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Investigation of Capital Works Options for the Management of Shoaling at the Mooloolah River Entrance

Reference: R.B20224.001.02.shoal_modelling.docx
Date: October 2014
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Investigation of Capital Works Options for the Management of Shoaling at the Mooloolah River Entrance

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	Title:	Investigation of Capital Works Options for the Management of Shoaling at the Mooloolah River Entrance
	Project Manager: **Author:**	personal information
	Client:	Department of Transport and Main Roads
	Client Contact:	Chris Voisey
	Client Reference:	TMRHMS003
Synopsis:		

REVISION/CHECKING HISTORY

Revision Number	Date	Checked by		Issued by	
0	30/08/2013	IAT		MPB	
1	31/03/2014	IAT		MPB	
2	02/10/2014	IAT	personal information	MPB	personal information

DISTRIBUTION

Destination	Revision										
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Executive Summary

This report describes the development and calibration of a numerical modelling tool designed to assess proposed capital works options to manage shoaling of the Mooloolah River Entrance.

Each capital works option is discussed relative to the baseline condition and assessed against the following criteria:

a) Maintenance of channel to a minimum depth of -3.0mLAT;

b) Impact on entrance wave conditions;

c) Impact to Mooloolaba Spit; and

d) Impact to Mooloolah River flood flows.

Initial assessment of the eastern breakwater sand trapping capacity suggests the following:

- The channel entrance design depth of -3mLAT (equivalent to -4mAHD) was maintained throughout the assessment period for each eastern breakwater extension. Sand bypassing to the entrance still occurred for Option 4 however at a slower rate than the baseline configuration.
- A decrease in wave height at the harbour entrance is predicted for each capital works configuration. The mildest wave conditions are associated with Option 4 and Option 3 where the predicted significant wave height was less than 0.5m throughout the assessment period.
- Natural sand bypassing of the entrance is significantly reduced for Option 1 & 2 and Option 3. These breakwater configurations redirect sand offshore to deeper water where it is less likely to move onshore toward the Mooloolaba Bay shoreline. The reduction in sand supply to Mooloolaba Bay would require mitigation via mechanical bypassing methods to avoid an undesirable shoreline recession response. Option 4 permits some natural bypassing of the entrance however at a slower rate than the baseline configuration.
- Assessment of flood flows suggests the proposed capital works options have an insignificant impact to peak water levels in the lower Mooloolah River.

Methods to mitigate the reduction in sand supply to Mooloolaba Bay and associated shoreline recession were also assessed, including:

- Dredging and placement;
- Sand shifter system similar to the Noosa Main Beach facility; and
- Crane-mounted jet pump system.

The high-level sand bypassing method assessments suggested a crane-mounted jet pump system to be the most viable sand bypassing method. If adopted in conjunction with a breakwater extension, the specifications and operational requirements of the preferred crane would need to form an essential component of the capital works detailed design.

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1 Introduction

Effective management of our coastal shorelines requires a good understanding of contemporary and likely future behaviour with respect to natural and man-induced changes. Management of the Mooloolah River Entrance aims to maintain a channel to 3m below LAT (or deeper). At times, this management objective is not met due to recurring entrance shoaling events.

The aim of the present study is to develop a modelling tool that simulates how the shoaling processes at the Mooloolah River Entrance behave with respect to four proposed modifications to the existing training walls. The tool is to be used to assess, refine and cost the best capital works solution to the shoaling problem. The underlining objectives of this investigation are to:

(1) Describe the existing shoaling processes at the Mooloolah River Entrance and adopt an appropriate historical shoaling event to guide the proposed capital works design refinement.

(2) Develop a numerical modelling system that combines hydrodynamic (tidal, meteorological, wave and flow forcing) and non-cohesive (sand) sediment transport models to predict shoaling at the harbour entrance. The modelling system must demonstrate sound predictive skill and be calibrated/verified using existing wave, tide and hydrographic survey data supplied by TMR.

(3) Use the modelling system and adopted design shoaling event to assess four (4) capital works options against pre-defined criteria:

  (a) Maintenance of channel to a minimum depth of -3.0mLAT;

  (b) Impact on entrance wave conditions;

  (c) Impact to Mooloolaba Spit; and

  (d) Impact to Mooloolah River flood flows.

(4) Through consultation with TMR propose option refinements.

(5) Assess and compare the performance of the refined options using the modelling system.

(6) Undertake detailed cost estimates for the refined options based on capital works and ongoing maintenance costs.

(7) Based on the coastal processes assessments, recommend the most appropriate option for further consideration by TMR.

1.1 Background

Training of the Mooloolah River Entrance occurred in 1968/69 and since this time has experienced episodic shoaling with notable events in the early 1970s, 1985 to 1987, 1996, 2003/2004, 2008 and 2011/12.

The sequence of coastal processes understood to cause a significant Mooloolah Entrance shoaling event were originally described by Department of Harbours and Marine (1987) who suggested sand bypassing mechanisms at Point Cartwright contributed to the “stockpiling” of sand deposits that can then move toward the entrance under certain wave conditions. This conceptual model was generally supported in a subsequent investigation by WBM Oceanics (2004).

Due to the relatively infrequent nature of the shoaling events, TMR adopted a reactive strategy to maintain the design depth of the entrance channel. The approach uses a shoaling prediction tool (WBM Oceanics 2004 and 2005) with monitoring of seabed changes via hydrographic surveys as an early warning system so that dredge equipment can be mobilised to mechanically move sand from the entrance.

Local geological constraints and wave conditions mean that the sand must enter the navigational channel before it can be intercepted effectively by a dredge. This weakness of the strategy was recently exposed (in terms of operation and cost) during a particularly persistent shoaling event that started during April 2011 and continued into early 2013.

A sand shifter trial operated by Slurry Systems Pty Ltd was commissioned by TMR during 2012 to investigate an alternative method to artificially bypass sand across the entrance. The sand shifter system was installed at Point Cartwright adjacent to the eastern breakwater where sand accumulation was anticipated. The system was designed to transfer accumulated sand via a pipeline from the eastern breakwater to the shoreline at Mooloolaba Bay (mimicking the “natural” entrance bypassing mechanisms). The trial showed that the system was not able to work efficiently due to the shallow thickness of sand across the rock shelf and inadequate sand trapping capacity of the present entrance configuration.

The weakness of the existing entrance shoaling management strategy and the failure of an alternative sand bypassing method with the present entrance configuration has prompted an investigation of alternative capital works options for the Mooloolah River Entrance. This report describes the numerical modelling tools developed to test for four modified entrance configurations and presents initial option assessment results.

2 Numerical Model Descriptions

Multiple numerical models have been used to undertake the hydrodynamic and shoaling process assessments of the capital works options at Mooloolah River Entrance, as described below.

2.1 Waves (SWAN)

The wave modelling component of these assessments has been undertaken using the spectral wave model SWAN.

SWAN (Delft University of Technology, 2006) is a third-generation spectral wave model, which is capable of simulating the generation of waves by wind, dissipation by whitecapping, depth-induced wave breaking, bottom friction and wave-wave interactions in both deep and shallow water. SWAN simulates wave/swell propagation in two-dimensions, including shoaling and refraction due to spatial variations in bathymetry and currents. This is a global industry standard modelling package that has been applied with reliable results to many such investigations worldwide.

The regional SWAN model (400m grid resolution) used in this study was previously developed as part of the Moreton Bay RWQMv3 project (CSIRO, 2012). This project included validation of the SWAN wave model predictions using wave buoy recording and ADCP measurements within Moreton Bay. Additional SWAN wave models ranging in resolution from 100m down to 25m were developed specifically for this study and nested within the existing regional model domain. The system of nested SWAN models is shown in Figure 2-1. The locations of Waverider buoys referred to throughout this report are also shown in Figure 2-1.

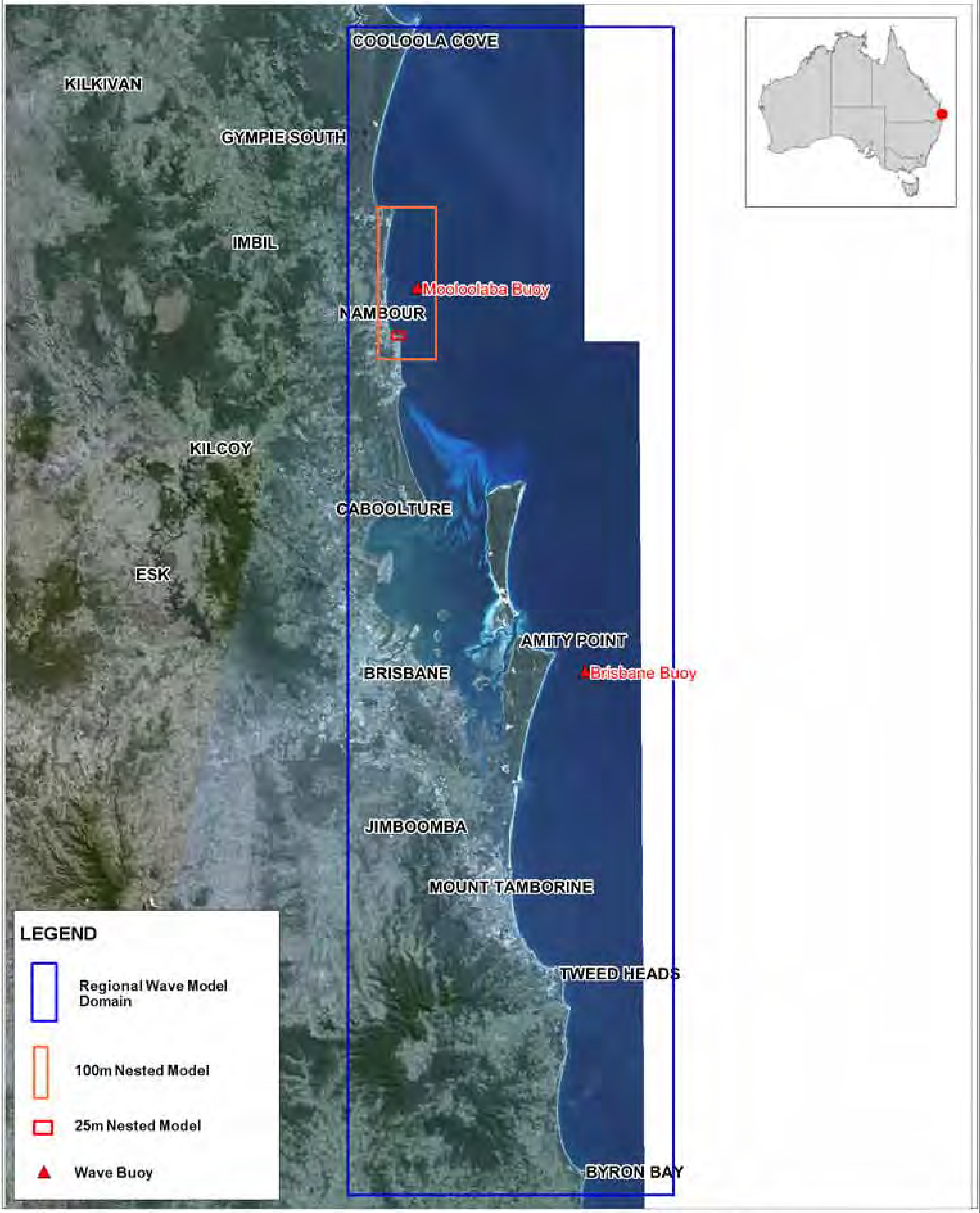
The model bathymetry has been derived from the following sources, listed in decreasing order of priority:

- SKM 2m Digital Elevation Model (DEM) created from a 2011 bathymetric LiDAR survey of the study area (Queensland Government, 2012); and
- BMT WBM 20m DEM developed for Moreton Bay RWQMv3 project (CSIRO, 2012).

In this particular study SWAN was used to model the transformation of incoming waves from offshore deep water into the nearshore Mooloolah River Entrance study area. Specifically the SWAN wave model was used to transform measured directional wave data from the Brisbane (Point Lookout) Waverider Buoy into shallow water.

The wind boundary condition applied to the 100m and 25m SWAN grids was based on the measured Cape Moreton (040043), Sunshine Coast Airport (040861) and Double Island Point (040068) wind records supplied by the Commonwealth Bureau of Meteorology (BOM). This data was converted to 10m above mean sea level following the log-law conversion described in the Coastal Engineering Manual (U.S. Army Corps of Engineers, 2002) and interpolated across the model domain.

Validation of the modelling approach using directional wave data from Mooloolaba Waverider Buoy and is presented in Section 3.1.



Title:
Wave Model Extents

Figure: 2-1

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2.2 Hydrodynamic (TUFLOW FV)

The hydrodynamic modelling component of these assessments has been undertaken using the TUFLOW FV software, which is developed and distributed by BMT WBM (http://www.tuflow.com/Tuflow%20FV.aspx). TUFLOW FV is a numerical hydrodynamic model for the two-dimensional (2D) and three-dimensional (3D) Non-Linear Shallow Water Equations (NLSWE). The model is suitable for solving a wide range of hydrodynamic systems ranging in scale from open channels and floodplains, through estuaries to coasts and oceans.

The Finite-Volume (FV) numerical scheme employed by TUFLOW FV is capable of solving the NLSWE on both structured rectilinear grids and unstructured meshes comprised of triangular and quadrilateral elements. The flexible mesh allows for seamless boundary fitting along complex coastlines or open channels as well as accurately and efficiently representing complex bathymetries with a minimum number of computational elements. The flexible mesh capability is particularly efficient at resolving a range of scales in a single model without requiring multiple domain nesting.

2.2.1 Model Domain, Mesh and Bathymetry

The model domain is shown in Figure 2-2 and extends from Marcoola Beach in the north to Warana in the south and includes the tidal extent of the lower Mooloolah River. Locations where hydrodynamic data has been recorded and used for model calibration are also shown in Figure 2-2. The model mesh resolution at the offshore boundary is up to approximately 500m (mesh cell side length), increasing to 10m in the vicinity of the Mooloolah River Entrance.

Figure 2-3 shows detail of the model mesh and the various capital works option layouts incorporated into the mesh design to allow the influence of the proposed structures to be assessed accurately.

The hydrodynamic model bathymetry relied exclusively on the SKM 2m DEM of the study area (Queensland Government, 2012).

2.2.2 Model Configuration

The hydrodynamic model validation and impact assessments described in this report have used a 2D configuration of TUFLOW FV. The model has used the following configurations and parameterisations:

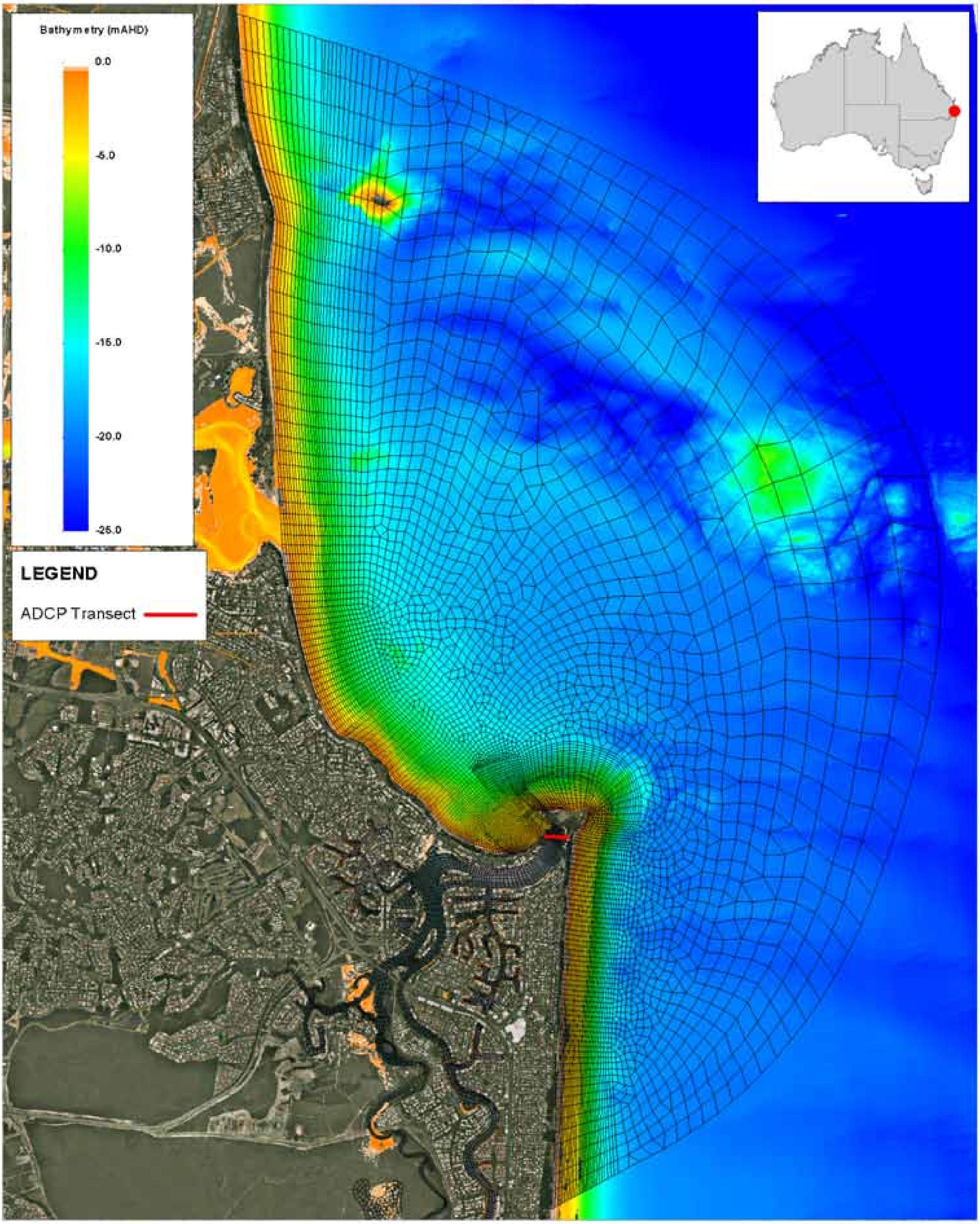
- Smagorinsky model to estimate horizontal turbulent and sub-grid mixing;
- Bottom drag derived from application of the “log-law”; and
- Bottom roughness length-scales between 0.05-0.5m.

2.2.3 Boundary Conditions

Tidal water level variation at the Mooloolaba tide gauge was predicted using a set of tide constituents and harmonic analysis. The water level time series was applied at the offshore boundary with a -15min temporal offset to account for the tidal phase difference between the offshore and prediction location.

Wave boundary conditions have been derived from the 100m and 25m resolution SWAN models described in Section 2.1. These are applied as spatially and temporally varying wave fields that are then interpolated onto the TUFLOW FV flexible mesh. Both un-coupled and fully-coupled wave models have been used, with the latter described in further detail in Section 2.3.1.1.

The same wind boundary condition developed for the 100m and 25m resolution SWAN models described in Section 2.1 was also applied to the hydrodynamic model.



Title:
Mooloolah Entrance Morphology Model Domain

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BMT WBM endeavours to ensure that the information provided in this map is correct at the time of publication. BMT WBM does not warrant, guarantee or make representations regarding the currency and accuracy of information contained in this map.

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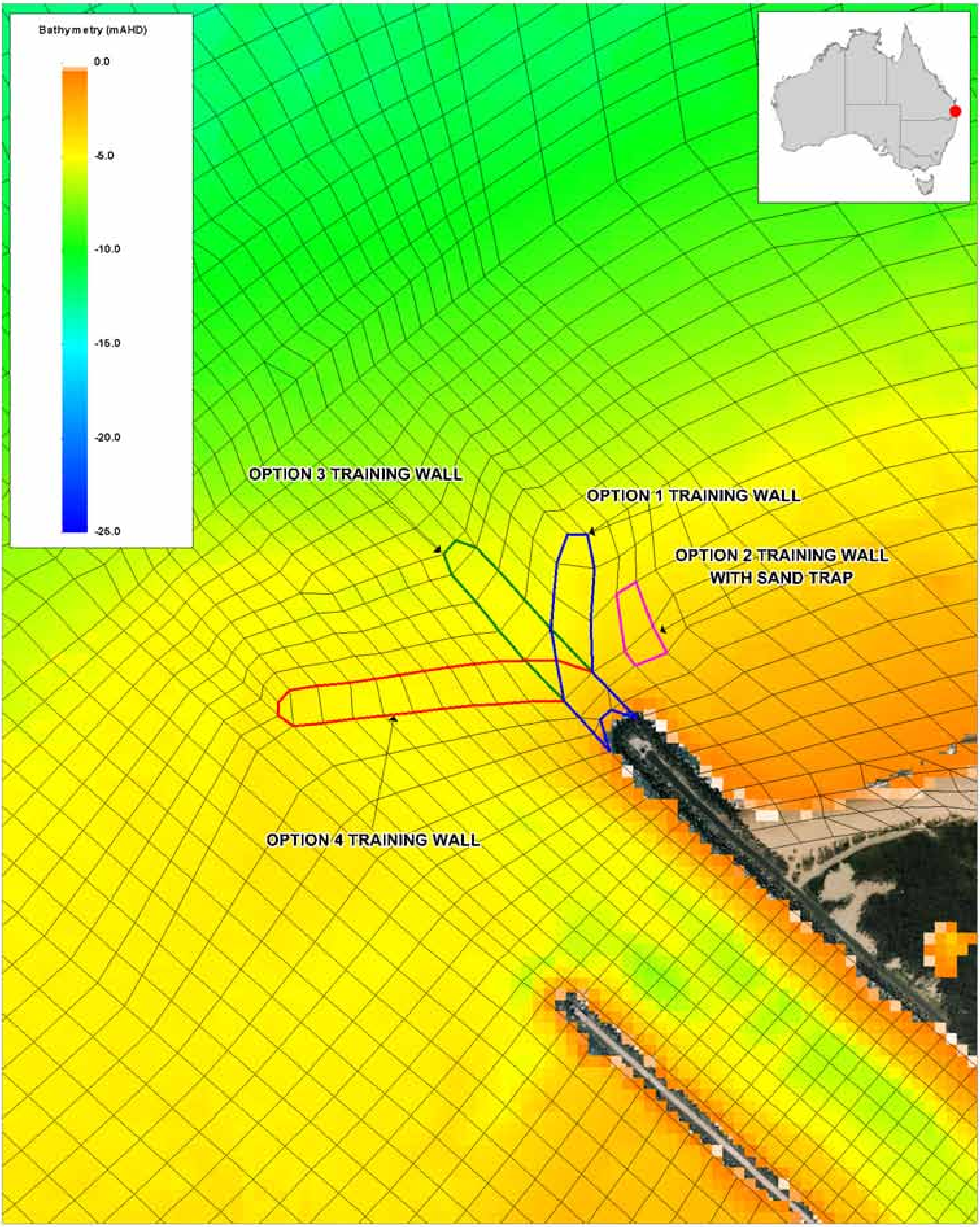

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Model Mesh Detail incorporating Proposed Capital Works Options

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2.3 Coastal Processes

2.3.1 TUFLOW FV

The Mooloolah River Entrance shoaling assessments in this study refer to the modelling of sediment transport driven by currents and waves. This has been undertaken using the sediment transport and morphology module within TUFLOW FV.

2.3.1.1 Wave Coupling

A dynamic 2-way coupling between the SWAN wave model and TUFLOW-FV has been implemented to provide the necessary littoral zone forcing of currents by the waves, as well as provide temporally and spatially varying bed elevation, water level and current fields to SWAN. The dynamic 2-way coupling of SWAN and TUFLOW FV occurs within the inner, 25m resolution nested SWAN model shown in Figure 2-1. Outside this region an un-coupled wave model forcing from the 100m resolution SWAN model has been applied, which does not feature dynamic variations in bed elevation, water level and current fields.

The short wave derived radiation stress gradients provide a source of momentum to the hydrodynamic model which primarily drives the longshore currents in the surfzone. In addition the short wave motion Stokes drift induces an additional mass transport in the direction of wave propagation that is applied to the hydrodynamic (long wave) model. Along an approximately straight and uniform coastline, the onshore mass transport is approximately balanced by an offshore directed current (or “undertow”). The short wave model also provides wave parameter fields (H_{sig}, T_p, Direction) to the TUFLOW FV sediment transport module.

2.3.1.2 Sediment Transport

The TRANSPOR model (van Rijn, 2004) has been used to predict sediment transport within TUFLOW FV. The TRANSPOR model is capable of representing multiple fraction sediment transport including wave and current related bedload and suspended load. The calculated bedload component is a direct input to the TUFLOW FV morphological bed update scheme, while the suspended load component is converted to an equivalent sediment pickup rate (Nielsen, 1992), which provides a suspended sediment source term to the TUFLOW FV water column advection-dispersion scheme (and corresponding sink term to the bed). Suspended sediment settling provides a sink term to the water column (and corresponding source term to the bed).

TRANSPOR represents the interaction of both current and wave related sediment transport. The presence of waves can enhance sediment pickup and therefore also the rate of transport by the local currents. TRANSPOR also includes the prediction of wave-related sediment transport due to processes such as wave velocity skewness and wave boundary layer streaming. These (and other) processes can generate a net transport in the direction of (or against) wave travel, even in the absence of a local current.

A single sand fraction with median grain size D_{50} = 0.22mm has been adopted for the modelling assessments. The internal routines in TRANSPOR have been used to calculate bed roughness values based on sediment and hydrodynamic parameters. All other parameters have adopted the default values described in van Rijn et al. (2004), except that a calibration factor has been applied directly to the total sediment transport as described in Section 3.3.

3 Numerical Model Calibration & Validation

3.1 Waves

The regional SWAN model has been previously validated against various measurements including:

- Moreton Bay Waverider buoy data (EHP, 2007-2009);
- Mooloolaba Waverider buoy data (EHP, 2007-2009); and
- Moreton Bay ADCP measurements (CSIRO, 2012) at the following locations:
  - Moreton Banks;
  - South West Spit; and
  - Beacon M3.

In all cases the model was forced with boundary wave parameters derived from the Brisbane (Point Lookout) Waverider buoy dataset and an interpolated wind field derived from BOM wind recordings.

Validation of the wave modelling system developed for the current study was undertaken using a subset of additional Mooloolaba Waverider buoy time series data supplied by EHP and TMR. The total dataset between October 2011 and April 2013 is summarised using a wave rose plot in Figure 3-1. The wave rose shows a significant wave height (H_{sig}) typically less than 4m and a prevailing south-easterly wave direction offshore from Mooloolaba (location indicated in Figure 2-1). Validation results for the 100m grid resolution SWAN model is provided in Figure 3-2 and demonstrates the good capability of the SWAN model at the Mooloolaba Waverider buoy location.

The late 2011 to early 2012 period shown Figure 3-2 corresponds to a persistent Mooloolah River Entrance shoaling event. This event is described further in Section 3.3.

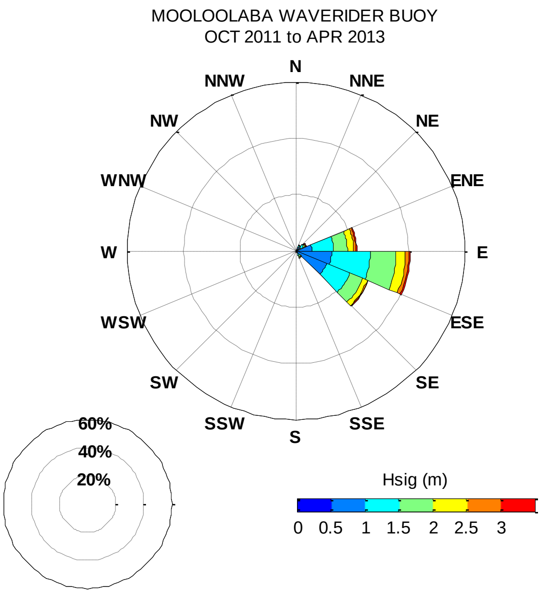


Figure 3-1 Mooloolaba Buoy Wave Climate October 2011 to April 2013

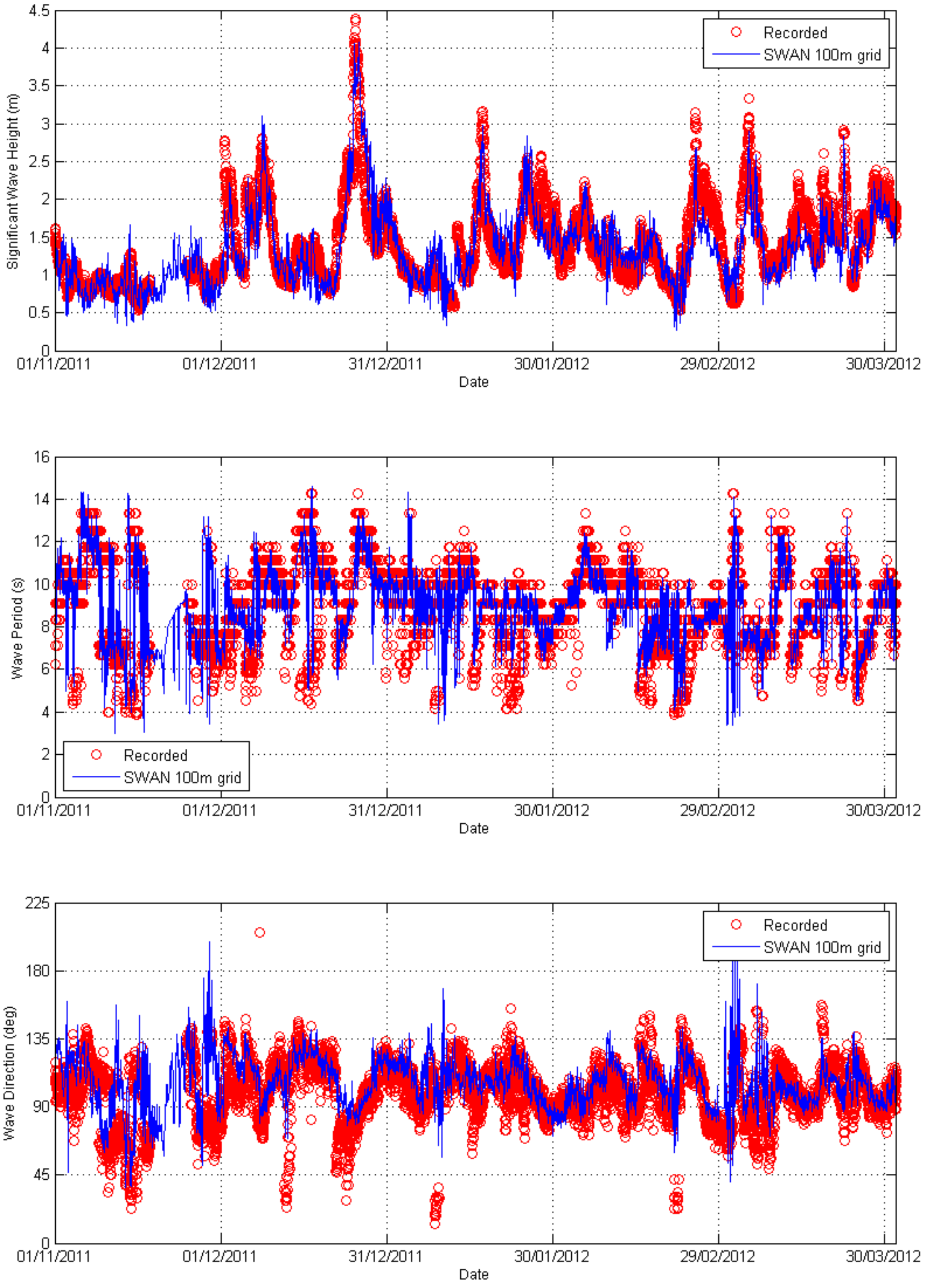


Figure 3-2 SWAN Model Validation to Mooloolaba Waverider Buoy Measurements

3.2 Tidal Hydrodynamics

The TUFLOW FV tidal model has been calibrated to stationary water level and boat-mounted ADCP flow measurement datasets obtained at the Mooloolah River Entrance during May 2005 (locations indicated in Figure 2-2). This data has been previously published (WBM Oceanics, 2005) and was collected as part of the Moreton Bay Waterways Catchment Partnership projects.

A tidal water level comparison is provided in Figure 3-4 and suggests the phase and amplitude of the tide is predicted well at the river entrance. The tidal flow comparison in Figure 3-5 verifies that the model is also reproducing the tidal exchange within the Mooloolah River estuary.

It is noted that two weirs within the waterways connected to the lower Mooloolah River influence the volume of water entering and leaving the estuary system. In order to achieve satisfactory model calibration to the 2005 datasets it was necessary to open the weirs which effectively increase the tidal exchange. The locations of the weirs are indicated in Figure 3-3 and recent aerial photography suggests the weirs are presently closed (inferred due to the distinctly different water colour either side of weirs). Consequently, the weirs were closed for the design shoal morphology calibration simulation described in Section 3.3.

