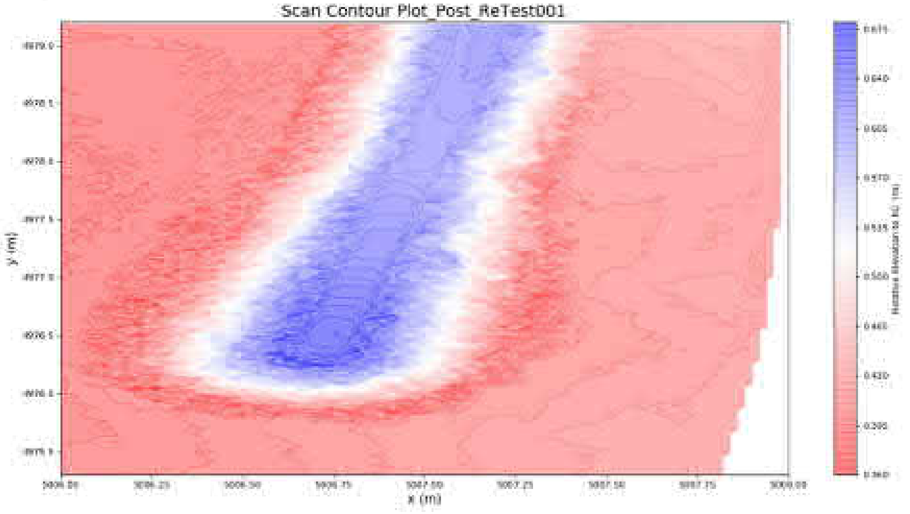
ReTest01_Pre



ReTest01_Post

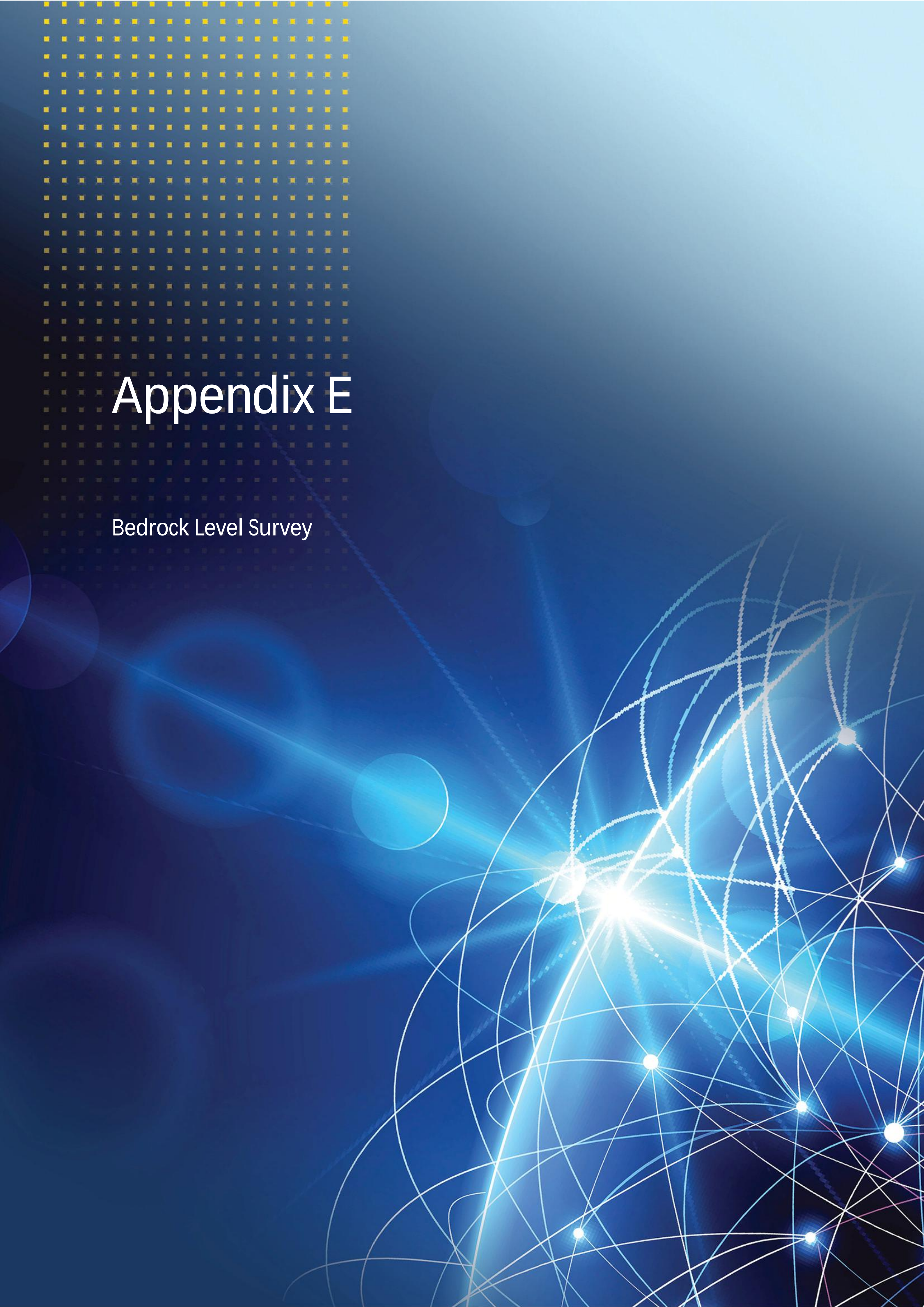






Appendix E

Bedrock Level Survey



Mooloolah River Entrance
Sand Depth Survey

Prepared for:

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Contents

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Figures

Summary

This report presents the results of the sediment sampling work completed by frc environmental on behalf of the Department of Transport and Main Roads (DTMR). The objective of the work was to determine the depth of sediment above bedrock at the mouth of the Mooloolah River, as DTMR are planning to place a sand extractor device on the eastern side of the river mouth to remove sand before it moves into the river mouth.

The depth of sediment varied from 0.2 m to 2.1 m above the bedrock. The greatest depth of sediment was found at points PT 4, PT 9 and PT 15 (closer to the middle of the river mouth). The depth of bedrock varied from 3.2 to 4.9 m below LAT.

1 Introduction

This report presents the results of the sediment sampling work completed by frc environmental on behalf of the Department of Transport and Main Roads (DTMR). The objective of the work was to determine the depth of sediment above bedrock at the mouth of the Mooloolah River, as DTMR are planning to place a sand extractor device on the eastern side of the river mouth to remove sand before it moves into the river mouth.

1.1 Background

The Mooloolah River entrance is between Point Cartwright and Mooloolaba Beach on the Sunshine Coast and is maintained by DTMR. The entrance periodically, and at short notice, requires maintenance dredging due to shoals of marine sand being transported around Point Cartwright. Siltation mechanisms at the entrance to the Mooloolah River are episodic, and are associated with longshore sediment transport around Point Cartwright, which is dependant on favourable wave conditions primarily from the south-easterly direction.

The Mooloolah River entrance is the main entrance for access to the state boat harbour facilities. In addition to these public facilities, the boat harbour is a port for pilot vessels that guide trade ships into the Port of Brisbane. Other industries also rely on the river entrance, most notably commercial fishing, recreational fishing and boating. The use of the harbour depends on the maintenance of navigable depths in the Mooloolah River entrance. It is therefore essential to continue to carry out periodic maintenance dredging to preserve the harbour’s function and preserve the viability of the industries it supports.

2 Methods

The depth of sediment and depth to bedrock was assessed at 12 points on the eastern side of the river mouth (Table 2.1 & Figure 2.1). The depth of sediment was measured by penetrating the sediment with a standard vibracorer, until it hit bedrock. The time and water depth was noted, and the depth of bedrock relative to Lowest Astronomical Tide (LAT) was calculated using tide gauge data from Mooloolaba provided by Maritime Safety Queensland.

The work was completed on 11 May 2012. Conditions were not optimal for vibracoring as there was an easterly swell of 0.5 to 1.2 m; however, work was completed between swell sets.

The vibracorer was unable to penetrate areas where rock or coarse gravel and boulders are present. In this case a gradated stainless steel rod was used to probe the depth of sediment.

Table 2.1 GPS position of each site.

Sample Point	Easting [a]	Northing
PT 1	513174	7049176
PT 2	513145	7049146
PT 3	513120	7049122
PT 4	513133	7049108
PT 5	513159	7049134
PT 6	513189	7049169
PT 7	513207	7049143
PT 8	513179	7049116
PT 9	513154	7049090
PT 10	513203	7049123
PT 11	513219	7049107
PT 12	513153	7049194
PT 13	513106	7049154
PT 14	513092	7049133
PT 15	513088	7049091

[a] UTM WGS 84 Zone 56; ± 4 m position accuracy

Figure 2.1 Sample points at the Mooloolah River mouth.

3 Results

3.1 Sediment Depth

The depth of sediment was greatest at PT 15 (2.1 m) to the west and shallowest at points PT 1 and PT 12 to the east. The depth of bedrock ranged from 3.2 to 4.9 m below LAT (Table 3.1). The eastern area in the vicinity of PT 1 has been assessed for sediment depth previously, and there was typically less than 0.5 m of sand covering the bedrock (frc environmental 2009; data from Site 4 in this survey which is 20 m northwest of PT 1 Appendix A). This eastern area typically has small rocky outcrops that extrude out of the sand and can be seen as darker patches on the map above (Figure 2.1).

Table 3.1 Depth of sediment and bedrock below LAT.

Sample Point	Depth of Sediment (m) (± 0.25 m)	Depth of Bedrock (m below LAT) (± 0.5 m)
PT 1 [a]	0.25	3.3
PT 2	0.75	4.0
PT 3	1.25	4.5
PT 4	2.00	4.3
PT 5	1.75	4.7
PT 6	1.25	4.3
PT 7	1.75	3.7
PT 8 [b]	–	–
PT 9	2.00	4.1
PT 10 [b]	–	–
PT 11 [b]	–	–
PT 12	0.25	4.5
PT 13	0.75	4.5
PT 14	1.00	4.3
PT 15	2.25	4.9

[a] area could not be sampled with the vibracorer due to the nature of the sediment; a gradated stainless steel rod was used to probe the depth of sediment

[b] point not sampled due to unsafe operating conditions: proximity to the breakwall, swell and breaking waves, or bedrock was at the surface

– not sampled

The depth of sediment at PT 4 was 2 m ± 0.25, which is similar to the depth of sediment previously found in that area in 2009 (frc environmental 2009; data from Site 2, approximately 15 metres north of PT 4). In contrast, sediment depth at PT 3 (also within 15 m of Site 2 surveyed in 2009) was slightly less at 1.25 m ± 0.25.

A reassessment of the depth of bedrock measured in the 2009 survey, accounting for potential error in the sampling location (± 4 m) and based on the May 2009 survey plan E251-365, indicates that the depth of bedrock ranges from 5.6 m to 6.1 m ± 0.2 m below LAT. Based on the depth of bedrock measured in the 2009 survey, there appears to be isolated patches of deeper sediment, particularly 15–20 m north of PT 4 (Appendix A, data from Site 2; frc environmental 2009). While this exact point was not surveyed in 2012, we would expect a sediment depth in this location to be greater than 3 m.

Limitations

The vibracore was not used at PT 1 due to the shallow depth of sediment and presence of rocks, which cannot be penetrated. In this case we used a narrow stainless steel probe, which could pass between some of the rocks in this area. However, the probe is not heavy and does not vibrate, so the measurement reported would be a minimum depth of sediment at that point.

If a layer of gravel or rock was present under the sand at any point, the depth of bedrock presented would be a minimum measurement below LAT.

Sampling at points PT 8, PT 10 and PT 11 was not completed due to unsafe operating conditions: swell and waves breaking on the point, or because the bedrock was at or near the surface.

4 Conclusion

The depth of sediment varied from 0.2 m to 2.1 m above the bedrock. The greatest depth of sediment was found at points PT 4, PT 9 and PT 15 (closer to the middle of the river mouth). The depth of bedrock varied from 3.3 to 4.9 m below LAT at the points surveyed.

Based on comparisons with previous surveys in 2009, the depth of sediment appears to be variable across the harbour entrance, with isolated sections of deeper bedrock, especially 15 m north of PT 4 where the sediment depth is expected to be greater than 3 m based on the results of the 2009 survey.

5 Reference

frc environmental, 2009, *Mooloolah River Entrance Sediment Sampling and Analysis*, report prepared for Queensland Department of Transport and Main Roads, November 2009.

Appendix A Sites Surveyed in 2009 by frc environmental

frc environmental
AQUATIC ECOLOGISTS
deep thinking.science.