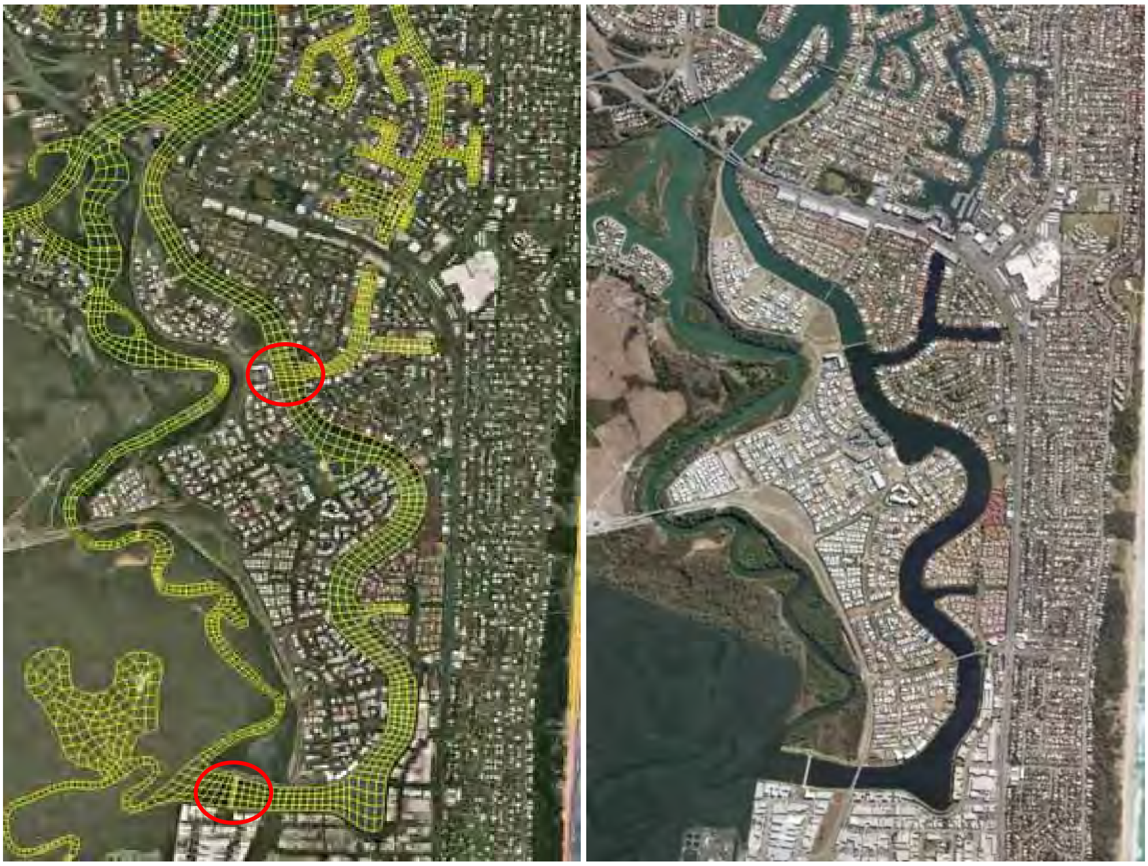


Figure 3-3 Weir Locations Shown within TUFLOW FV Model Mesh (left) and Recent Aerial Photography Indicating Weir Closure (right)

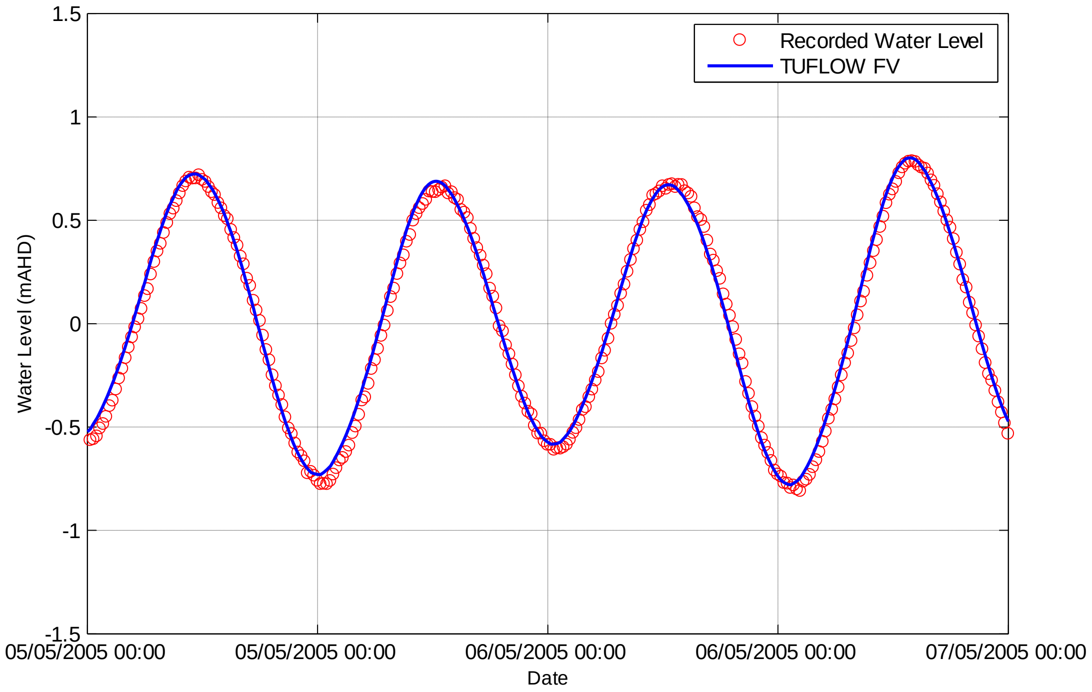


Figure 3-4 Tide Recorder Comparisons from May 2005 Monitoring

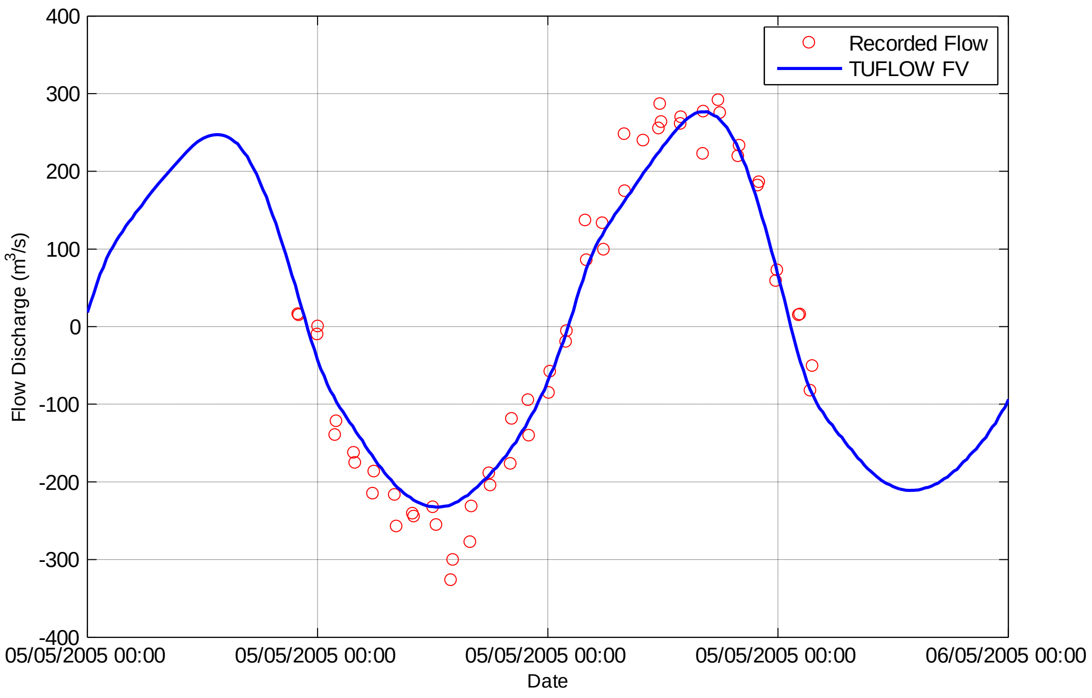


Figure 3-5 ADCP Flow Comparisons from May 2005 Monitoring

3.3 Shoal Morphology Calibration

A persistent Mooloolah River Entrance shoaling event observed between December 2011 and May 2012 was used to calibrate the TUFLOW FV morphology module (incorporating the TRANSPOR model). A number of hydrographic surveys were undertaken by TMR during the shoaling event to identify a navigable channel and guide ongoing maintenance dredging. A sequence of DEMs created from the hydrographic survey data have been used calculate the instantaneous shoal volume. This information provided a means to quantitatively assess the predictive skill of the morphology model.

A sequence of selected shoal DEMs are shown with the corresponding hydrographic survey chart in Appendix A. The instantaneous shoal volume within the polygon shown in each figure was calculated and compared to predicted shoal volume at the corresponding time. The sequence of observed and predicted shoal volumes is summarised in Table 3-1 and the predicted shoal volume time series is compared to the observed volumes in Figure 3-6. The observed and predicted shoal morphology near the end of the assessment period is also qualitatively compared in Figure 3-7.

The morphology model predicts the general trend of sand accumulation during the calibration period. The over prediction of shoal volume (up to 25%) at certain times is likely to be associated with the sand removed by maintenance dredging. This sediment sink was not considered in the modelling but is inherently accounted for in the hydrographic survey data.

Table 3-1 Shoal Morphology Calibration Results

Survey Date	Observed Volume* (m^3)	Predicted Volume (m^3)	Difference (%)
20 Dec 2011	4046	4098	1
22 Dec 2011	3812	4139	8
29 Dec 2011	9580	9492	1
09 Feb 2012	14417	17854	21
17 Feb 2012	14188	18238	25
01 Mar 2012	17406	19328	10
13 Mar 2012	16973	20242	18
27 Mar 2012	22400	21360	5
04 Apr 2012	24259	23268	4
24 Apr 2012	20638	24847	19

*calculated from DEMs created from hydrographic survey data

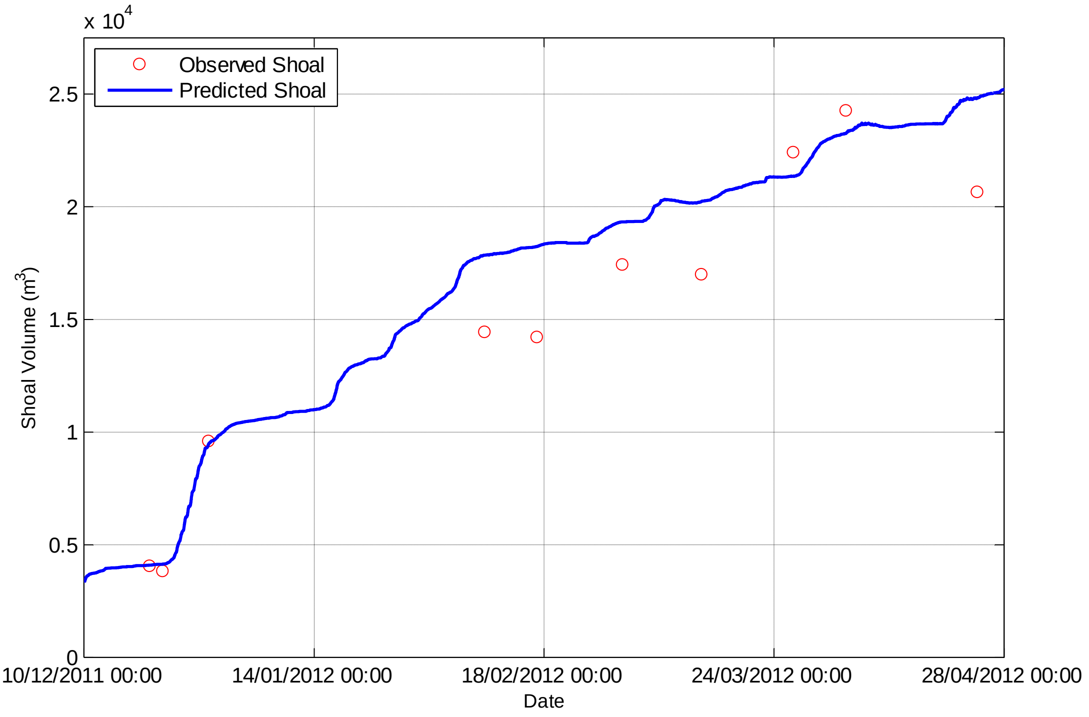
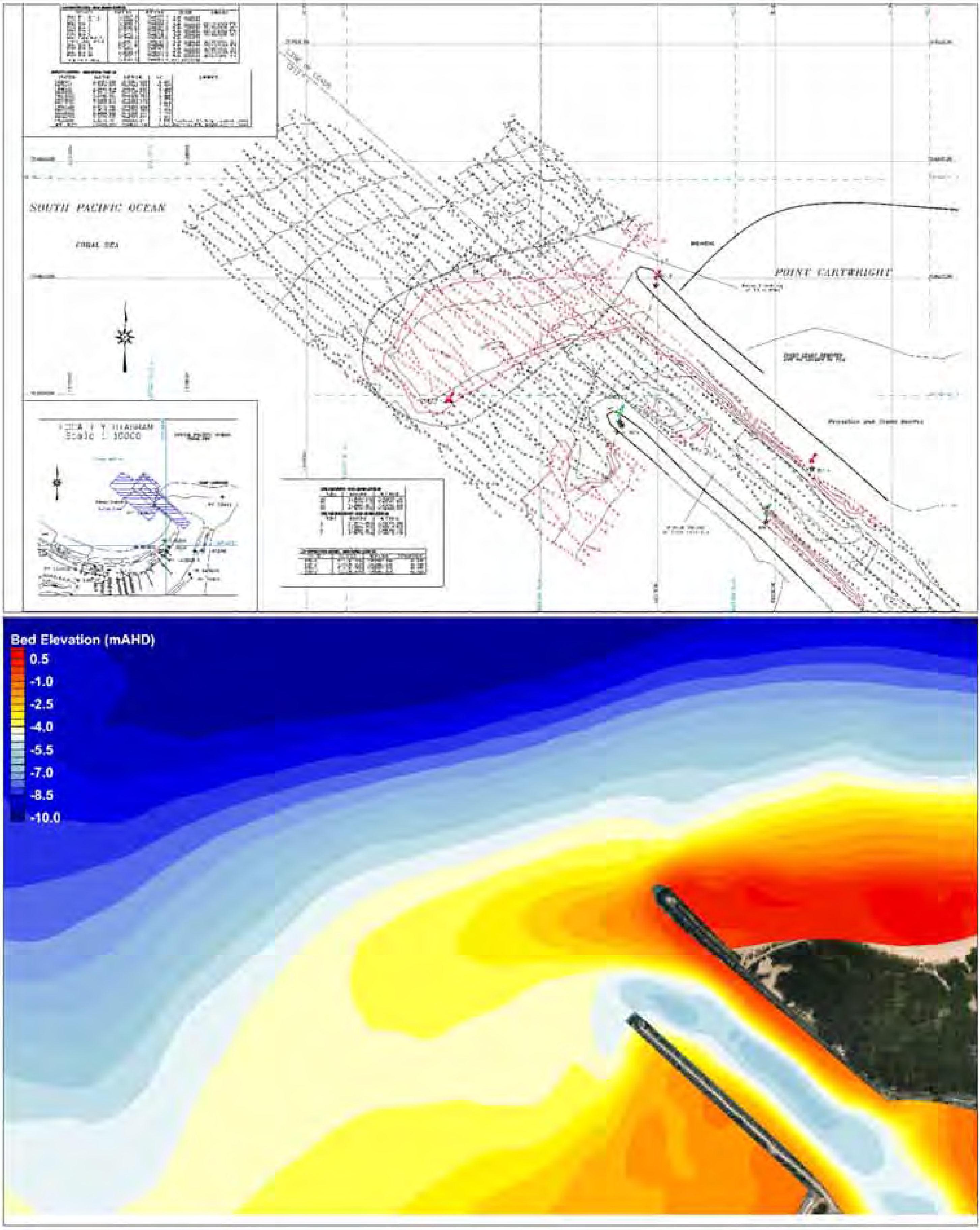


Figure 3-6 **Shoal Volume Comparison between December 2011 and April 2012**



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Observed and Predicted Shoal Comparison 24 April 2012

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4 Capital Works Options Assessment

The calibrated Mooloolah River Entrance shoal modelling system described in Sections 2 and 3 was used as the primary tool to assess four capital works options:

(1) Option 1 – curved eastern breakwater extension to create sand trap;;

(2) Option 2 – curved eastern breakwater extension with rock extraction to create sand trap

(3) Option 3 – straight eastern breakwater extension to deflect sand offshore; and

(4) Option 4 – eastern breakwater extension with change in alignment that follows existing sand transport pathway.

Conceptual layouts of the four options are shown in Figure 4-1 to Figure 4-4.

The intention of the Option 1 and Option 2 configurations is to trap sand in a manner that allows a mechanical bypassing system to operate effectively. The Option 3 and Option 4 configurations are intended to divert sand offshore and/or sufficiently delay any substantial entrance shoaling. Maintenance dredging or alternative mechanical bypassing of accumulated sand would be undertaken to avoid shoaling of the navigation channel.

The initial assessments described in the following sections primarily focus on the breakwater extension sand trapping capacity and associated impacts. This assessment has not distinguished between Option 1 and Option 2 and therefore these options have been reported together. The efficiency of methods to mechanically bypass the intercepted sand will be included as part of subsequent assessments and will consider the required rock excavation depth associated with the Option 2 capital works.

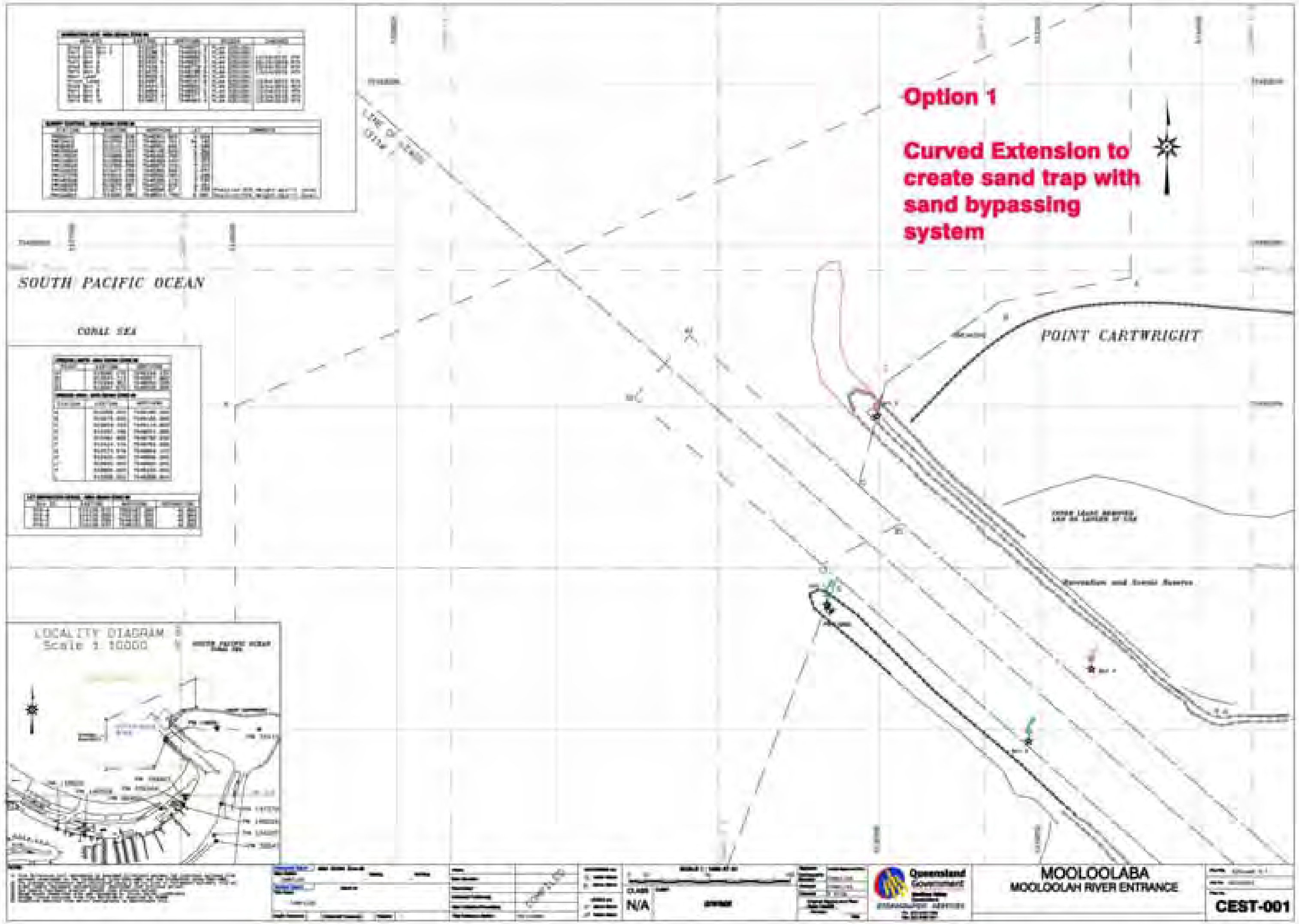


Figure 4-1 Option 1 Conceptual Layout

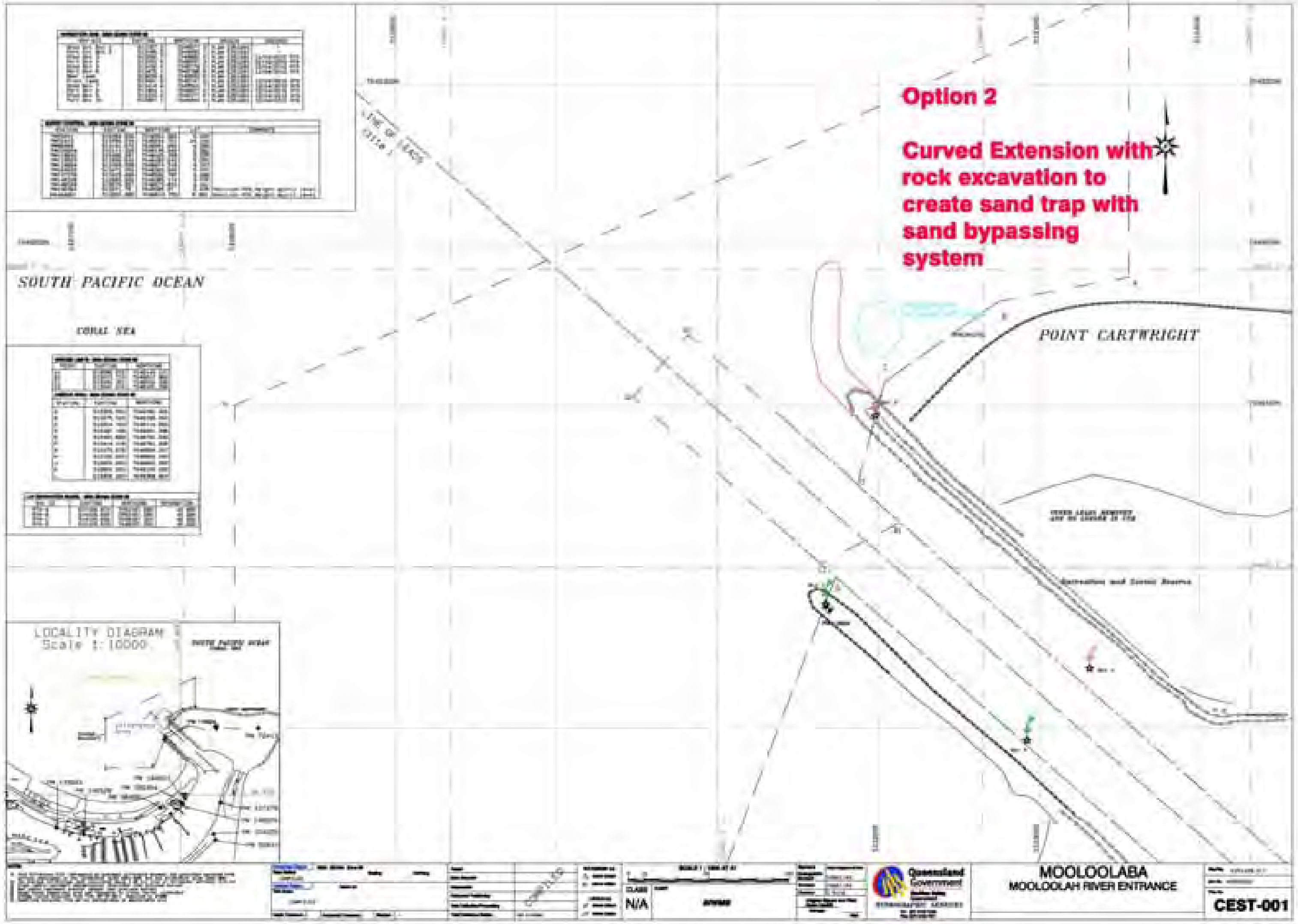


Figure 4-2 Option 2 Conceptual Layout

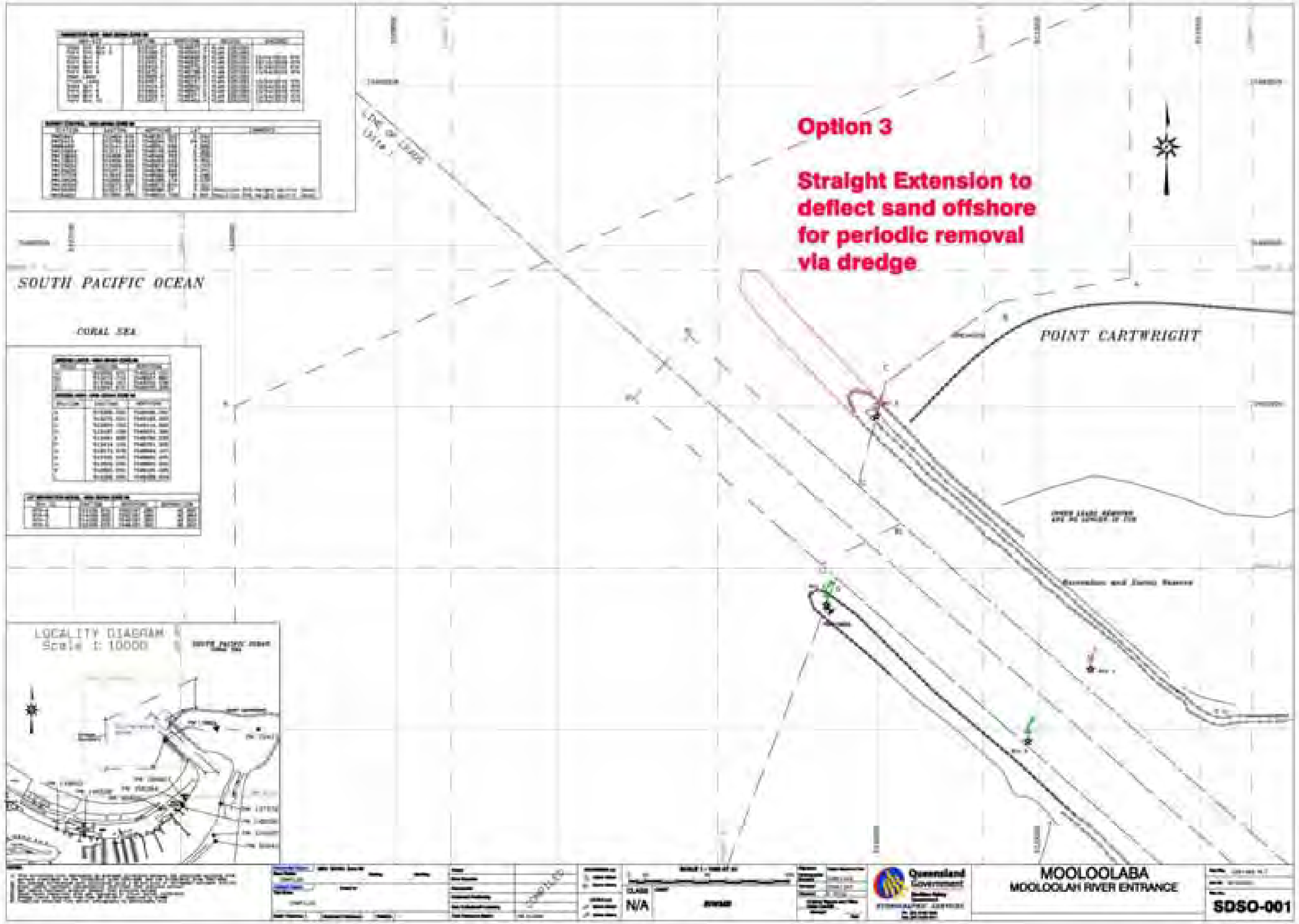


Figure 4-3 Option 3 Conceptual Layout

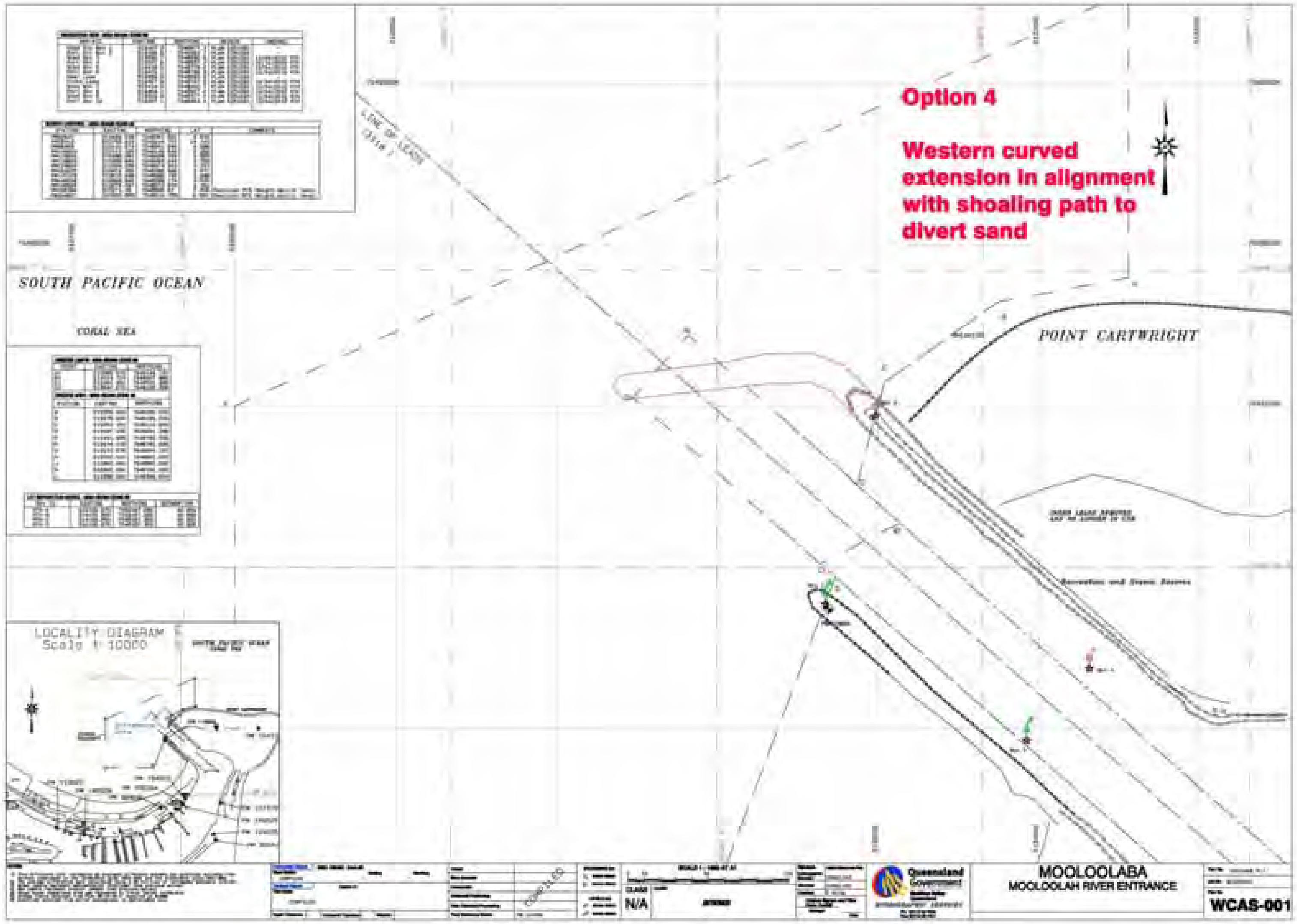


Figure 4-4 Option 4 Conceptual Layout

4.1 Assessment Approach

The persistent shoaling period between November 2011 and May 2012 adopted for model calibration (described in Section 3) was also used to assess the capital works options. The primary assessment criteria included:

- Maintenance of navigation channel to a minimum depth of -3.0mLAT (equivalent to -4.0mAHD);
- Impact on entrance wave conditions;
- Impact to Mooloolaba Spit and “natural” sand bypassing of the entrance; and
- Impact to Mooloolah River flood flows.

In addition, an assessment of sand accumulation at the eastern breakwater was also completed to establish whether sand was being trapped in a manner that would allow mechanical bypassing to occur.

In order to establish the “baseline” conditions, the assessment criteria were first applied to the existing entrance configuration. The baseline assessment results provide the basis for the capital works options to be assessed against.

4.2 Baseline Conditions

4.2.1 Navigation Channel Depth

The entrance channel to Mooloolaba Harbour was developed with a navigable depth of -3mLAT (equivalent to -4.0mAHD). In order to maintain this design depth, TMR monitors seabed changes via regular hydrographic surveys and commissions dredging when required to manage the episodic shoaling of the entrance. This reactive management strategy was recently challenged (in terms of operation and cost) by the most persistent shoaling event experienced since the entrance was trained in the late 1960s.

Figure 4-5 shows the predicted baseline shoal and choked entrance channel at the end of the 2011/12 assessment period. The shoal position and alignment is typical of historical shoaling events that occurred in 1985 to 1987, 2003/04 and 2008 however the shoal volume during this more recent event was larger than previously observed. During such events the navigation channel has been re-aligned to the west of the entrance in order to maintain navigable depths. This channel configuration is operationally difficult for harbour users, particularly when wave breaking occurs across the shoal.

The primary objective of a modified Mooloolah River Entrance configuration is to manage the shoaling potential in a manner that maintains a navigable channel to -3mLAT or deeper. Capital works options navigational channel depth assessments are presented in Sections 4.3.1, 4.4.1 and 4.5.1.

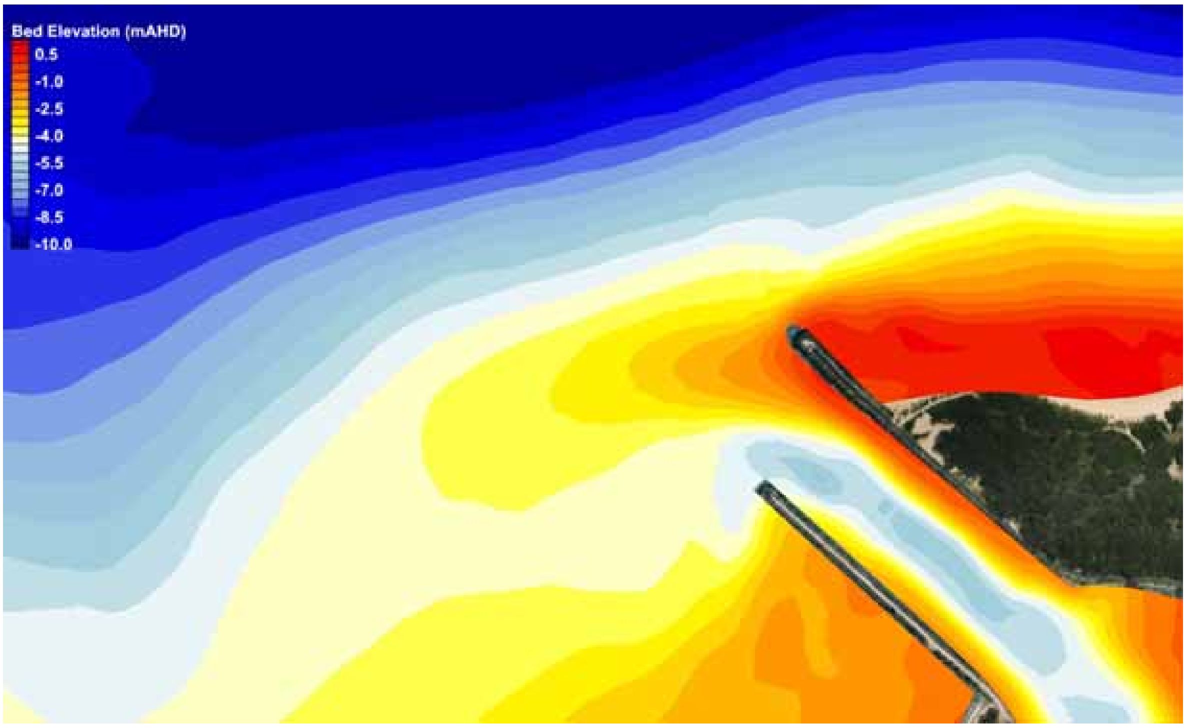


Figure 4-5 Baseline Bed Elevation at the end of Assessment Period

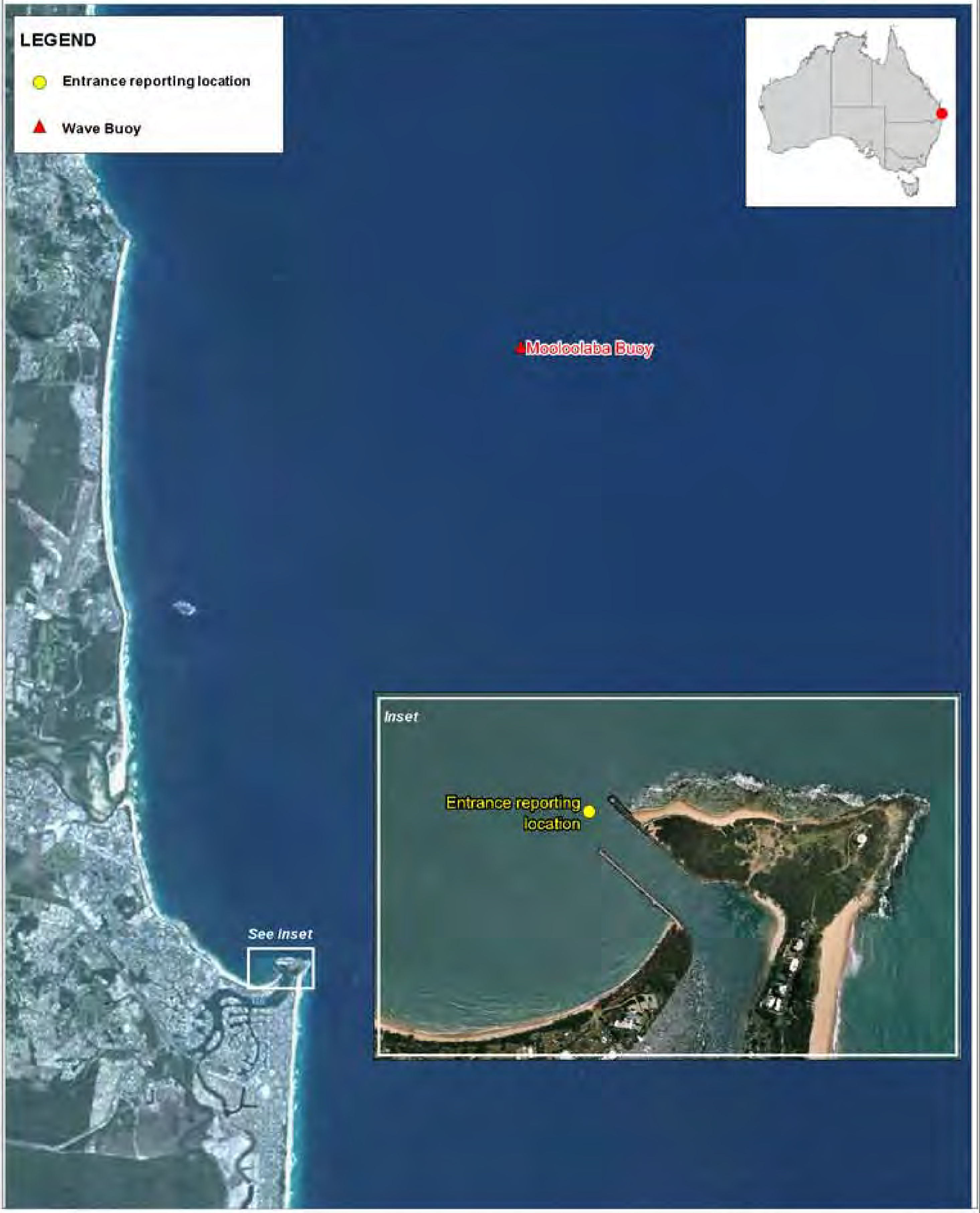
4.2.2 Entrance Wave Conditions

The 2011/12 assessment period was characterised by considerable wave energy with recordings from the Mooloolaba waverider buoy showing at least eight events with significant wave heights exceeding 2.5m (refer Figure 3-2). The wave rose previously shown in Figure 3-1 illustrates the dominance of wave energy from the south to south easterly directional sector.