Mooloolah River Entrance Sediment Sampling & Analysis

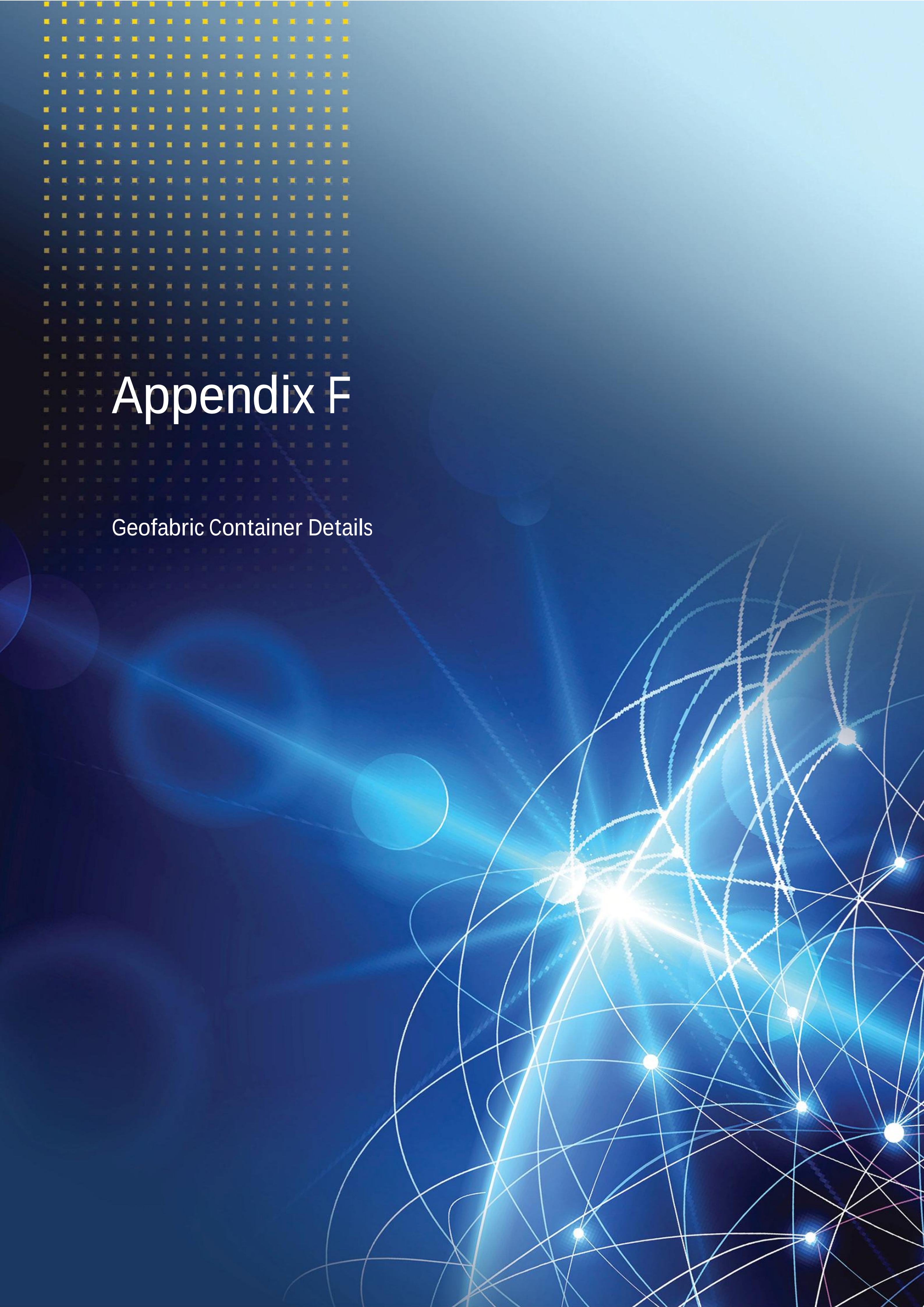
Figure A.1 Sites surveyed in by frc environmental in 2009.

Google Earth 2009

November 2009

Appendix F

Geofabric Container Details



ENGINEERED GEOSYNTHETICS

Heavy Duty Marine Geobags

For Applications Requiring Precise Underwater Placement

TENCATE
materials that make a difference

Heavy Duty Marine Geobags

For Applications Requiring Precise Underwater Placement

- Manufactured from robust heavy duty woven PP fabrics designed for use in marine applications.
- Available in standard sizes or supplied as bespoke units according to engineers design
- Designed to be lifted into position and precisely placed.
- Suitable for filling with a wide range of fill material including sharp coral stone.
- Designed to withstand heavy marine sea conditions and robust site construction practice.
- Easy to fill and install.

Heavy Duty Marine Geobags

Standard Items – Custom sized bags can be fabricated

		1.0 m^3 bag		**2.5 m^3 bag**		**4.0m^3 bag**	
		Type 1	Type 2	Type 1	Type 2	Type 1	Type 2
Robustness		Robust	Extremely Robust	Robust	Extremely Robust	Robust	Extremely Robust
Item Code		GB580MSS1	GB600MSS1	GB580MSS2	GB600MSS2	GB580MSS3	GB600MSS3
UV Stability		Highly Stabilized	Extreme High Stability	Highly Stabilized	Extreme High Stability	Highly Stabilized	Extreme High Stability
Width	m	1	1	1	1	1.6	1.6
Length	m	1	1	2.5	2.5	1.6	1.6
Height	m	1	1	1	1	1.6	1.6
Capacity	m^3	1.0	1.0	2.5	2.5	4.0	4.0
Weight of bag = Filled Volume x 1.8 ton/m^3	ton	2	2	4.50	4.50	7.37	7.37
Fill Ports	nos.	1 x 0.7m	1 x 0.7m	2 x 0.7m	2 x 0.7m	1 x 0.7m	1 x 0.7m
Tensile strength of the strap	tons	2.5	2.5	4	4	5.5	5.5
Nos. of Lifting Straps	nos.	4	4	6	6	8	8
Strength of Lifting Strap	ton	10	10	24	24	44	44
FOS*		5.6	5.6	5.3	5.3	6.0	6.0