The Mooloolah entrance is largely sheltered from the prevailing south to south easterly offshore wave climate, with wave refraction at Point Cartwright and wave breaking in shallow water acting to dissipate wave energy prior to reaching the harbour entrance.

Figure 4-6 shows a point location at the harbour entrance selected for baseline wave climate reporting. A wave rose based on model output at the entrance reporting location is presented in Figure 4-7 and shows the significant wave height is typically less than 1.5m. The wave direction is confined to the north easterly sector due to the before mentioned regional wave refraction at Point Cartwright and the more localised refraction at the eastern breakwater. Figure 4-8 compares time series of the offshore significant wave height recorded at the Mooloolaba buoy with the milder wave conditions at the entrance reporting location.

The baseline results suggest the significant wave height at the harbour entrance exceeded 0.5m approximately 65% of the time during the assessment period. The baseline wave climate results are considered relative to the entrance wave climate associated with the capital works options in Sections 4.3.2, 4.4.2 and 4.5.2



Title:

Entrance Wave Climate Reporting Location

Figure: 4-6

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0km 2.5 5

Approx. Scale Main Map

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Filepath : I:\B20224_I_mpb_Mooloolah_Entrace\DRG\COA_002_130627_SWAN_extents.wor

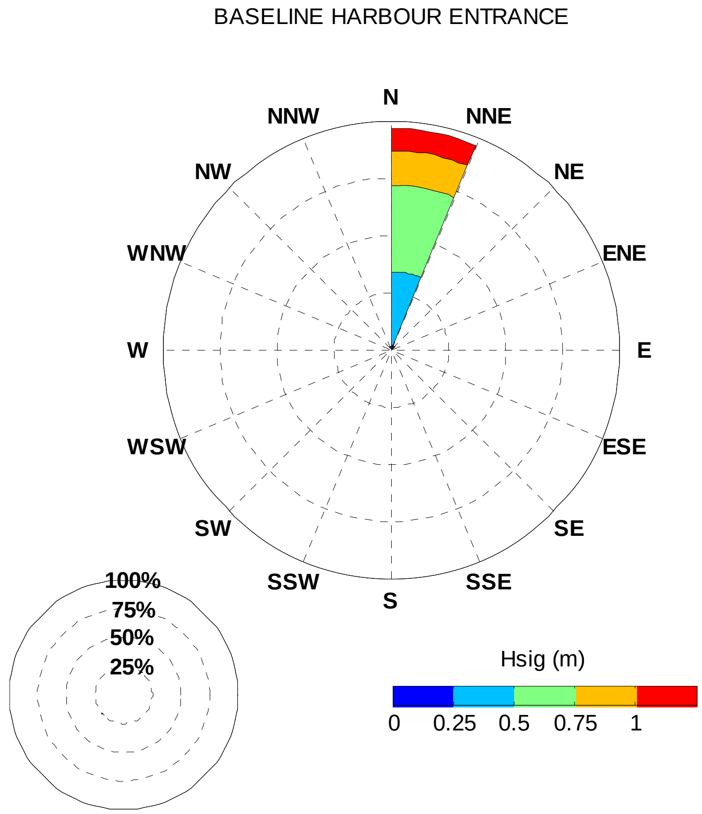


Figure 4-7 **Baseline Harbour Entrance Wave Climate November 2011 to May 2012**

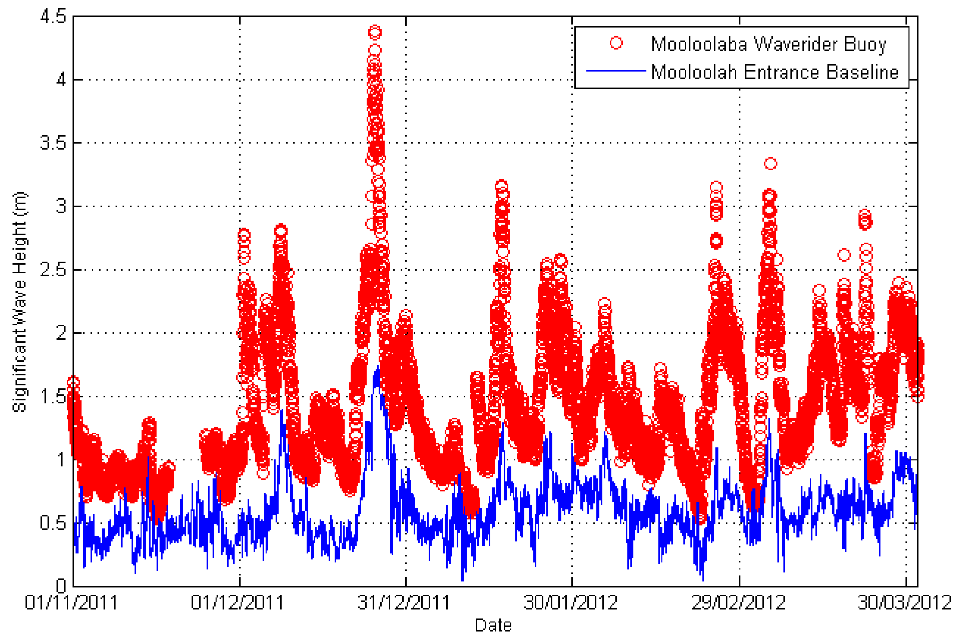


Figure 4-8 **Offshore and Baseline Harbour Entrance Significant Wave Height Time series Comparison**

4.2.3 Natural Sand Bypassing of Entrance

Previous studies suggest that headland sand bypassing mechanisms at Point Cartwright contribute to the "stockpiling" of sand deposits that may move toward the entrance under certain wave conditions (e.g. Department of Harbours and Marine, 1987; WBM Oceanics, 2004). This hypothesis is supported by historical aerial photography where sand deposit areas are clearly visible and is conceptualised in Figure 4-9.

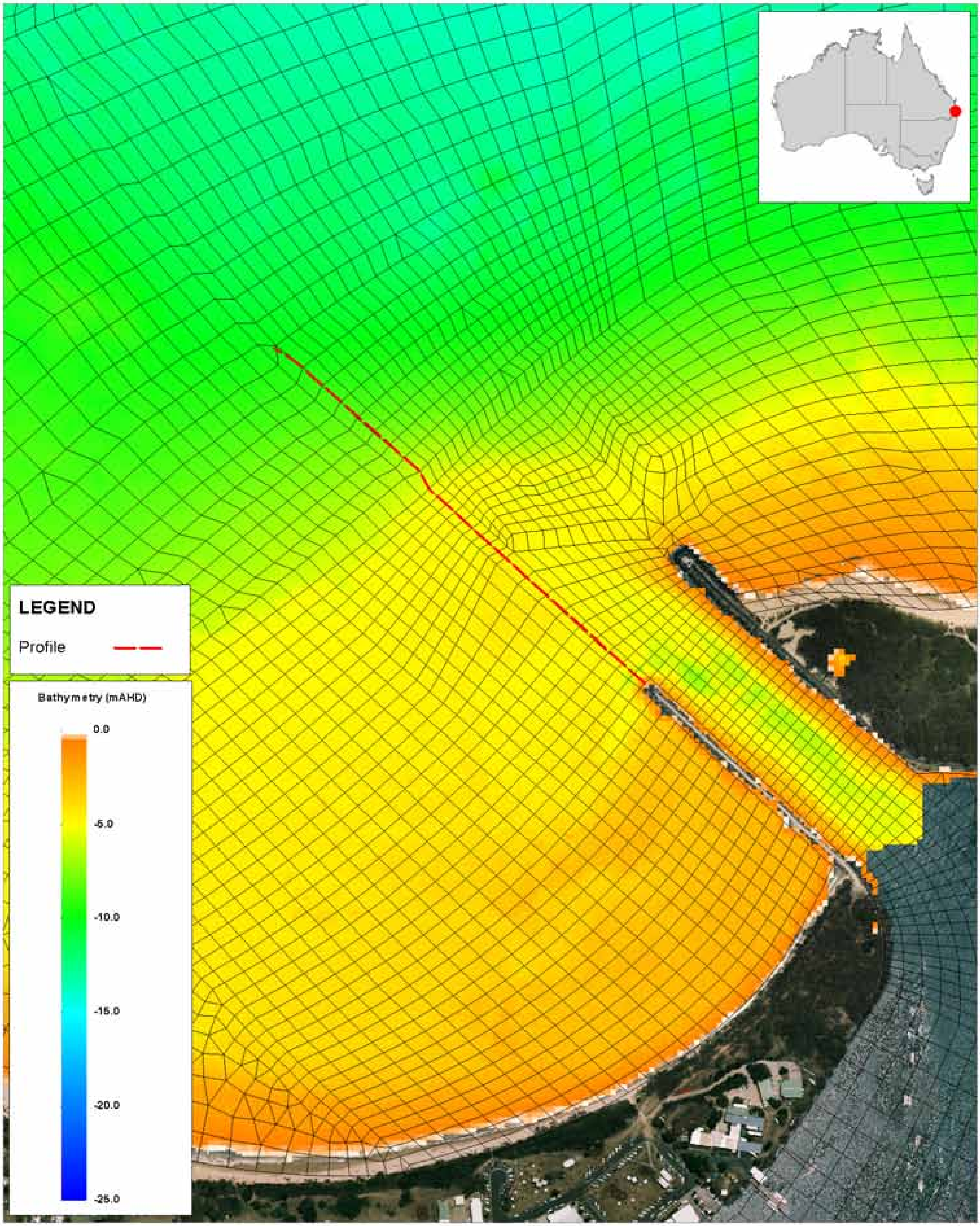
Figure 4-9 Natural Sand Bypassing of Point Cartwright and Entrance (modified from Department of Harbours and Marine, 1987)

The longshore sand transport processes that contribute to bypassing of Point Cartwright and infrequent shoaling at the Mooloolah River entrance also supply sand to Mooloolaba Bay. Modification of the entrance has the potential to interrupt the sediment transport pathway to Mooloolaba Bay, which unmitigated is likely to cause undesirable shoreline recession impacts. The annual average net volume of sand supply to Mooloolaba Bay is expected to be 5,000-10,000m^3 (e.g. BMT WBM, 2013), however, significantly greater bypass volumes are observed during persistent shoal events.

In order to consider the potential changes in sand transport rates to Mooloolaba Bay, the volume of sand passing the western breakwater (to an offshore depth of -10mAHD) during the December 2011 to May 2012 assessment period was calculated using the numerical modelling tools. The profile used to estimate the natural sand bypassing volume is shown in Figure 4-10 and the baseline result is summarised in Table 4-1. The baseline result is used in Sections 4.3.3, 4.4.3 and 4.5.3 to assess the relative impact of the capital works options. It should be noted that the sand transport potential during the assessment period is likely to be significantly higher than the long term average.

Table 4-1 Predicted Natural Sand Bypassing to Mooloolaba Bay during Assessment Period

Scenario	Natural Sand Bypass Volume (m^3)
Baseline (existing)	96,500



Title:
Profile used for Natural Sand Bypass Estimate

Figure:
4-10

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A

N

0 75 150m
Approx. Scale

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Filepath : I:\B20224_I_mpb_Mooloolah_Entrace\DRG\COA_011_141001_sedflux_polyline.wor

4.2.4 Sand Accumulation at Eastern Breakwater

A sand shifter trial operated by Slurry Systems Pty Ltd was recently commissioned by TMR to investigate the effectiveness of artificially bypassing the entrance. The sand shifter system was installed at Point Cartwright adjacent to the eastern breakwater where sand accumulation was anticipated. The system was designed to transfer accumulated sand via a pipeline from the eastern breakwater to the shoreline at Mooloolaba Bay. The trial showed that the system was not able to work efficiently due to the shallow sand thickness across the rock shelf and inadequate sand trapping capacity of the present entrance configuration. The potential for a sand shifter system to work efficiently may improve if the sand trapping capacity of the entrance configuration is enhanced via the proposed capital works options.

Sand accumulation at the eastern breakwater has been assessed by considering the spatial total bed elevation change for the assessment period and a time series of bed elevation change at the location adjacent to the eastern breakwater where maximum sand accumulation is predicted. The baseline assessment results are presented in Figure 4-11 and Figure 4-12 and support the relatively poor sand trapping capacity of the present eastern breakwater configuration. Figure 4-11 shows the majority of sand accumulation occurs within the entrance where changes in bed elevation up to 4m are observed. Outside of the entrance and adjacent to the eastern breakwater the predicted peak in sand accumulation for the baseline configuration slightly exceeds 2m (Figure 4-12). Sand accumulation assessment for the capital works options is presented in Sections 4.3.4, 4.4.4 and 4.5.4.

Figure 4-11 Baseline Total Bed Elevation Difference

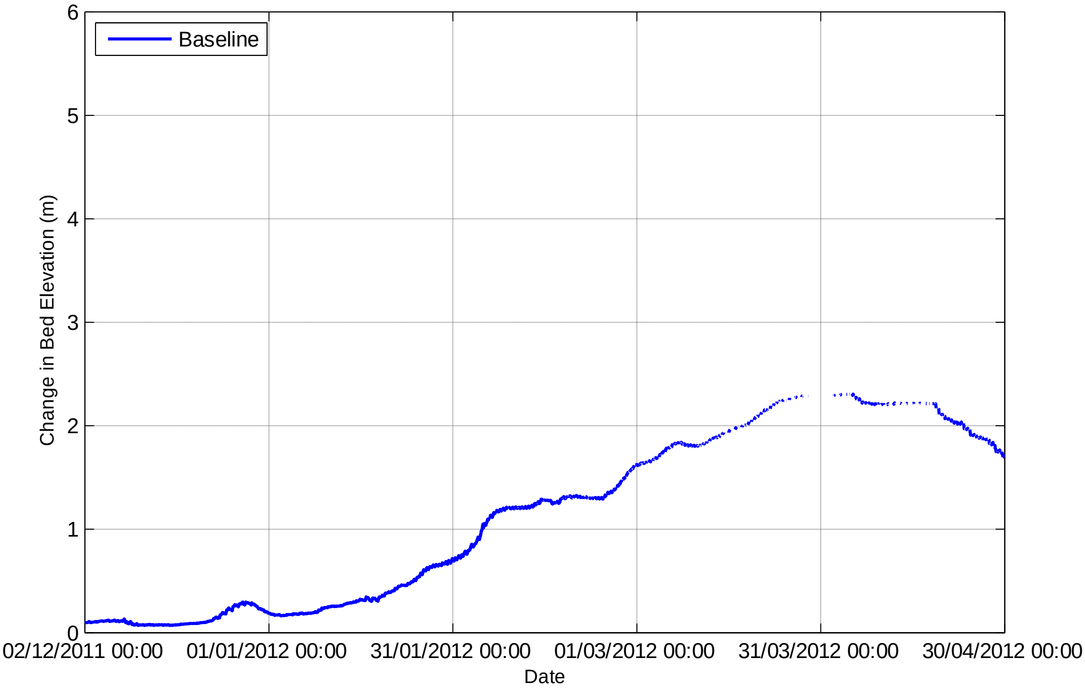

Figure 4-12 Baseline Bed Elevation Difference Time series

4.2.5 Mooloolah River Flood Flows

A baseline assessment of flood flows within the lower Mooloolah estuary was completed in order to investigate whether any of the capital works options adversely affect flood water levels. This preliminary assessment does not constitute a detailed flood study and the levels presented in this report are not to be used for other purposes.

The flood flow assessment involved applying a constant base flow of $500m^3/s$ to the TUFLOW FV model upstream boundary. A simulation covering a full neap to spring tidal cycle was completed and the maximum water level for the flood assessment period was extracted along the section indicated in Figure 4-13. The maximum water level along the section for the baseline entrance configuration is presented in Figure 4-14. It is noted that flood plain areas are not included in the model domain (the flood flow is confined to the river channel) and therefore it is expected that the predicted peak water levels are conservatively high. Nevertheless, this output is considered suitable for option assessment purposes. Baseline and capital works option maximum water level comparisons are presented in Sections Figure 4-21, Figure 4-28 and Figure 4-35.

Title:

Flood Assessment Long Section

Figure: 4-13

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N

0 250 500m

Approx. Scale

BMT WBM

www.bmtwbm.com.au

Filepath : I:\B20224_I_mpb_Mooloolah_Entrace\DRG\COA_010_130819_flood_assessment_profile.wor

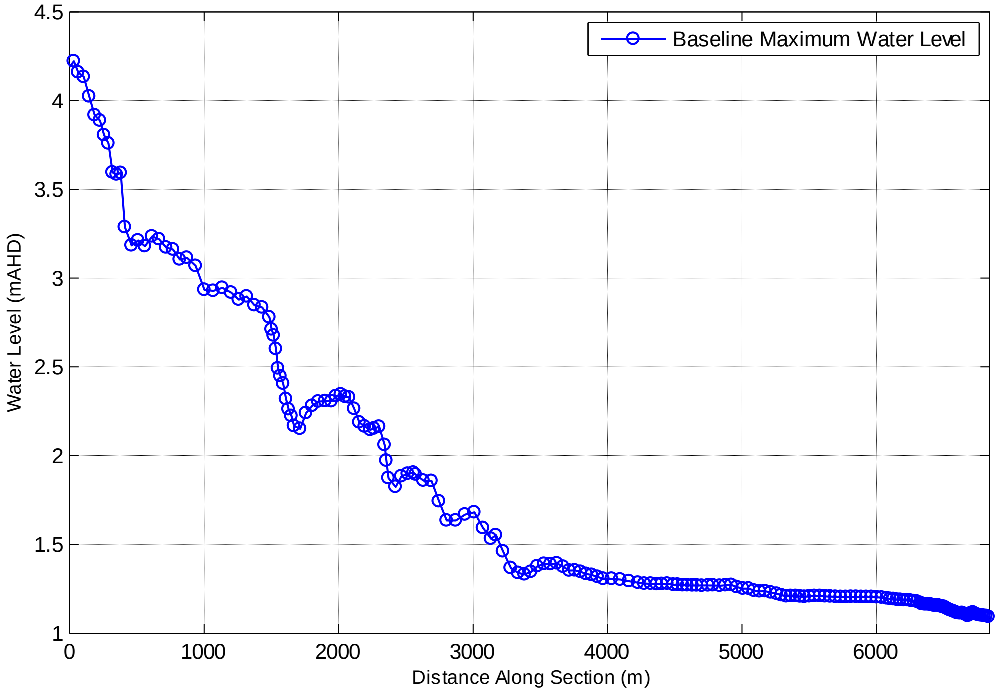


Figure 4-14 Baseline Flood Assessment Maximum Water Level

4.3 Option 1 & 2 Assessment

4.3.1 Navigation Channel Depth

The Option 1 & 2 entrance configuration assessment suggests a navigable channel with depth greater than -3mLAT (equivalent to -4mAHD) is maintained throughout the November 2011 to May 2012 simulation period. Figure 4-15 shows an accumulation of sand at the eastern breakwater and shoal development is predicted offshore from the extended groyne with little bypassing of sand to the entrance.

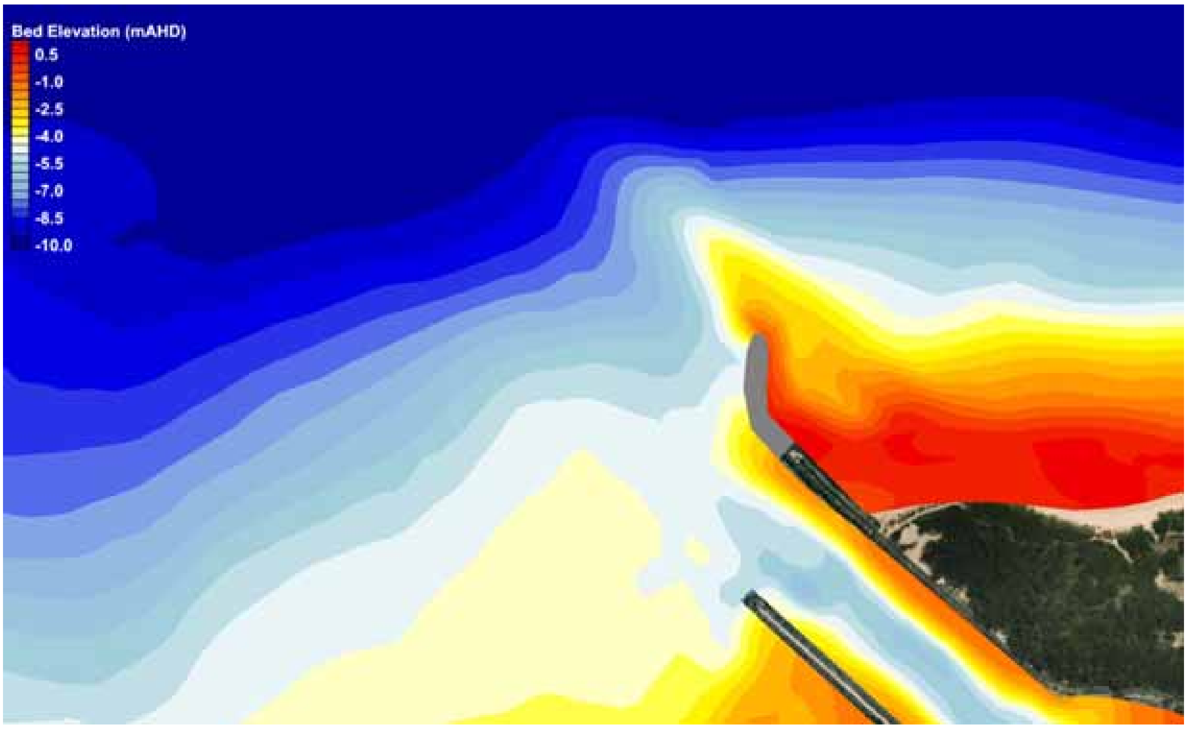


Figure 4-15 Option 1 & 2 Bed Elevation at the End of Assessment Period

4.3.2 Entrance Wave Conditions

A time series of significant wave height at the harbour entrance for the Option 1 & 2 capital works configuration is compared to the baseline wave condition in Figure 4-16. The breakwater extension enhances sheltering from the open coast wave conditions and reduces entrance wave heights by approximately 80%

The Option 1 & 2 wave climate for the assessment period is presented as a wave rose in Figure 4-17 and shows the majority of wave energy is from the N to NNE sector (similar to the baseline wave direction conditions). The results suggest the significant wave height at the harbour entrance exceeded 0.5m approximately 8% of the time during the assessment period.

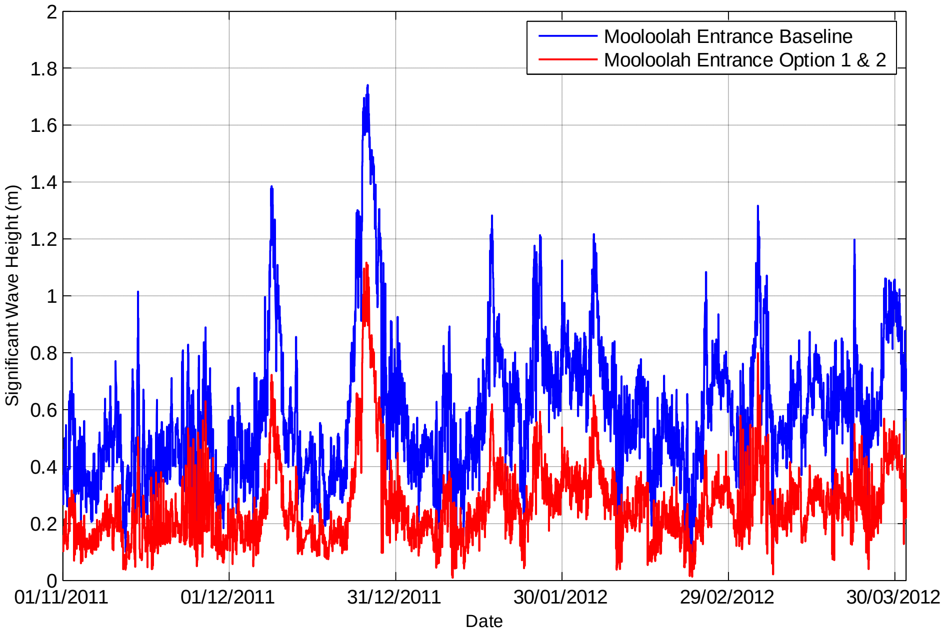


Figure 4-16 Option 1 & 2 and Baseline Harbour Entrance Significant Wave Height Time series Comparison

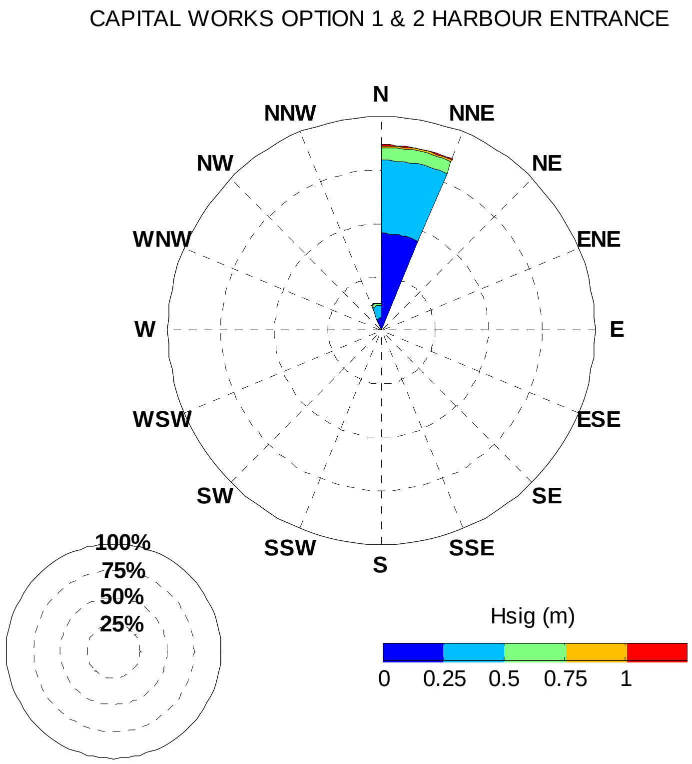


Figure 4-17 Capital Works Option 1 & 2 Harbour Entrance Wave Climate November 2011 to May 2012

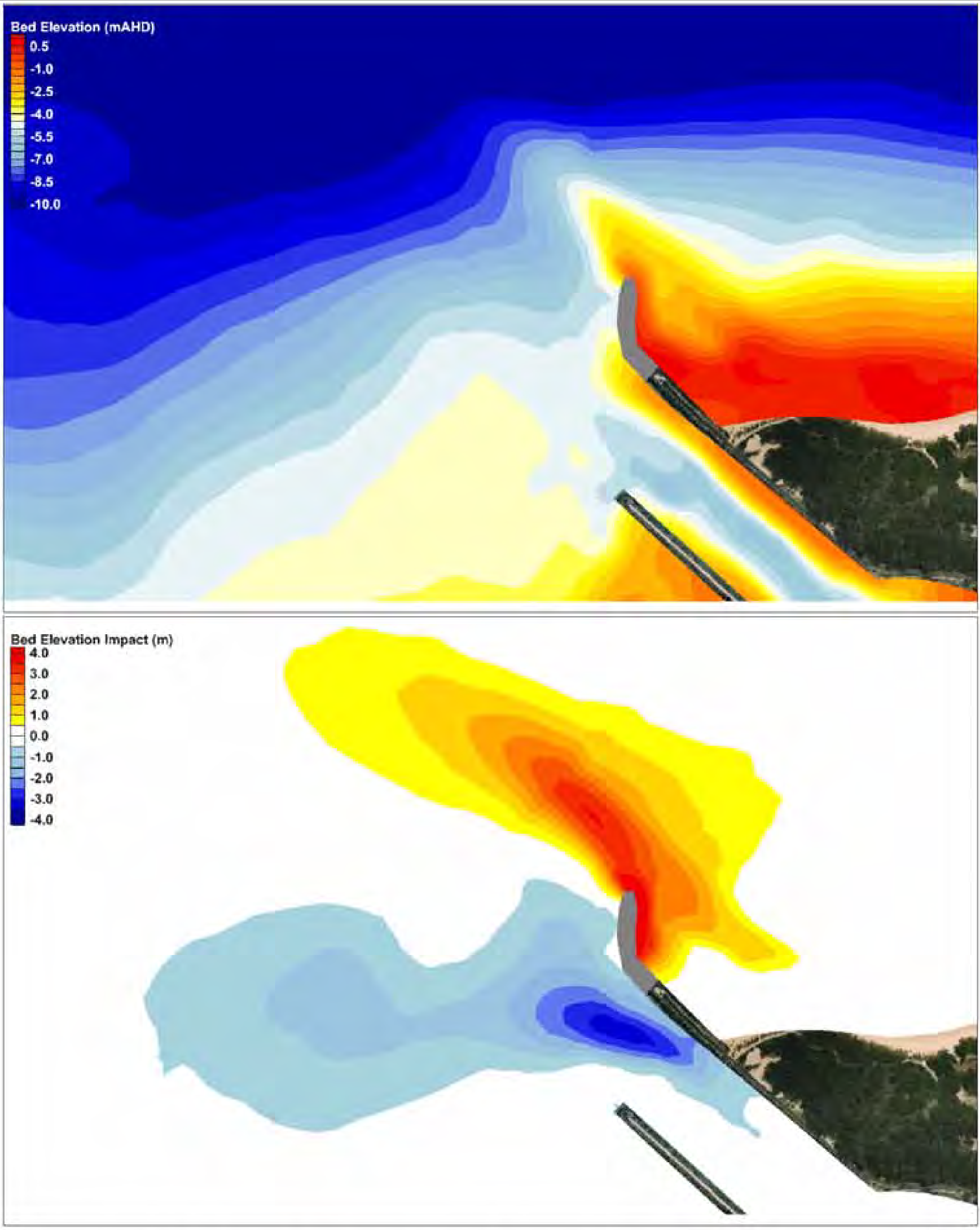
4.3.3 Natural Sand Bypassing of Entrance

The Option 1 & 2 configuration assessment suggests the breakwater extension will successfully intercept sand before reaching the entrance and therefore significantly reduce the volume sand naturally bypassing the entrance to Mooloolaba Bay. Table 4-2 provides estimates of the sand volume that passes the western breakwater (to an offshore depth of -10mAHD) during the simulation period. The Option 1 & 2 natural sand bypass volume is less than 10% of the volume predicted for the baseline configuration. This reduction in sand supply to Mooloolaba Bay would require mitigation via mechanical bypassing methods to avoid an undesirable shoreline recession response.

Table 4-2 Predicted Natural Sand Bypassing to Mooloolaba Bay during Assessment Period

Scenario	Natural Sand Bypass Volume (m^3)
Capital Works Option 1 & 2	8,600
Baseline (existing)	96,500
Impact	-87,900

Figure 4-18 shows the Option 1 & 2 shoal morphology and presents the spatial bed elevation impact (the difference between the baseline and Option 1 & 2 final bed elevation result). The blue contours represent sand deficit areas and clearly indicate a reduction in sand bypassing the entrance. The bed elevation impact results suggest the Option 1 & 2 capital works configuration redirects sand offshore to deeper water where it is less likely to move onshore toward Mooloolaba Bay shoreline.



Title:

Capital Works Option 1 & 2: Shoal Morphology (top) and Bed Elevation Impact (bottom)

Figure: 4-18

Rev: A

N

0 75 150m

Approx. Scale

BMT WBM

www.bmtwbm.com.au

Filepath : I:\B20224_I_mpb_Mooloolah_Entrace\DRG\COA_008_130802_DEV1_shoal_impact.wor

4.3.4 Sand Accumulation at Eastern Breakwater

The Option 1 & 2 entrance configuration assessment suggests the extension enhances sand trapping and accumulation at the eastern breakwater. Figure 4-19 shows the spatial total bed elevation change result. Figure 4-20 provides a time series of bed change due to sand accumulation at the location adjacent to eastern breakwater where the maximum bed elevation change occurs (indicated in Figure 4-19). Relative to the baseline configuration, the Option 1 & 2 sand trapping capacity is significantly enhanced with a peak sand depth close to 4.5m predicted.

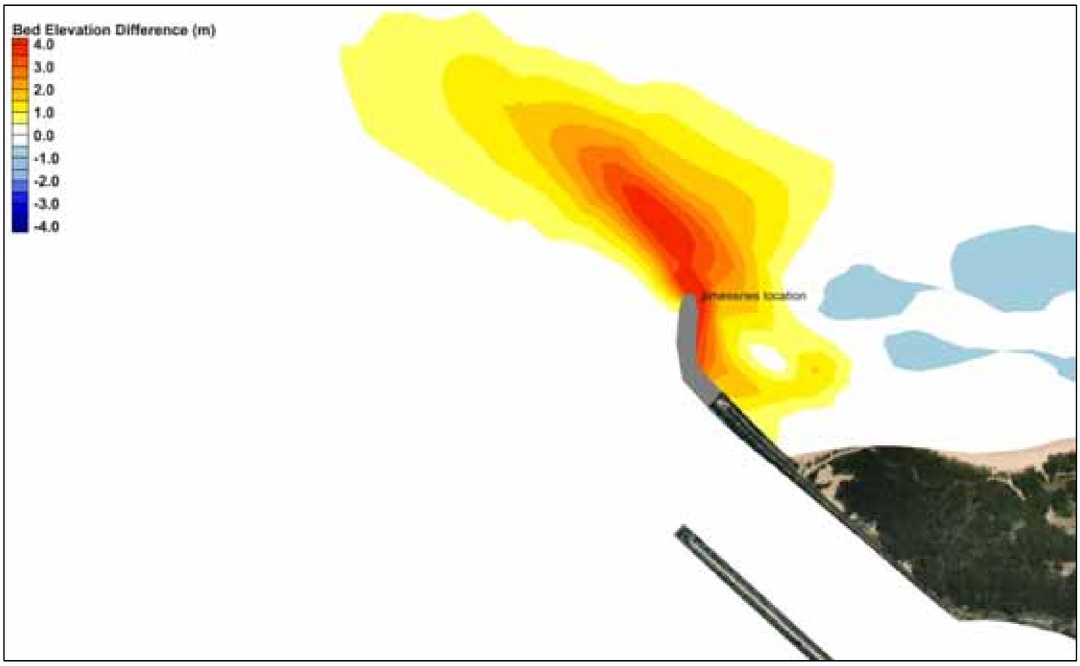


Figure 4-19 Option 1 & 2 Total Bed Elevation Difference

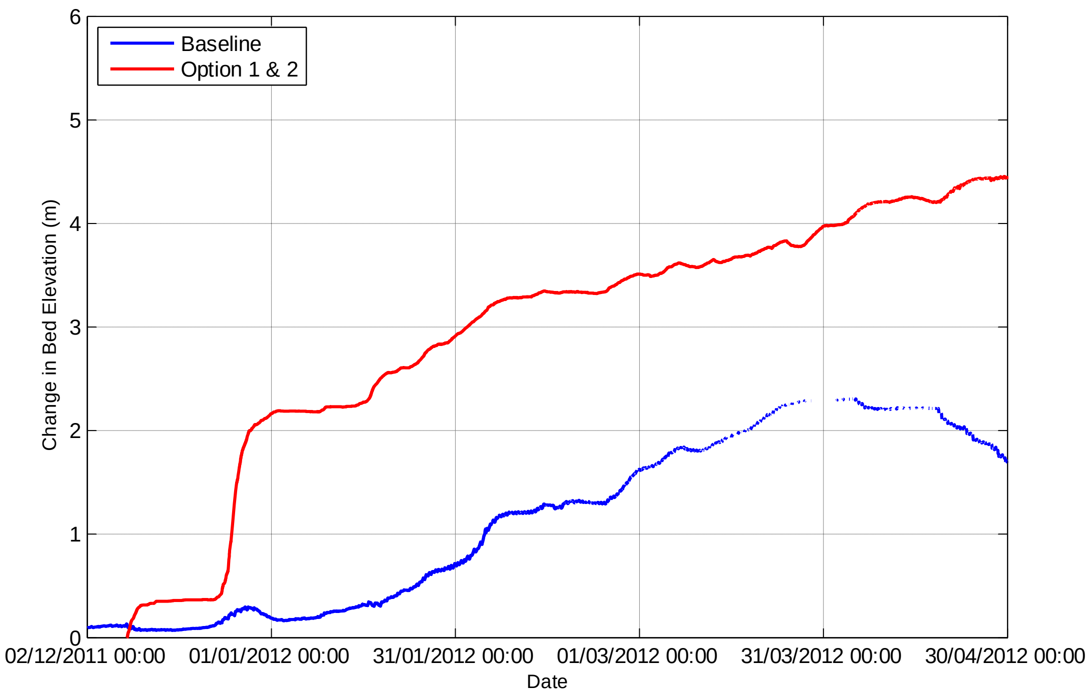


Figure 4-20 Option 1 & 2 and Baseline Bed Elevation Difference Time Series Comparison

4.3.5 Mooloolah River Flood Flows

The flood assessment result for the Option 1 & 2 capital works configuration is compared to the baseline result in Figure 4-21. The assessment result suggests the capital works option has an insignificant impact on peak water levels within the lower Mooloolah River under design flow.

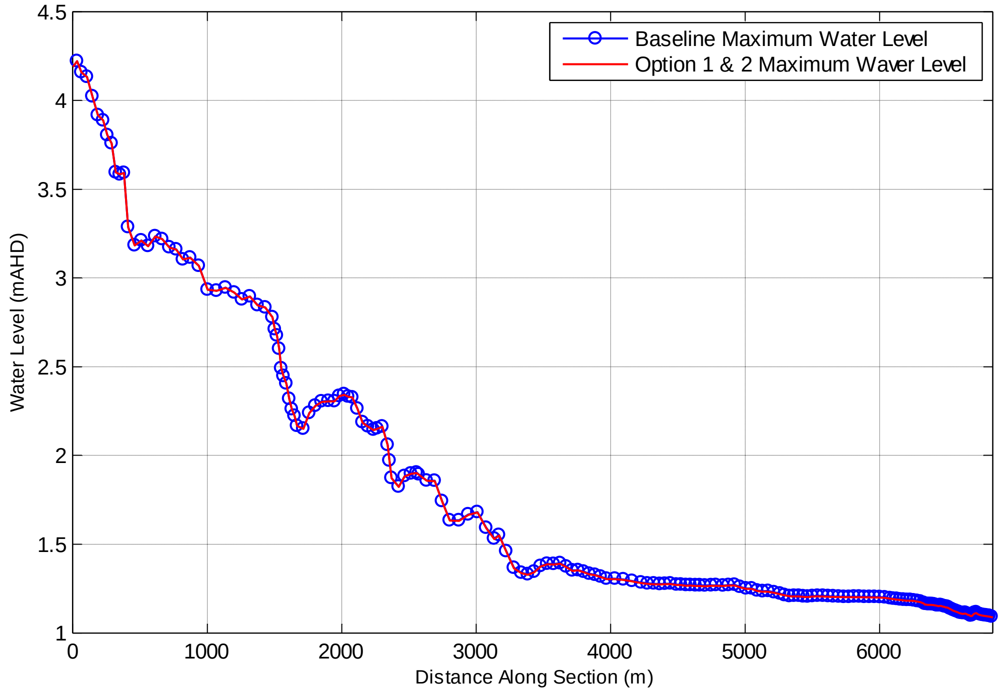


Figure 4-21 Option 1 & 2 and Baseline Flood Assessment Maximum Water Level Comparison

4.4 Option 3 Assessment

4.4.1 Navigation Channel Depth

The Option 3 entrance configuration assessment suggests a navigable channel with depth greater than -3mLAT (equivalent to -4mAHD) is maintained throughout the November 2011 to May 2012 simulation period. Figure 4-22 shows an accumulation of sand at the eastern breakwater and shoal development is predicted offshore from the extended groyne with minor bypassing of sand to the entrance.

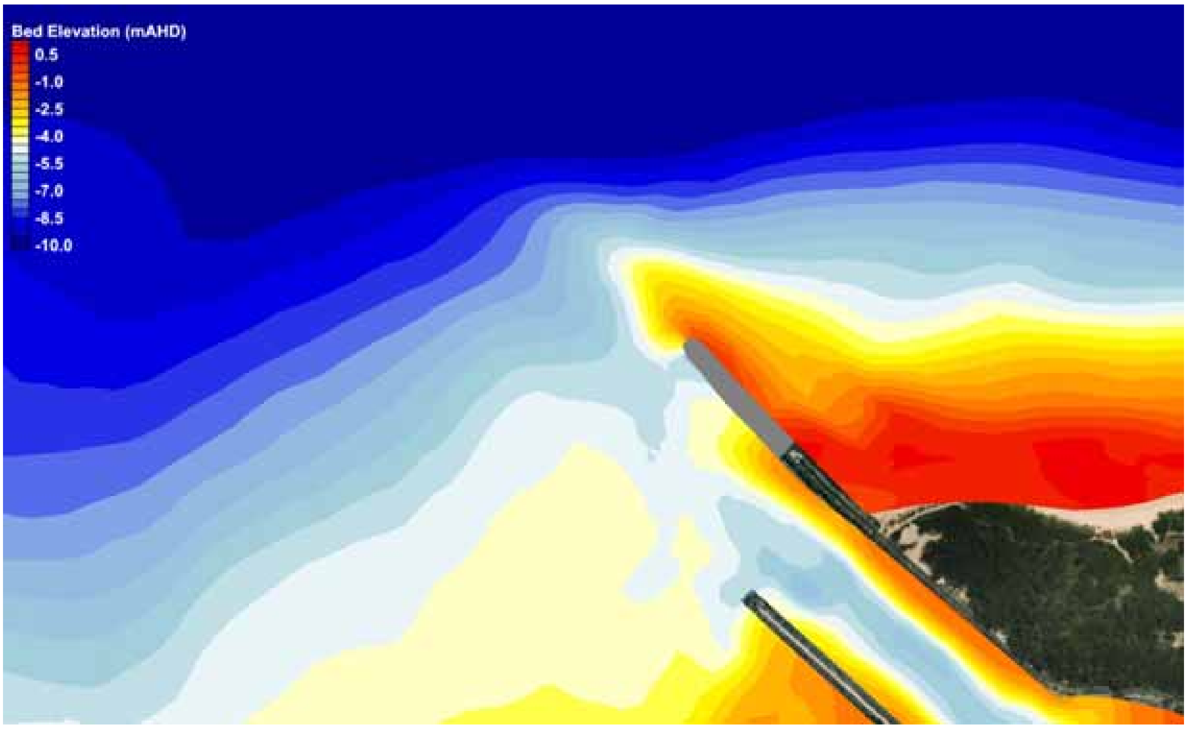


Figure 4-22 Option 3 Bed Elevation at the end of Assessment Period

4.4.2 Entrance Wave Conditions

A time series of significant wave height at the harbour entrance for the Option 3 capital works configuration is compared to the baseline wave condition in Figure 4-23. The breakwater extension enhances sheltering from the open coast wave conditions and reduces entrance wave heights by approximately 140%.

The Option 3 wave climate for the assessment period is presented as a wave rose in Figure 4-24 and shows the majority of wave energy is from the NNW to N sector, representing a minor westerly shift in wave direction relative to the baseline conditions. The predicted wave height within the harbour entrance did not exceed 0.5m during the assessment period.

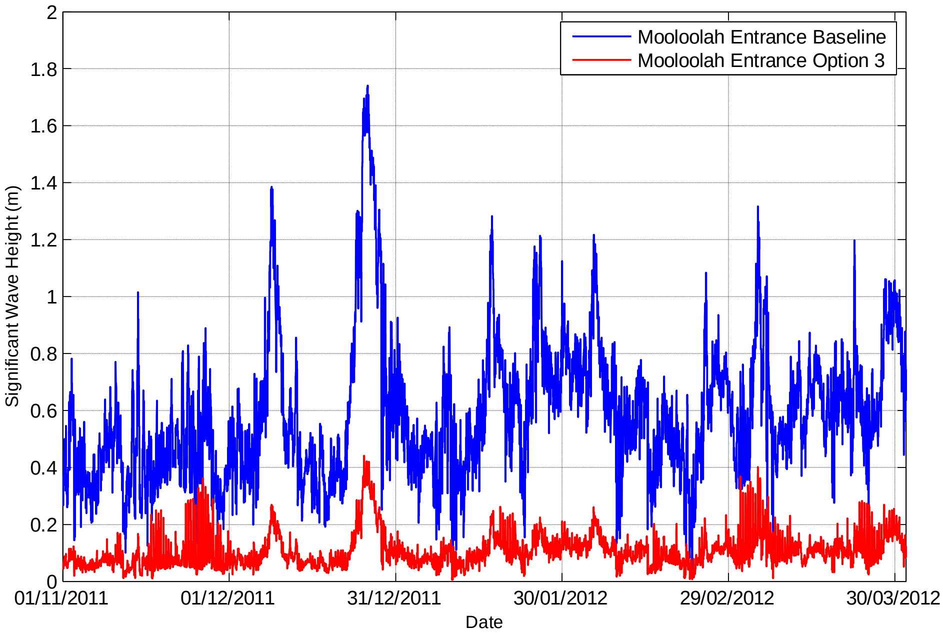


Figure 4-23 Option 3 and Baseline Harbour Entrance Significant Wave Height Time series Comparison

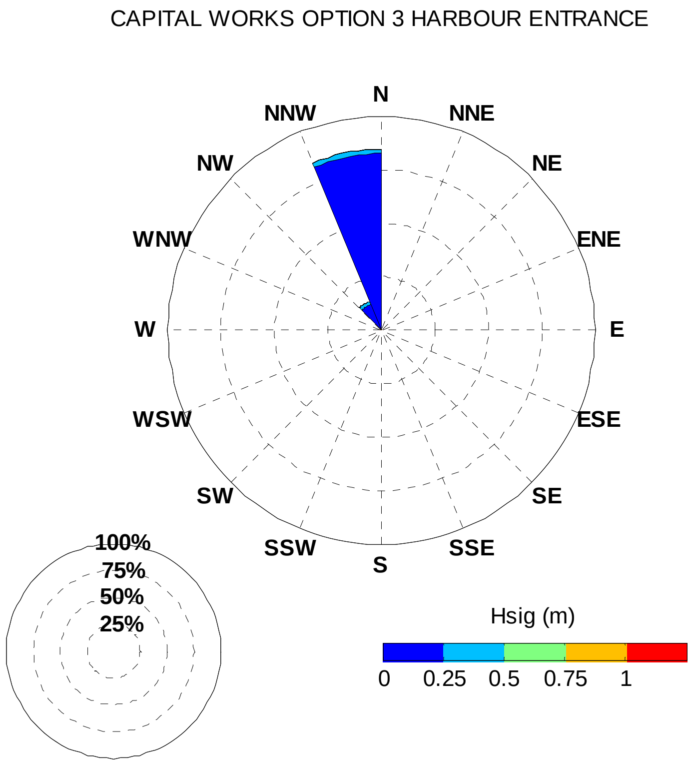


Figure 4-24 Capital Works Option 3 Harbour Entrance Wave Climate November 2011 to May 2012

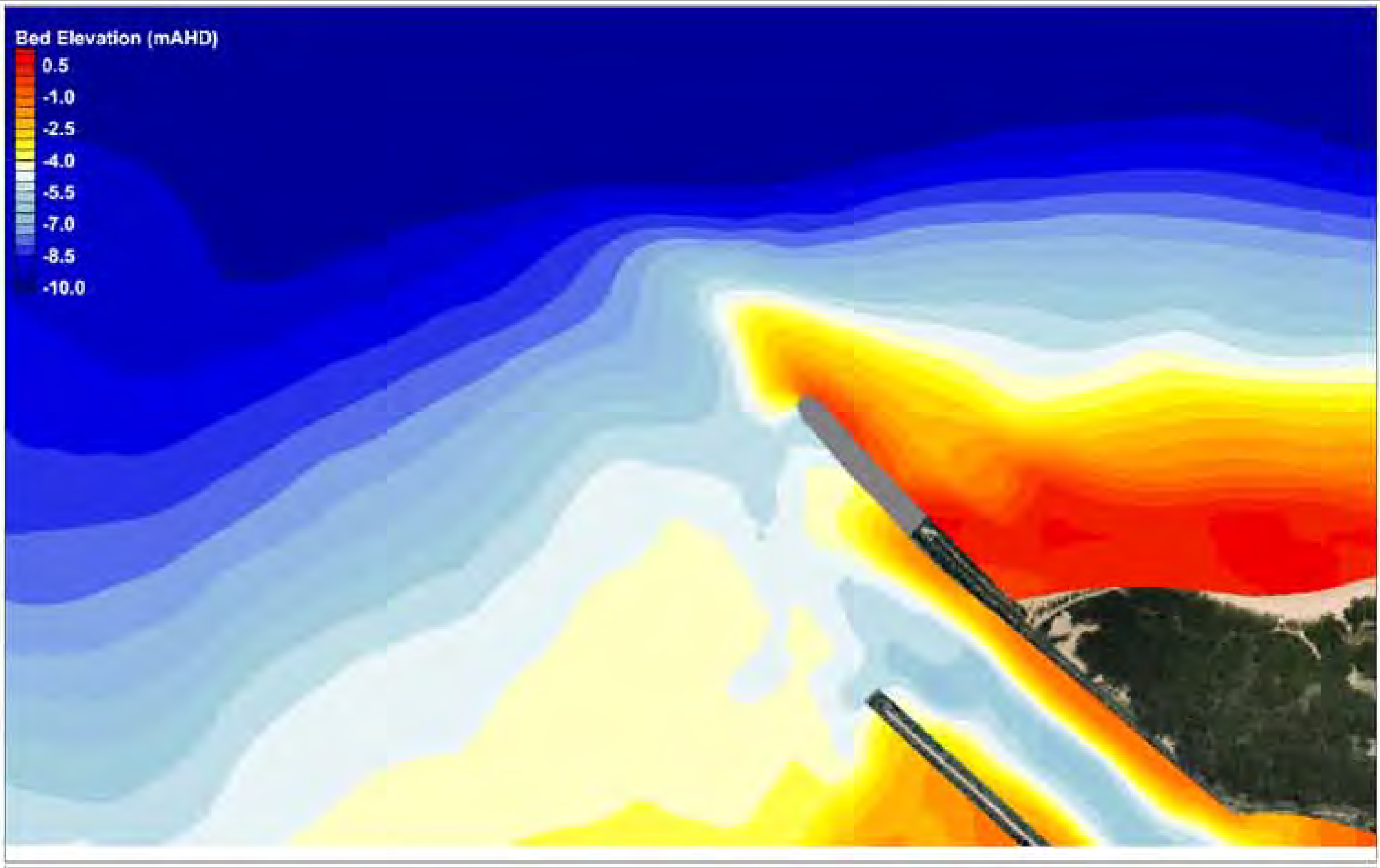
4.4.3 Natural Sand Bypassing of Entrance

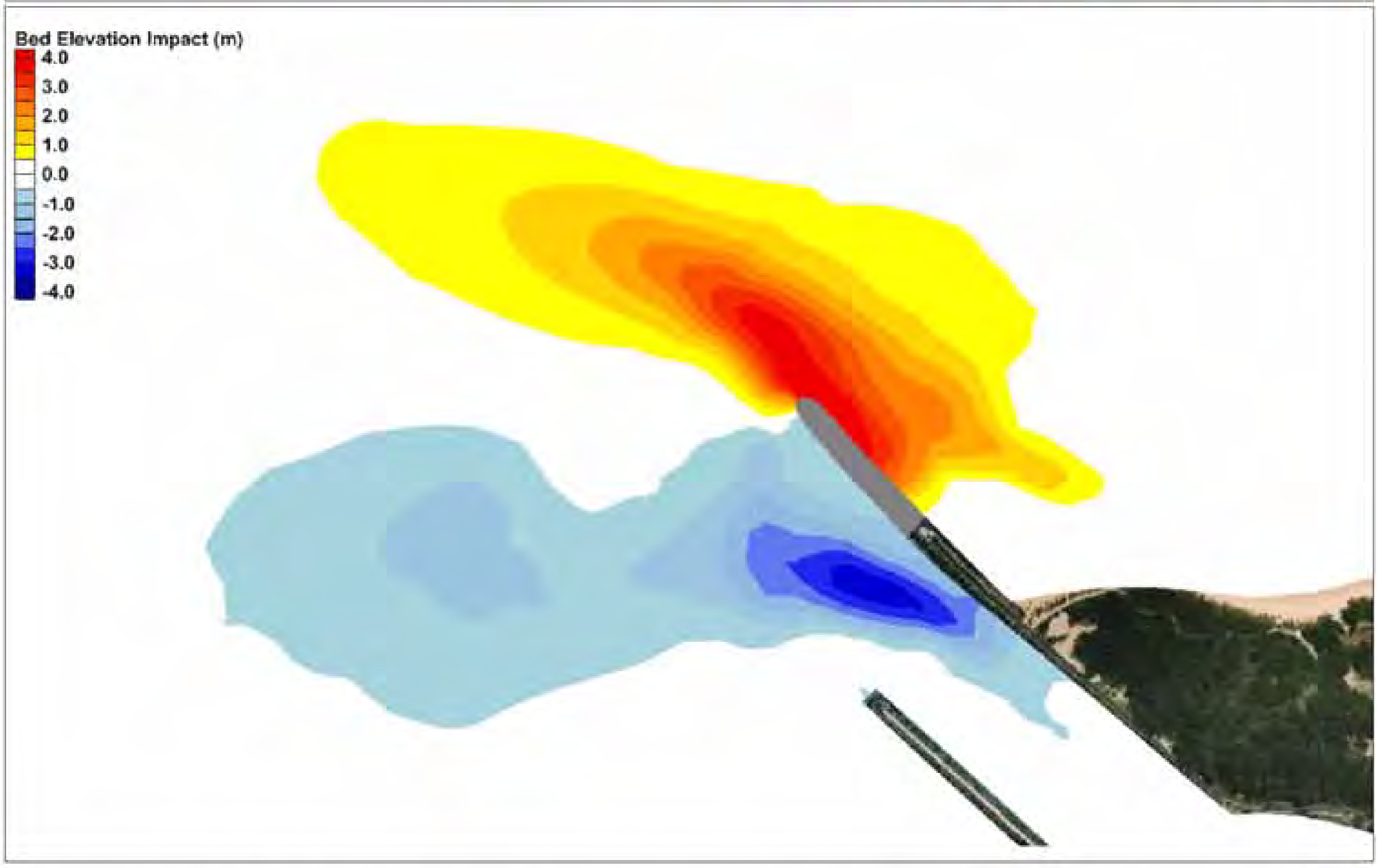
The Option 3 configuration assessment suggests the breakwater extension will successfully intercept sand before reaching the entrance and therefore significantly reduce the volume of sand naturally bypassing the entrance to Mooloolaba Bay. Table 4-3 provides estimates of the sand volume that passes the western breakwater (to an offshore depth of -10mAHD) during the simulation period. The Option 3 natural sand bypass volume is less than 15% of the volume predicted for the baseline configuration. This reduction in sand supply to Mooloolaba Bay would require mitigation via mechanical bypassing methods to avoid an undesirable shoreline recession response.

Table 4-3 Predicted Natural Sand Bypassing to Mooloolaba Bay during Assessment Period

Scenario	Natural Sand Bypass Volume (m^3)
Capital Works Option 3	12,100
Baseline (existing)	96,500
Impact	-84,400

Figure 4-25 shows the Option 3 shoal morphology and presents the spatial bed elevation impact (the difference between the baseline and Option 3 final bed elevation result). The blue contours represent sand deficit areas and clearly indicate a reduction in sand bypassing the entrance. The bed elevation impact results suggest the Option 3 capital works configuration redirects sand offshore to deeper water where it is less likely to move onshore toward Mooloolaba Bay shoreline.





Title:

Capital Works Option 3: Shoal Morphology (top) and Bed Elevation Impact (bottom)

Figure: 4-25

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N

0 75 150m

Approx. Scale

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www.bmtwbm.com.au

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4.4.4 Sand Accumulation at Eastern Breakwater

The Option 3 entrance configuration assessment suggests the extension enhances sand trapping and accumulation at the eastern breakwater. Figure 4-26 shows the spatial total bed elevation change result. Figure 4-27 provides a time series of bed change due to sand accumulation at the location adjacent to eastern breakwater where the maximum bed elevation change occurs (indicated in Figure 4-26). Relative to the baseline configuration, the Option 3 sand trapping capacity is significantly enhanced with a peak sand depth in excess of 5m predicted.

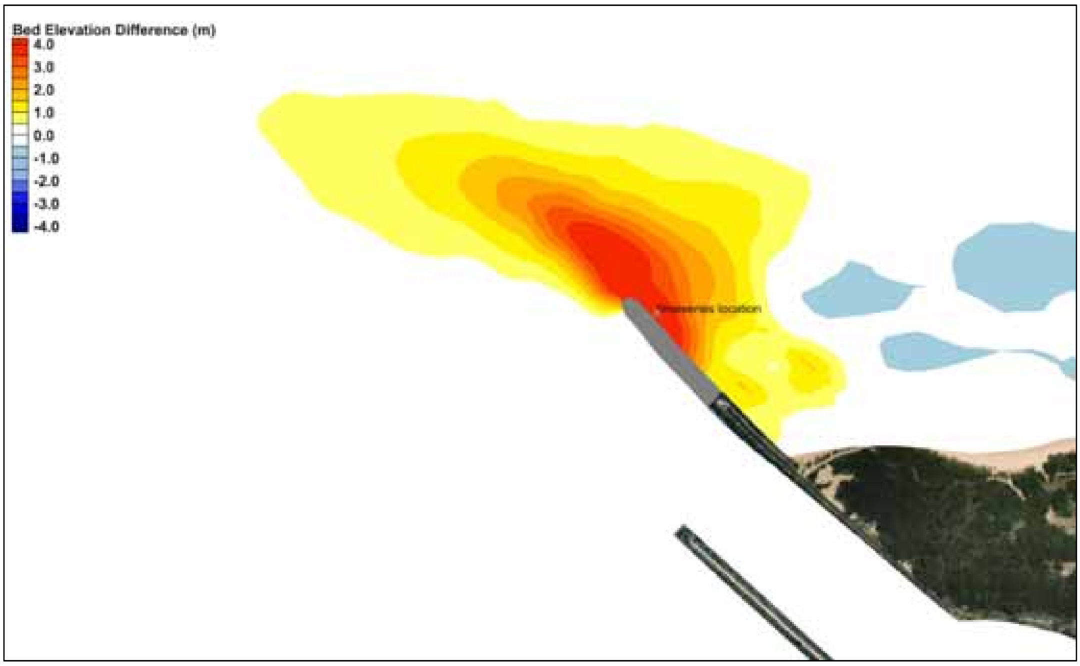


Figure 4-26 Bed Elevation Difference – Option 3

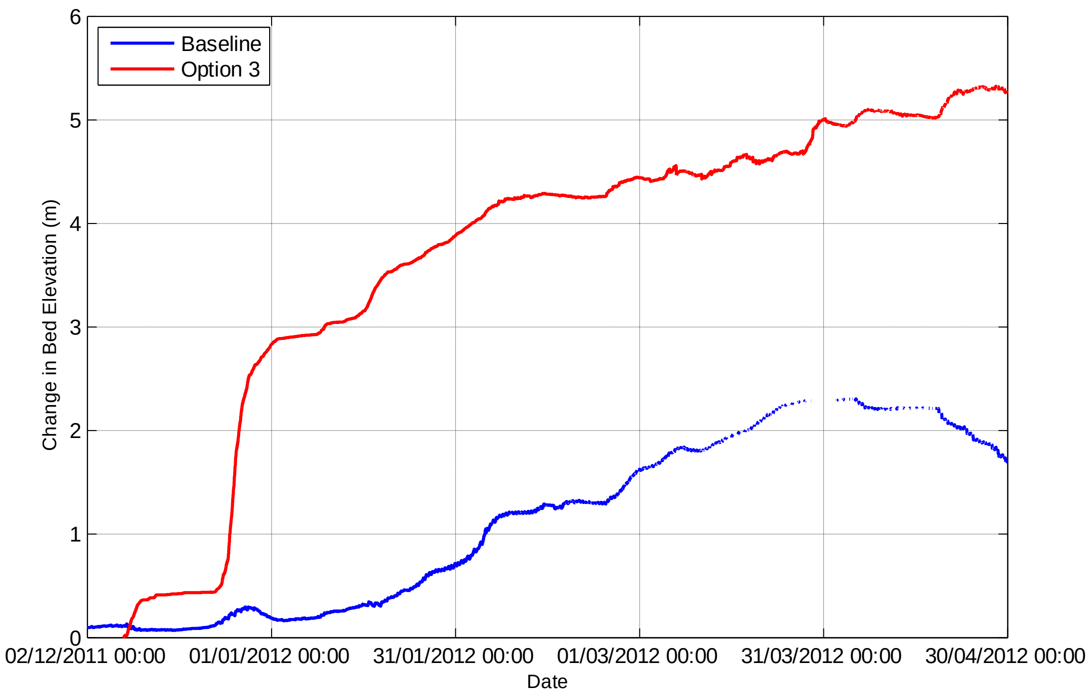


Figure 4-27 Bed Elevation Difference Time series – Option 3

4.4.5 Mooloolah River Flood Flows

The flood assessment result for the Option 3 capital works configuration is compared to the baseline result in Figure 4-28. The assessment result suggests the capital works option has an insignificant impact on peak water levels within the lower Mooloolah River under design flow.

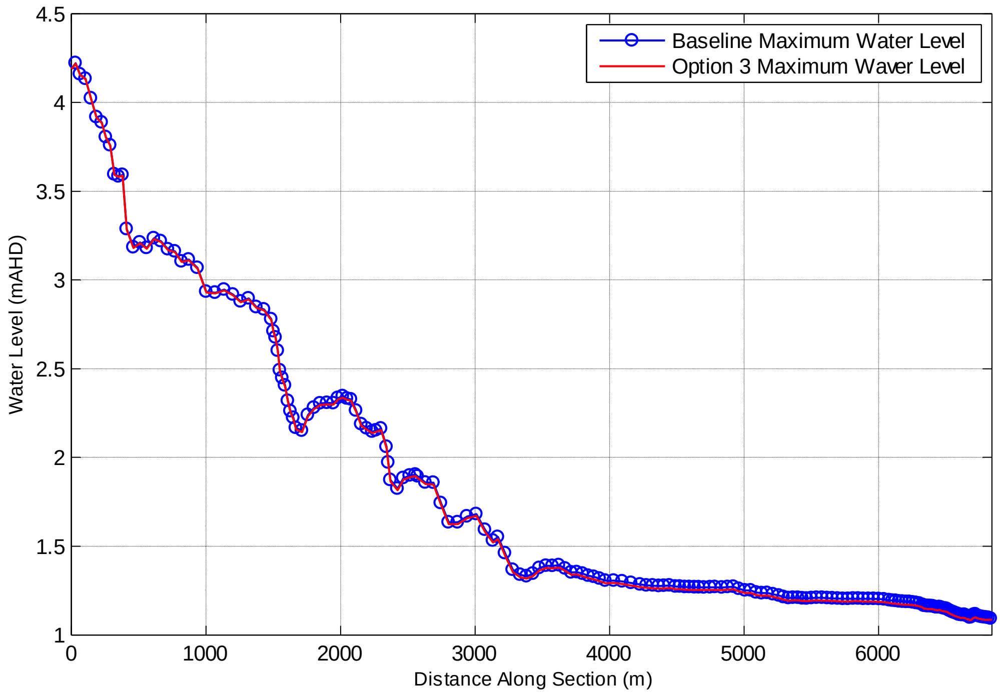


Figure 4-28 Option 3 and Baseline Flood Assessment Maximum Water Level Comparison

4.5 Option 4 Assessment

4.5.1 Navigation Channel Depth

The Option 4 entrance configuration assessment suggests a navigable channel with depth greater than -3mLAT (equivalent to -4mAHD) is maintained throughout the November 2011 to May 2012 simulation period. It is noted that sand bypassing to the entrance is predicted however at slower rates compared to the baseline entrance configuration. Figure 4-29 shows an accumulation of sand at the eastern breakwater and shoal development along the north face of the breakwater extension.

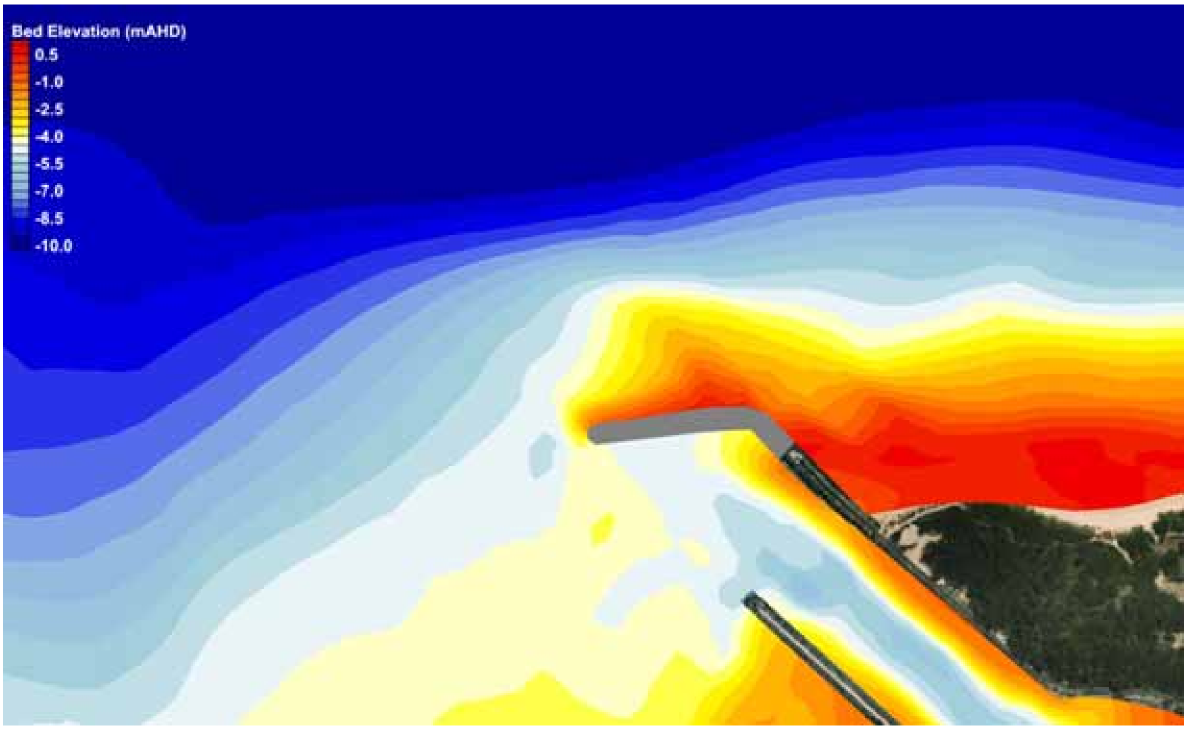


Figure 4-29 Option 4 Bed Elevation at the end of Assessment Period

4.5.2 Entrance Wave Conditions

A time series of significant wave height at the harbour entrance for the Option 4 capital works configuration is compared to the baseline wave condition in Figure 4-23. The breakwater extension enhances sheltering from the open coast wave conditions and reduces entrance wave heights by approximately 190%.

The Option 4 wave climate for the assessment period is presented as a wave rose in Figure 4-31 and shows the majority of wave energy is from the NW to NNW sector, representing a westerly shift in wave direction relative to the baseline conditions. The predicted wave height within the harbour entrance did not exceed 0.2m during the assessment period.