* Lifting strap FOS compliant with Australian regulations

ENGINEERED GEOSYNTHETICS

Case Study

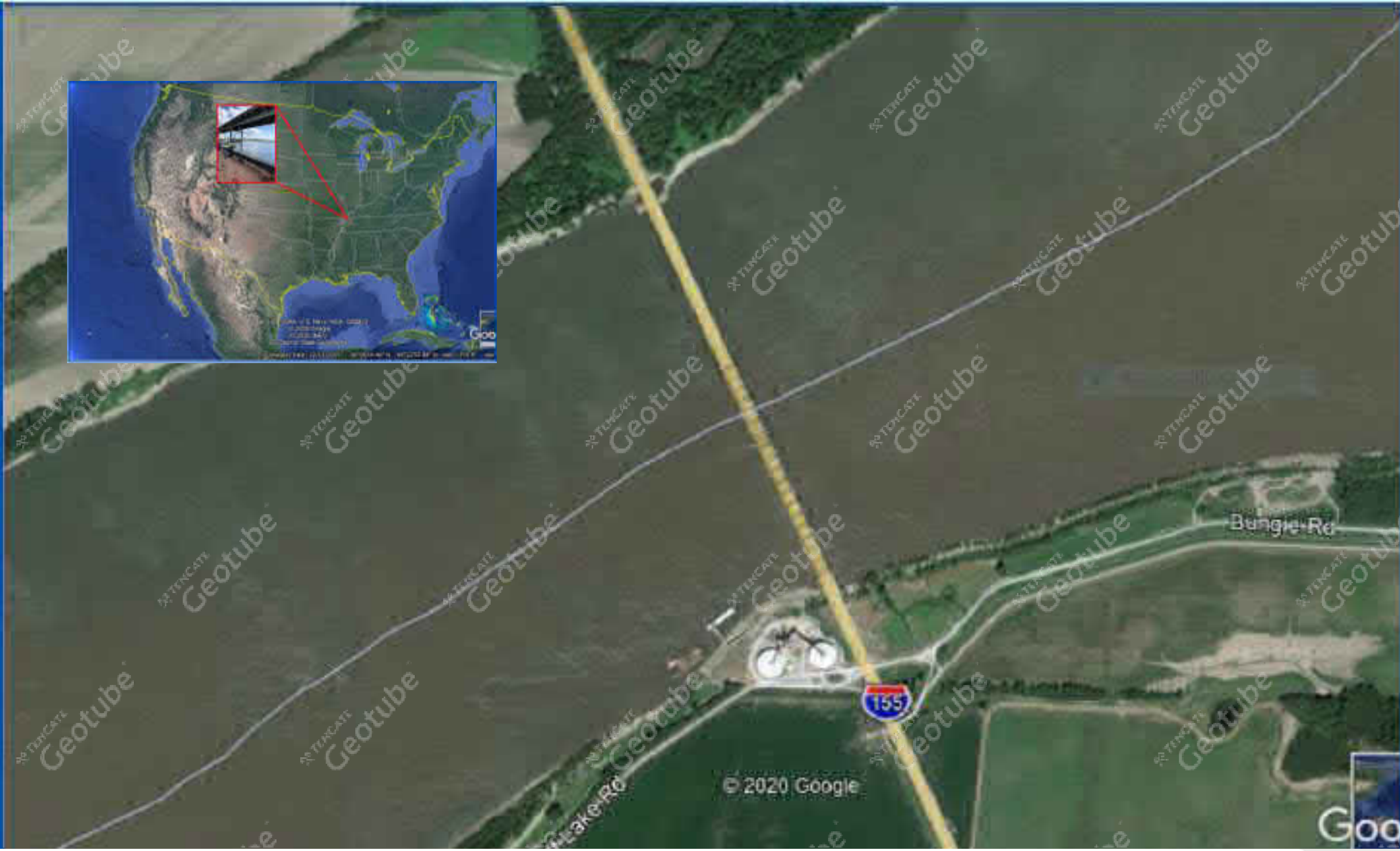
The application of TenCate Geobags for bridge pier scour protection

I - 155 Mississippi River, Dyer County, Tennessee USA

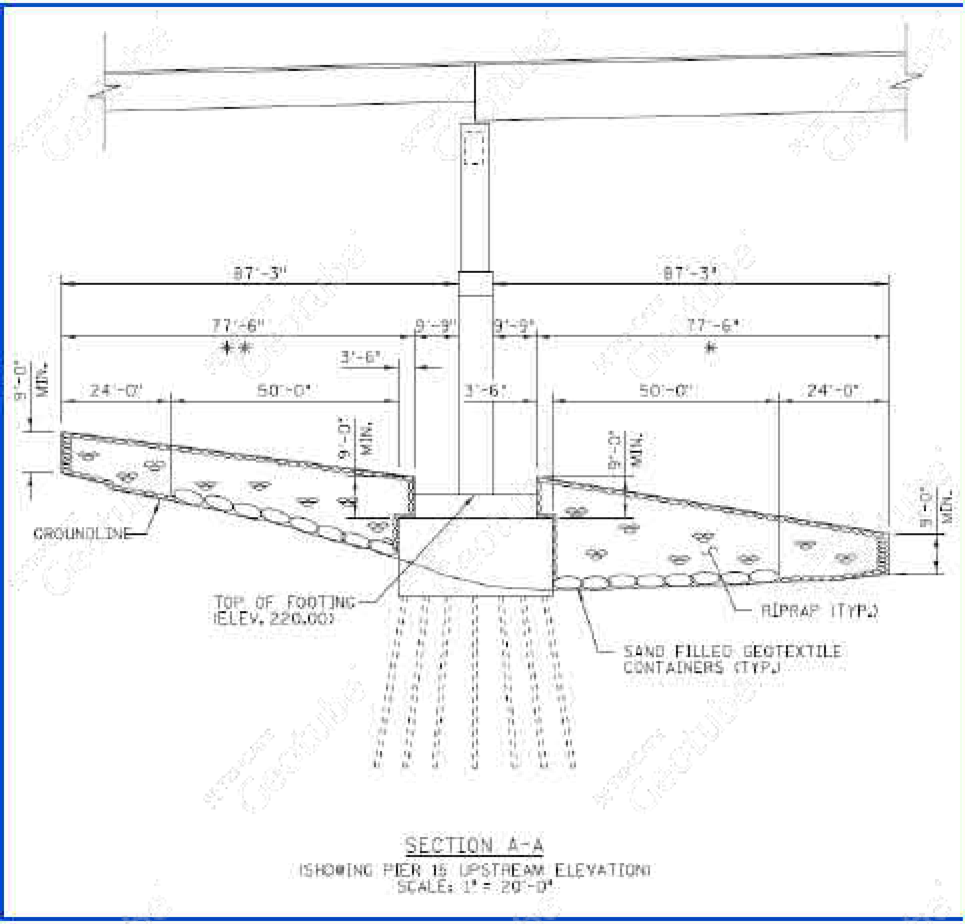
TENCATE
materials that make a difference

Application of TenCate Geobags for bridge pier scour protection,

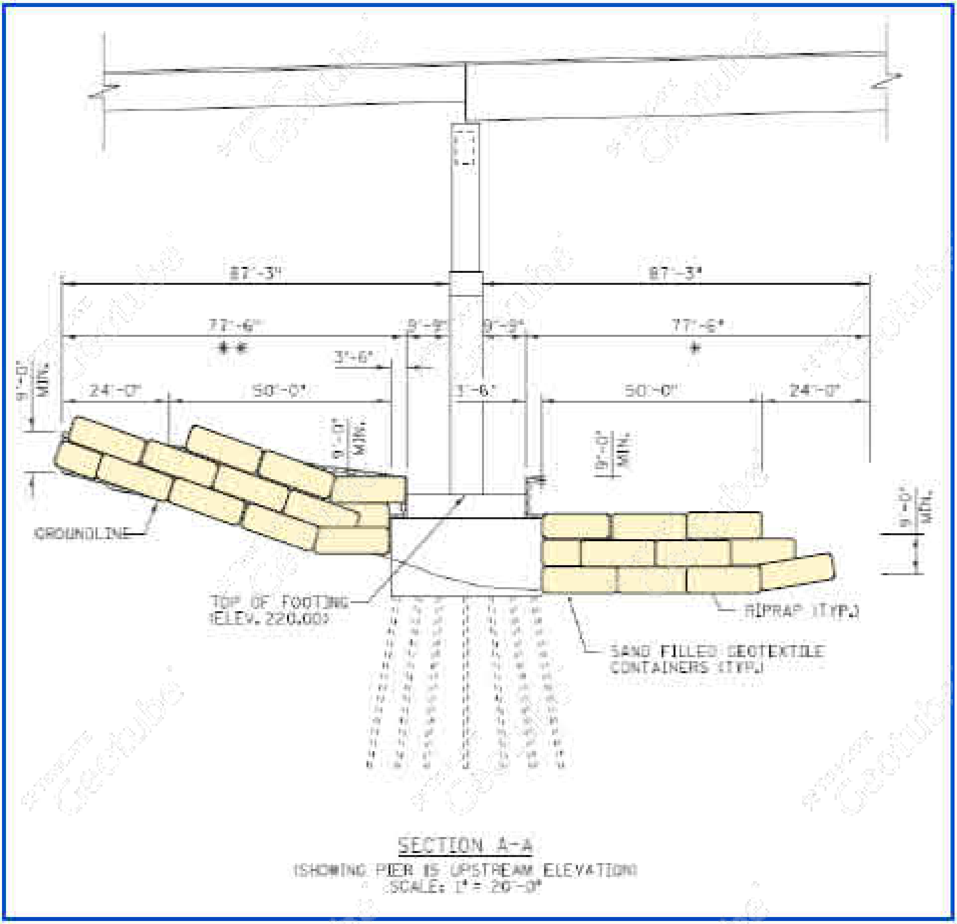
Tennessee USA



Proposed Deployment Location of TenCate Geobags Scour Protection

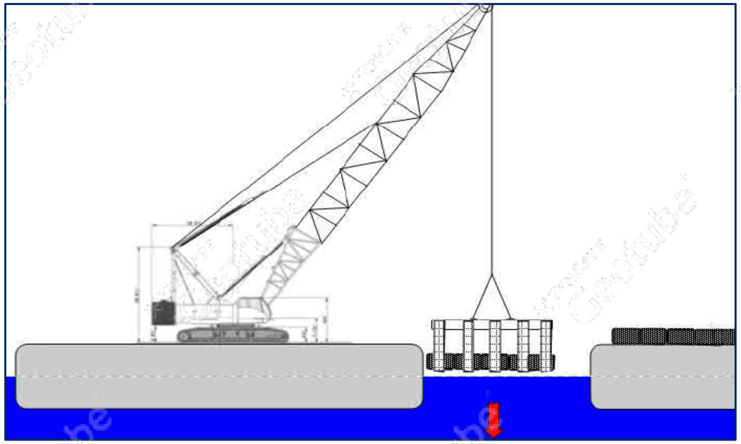


SECTION A-A
(SHOWING PIER IS UPSTREAM ELEVATION)
SCALE: 1" = 20'-0"

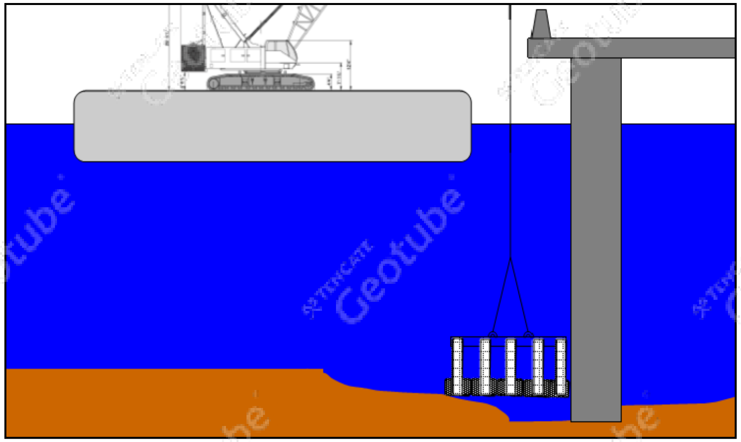


SECTION A-A
(SHOWING PIER IS UPSTREAM ELEVATION)
SCALE: 1" = 20'-0"

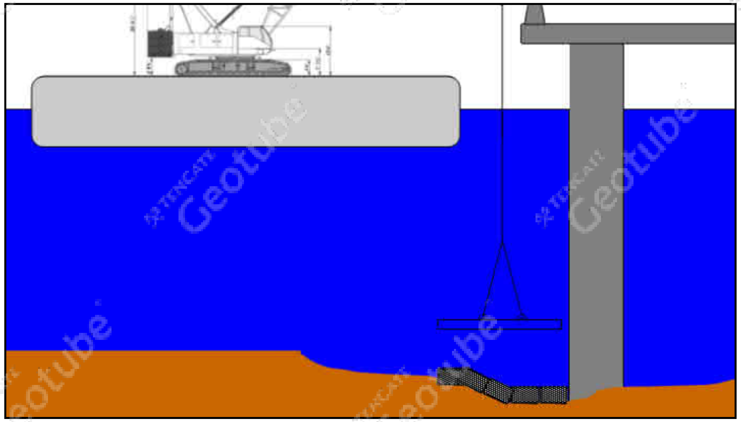
Placement Technique Applied by Contractor