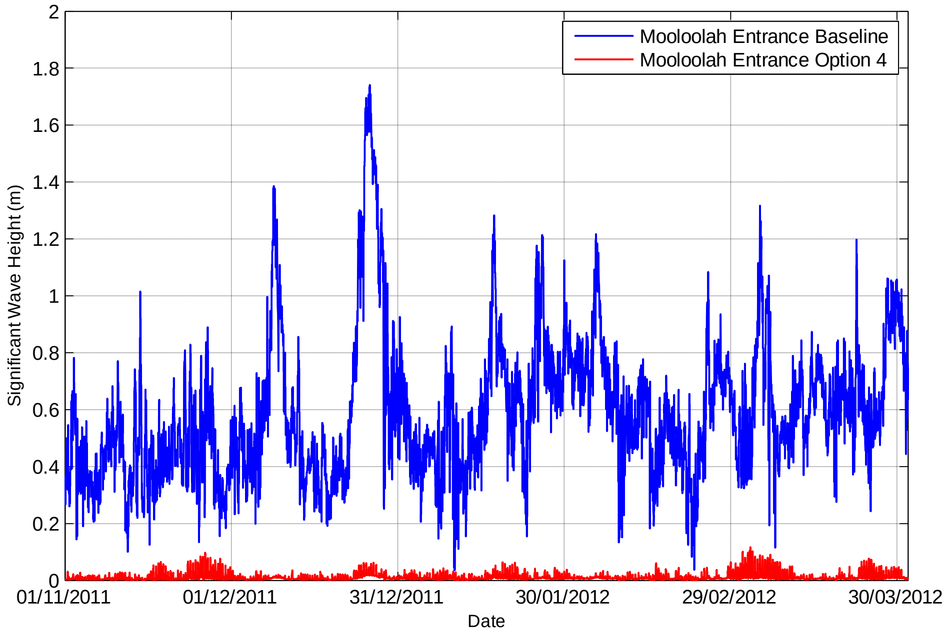


Figure 4-30 Option 4 and Baseline Harbour Entrance Significant Wave Height Time Series Comparison

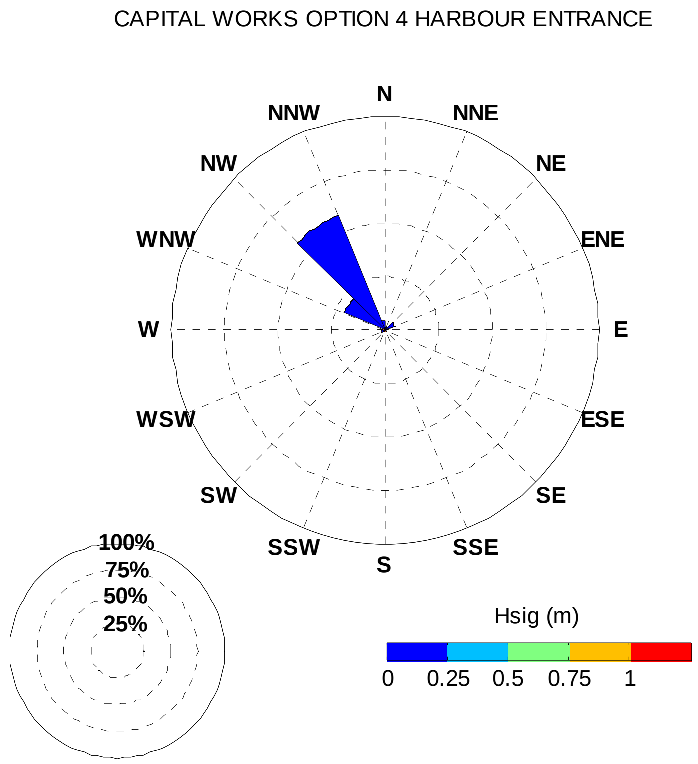


Figure 4-31 Capital Works Option 4 Harbour Entrance Wave Climate November 2011 to May 2012

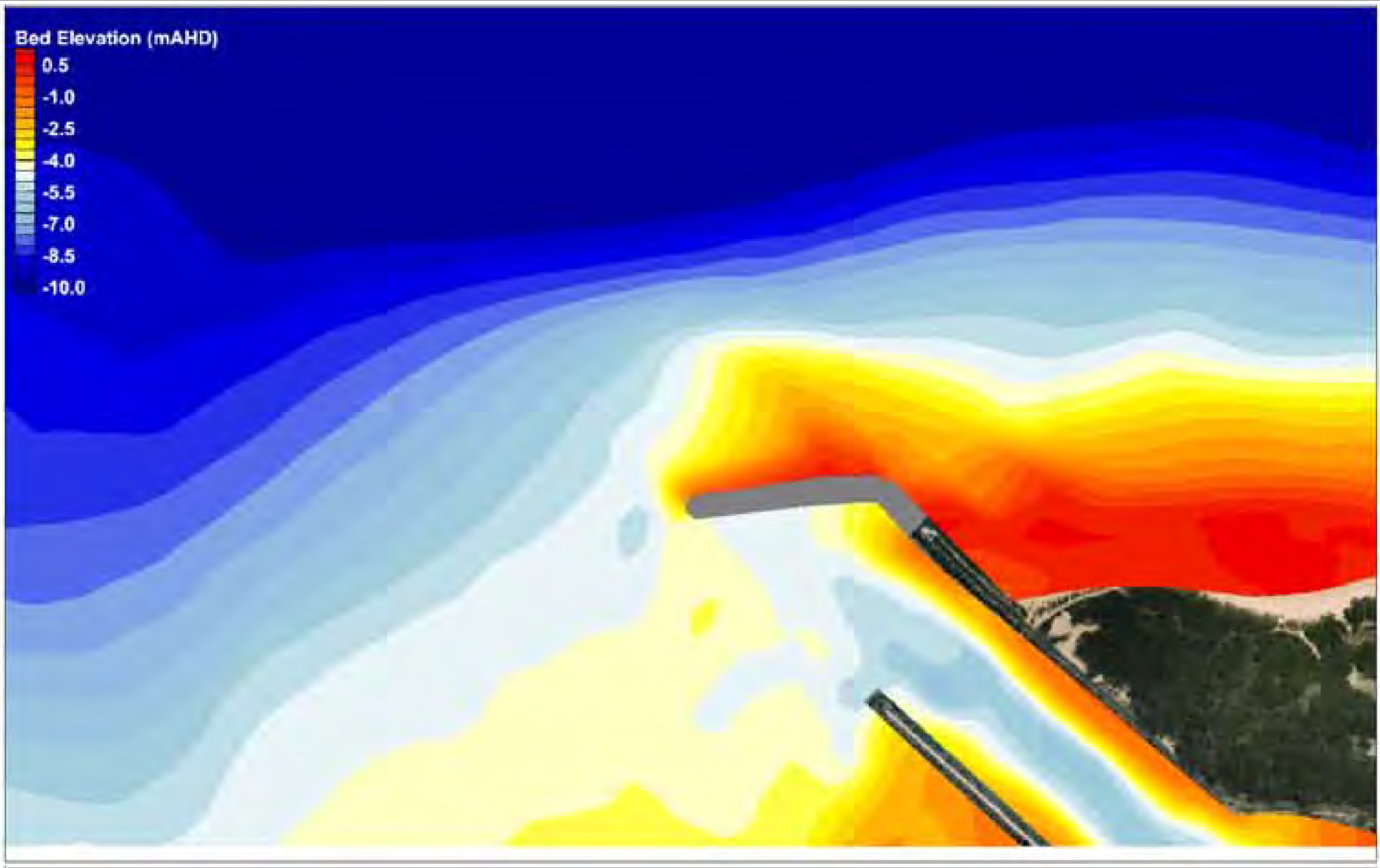
4.5.3 Natural Sand Bypassing of Entrance

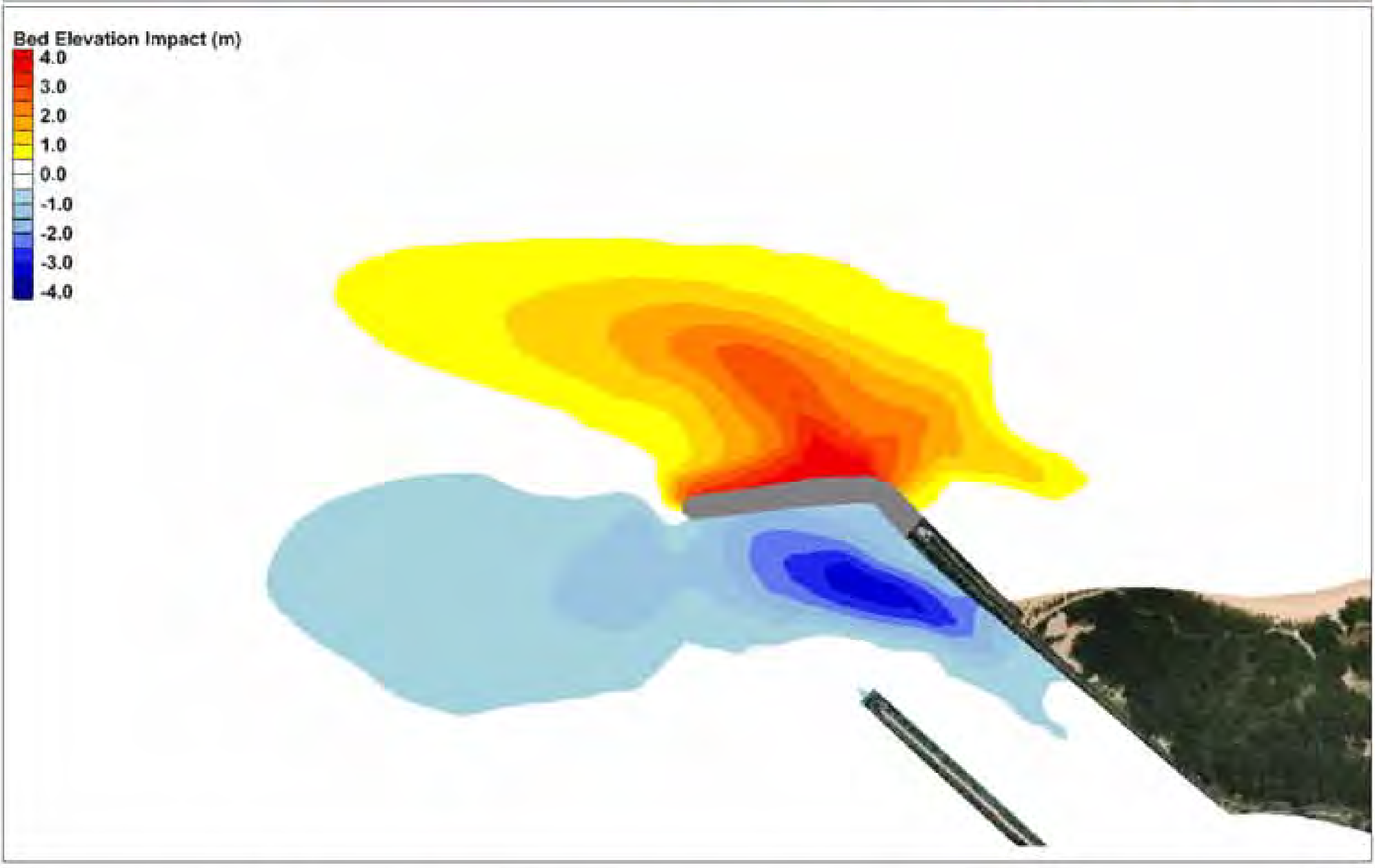
The Option 4 configuration assessment suggests the breakwater extension will reduce the rate at which sand naturally bypassing the entrance to Mooloolaba Bay. Table 4-4 provides estimates of the sand volume that passes the western breakwater (to an offshore depth of -10mAHD) during the simulation period. The Option 4 natural sand bypass volume is approximately 40% of the volume predicted for the baseline configuration. This reduction in sand supply to Mooloolaba Bay would require mitigation via mechanical bypassing methods to avoid an undesirable shoreline recession response.

Table 4-4 Predicted Natural Sand Bypassing to Mooloolaba Bay during Assessment Period

Scenario	Natural Sand Bypass Volume (m^3)
Capital Works Option 4	36,500
Baseline (existing)	96,500
Impact	-60,000

Figure 4-32 shows the Option 4 shoal morphology and presents the spatial bed elevation impact (the difference between the baseline and Option 4 final bed elevation result). The blue contours represent sand deficit areas and clearly indicate a reduction in sand bypassing the entrance. The bed elevation impact results suggest the Option 4 capital works configuration reduces the rate at which sand bypasses the entrance and moves toward the Mooloolaba Bay shoreline.





Title:

Capital Works Option 4: Shoal Morphology (top) and Bed Elevation Impact (bottom)

Figure: 4-32

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N

0 75 150m

Approx. Scale

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www.bmtwbm.com.au

Filepath : I:\B20224_I_mpb_Mooloolah_Entrace\DRG\COA_008_130802_DEV4_shoal_impact.wor

4.5.4 Sand Accumulation at Eastern Breakwater

The Option 4 entrance configuration assessment suggests the extension enhances sand trapping and accumulation along the north face of the breakwater extension. Figure 4-34 shows the spatial total bed elevation change result. Figure 4-35 provides a time series of bed change due to sand accumulation at the location adjacent to eastern breakwater where the maximum bed elevation change occurs (indicated in Figure 4-34). Relative to the baseline configuration, the Option 4 sand trapping capacity is significantly enhanced with a peak sand depth close to 5m predicted.

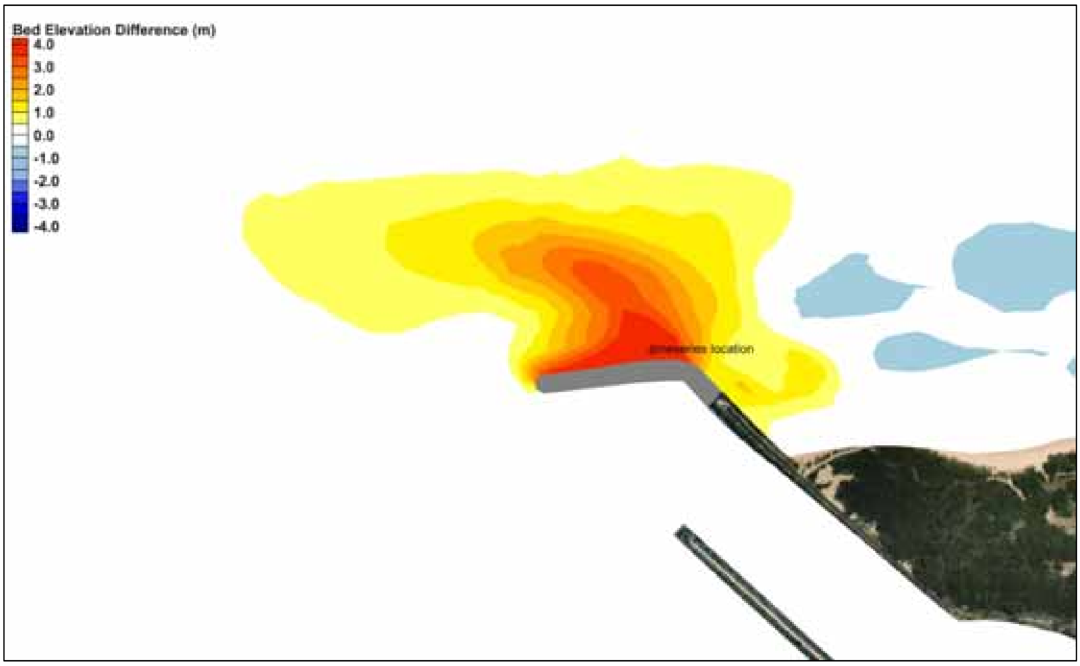

Figure 4-33 Bed Elevation Difference – Option 4

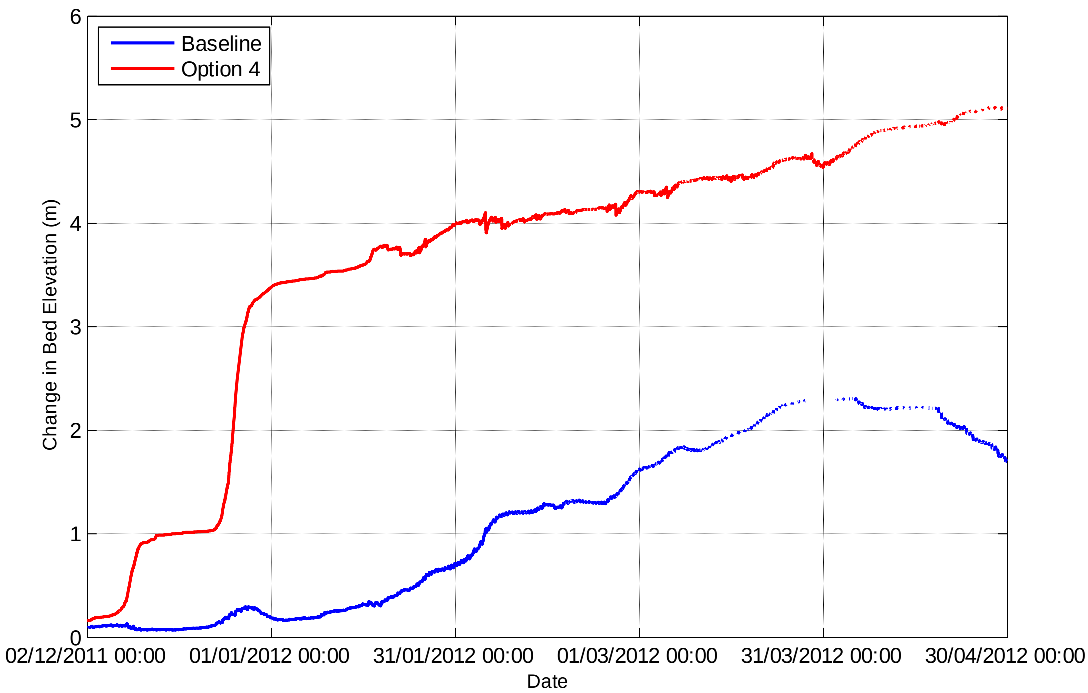

Figure 4-34 Bed Elevation Difference Time series– Option 4

4.5.5 Mooloolah River Flood Flows

The flood assessment result for the Option 4 capital works configuration is compared to the baseline result in Figure 4-35. The assessment result suggests the capital works option has an insignificant impact on peak water levels within the lower Mooloolah River under design flow.

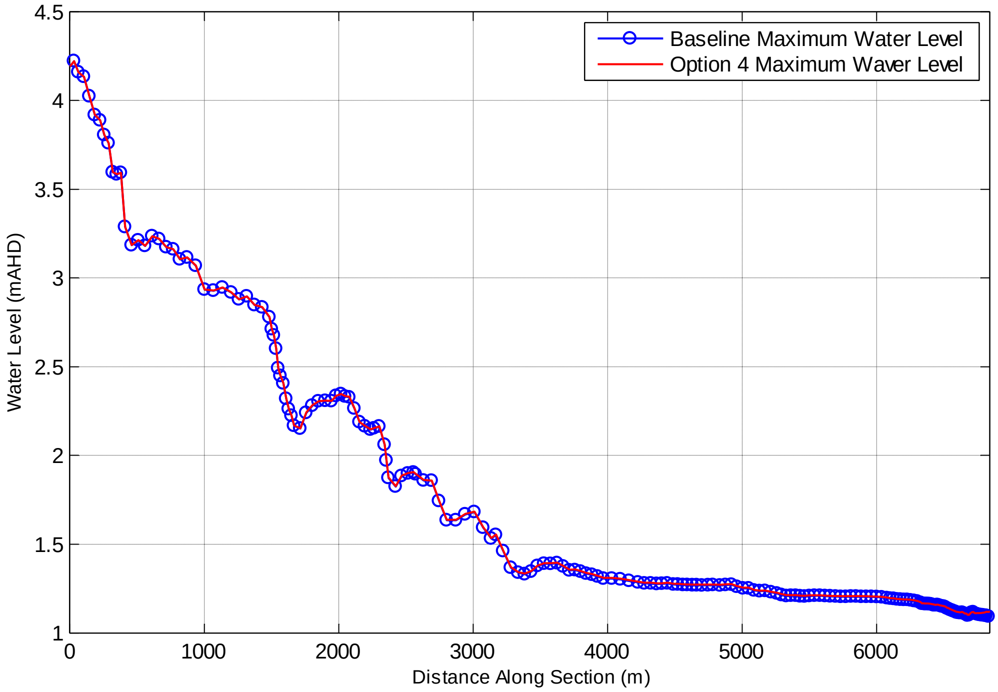

Figure 4-35 Option 4 and Baseline Flood Assessment Maximum Water Level Comparison

4.6 Breakwater Configuration Options Assessment Result Summary

The initial assessments of capital works options for the management of shoaling at the Mooloolah River Entrance have identified the following:

- The channel entrance design depth of -3mLAT (equivalent to -4mAHD) was maintained throughout the assessment period for each eastern breakwater extension. The Option 1 & 2 and Option 3 configurations direct sand into deep water offshore from the eastern breakwater. Sand bypassing to the entrance still occurred for Option 4 however at a slower rate than the baseline configuration. Over time, natural bypassing of the Option 4 breakwater is expected to result in shoaling of the entrance.
- A decrease in wave height at the harbour entrance is predicted for each capital works configuration. The mildest wave conditions are associated with Option 4 and Option 3 where the predicted significant wave height was less than 0.5m throughout the entire assessment period.
- Natural sand bypassing of the entrance is significantly reduced for Option 1 & 2 and Option 3. These breakwater configurations redirect sand offshore to deeper water where it is less likely to move onshore toward the Mooloolaba Bay shoreline. The reduction in sand supply to Mooloolaba Bay would require mitigation via mechanical bypassing methods to avoid an undesirable shoreline recession response. Option 4 permits some natural bypassing of the entrance however at a slower rate than the baseline configuration. Impacts to the Mooloolaba Bay sand supply would still require mitigation via mechanical sand bypassing for the Option 4 breakwater configuration.
- Sand accumulation at the eastern breakwater is enhanced for each capital works option. A peak sand accumulation depth of between 4-5m was predicted during the assessment period for Option 1 & 2 and 3. A similar sand accumulation depth was predicted at the north face of the Option 4 breakwater.
- Assessment of flood flows suggests the proposed capital works options have an insignificant impact to peak water levels in the lower Mooloolah River.

Table 4-5 Summary of Breakwater Options Assessment Results Summary

Breakwater Configuration	Maintenance of Navigation Channel	Significant Wave Height <0.5m (% of time)	Natural Sand Bypass Volume (% of Baseline)	Impact to river flow
Option 1 & 2	Yes	92	9	Negligible
Option 3	Yes	100	13	Negligible
Option 4	Yes*	100	38	Negligible

*Channel maintained during design event; entrance shoaling expected over time.

5 Capital Works Option Refinements

5.1 Introduction

Refinement of the capital works options, such as a reduction in length of the proposed breakwater extensions, may offer the following advantages:

- Reduced materials quantities, thereby reducing disturbances caused by rock extraction, delivery and placement;
- Reduced capital works costs; and
- Reduced ongoing maintenance requirements and costs.

Any refinement to the proposed breakwater extension design would still need to achieve the key objectives of an enhanced shoal management strategy, namely:

a) Maintenance of channel to a minimum depth of -3.0mLAT;

b) No adverse impact on entrance wave conditions;

c) Impact to Mooloolaba Spit; and

d) Impact to Mooloolah River flood flows.

The assessments presented in Chapter 4 suggest each of the eastern breakwater extensions successfully achieve (a), (b), and (d) for the design shoaling event however would fail to achieve (c) in the absence of additional mechanical sand bypassing. In this Chapter the consequence of reducing the length of the Option 3 eastern breakwater is considered. Potential mechanical sand bypassing methods intended to mitigate sand supply impacts are assessed in Chapter 7.

5.2 Option 3b Assessment

A refinement of the Option 3 breakwater configuration was investigated using the modelling system described in Section 4.1. The refined configuration considered a 60m extension of the eastern breakwater (reduced from 100m) and is referred to herein as Option 3b.

5.2.1 Option 3b Navigational Channel Depth Assessment

Capital works option assessment results presented in Section 4 suggest all proposed breakwater extensions successfully maintain a navigational channel during the design shoaling event. The risk associated with a reduction in breakwater length is failure to meet this key criterion.

The Option 3b entrance configuration assessment suggests a navigable channel with depth greater than -3mLAT (equivalent to -4mAHD) is maintained throughout the design event. Figure 5-1 shows an accumulation of sand at the eastern breakwater and shoal development is predicted offshore from the 60m breakwater extension. In comparison to Option 3 (i.e. the 100m breakwater extension described in Section 4.4), additional sand bypassing of the breakwater occurs and increased sand accumulation within the channel is predicted.

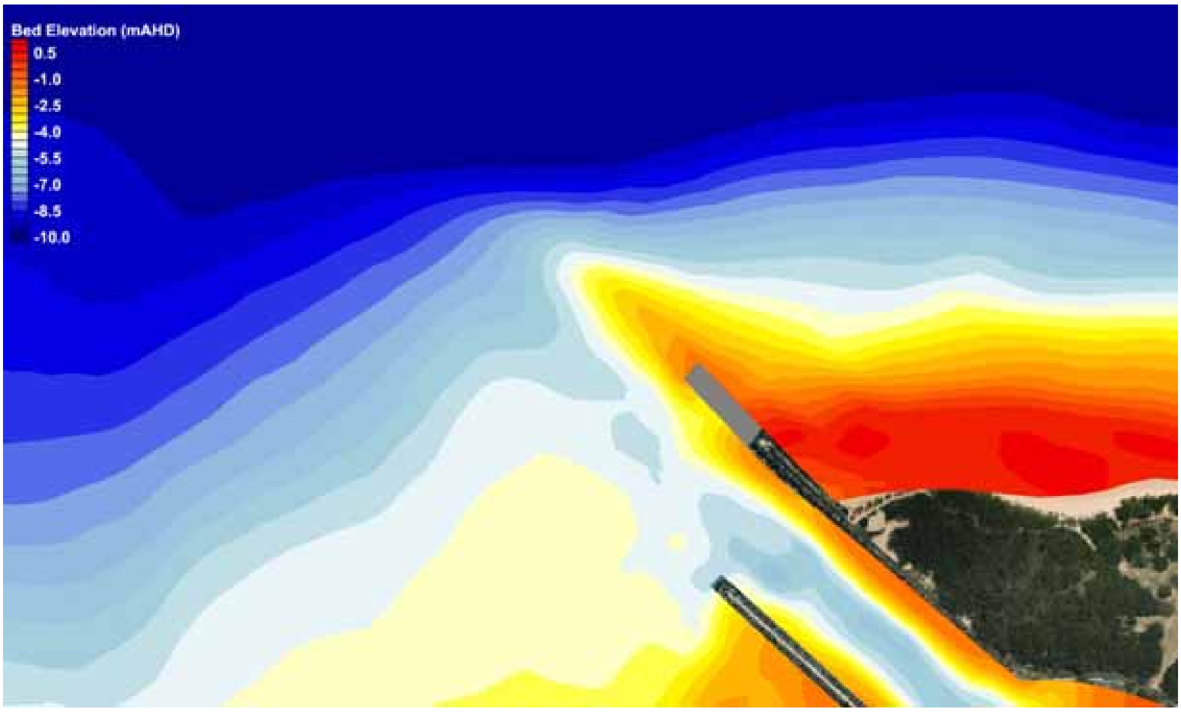

Figure 5-1 Option 3b Bed Elevation at the end of Assessment Period

5.2.2 Option 3b Natural Sand Bypassing of Entrance

The Option 3b configuration assessment suggests the 60m breakwater extension will successfully intercept a significant proportion of sand associated with the design shoaling event. Table 5-1 provides estimates of the sand volume that passes the western breakwater (to an offshore depth of -10mAHD) during the simulation period. The Option 3b natural sand bypass volume is less than 28% of the volume predicted for the baseline configuration. The reduction in sand supply to Mooloolaba Bay is less than the 100m breakwater extension (Option 3), however, it is expected that this reduced impact would still need to be mitigated via mechanical bypassing methods.

Table 5-1 Option 3b Predicted Natural Sand Bypassing to Mooloolaba Bay during Assessment Period

Scenario	Natural Sand Bypass Volume (m^3)
Capital Works Option 3b	26,600
Baseline (existing)	96,500
Impact	-69,900

6 Breakwater Extension Capital Works Assessments

6.1 Introduction

The quantity of armour stone and core material required for each capital works option is estimated in this Chapter. The estimates are based on design cross sections of the existing eastern breakwater. Commentary on design wave heights and appropriate armour layer characteristics is provided. Additional design wave and armour layer assessments may be required as part of the detailed design of a preferred option.

Capital works cost estimates are also provided and assume rock would be sourced from a state-owned quarry on Commercial Road, Kuluin. The existing breakwaters were constructed using rock from this source. It is estimated that approximately 9,000m^3 (20,000t) of blasted material is currently available on site and additional material could be released if required (pers. comm. TMR, 2013). Capital works costs would therefore be primarily associated with the transport, delivery and placement of material.

6.2 Existing Eastern Breakwater

Details of the existing eastern breakwater design were provided by TMR. The general arrangement, longitudinal section and cross section drawings are shown in Figure 6-1 and Figure 6-2 (noting imperial units). The offshore extent of the existing breakwater has the following design characteristics:

- Crest elevation = 5.2 meters above LAT
- Crest width = 4.9 meters
- Armour stone weight = 4.9 tonne
- Armour stone diameter = 1.2 meters
- Channel side armour rock layer thickness = 2.4 meters
- Ocean side armour rock layer thickness = 3.7 meters
- Core = quarry run material from 0.2m diameter up to 0.8m diameter adjacent to armour layer

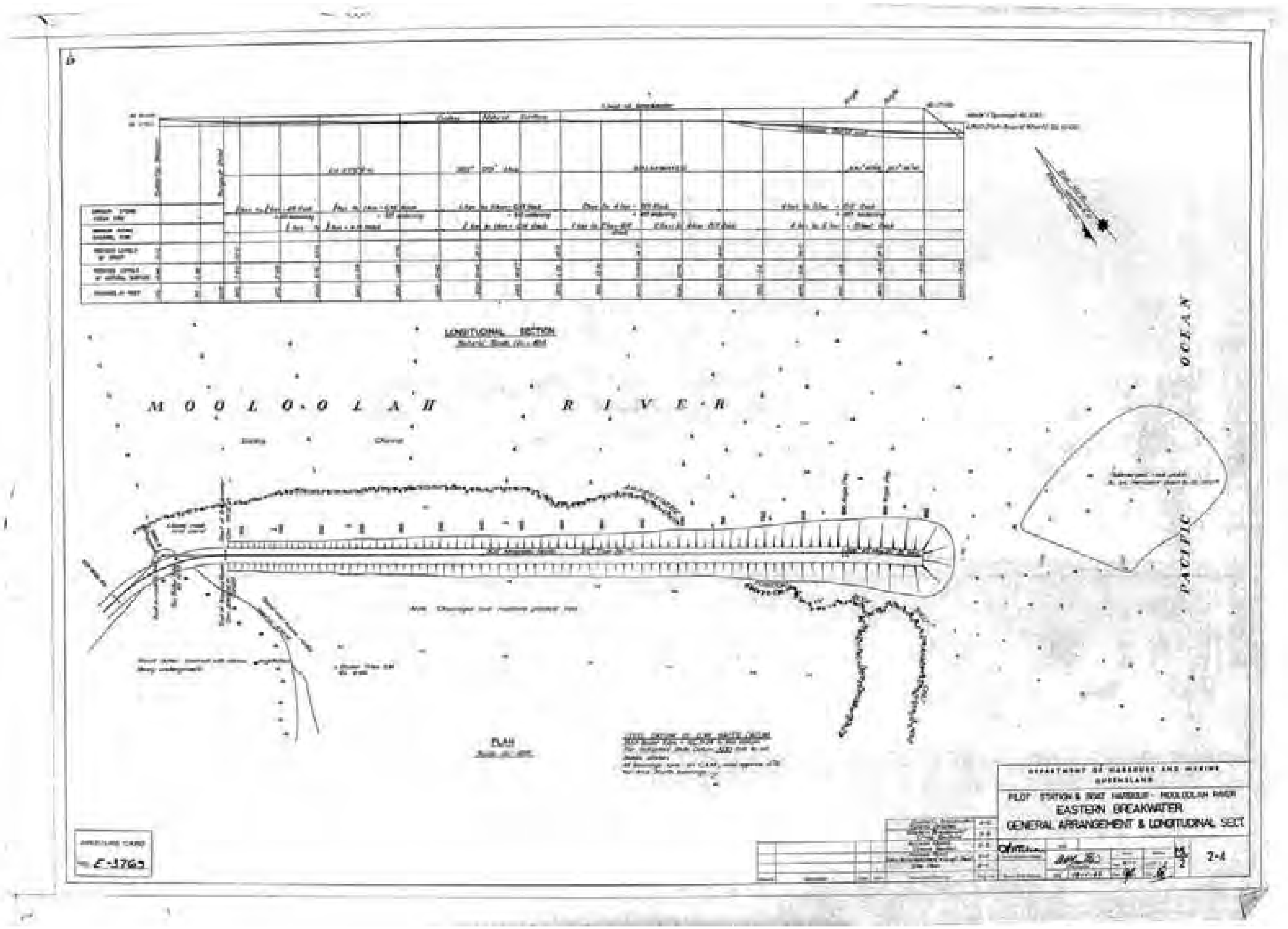

Figure 6-1 Existing Eastern Breakwater Longitudinal Section (provided by TMR)

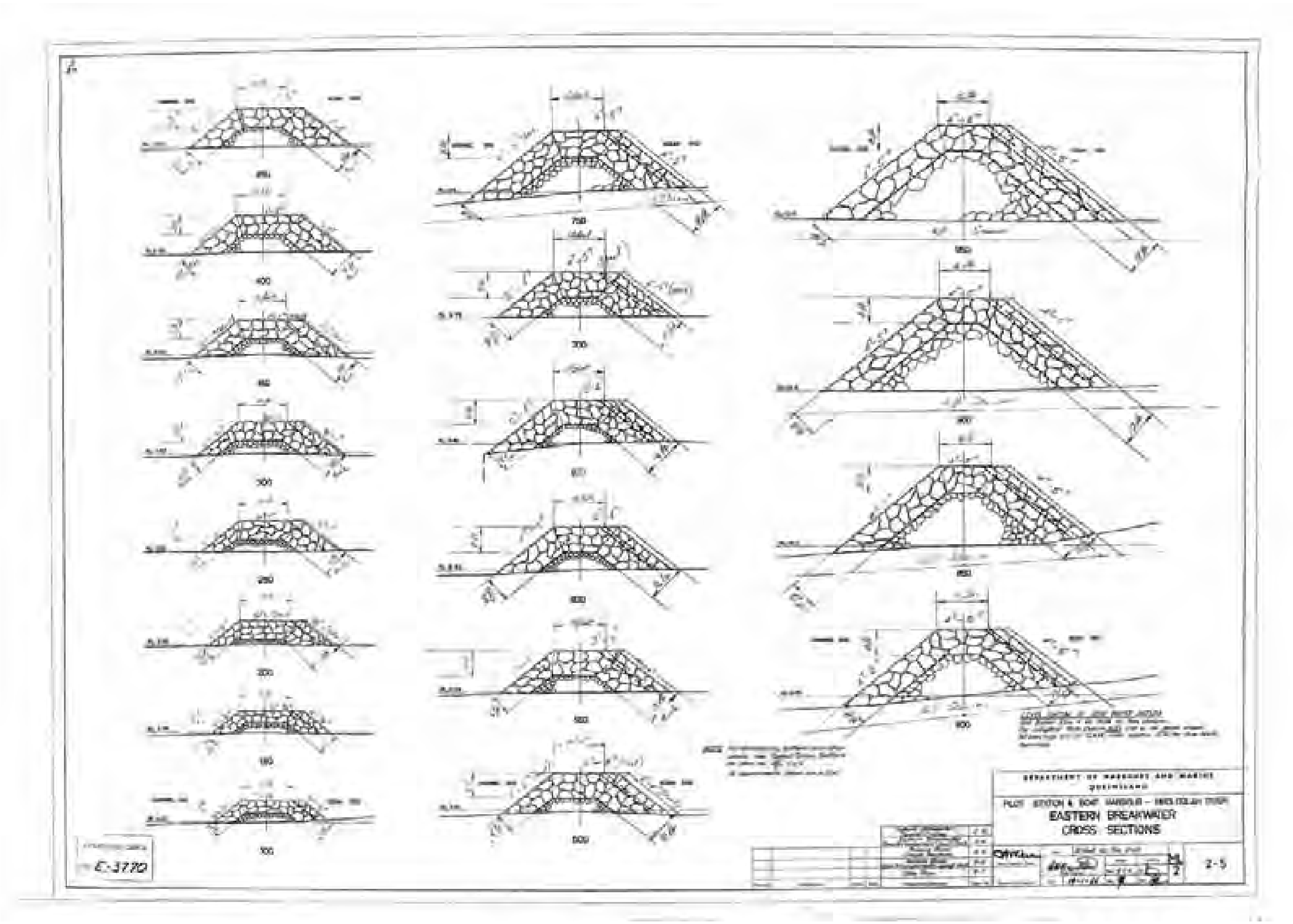

Figure 6-2 Existing Eastern Breakwater Cross Sections (provided by TMR)

Following Hudson's (1953, 1959) design methods, the seaward extent of the existing breakwater would be expected to remain stable under wave attack for nearshore significant wave heights up to approximately 3m. The relationships between individual armour stone mass, size and significant wave height are shown in Figure 6-3.

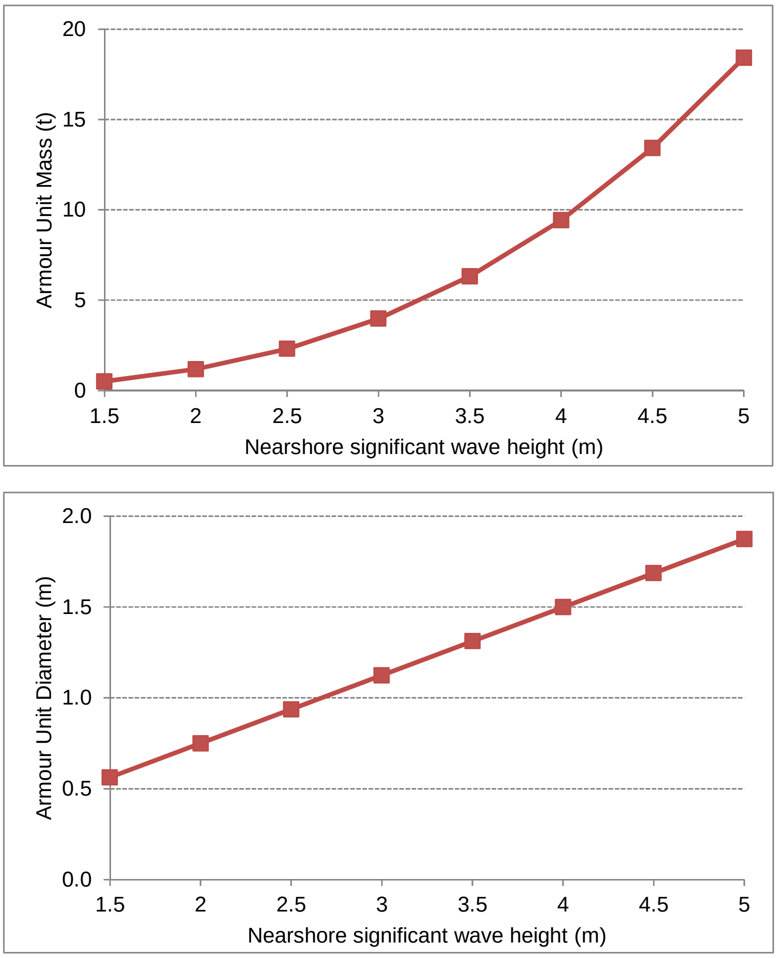

Figure 6-3 Relationship between Armour Stone Characteristics and Significant Wave Height – 2:1 Sloping Permeable Structure (Hudson 1953, 1959)

The Australian Standard Guidelines for the design of maritime structures (AS 4997-2005) suggest the design wave for "normal maritime structures" should be based on the highest 1% of waves (H_1) associated with the 500 year Annual Recurrence Interval (ARI) design storm event. Considering this criterion and the conservative relationship $H_1 \approx 2H_{sig}$, the existing breakwater is designed to remain stable for up to $H_1 \approx$ 6m. If a capital works option was adopted, assessment of design wave heights associated with 500 year ARI design storm and appropriate armour layer characteristics would need to be confirmed as part of a detailed design of the preferred breakwater extension.

For the purpose of rock quantity estimates provided in the following Section, it has been assumed similar armour stone characteristics (i.e. 4.9 tonne, 1.2m diameter) would be suitable for an extension of the eastern breakwater.

6.3 Breakwater Extension Capital Works Options Assessments

An indicative cross sectional area of armour stone and core material was developed to allow an estimate of the rock quantity required for the breakwater extension options. The cross sectional area was approximated from the existing offshore cross section shown in Figure 6-2 (chainage 950) and conservatively assumed a constant bed elevation of -7mAHD in the proposed breakwater extension area[1]. The following cross sectional armour stone and core material requirements were derived:

- Armour stone layer area ≈ 86m^2
- Core material area ≈ 148m^2

These cross sectional areas are used below estimate the required quantity of armour stone and core material for each breakwater extension option. The adopted bed elevation is expected to lead to “upper estimates” of rock quantity. The conservative bed elevation assumption is also intended to capture other uncertainties such as the design wave height criterion and required armour stone mass (refer Section 6.2)

6.3.1 Breakwater Extension Rock Quantities

Armour stone and core material quantity estimates for each breakwater extension option are presented in Table 6-1. The cubic meter quantities are based on the length of the extension and the design cross sectional area at the offshore extent of the existing structure (refer Section 6.2). The total mass estimates assume basalt rock with mass density 2800kg/m^3 and a notional permeability value of 0.4 for a rock structure with a relatively impermeable core (e.g. CIRIA, 2007).

Table 6-1 Breakwater Extension Rock Quantity Estimates

Extension	Length (m)	Core Material (m^3)	Armour Material (m^3)	Total (m^3)	Core Material (tonne)	Armour Material (tonne)	Total (tonne)
Option 1 & 2	85	12,603	7,297	19,900	21,173	12,258	33,432
Option 3	100	14,827	8,584	23,412	24,910	14,422	39,331
Option 3b	60	8,896	5,151	14,047	14,946	8,653	23,599
Option 4	120	17,793	10,301	28,094	29,892	17,306	47,198

6.3.2 Breakwater Extension Capital Costs

Capital cost estimates based on the quantity of rock required for each breakwater extension option are summarised in Table 6-2 and consider the following:

- Total tonne rock quantity estimates provided in Table 6-1
- \$35/t to transport and deliver rock
- \$20/t to place rock (assuming placement with land-based equipment)

[1] Bedrock level estimates previously obtained on behalf of TMR suggest elevations between -5.5mAHD and -6.9mAHD in the breakwater extension area.

Table 6-2 Breakwater Extension Rock Transport, Delivery and Placement Cost Estimates

Extension	Rock Quantity (tonne)	Rock Cost ($ inc GST)
Option 1 & 2	33,432	1,838,744
Option 3	39,331	2,163,229
Option 3b	23,599	1,297,937
Option 4	47,198	2,595,874

In additional to the above, TMR have also provided an indicative cost of $455,000 to for activities associated with the proposed capital works, including (pers. comm. TMR, 2014):

- $250,000 for rock blasting works at the Commercial Road quarry (up to 55,000t)
- $300,000 for road access to site and repair/rehabilitation after completion of capital works
- $100,000 for sorting and crushing rock onsite
- $25,000 for re-establishing navigational aids

7 Mechanical Relocation of Accumulated Sand

7.1 Introduction

On average, approximately 5,000-10,000m^3/year of sand is estimated to bypass the Mooloolah Entrance and enter the Mooloolaba Bay beach system (e.g. BMT WBM, 2013). The annual bypassing volume is observed to be significantly greater during episodic shoaling events.

The modelling assessments presented in Chapter 4 and 5 suggest each breakwater extension reduces the rate at which sand naturally bypasses the entrance to Mooloolaba Bay. This reduction in sand supply has the potential to cause undesirable shoreline erosion impacts to Mooloolaba beaches and is expected to require mitigation via mechanical bypassing methods. Three potential sand bypassing methods have been considered and assessed:

- Dredging and placement
- Sand shifter system similar to the Noosa Main Beach facility
- Crane-mounted mobile jet pump

Discussion of the logistics and cost to implement these sand supply management strategies is provided in this Chapter. It is noted that the success of these methods being used in conjunction with an eastern breakwater extension remains uncertain and may need additional design considerations and optimisation through field trials.

7.2 Dredging and Placement Sand Bypass Method

Since training of the Mooloolah River, TMR has followed a reactive management strategy to maintain the entrance channel design depth. The approach uses a shoaling prediction tool (WBM Oceanics 2004 and 2005) with monitoring of seabed changes via hydrographic surveys as an early warning system so that dredge equipment can be mobilised to mechanically move sand from the entrance. The aerial image in Figure 7-1 shows a Cutter Suction Dredge (CSD) operating offshore from the western breakwater and transferring dredged sand via a pipeline to the eastern corner of Mooloolaba Beach.

Figure 7-1 Dredging at the Mooloolah Entrance January 2010 (image source: NearMap)

Dredging of accumulated sand from the updrift side of the eastern breakwater and placement on Mooloolaba Bay beaches via a pipeline is a potential method to mitigate a reduced rate of natural sand bypassing caused by the proposed breakwater extension. It is noted that the location where sand accumulates (i.e. in an exposed wave climate, including wave reflection from the extended breakwater structure) may present conditions that challenge standard dredging techniques and therefore this sand bypassing option carries a significant risk of cost overruns due to operational delays.

Cost estimates provided in the following section assume that the dredging can be undertaken using conventional methods. The operational feasibility of this potential sand bypassing approach would need to be assessed by a qualified dredging consultant, with consideration given to the required dredge equipment and anticipated local conditions.

7.2.1 Dredging and Placement Costs

Assuming that dredging from the updrift side of the extended breakwater is operationally feasible, it is expected that works would need to be carried out at 2-year intervals in order to mitigate sand supply impacts to Mooloolaba Bay. Each dredge campaign would seek to relocate up to 30,000m^3 of sand from the eastern breakwater to Mooloolaba Bay. The estimated sand dredging and placement costs are presented in Table 7-1. Based on previous dredging experience at the Mooloolah Entrance, TMR has raised concern regarding the operational feasibility of this proposed bypassing method. Consequently, the cost estimate in Table 7-1 includes a substantial standby allowance.

Table 7-1 Dredge Sand Bypassing Method Cost Estimate per Campaign

Activity	Assumptions	Quantity	Cost
Dredge mobilisation	Cutter Suction Dredge 400mm	1	$250,000
Dredge de-mobilisation	-	1	$25,000
Sand volume dredged, delivered and re-profiled on beach	$7/m^3 for delivery distances less than 1km (assumed existing pipeline adequate)	30,000 m^3	$210,000
Standby allowance	$750 per hour standby rate	480 hrs	$360,000
Dredge Campaign Total Cost (per two years)			**$845,000**

7.3 Sand Shifter Sand Bypass Method

As briefly discussed in Section 1.1, a sand shifter trial operated by Slurry Systems Marine Pty Ltd was commissioned by TMR during 2012 to investigate the potential to intercept and artificially bypass sand across the Mooloolah River entrance. The sand shifter system was installed at Point Cartwright in the lee of the eastern breakwater where sand accumulation was anticipated. A successful sand bypassing system would transfer accumulated sand via a pipeline from the eastern breakwater to the shoreline at Mooloolaba Bay, thereby mimicking the “natural” entrance bypassing mechanisms and reduce shoaling and maintenance dredging requirements.

The 2012 trial showed that the system was not able to work efficiently due to the shallow thickness of sand across the rock shelf and inadequate sand trapping capacity of the present entrance configuration. The potential for a sand shifter system to operate in conjunction with an eastern breakwater extension is explored in the following sections.

7.3.1 Existing Sand Shifter System - Noosa Main Beach

Slurry Systems Marine and Sunshine Coast Council currently operate a sand shifter system at Noosa Main Beach, Queensland. Following a number of successful trials of the system (commencing in 2004) a permanent installation of a dual sand shifter system was completed in 2012. The sand shifter units are installed in the lee of the eastern Noosa River breakwater and permanently buried (typically beneath 4.5 to 6m of sand) at a location where sand transported by littoral drift processes tends to accumulate. During shifter operation a sand/seawater mixture is pumped updrift to eroded beach sections. The system thereby assists in maintaining a beach by recycling sand that would otherwise be lost northwards from the beach unit. The existing system has the capacity to pump up to 80,000m^3 of sand per annum. A sand shifter unit and sand/seawater pumping at Noosa Main Beach is shown in Figure 7-2.

Figure 7-2 Noosa Main Beach Sand Shifter System: a) Sand Shifter Unit before burial; b) Sand/Seawater Mixture Pumping (Slurry Systems Marine)

Details of the Noosa Main Beach sand shifter system operational performance during early 2013 was provided by Slurry System Marine for consideration in this project. Production from late January to mid-May 2013 is show in Figure 7-3. After this time operation was suspended for an extended period, to be reinstated at a later date as required. During this period approximately 33,300m^3 of sand was relocated from the eastern breakwater back to the beach, corresponding to an average of approximately 325m^3/day.

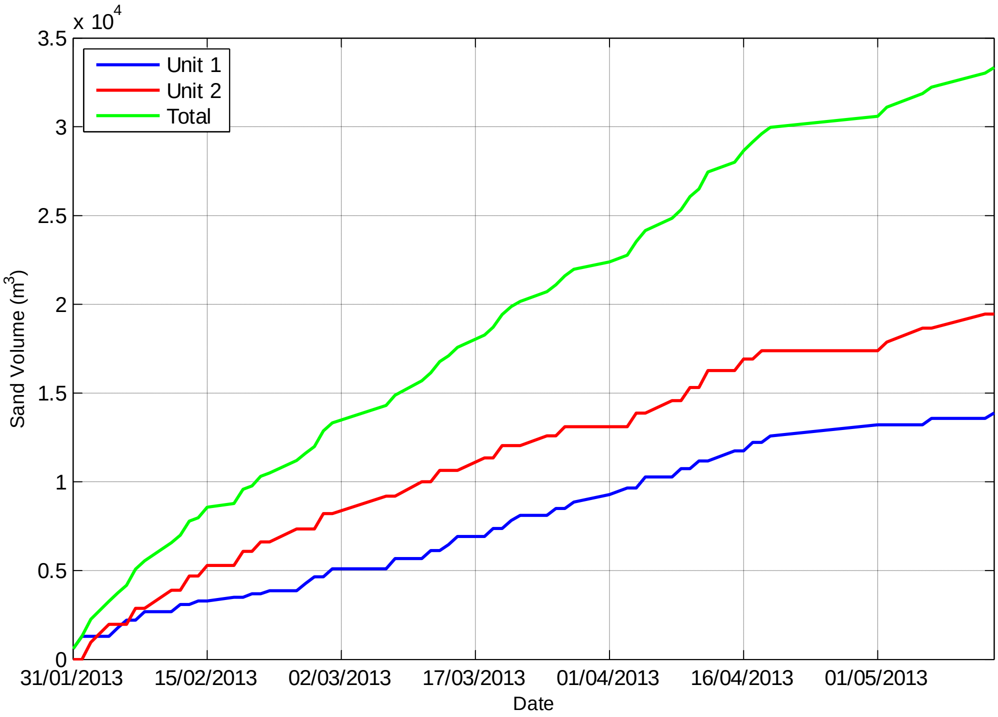

Figure 7-3 Noosa Main Beach Sand Shifter System Production during 2013 (Slurry Systems Marine)

7.3.2 Mooloolah Entrance Sand Shifter System Potential

If demonstrated to be operationally viable, a sand shifter system with similar production capacity to Noosa Main Beach could be installed at the Mooloolah entrance. The intention of this system would be to bypass sand from the eastern breakwater to Mooloolaba Bay to mitigate the impact to sand supply associated with a breakwater extension.

A high-level numerical assessment of a sand shifter operating in conjunction with the Option 2, Option 3 and Option 3b breakwater configuration was undertaken for the design shoal event. It was assumed a sand shifter system could not operate effectively with the Option 4 breakwater (pers. comm. Lex Nankervis, 2013) and therefore this configuration was not assessed. Key simplifications of the assessment include:

- An assumption that the sand shifter could be installed at the location where peak sand accumulation occurs.
- The number of sand shifter units simulated depended on the size of the model cell where the peak sand accumulation was predicted. For the Option 2 breakwater configuration, a single sand shifter unit was simulated with a maximum production rate of 800m^3/day. For the Option 3 and Option 3b breakwater configurations, two sand sifter units (located in adjacent model cells) were simulated, each with a maximum production rate of 400m^3/day. The total sand shifter production rate of up to 800m^3/day depends on the availability of sand within the model cell where the sand shifter unit is located.

- The sand shifter could operate continuously for a five month period (December 2011 to April 2012).
- Sand could be extracted to the bed rock level. In reality, a sand shifter unit would be situated approximately 1m above the bed rock and therefore could not extract sand from below this level.

The final shoal morphology results are shown in Figure 7-4, Figure 7-5 and Figure 7-6. For each configuration, the assessment suggests the sand shifter intercepts a relatively small fraction of the shoal with the majority of the sand moving past the units further offshore.

The volume of sand bypassed for each breakwater and sand shifter configuration is summarised in Table 7-2.

The assessments suggest relatively low daily transport rates and total bypass volumes would be realised with a sand shifter system, with production limited by a general lack of sand accumulation in the vicinity of the sand shifter units. Considering the natural bypassing volume associated with the design event and the baseline scenario (approximately 96,500m^3), the predicted sand bypassing volume achieved with the sand shifter system is less than 10% of the existing case. It is assumed that the sand shifter efficiency and production rates could be improved through optimisation of the unit locations. Given the limitations and uncertainty in representing a sand shifter numerically, this would be better explored through field trials following the proposed capital works.

Table 7-2 Sand Shifter Assessment Results

Breakwater Configuration	Average Production Rate* (m^3/day)	Sand Shifter Bypass Volume (m^3)
Option 2	35	5,190
Option 3	60	8,800
Option 3b	65	9,570

*Based on continuous operation for assessment period

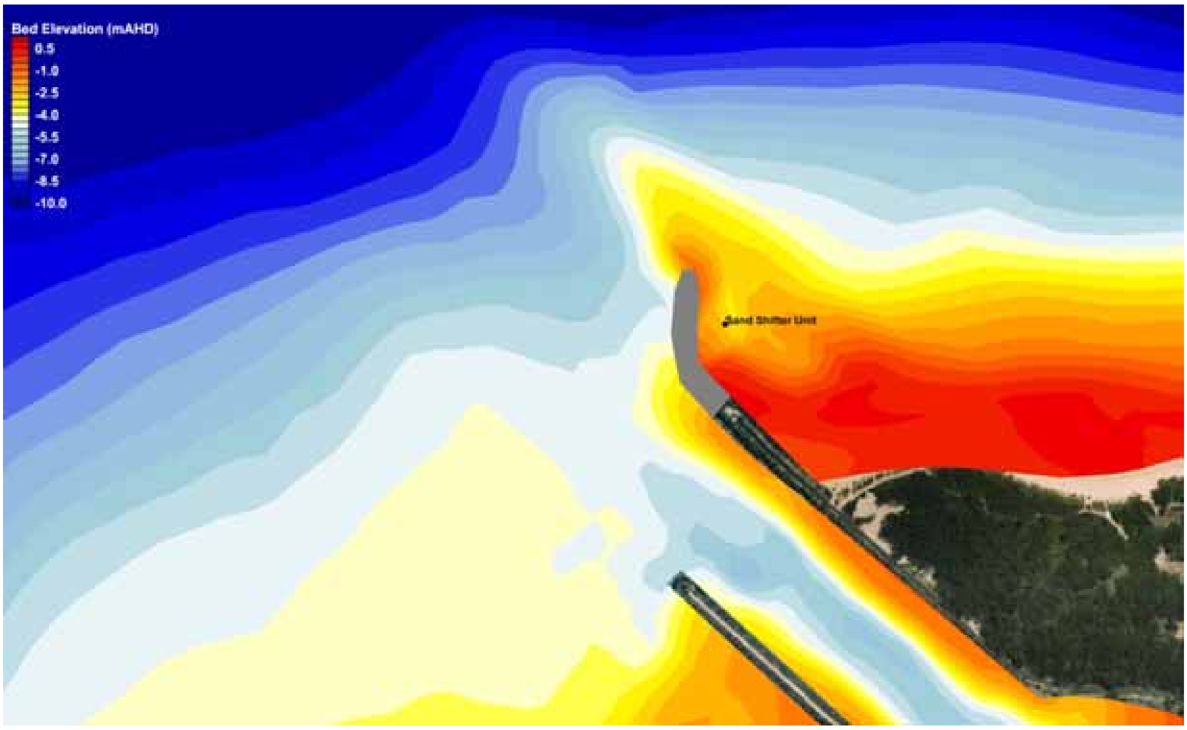

Figure 7-4 Option 2 with Sand Shifter Bed Elevation at the end of Assessment Period

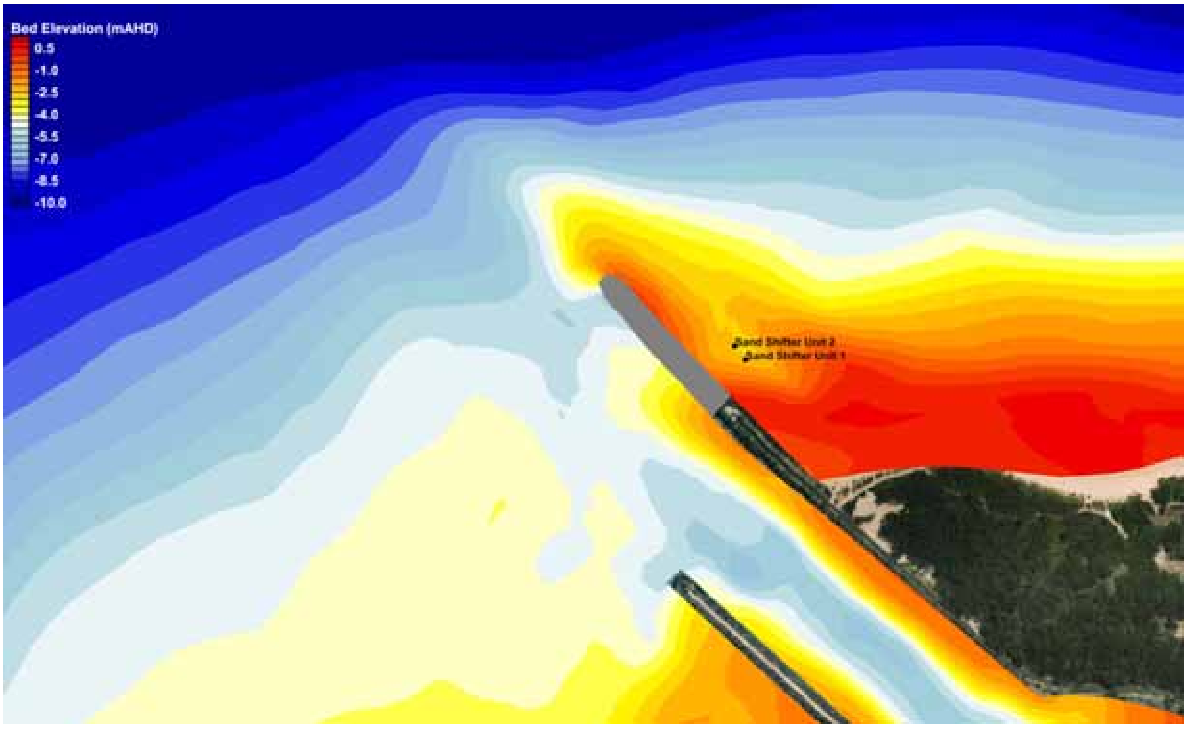

Figure 7-5 Option 3 with Sand Shifter Bed Elevation at the end of Assessment Period

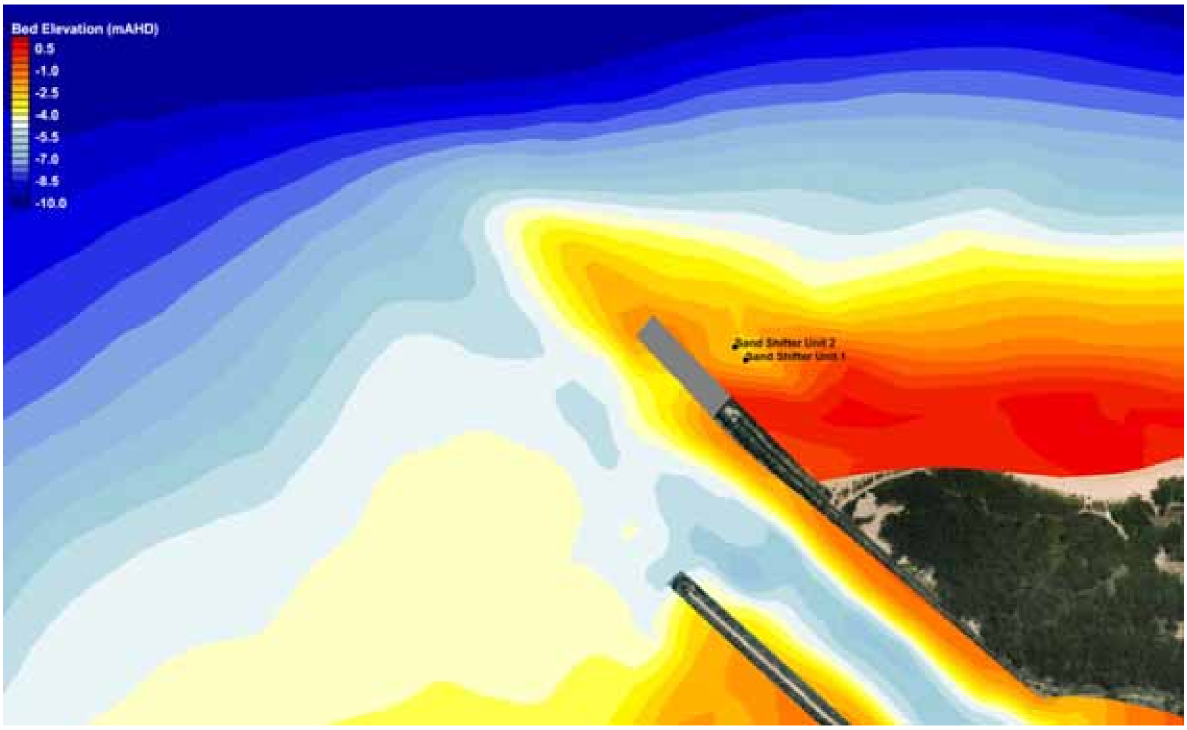

Figure 7-6 Option 3b with Sand Shifter Bed Elevation at the end of Assessment Period

7.3.3 Mooloolah Entrance Sand Shifter System Costs

Capital and operational costs associated with a sand shifter unit at the Mooloolah Entrance eastern breakwater have been developed by Slurry Systems Marine and are provided in Table 7-3 and Table 7-4. In developing cost estimates, the system was assumed to have a similar production capacity to the Noosa Main Beach facility; however, the numerical assessments presented in Section 7.3.2 suggest lower actual production rates may be realised.

Table 7-3 Sand Shifter Capital Cost Estimate provided by Slurry Systems Marine

Sand Shifter Capital Item	Cost
Site Establishment	$175,000
Site and Civils	$308,000
Structural	$257,000
Mechanical Equipment	$359,000
Buildings	$169,000
Electrical	$362,000
Conduit installation for HV power line extension 945m	$52,000
HV power line cable	$68,000
Instrumentation & Control	$117,000
Piping	$210,000
Commissioning	$16,000
Engineering and Design Services	$190,000
Sand Shifter Capital Total Cost	**$2,283,000**

Table 7-4 Sand Shifter Annual Operational Cost Estimate provided by Slurry Systems Marine

Activity	Assumptions	Quantity	Cost
Sand shifter operation (appointed contractor to manage the facility)	Monthly operational cost at $10,800/month, including water pumping equipment supplied by contractor	12 months	$129,600
Sand shifter volume rate	Volume rate $3.50/m^3 assuming 20,000 m^3/year	20,000 m^3	$70,000
Sand shifter power cost	Power cost $0.45/m^3 (off-peak power)	20,000 m^3	$9,000
Sand shifter maintenance	-	NA	$10,000
		Sand Shifter Annual Total Cost	**$218,600**

7.4 Crane with Mobile Jet Pump Sand Bypass Method

A jet pump, or "eductor", deployed by a crawler crane has been demonstrated to be an effective sand bypassing method. The system relies on a supply pump to deliver water to the eductor via a high-density polyethylene (HDPE) pipeline. The eductor is deployed by a crane to the target area where it excavates the sand and draws a sand/seawater mixture. The slurry is then pumped to the discharge location. This is the permanent sand bypassing method used at the Indian River Inlet, Delaware (refer Section 7.4.1), and has been successfully trialled by Slurry Systems Marine at Lakes Entrance, Victoria.

7.4.1 Existing Crane with Mobile Jet Pump System – Indian River Inlet, Delaware

The Indian River Inlet has a trained entrance and shows the classic updrift accretion and downdrift erosion pattern associated with a dominant net sediment transport direction. Beach nourishment to protect infrastructure and recreational values associated with the downdrift shoreline has been undertaken since the mind 1950's (USACE, 1994).

An eductor deployed from a crawler crane forms part of a fixed-sand bypassing system at the Indian River Inlet and has operated since 1990. The supply and booster pumps are contained within a pump house which is permanently situated behind the sand dunes. The sand/seawater slurry flows to the booster pump via a HDPE pipeline and is pumped across the bridge to the downdrift side of the inlet. An aerial photograph of the bypassing system is shown in Figure 7-7.

Figure 7-7 Crane-mounted Mobile Jet Pump Sand Bypass System - Indian River Inlet, Delaware (USACE, 2013)

7.4.2 Mooloolah Entrance Crane with Mobile Jet Pump System Potential

A high-level assessment of a crane-mounted jet pump sand bypass system the Mooloolah Entrance was undertaken using the capital works options shoal morphology results presented in Section 4 (Option 1 and Option 3) and Section 5 (Option 3b). An important parameter when assessing this bypassing method is the working range of the crane. To this end, the assessment considered the approximate sand volume that could be accessed using 50t and 90t crawler cranes with working ranges of 30m and 50m respectively. Key assumptions of the assessment included:

- The crawler cranes could access and be transported along the existing breakwater and the potential capital works extensions.
- The 50t crane would require a 4m wide area to operate and that this area was available at any position along the breakwater. Operating positions at the head of the breakwater and at a mid-point of the breakwater were adopted for the assessment.
- The 90t crane would require a 6m wide area to operate, necessitating a purpose built operating platform towards the head of the breakwater. The additional capital works costs associated with this breakwater modification is considered in Section 7.4.3.1.
- Shoreline accretion on the updrift side of the breakwater was sufficient to allow the crane to also operate from a beach position (close to the 0mAHD contour).
- Bypassing volumes are based on the static shoal morphology at the end of the design period.

The adopted crawler crane positions and working ranges for each breakwater configuration and crane size are illustrated in Figure 7-8 to Figure 7-13. For the 50t crane assessment three operating positions were assumed (breakwater head, mid-breakwater and beach). Only two positions were assumed feasible with the 90t crane (breakwater head and beach).

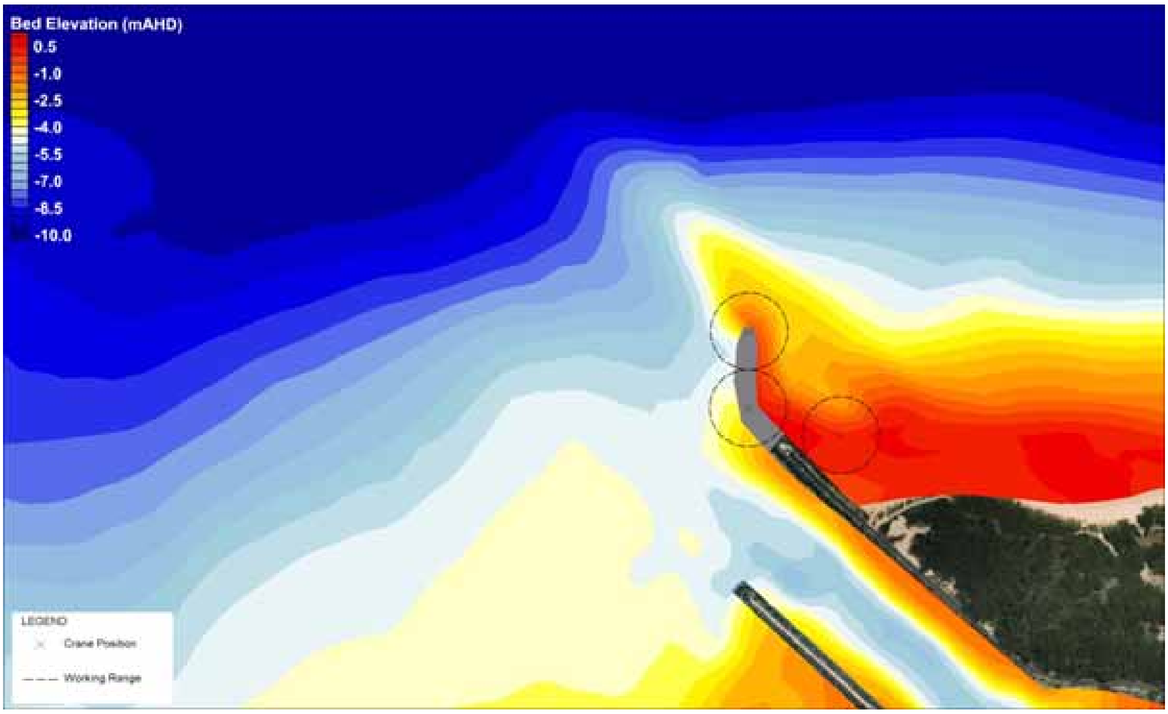

Figure 7-8 Option 2 50t Crawler Crane Positions and Working Ranges used to Estimate Bypass Volume

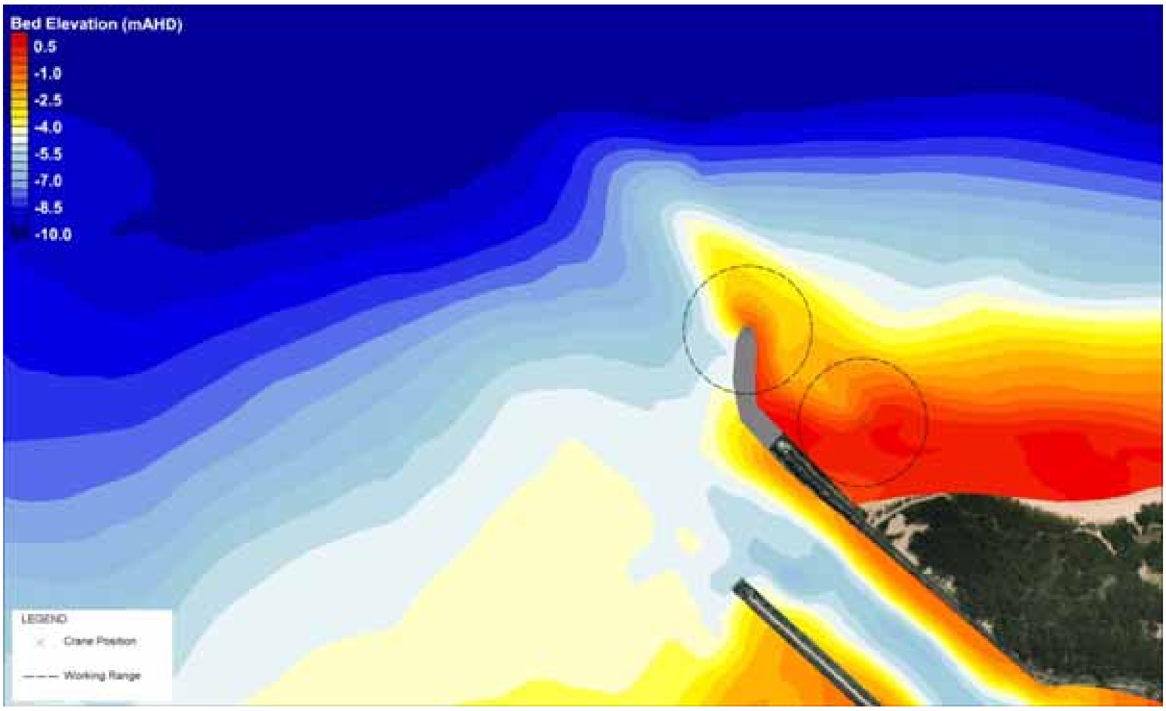

Figure 7-9 Option 2 90t Crawler Crane Positions and Working Ranges used to Estimate Bypass Volume

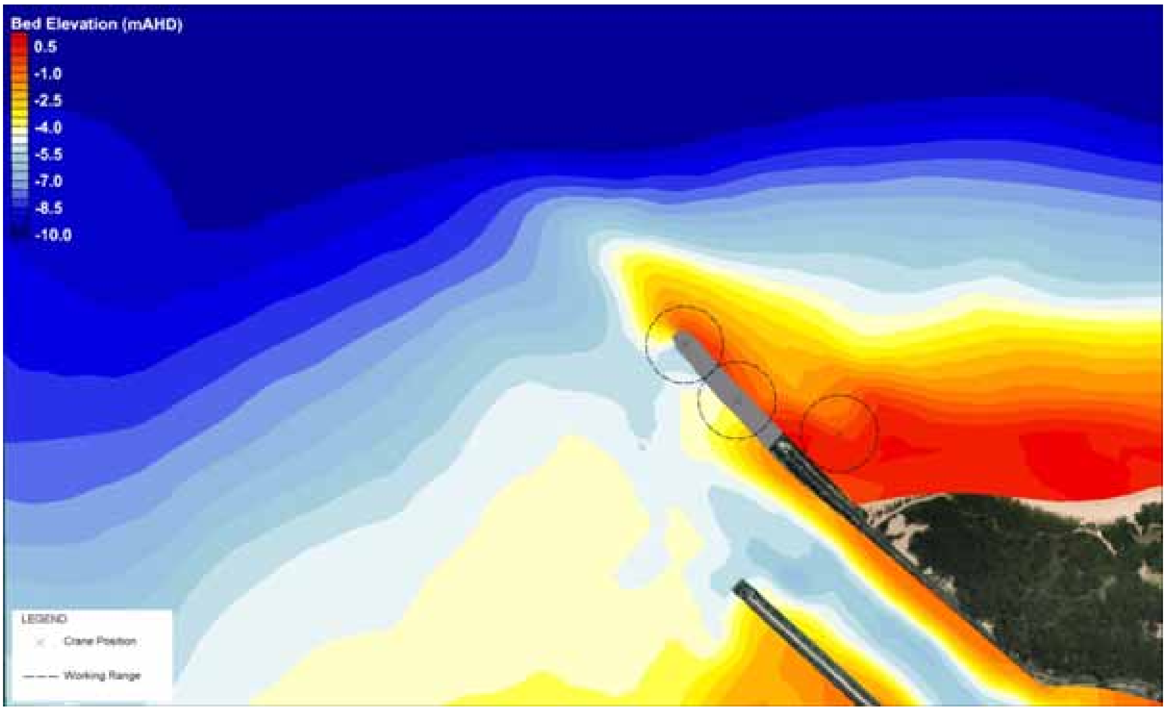

Figure 7-10 Option 3 50t Crawler Crane Positions and Working Ranges used to Estimate Bypass Volume