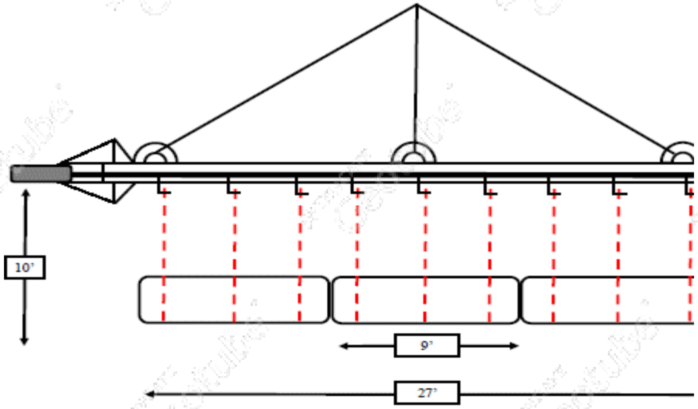
Positioning of Geobags fixed to deployment sling



Positioning of Geobags fixed to deployment sling

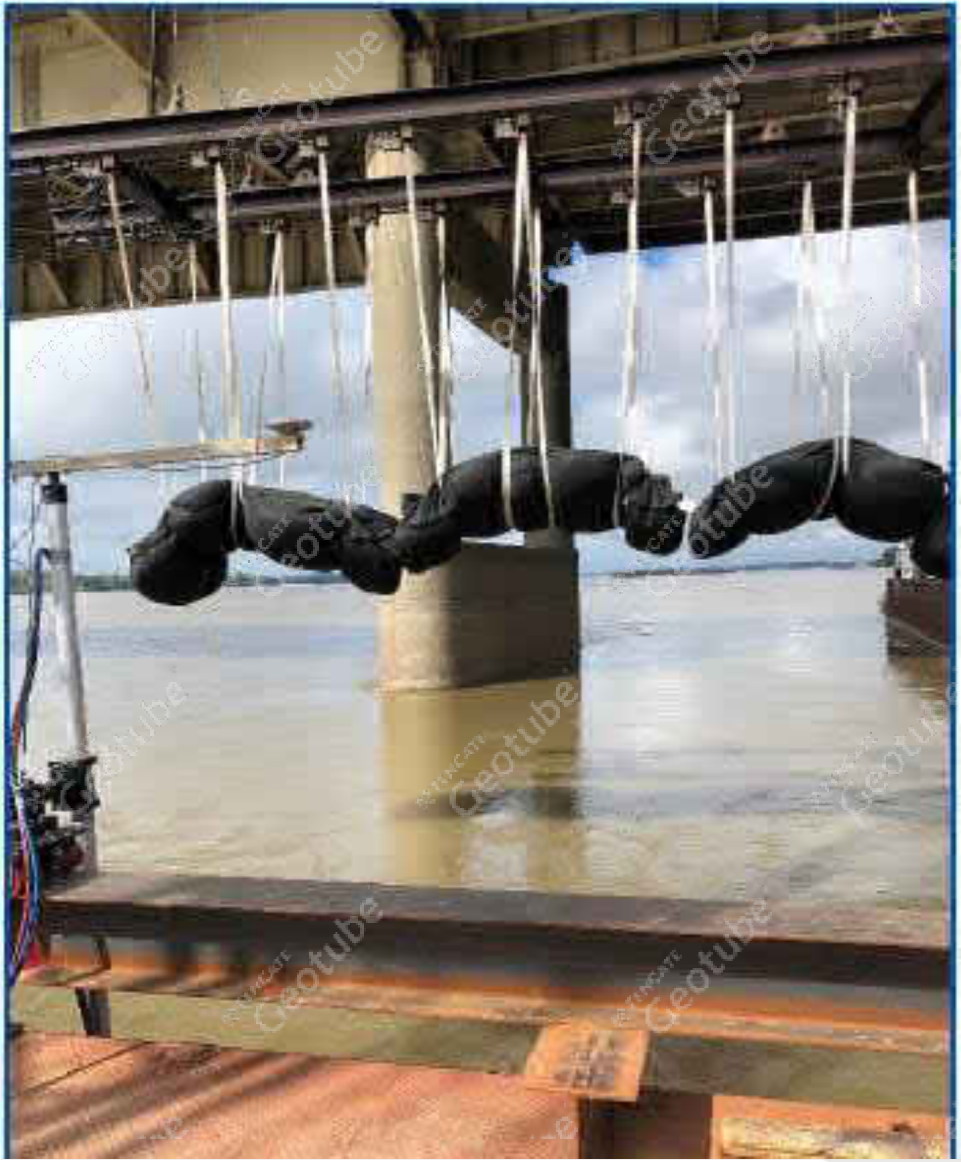


Release of Bags on riverbed

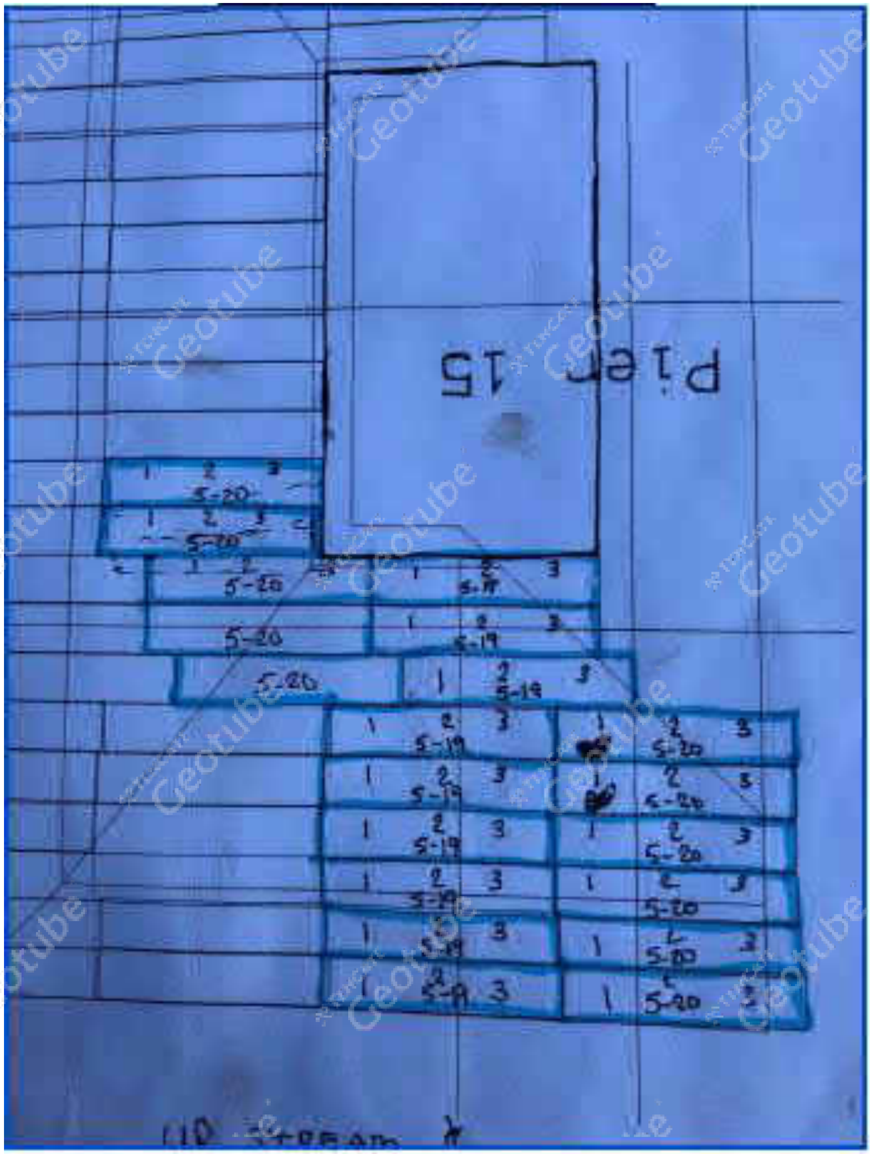


Deployment Sling & Bag Configuration

GPS Locator and Geobag Placement



Engineers Geobag Placement Plan



ENGINEERED GEOSYNTHETICS

Case Study

Application of TenCate Geobags as a platform layer for a Geotube® bunding structure over soft marine clay

Matabari Power Plant, Bangladesh

TENCATE
materials that make a difference

Matabari Power Plant, Bangladesh

Tencate Geobags were utilized as part of an integrated Geotube® system in the construction of Temporary Dyke and Breakwaters.

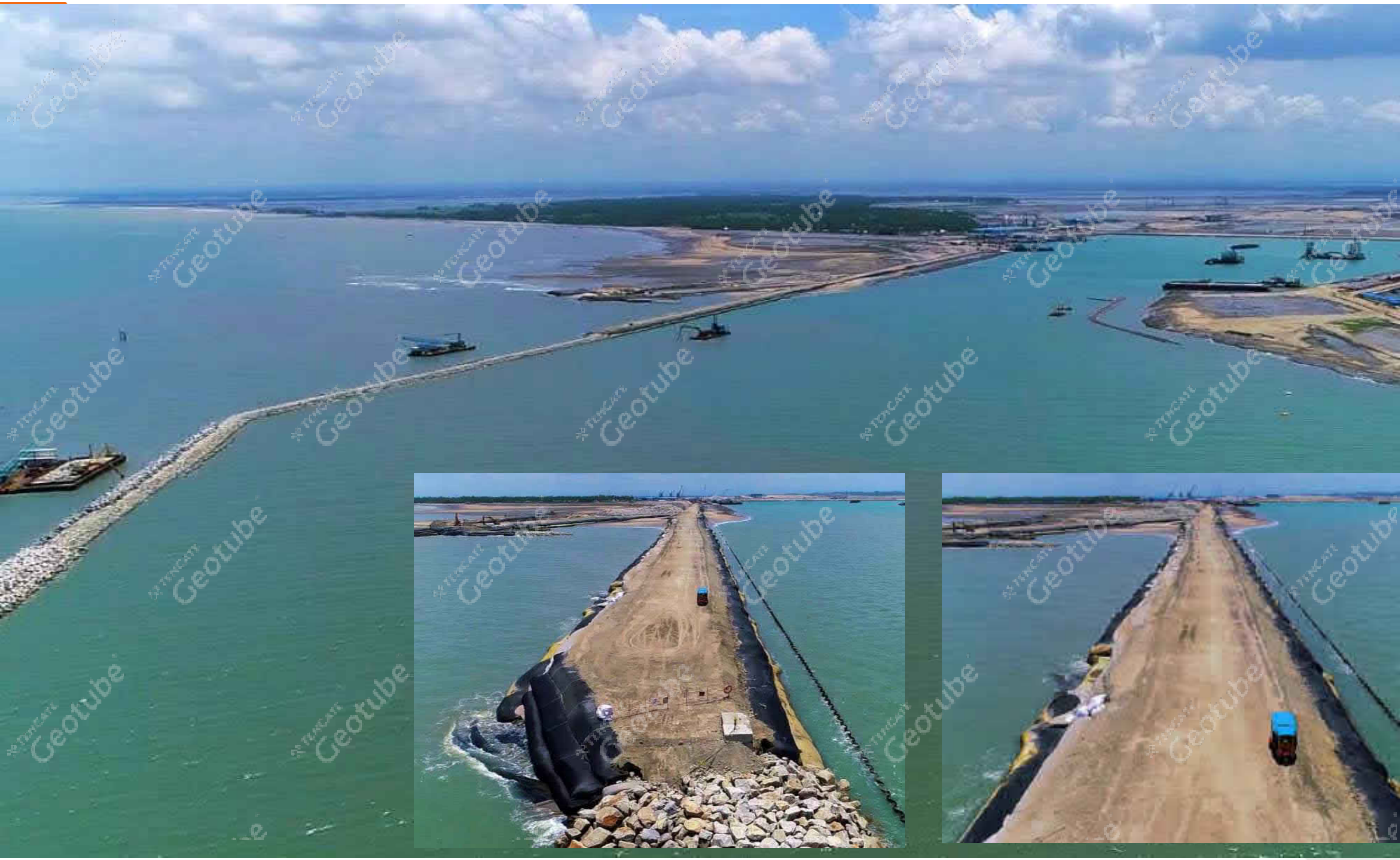
Temporary Dykes

- Placement and pumping of multiple stacked units of TenCate Geotube® GT750M 12.60m circumference x 25m length, inflated to 2.0m height.