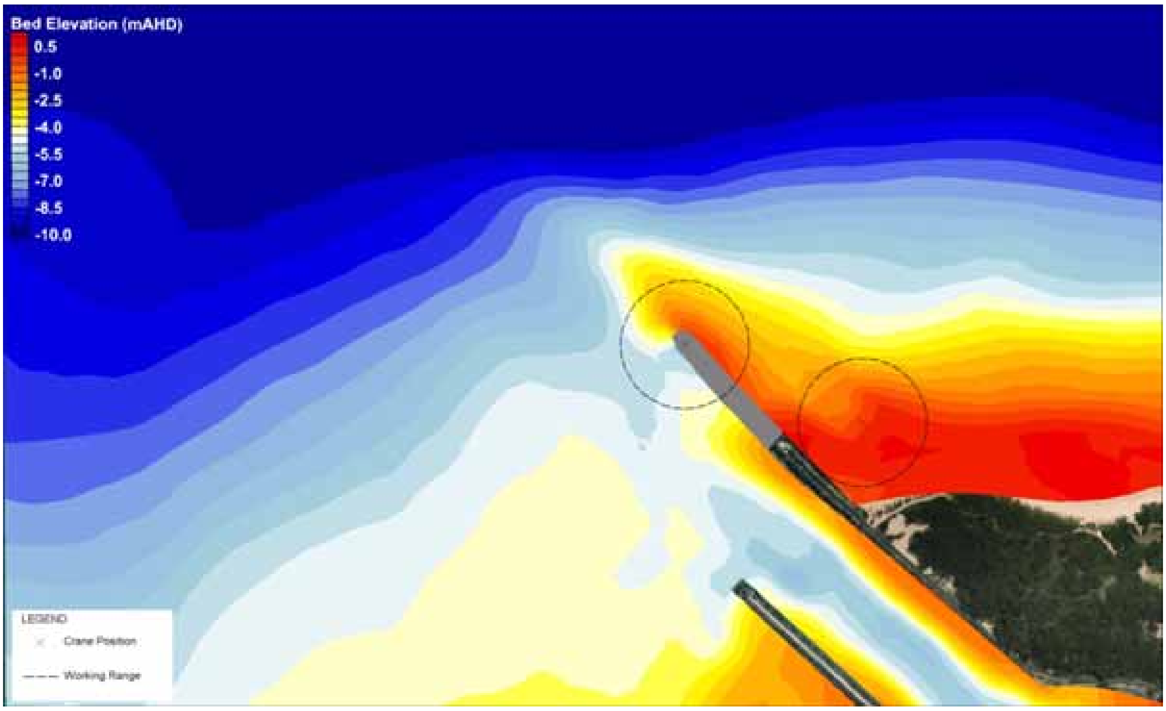

Figure 7-11 Option 3 90t Crawler Crane Positions and Working Ranges used to Estimate Bypass Volume

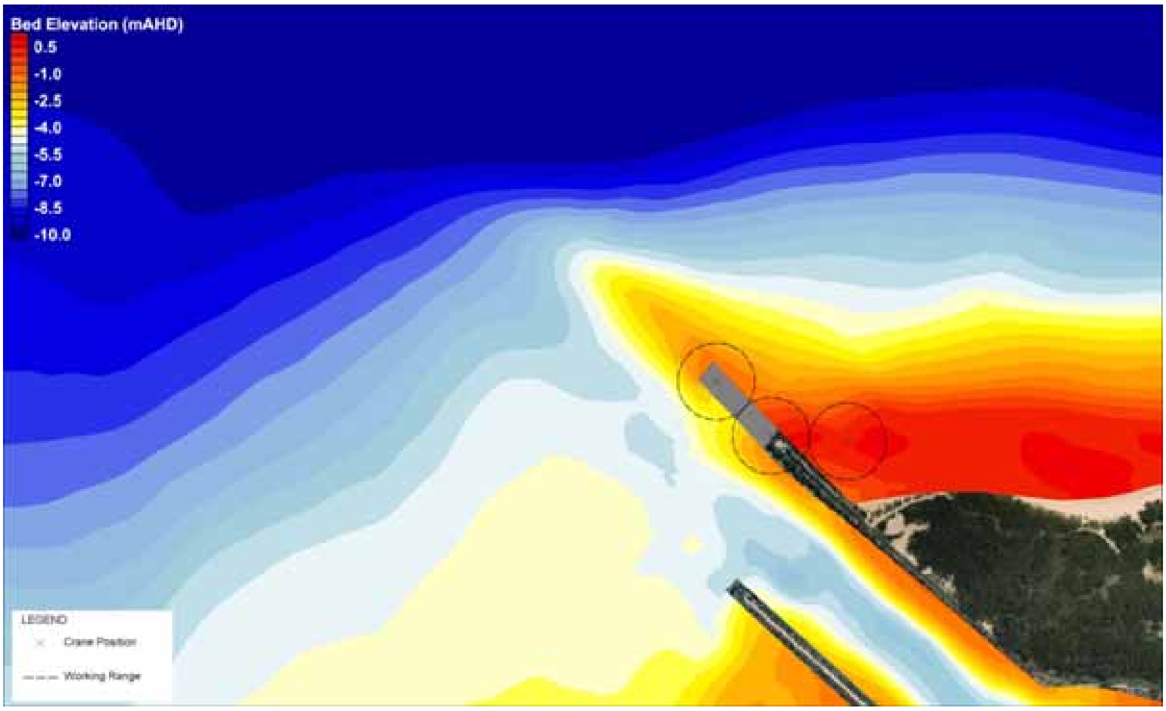

Figure 7-12 Option 3b 50t Crawler Crane Positions and Working Ranges used to Estimate Bypass Volume

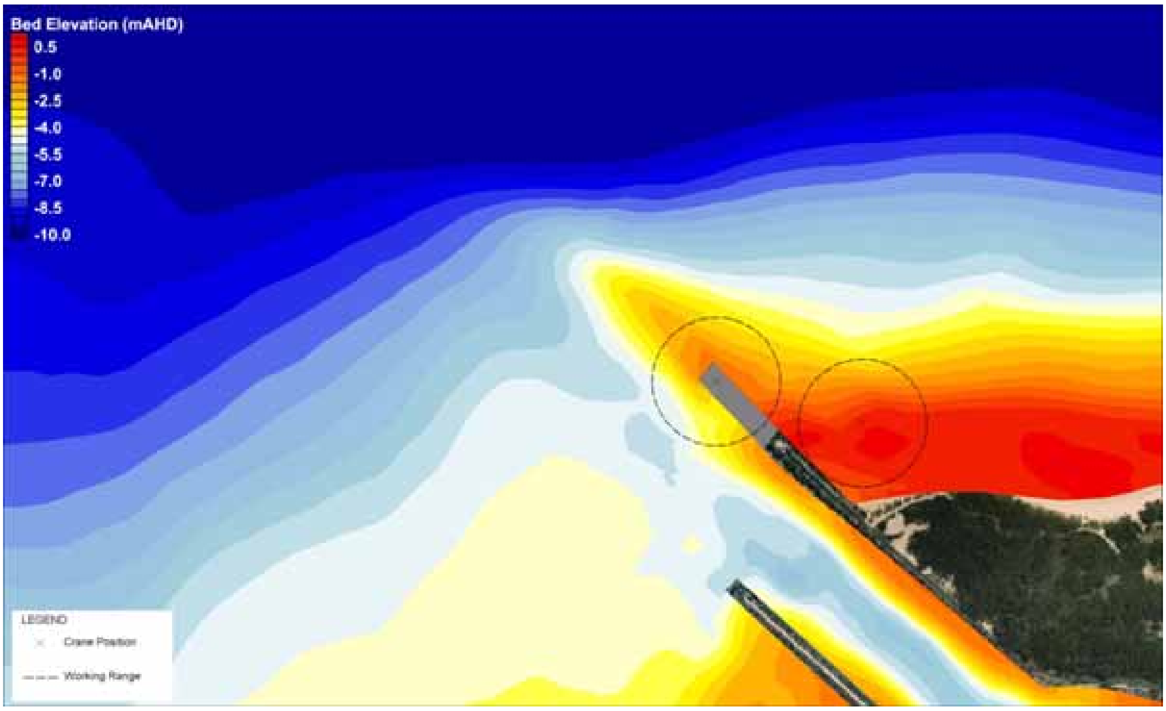

Figure 7-13 Option 3b 90t Crawler Crane Positions and Working Ranges used to Estimate Bypass Volume

The estimated bypassing volumes achieved with the 50t (30m working range) and 90t (50m working range) crane scenarios are presented in Table 7-5 and Table 7-6. These volumes have been estimated from the sand available at the end of the design event simulation period (above the bed rock layer) within the working range of each crane position.

Table 7-5 50t Crane with Mobile Jet Pump Assessment Results

Breakwater Configuration	Breakwater Head Crane Position (m^3)	Mid-Breakwater Crane Position (m^3)	Beach Crane Position (m^3)	Total Bypass Volume (m^3)
Option 2	8,950	3,200	3,840	15,990
Option 3	8,660	3,530	3,640	15,830
Option 3b	7,640	2,870	3,970	14,810

Table 7-6 90t Crane with Mobile Jet Pump Assessment Results

Breakwater Configuration	Breakwater Head Crane Position (m^3)	Beach Crane Position (m^3)	Total Bypass Volume (m^3)
Option 2	17,490	7,310	24,800
Option 3	19,040	6,460	25,500
Option 3b	15,670	7,010	22,680

The crane-mounted mobile jet pump assessments suggest the potential bypass volume is relatively insensitive to the capital works options considered. Considering the annual average natural sand supply to Mooloolaba Bay to be 5,000-10,000m^3 (e.g. BMT WBM, 2013), the 50t crane system may only alleviate sand supply impacts over the long term. The greater sand volumes accessed with the 90t crane suggests this system would effectively mitigate sand supply impacts to the Mooloolaba shoreline. It is noted that sufficient sand accumulation to allow bypassing using the crane-mounted mobile jet pump method may not occur for a number of years (depending on natural sand bypassing at Point Cartwright and sand transport rates). During this period sand supply impacts to Mooloolaba Bay may need to be mitigated using another method and material from an alternative nearby location.

For all capital works options a significant volume of sand is directed offshore and beyond the working range of the crawler crane sizes considered. The range of the system could potentially be improved by incorporating a boat to tow the hose and jet to positions beyond the working range of the crane. Complementary boat work would only be possible under favourable sea conditions. Furthermore, the non-static morphology during actual operations is also likely to enhance production due to sand being continually redistributed by the coastal processes and a tendency to infill extraction areas.

7.4.3 Mooloolah Entrance Crane with Mobile Jet Pump System Costs

Mobilisation/de-mobilisation and operational costs associated with a jet pump system at the Mooloolah Entrance eastern breakwater have been developed by Slurry Systems Marine and are provided in Table 7-7 and Table 7-8. For the purposes of providing a cost estimate, the sand bypassing production rate is assumed to be 100m^3/hr. It is noted that the actual production rate would be higher when using the system in areas with deep sand deposits (pers. comm. Lex Nankervis, 2014).

Consistent with the dredge frequency and volume assumptions in Section 7.2.1, it is expected that the mobile jet pump works would need to be carried out at 2-year intervals to mitigate the reduction in sand supply to Mooloolaba Bay beaches. Each campaign would seek to relocate up to 30,000m3 of sand from the eastern breakwater to Mooloolaba Bay. The estimated cost of this sand bypassing method per campaign is presented in Table 7-9.

Table 7-7 Mobile Jet Pump Sand Bypassing Method Mobilisation/De-mobilisation Cost Estimate provided by Slurry Systems Marine

Plant Mobilisation/De-mobilisation	Cost
Crawler crane 50t / 90t	$7,500 / $12,500
Booster pump	$4,500
Hydraulic Power Unit (HPU) and submersible pump	$4,500
Pipe and hose (connection between submersible pump and booster pump)	$6,000
Labour	$9,000
De-mobilisation	$30,000
Mobilisation and De-mobilisation Total Cost	**$61,500 / $66,500**

Table 7-8 Mobile Jet Pump Sand Bypassing Method Daily Operational Cost Estimate provided by Slurry Systems Marine

Daily (10 hour) Operating Cost	Cost
Crawler crane (including operator) 50t / 90t	$2,500 / $3,500
Booster pump	$600
HPU and submersible pump	$1,200
Fuel	$2,600
Labour (two person)	$2,000
Contingency	$1,000
Daily Operating Total Cost	**$9,900 / $10,900**

Table 7-9 Mobile Jet Pump Sand Bypassing Method Cost Estimate per Campaign

Activity	Assumptions	Quantity	Cost
Mobilisation/de-mobilisation	Delivery pipeline permanently installed	1	$61,500
Sand volume bypassed and re-profiled on beach	Production rate 100m^3/hr at \$9.9/m^3 (50t crane) or \$10.9/m^3 (90t crane) for delivery distances less than 1km	30,000 m^3	$297,000 / $327,000
Mobile Jet Pump Campaign Total Cost (per two years)			**$358,500 / $388,500**

7.4.3.1 90t Crawler Crane Operating Platform Costs

It has been assumed that the existing breakwater crest width of approximately 4m would provide sufficient transportation access for a 90t crawler crane. During operation, the crane would require an additional working area to accommodate the stabilising legs. The mobile jet pump assessments presented in Section 7.4.2 assumed an operating platform for the 90t could be accommodated at the head of the breakwater extension.

The rock quantity estimates presented in Section 6.3.1 were based on a breakwater crest width of 4.9m. Additional rock would be required to incorporate an operating platform for a 90t crawler crane at the head of the breakwater. Table 7-10 provides the rock quantity for an operating platform with a crest width of 7.0m. It has been assumed this increased crest width would be required over a 10m length at the seaward extent of the breakwater.

Table 7-10 Operating Platform Rock Quantity Estimates

Extension	Length (m)	Core Material (m^3)	Armour Material (m^3)	Total (m^3)	Core Material (tonne)	Armour Material (tonne)	Total (tonne)
Operating platform 7.0m crest width	10	1,665	910	2,575	2,797	1,530	4,326

The additional capital for the crane operating platform is approximately $181,500. Rock quantity and cost estimates are summarised in Table 7-11 and consider transport, delivery and placement costs previously presented in Section 6.3.2:

- $35/t to transport and deliver rock
- $20/t to place rock (assuming placement with land-based equipment)

Table 7-11 Operating Platform Rock Transport, Delivery and Placement Cost Estimates

Extension	Rock Quantity (tonne)	Rock Cost ($ inc GST)
Operating Platform	4,326	$237,955

8 Capital Works Options Assessment and Cost Summary

A number of potential enhanced shoal management strategies at the Mooloolah Entrance have been assessed. Each strategy considers an eastern breakwater extension in combination with a mechanical sand bypassing method required to mitigate a reduction in sand supply to Mooloolaba Bay.

The costs associated with each combination of breakwater extension and bypassing method are compared in Table 8-1. The cost estimates consider capital works, 20 year operational costs and a 25% contingency.

There is some uncertainty associated with the success of the bypassing methods and consequently the preferred sand supply mitigation strategy would require further testing prior to permanent implementation. Some uncertainties and further considerations include:

- The location where sand accumulates at the extended eastern breakwater may present conditions that challenge standard dredging techniques. If sand supply mitigation was to rely on dredging, further advice from a qualified dredge consultant should be sought regarding this matter. This option is likely to present a significant risk of cost overruns as results of operational delays.
- The episodic nature of littoral sand transport may not provide suitable conditions for a sand shifter system. Sand shifter trials in 2012 showed that the system was not able to work efficiently due to the shallow thickness of sand across the rock shelf and inadequate sand trapping capacity of the present entrance configuration. High-level numerical assessments suggest these issues may still be encountered with an eastern breakwater extension. Sand shifter potential could be further explored through field trials following eastern breakwater extension works.
- The crane-mounted jet pump system assessments assumed a crawler crane could access and operate from the eastern breakwater. Initial advice regarding this activity suggests it would be feasible. If this bypassing method is to be adopted in conjunction with a breakwater extension, the specifications and operational requirements of the preferred crane would need to form an essential component of the capital works detailed design.

Table 8-1 Capital Work Option Cost Comparison

Breakwater Configuration	Capital Cost	Sand Bypassing Method	Bypassing Method Capital Cost	Bypassing Operational Cost (over 20 years)	Bypassing Operation Cost Assumption	Total Cost over 20 years (Capital and Operational)	Total Cost with 25% Contingency
Option 1	$ 2,513,500	Sand Shifter	$ 2,283,000	$ 3,672,000	Annual operating cost $183,600	$ 8,468,500	$ 10,585,625
Option 1	$ 2,513,500	50t Crane with Mobile Jet Pump	NA	$ 3,585,000	2-yearly operating cost $358,500	$ 6,098,500	$ 7,623,125
Option 1 with crane platform	$ 2,751,500	90t Crane with Mobile Jet Pump	NA	$ 3,885,000	2-yearly operating cost $388,500	$ 6,636,500	$ 8,295,625
Option 3	$ 2,838,000	Dredge	NA	$ 8,450,000	2-yearly operating cost $845,000	$ 11,288,000	$ 14,110,000
Option 3	$ 2,838,000	Sand Shifter	$ 2,283,000	$ 3,672,000	Annual operating cost $183,600	$ 8,793,000	$ 10,991,250
Option 3	$ 2,838,000	50t Crane with Mobile Jet Pump	NA	$ 3,585,000	2-yearly operating cost $358,500	$ 6,423,000	$ 8,028,750
Option 3 with crane platform	$ 3,076,000	90t Crane with Mobile Jet Pump	NA	$ 3,885,000	2-yearly operating cost $388,500	$ 6,961,000	$ 8,701,250
Option 3b	$ 1,973,000	Dredge	NA	$ 8,450,000	2-yearly operating cost $845,000	$ 10,423,000	$ 13,028,750
Option 3b	$ 1,973,000	Sand Shifter	$ 2,283,000	$ 3,672,000	Annual operating cost $183,600	$ 7,928,000	$ 9,910,000
Option 3b	$ 1,973,000	50t Crane with Mobile Jet Pump	NA	$ 3,585,000	2-yearly operating cost $358,500	$ 5,558,000	$ 6,947,500

Breakwater Configuration	Capital Cost	Sand Bypassing Method	Bypassing Method Capital Cost	Bypassing Operational Cost (over 20 years)	Bypassing Operation Cost Assumption	Total Cost over 20 years (Capital and Operational)	Total Cost with 25% Contingency
Option 3b with crane platform	$ 2,211,000	90t Crane with Mobile Jet Pump	NA	$ 3,885,000	2-yearly operating cost $388,500	$ 6,096,000	$ 7,620,000
Option 4	$ 3,271,000	Dredge	NA	$ 8,450,000	2-yearly operating cost $845,000	$ 11,721,000	$ 14,651,250

9 Recommendations and Conclusions

The initial assessments described in this report considered four Mooloolah Entrance eastern breakwater capital works options. Using calibrated numerical modelling tools, each breakwater extension was assessed against the following criteria (refer Section 4):

a) Maintenance of channel to a minimum depth of -3.0mLAT (equivalent to -4.0mAHD);

b) Impact on entrance wave conditions;

c) Impact to Mooloolaba Spit; and

d) Impact to Mooloolah River flood flows.

The initial assessments found that all breakwater extensions successfully intercepted the design shoal and therefore achieved the key objective of maintaining a minimum channel depth of -3mLAT. The assessments also indicated that the four breakwater options are unlikely to cause an undesirable impact to entrance wave conditions or Mooloolah River flood flows. Each breakwater option was also found to reduce sand supply to Mooloolaba Spit. Without mitigation this impact would be expected to cause undesirable recession of the Mooloolaba shoreline.

The rock quantity required for each breakwater extension and the associated cost is a primary consideration given the general consistency in performance against the above criteria. In an effort to minimise capital works costs, a design refinement that considered a reduced breakwater extension length was also assessed (Option 3b, refer Section 5). The refined design was also found to meet the primary objectives of the breakwater extension.

If an eastern breakwater extension is adopted, there will be an ongoing need to mechanically bypass intercepted sand to Mooloolaba spit in perpetuity. The ultimate sand bypassing strategy would need to be developed following trails and may include a combination of options. For this reason, it is recommended that adequate contingency is allowed for in any capital works project to enable the effective development of the most efficient management strategy. Of the various mechanical bypassing options assessed in this study (refer Section 7), a crawler crane and jet pump is expected to be the most economically and operationally viable method.

10 References

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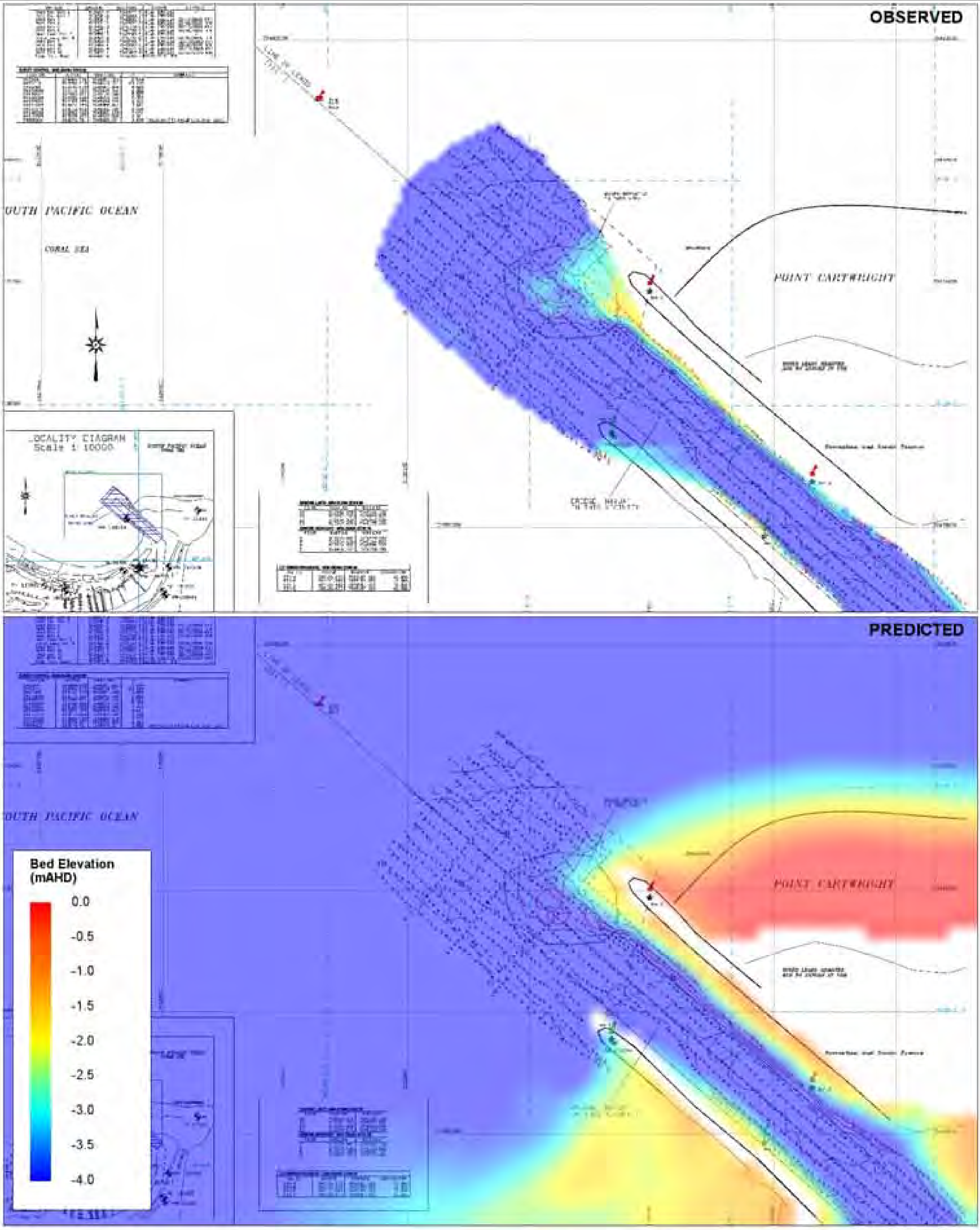
Appendix A Observed and Predicted Shoal DEMs



Title:
Observed and Predicted Shoal 22 December 2011

Figure: A-1

Rev: A

BMT WBM endeavours to ensure that the information provided in this map is correct at the time of publication. BMT WBM does not warrant, guarantee or make representations regarding the currency and accuracy of information contained in this map.

N

0 75 150m
Approx. Scale

BMT WBM
www.bmtwbm.com.au

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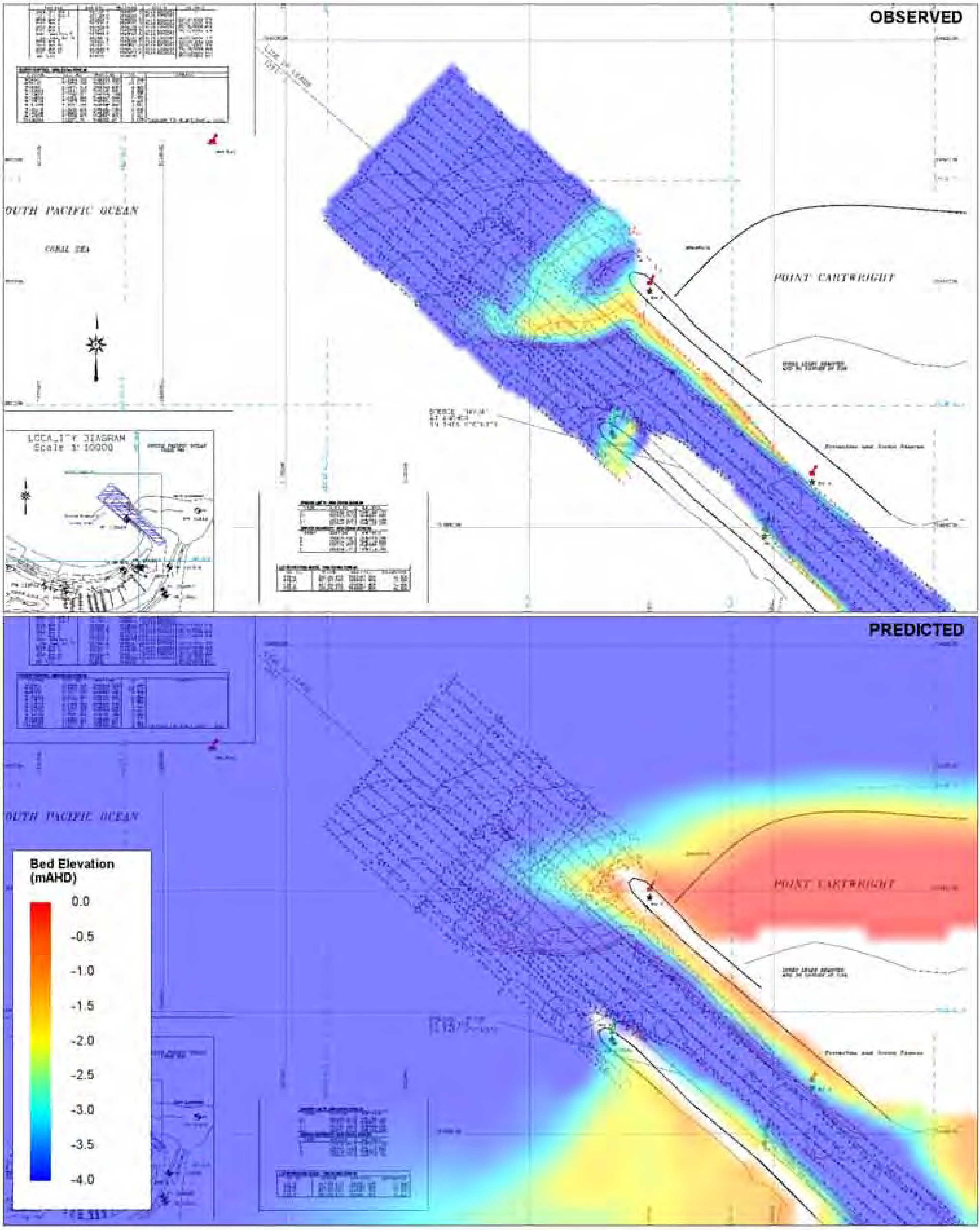

Title:

Observed and Predicted Shoal 29 December 2011

Figure: A-2

Rev: A

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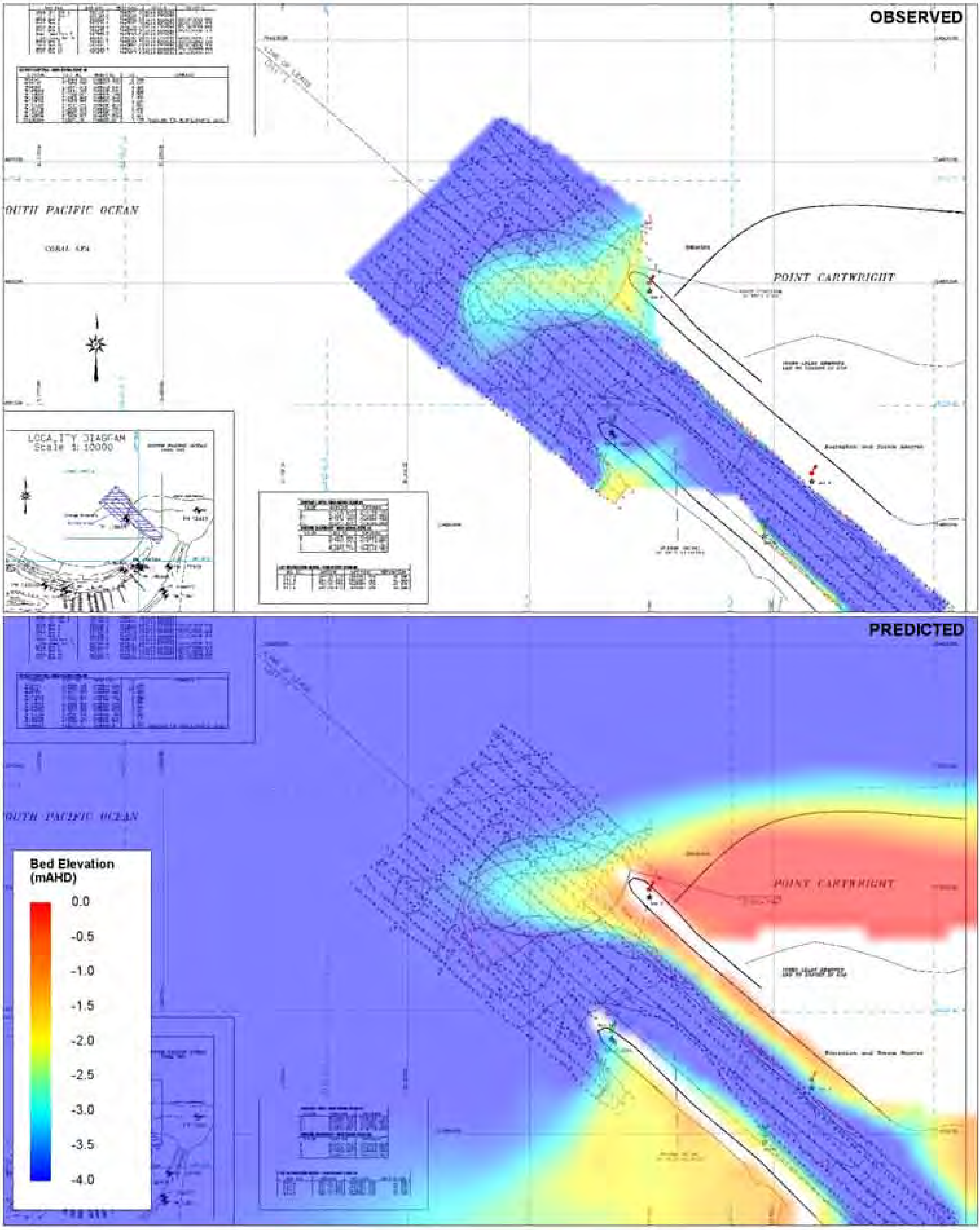

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Observed and Predicted Shoal 17 February 2012

Figure: **A-3**

Rev: **A**

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Approx. Scale

BMT WBM

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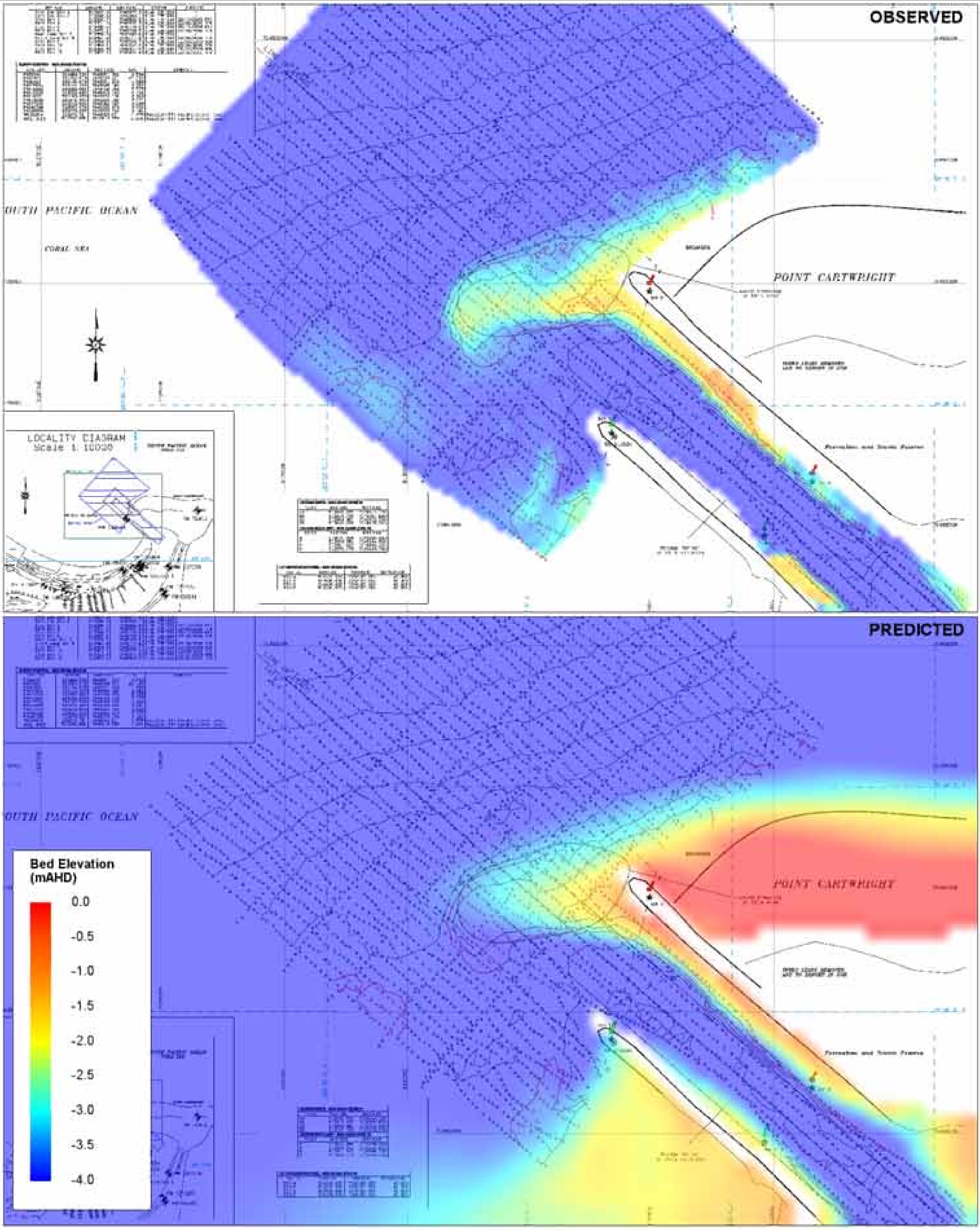

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Observed and Predicted Shoal 01 March 2012