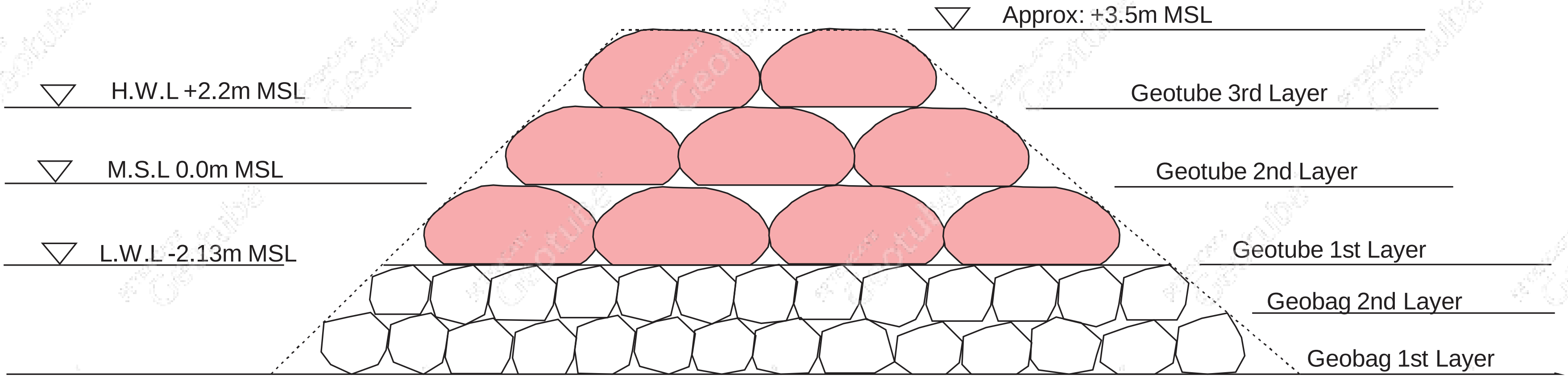
Breakwaters

- Placement of TenCate Heavy Duty Geobags over soft marine clay as a platform for Tencate Geotube® units.
- Placement and pumping of multiple stacked units of Tencate Geotube® GT750M 9.50m circumference x 25m length, inflated to 1.5m height

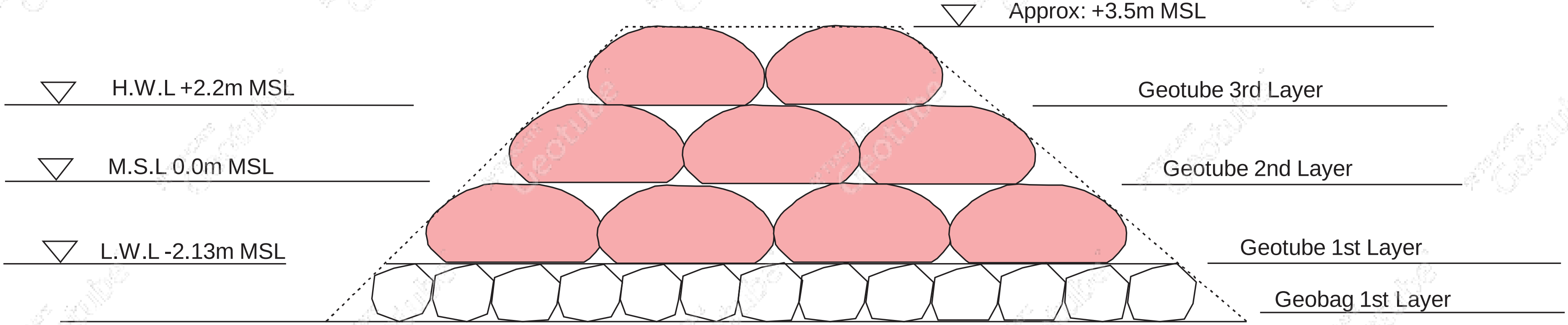
Matabari Power Plant, Bangladesh



Matabari Power Plant, Bangladesh

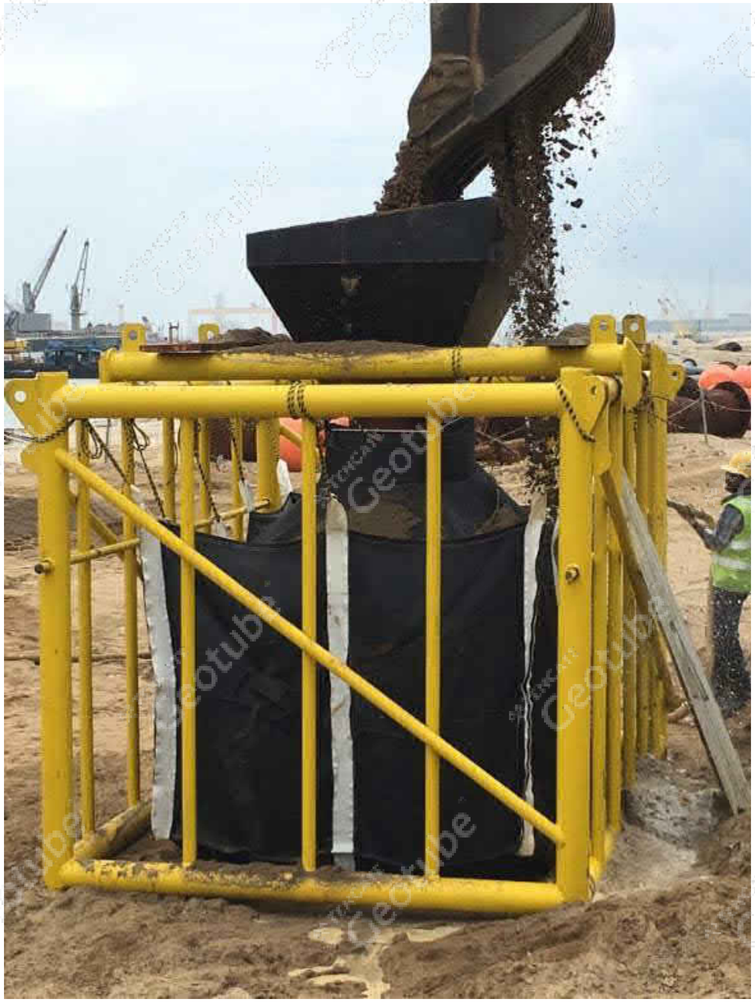


Section A-A



Section B-B

Typical Filling & Handling



Large Geobags require fabrication of a customized filling frame.

The Geobags are held within the frame by the bag straps and filled using a hopper.

Once filled the straps are released and the bags lifted out and placed.

Typical Filling & Handling Frame

ENGINEERED GEOSYNTHETICS

Case Study

Lifting & Placement of Large TenCate Marine Geobags using Geogrid Slings

TENCATE
materials that make a difference

Lifting & Placement of Large Geobag Units using Geogrid Slings


Standard drilling pipe
Diameter: 4” to 6”

Tencate Mirafi GX Geogrid Strength Range 40kN/m – 400kN/m

Lifting & Placement of Large Geobag Units using Geogrid Slings

Lifting & Placement of Large Geobag Units using Geogrid Slings