Figure: **A-4**

Rev: **A**

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Approx. Scale

BMT WBM

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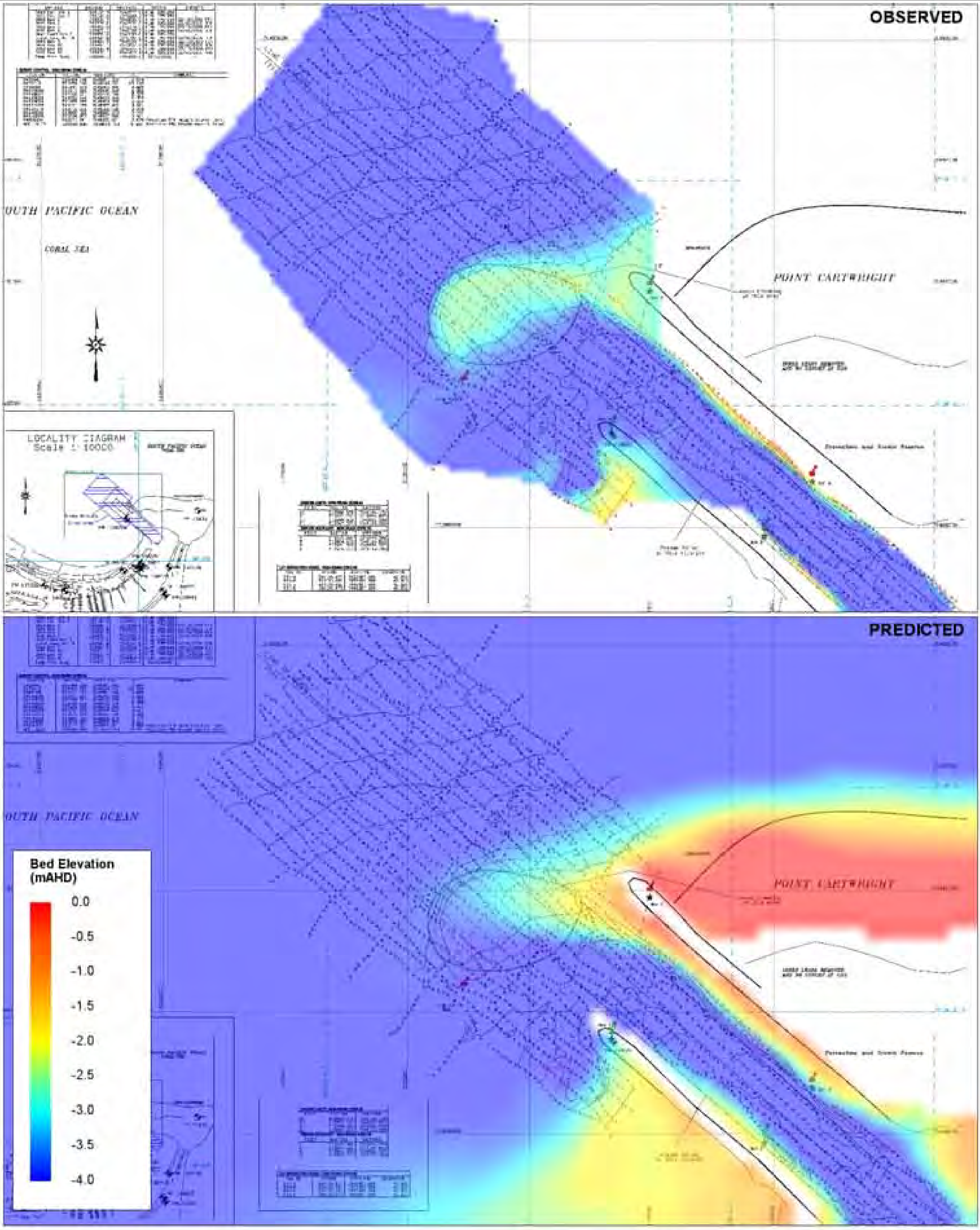

Title:

Observed and Predicted Shoal
13 March 2012

Figure:
A-5

Rev:
A

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Approx. Scale

BMT WBM
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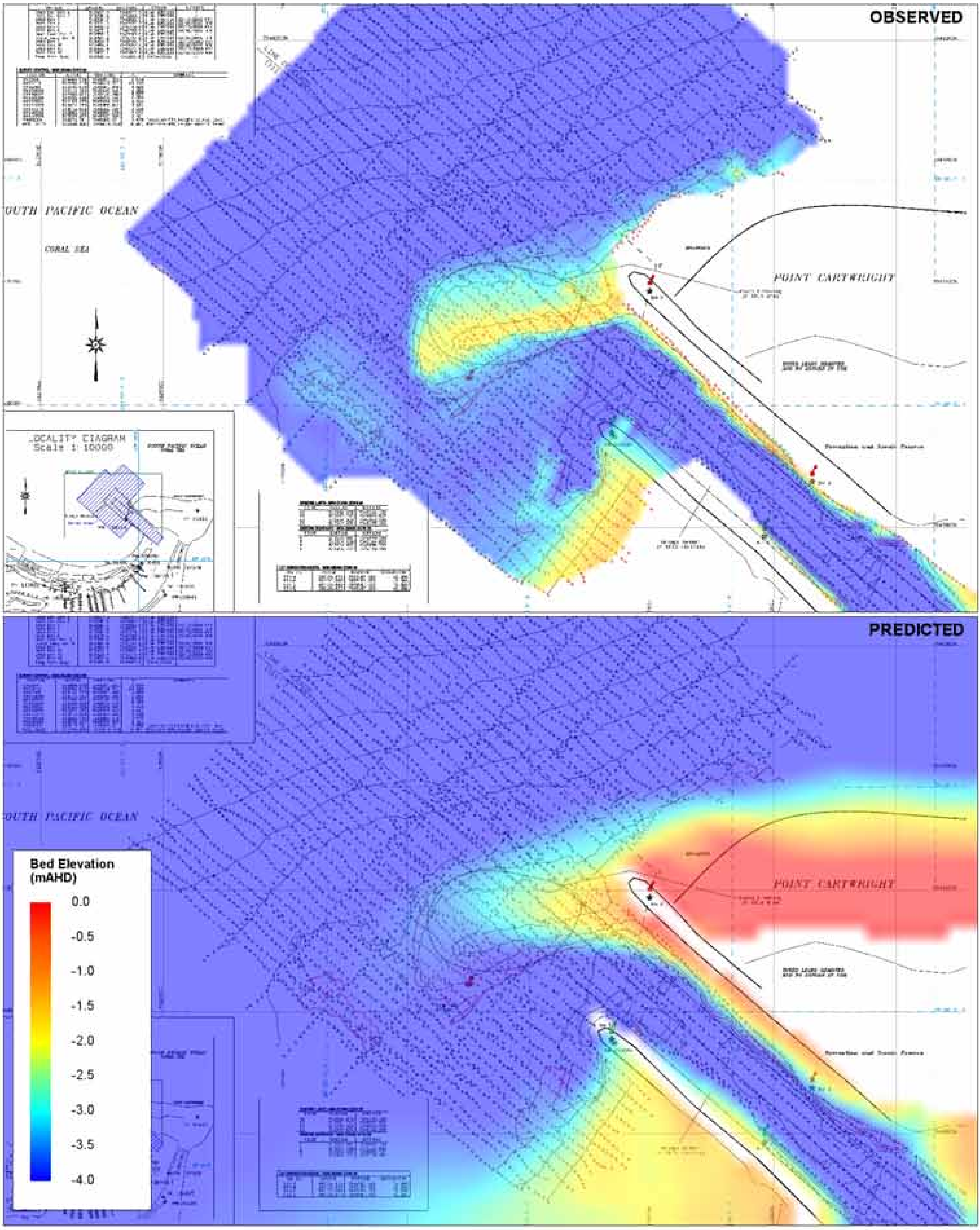

Title:

Observed and Predicted Shoal 04 April 2012

Figure: A-6

Rev: A

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Approx. Scale

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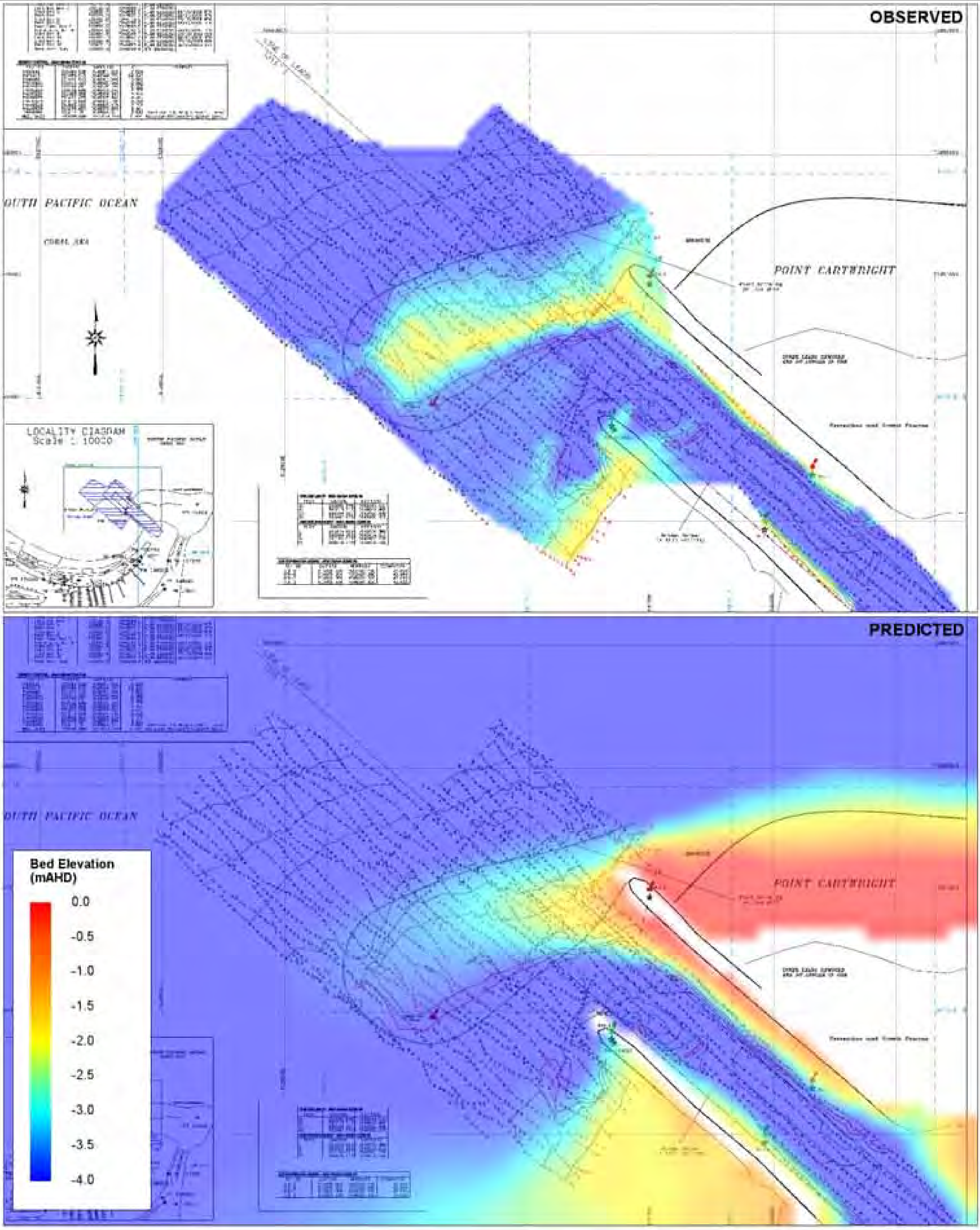

Title:

Observed and Predicted Shoal 24 April 2012

Figure: **A-7**

Rev: **A**

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Approx. Scale

BMT WBM

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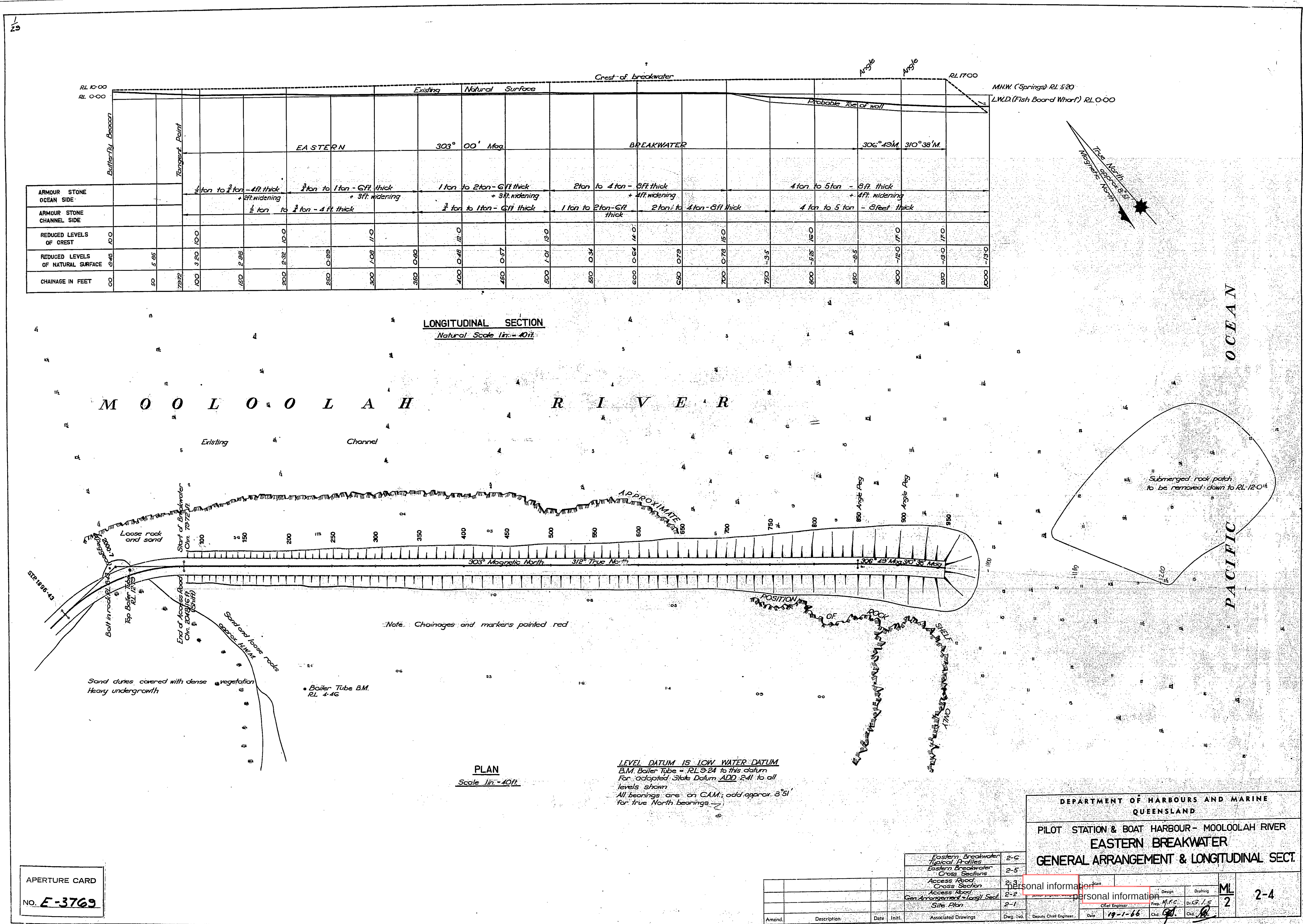
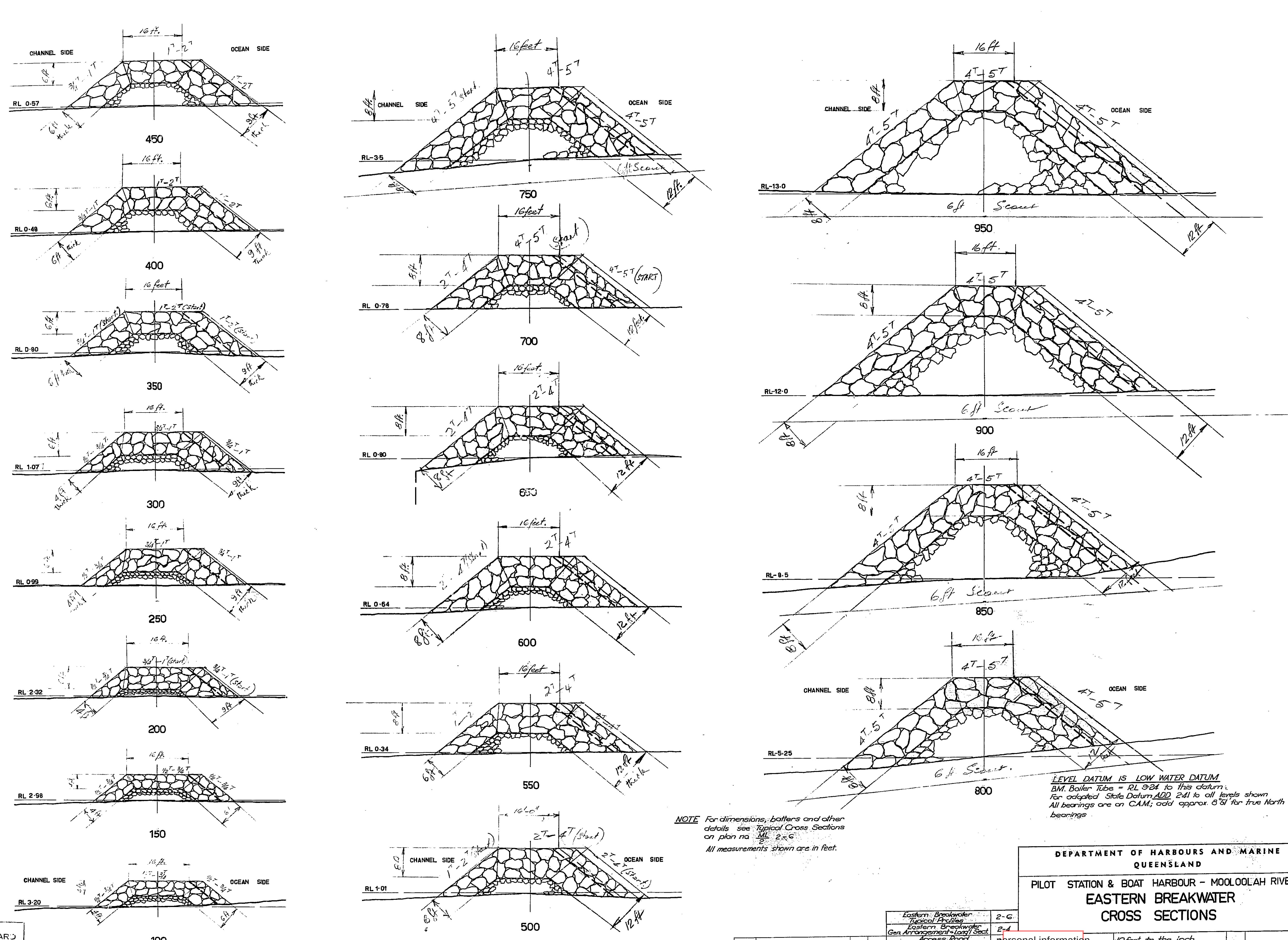
Appendix B Existing Breakwater Drawing Set



Crest of breakwater
Existing Natural Surface
Probable Toe of wall
RL 10·00
RL 0·00
RL 17·00
M.H.W. (Springs) RL 5·20
L.W.D. (Fish Board Wharf) RL 0·00
Butterfly Beacon
Tangent Point
EASTERN 303° 00' Mag. BREAKWATER
306°49'M. 310°38'M.
Angle
ARMOUR STONE OCEAN SIDE
ARMOUR STONE CHANNEL SIDE
REDUCED LEVELS OF CREST
REDUCED LEVELS OF NATURAL SURFACE
CHAINAGE IN FEET
1 ton to 2 ton - 6ft thick + 3ft widening
2 ton to 4 ton - 8ft thick + 4ft widening
4 ton to 5 ton - 8ft thick + 4ft widening
1 ton to 2 ton - 6ft thick
2 ton to 4 ton - 8ft thick
4 ton to 5 ton - 8feet thick
LONGITUDINAL SECTION
Natural Scale 1in. = 40ft.
True North
Magnetic North
approx 8°51'
M O O L O O L A H R I V E R
Existing Channel
PACIFIC OCEAN
Submerged rock patch to be removed down to RL -12·0
APPROXIMATE
POSITION OF ROCK SHELF ONLY
Loose rock and sand
Start of Breakwater
End of Access Road
Bolt in rock
Top Boiler Tube
Sand and loose rocks approx. H.W.M.
303° Magnetic North 312° True North
306° 49' Mag. 310° 38' Mag.
850 Angle Peg
900 Angle Peg
Note. Chainages and markers painted red
Sand dunes covered with dense vegetation
Heavy undergrowth
Boiler Tube B.M. RL 4·46
PLAN
Scale 1in. = 40ft.
LEVEL DATUM IS LOW WATER DATUM
B.M. Boiler Tube = RL 9·24 to this datum
For adopted State Datum ADD 2·41 to all levels shown
All bearings are on C.A.M.; add approx. 8°51' for true North bearings
APERTURE CARD
NO. E-3769
DEPARTMENT OF HARBOURS AND MARINE
QUEENSLAND
PILOT STATION & BOAT HARBOUR - MOOLOOLAH RIVER
EASTERN BREAKWATER
GENERAL ARRANGEMENT & LONGITUDINAL SECT.
Eastern Breakwater Typical Profiles 2-6
Eastern Breakwater Cross Sections 2-5
Access Road Cross Section 2-3
Access Road Gen. Arrangement + Long'l Sect. 2-2
Site Plan 2-1
Amend. Description Date Initl. Associated Drawings Dwg. No.
[redacted]
[redacted]
Date 19-1-66
ML 2
2-4



APERTURE CARD
NO. E-3770

NOTE For dimensions, batters and other details see Typical Cross Sections on plan no. $\frac{ML}{2}$ 2-6
All measurements shown are in feet.

LEVEL DATUM IS LOW WATER DATUM
B.M. Boiler Tube = R.L. 9·24 to this datum.
For adopted State Datum ADD 2·41 to all levels shown
All bearings are on C.A.M.; add approx. 8°51' for true North bearings

DEPARTMENT OF HARBOURS AND MARINE
QUEENSLAND

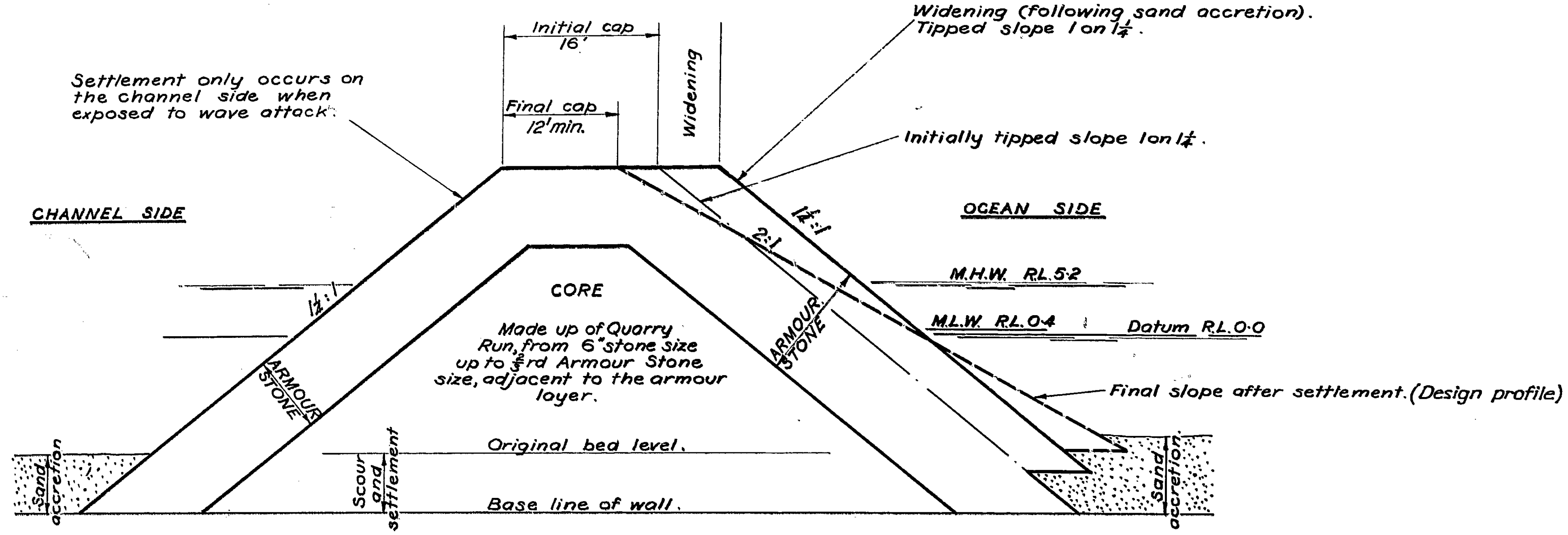
PILOT STATION & BOAT HARBOUR – MOOLOOLAH RIVER
EASTERN BREAKWATER
CROSS SECTIONS

Associated Drawings	Dwg. No.
Eastern Breakwater Typical Profiles	2-6
Eastern Breakwater Gen. Arrangement & Long'l Sect.	2-4
Access Road Cross Sections	[redacted]
Access Road Gen. Arrangement & Long'l Sect.	2-2
Site Plan	2-1

Amend.	Description	Date	Initl.

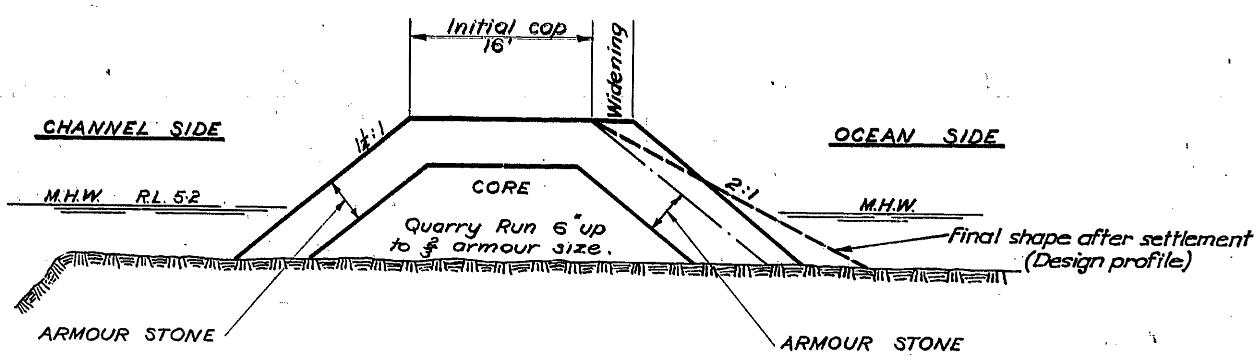
Scale: 10 feet to the inch
Senior Engineer Design
[redacted]
Deputy Chief Engineer
Date: 19-1-66
Design: Prep. M.F.C. Ckd.
Drafting: Dr. Ckd.
ML / 2
2-5

1/20



TYPICAL PROFILE – OUTER SECTION

Scale:- 1"=10'



TYPICAL PROFILE – ROCK SHELF SECTION

Scale:- 1"=10'

NOTES ON CONSTRUCTION.

1. The breakwater is to be constructed initially to the cap width shown.
2. The core is to be formed by direct end tipping of the smaller stone.
3. In the core segregation of sizes is important to produce as permeable a mass as possible. Smaller sizes to the base, larger sizes adjacent to the armour layer.
4. The large armour stone is to be skew tipped from the end of the advancing core.
5. It is important that the full thickness of armour stone be placed on the crown of the initial section, even though the smaller sizes would give a better surface for trucks.
6. On completion of primary armouring, breakwater is to be widened on seaward side, using armour stone sizes.
7. Turnouts of an erodible material may be constructed as directed by the Engineer.

INDIVIDUAL STONE WEIGHT			STONE SIZES BASED ON A CUBE
lb	cwt.	ton.	3 equal sides of:-
170	1·52		1'-0"
582	5·19	0·26	1'-6"
1,360		0·6	2'-0"
2,650		1·2	2'-6"
4,590		2·1	3'-0"
7,320		3·3	3'-6"
10,900		4·9	4'-0"

LEVEL DATUM IS LOW WATER DATUM
B.M. Boiler Tube – R.L. 9·24 to this datum — Ignore
For adopted State Datum ADD 2·41 to all levels shown
All bearings are on C.A.M.; add 8°51' for true North bearings.

[redacted]

DEPARTMENT OF HARBOURS AND MARINE
QUEENSLAND

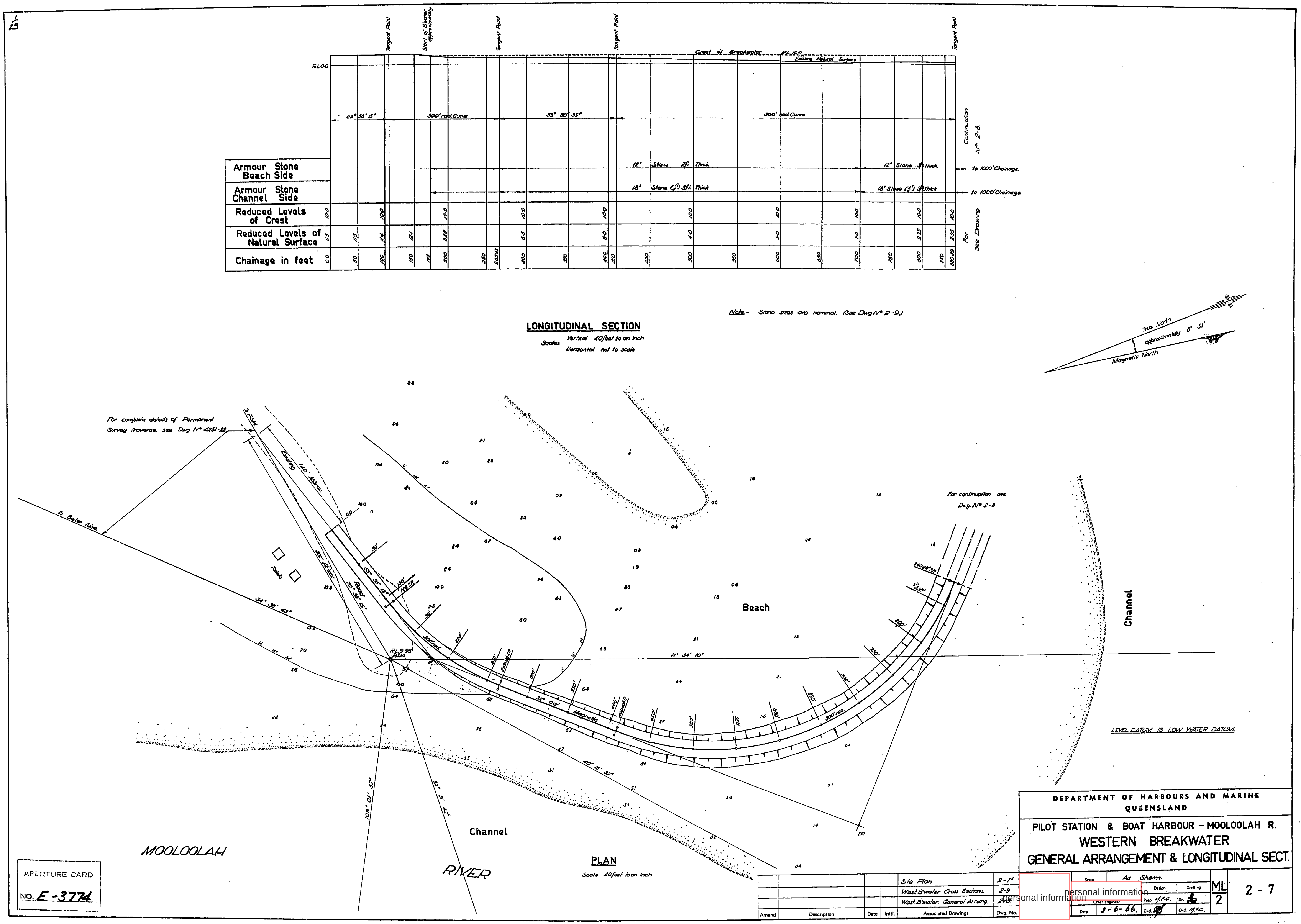
PILOT STATION & BOAT HARBOUR – MOOLOOLAH RIVER

EASTERN BREAKWATER
TYPICAL PROFILES

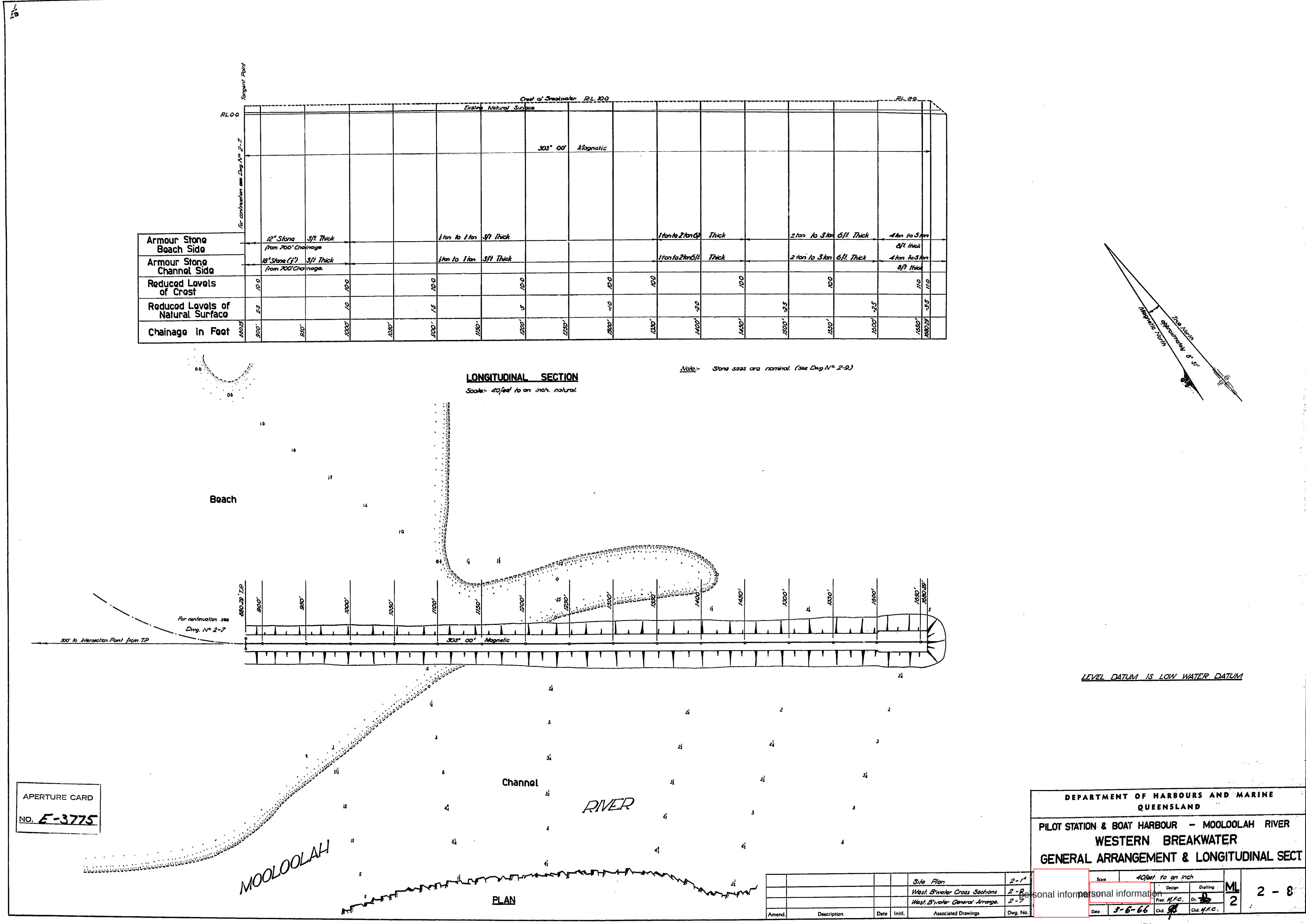
Associated Drawings	Dwg. No.
Eastern Breakwater Cross Sections	2-5
Eastern Breakwater Gen Arrangement & Long'l Sect.	2-4
Access Road Cross Sections	2-3
Access Road Gen Arrangement & Long'l Sect.	2-2
Site Plan	2-1

Amend.	Description	Date	Initl.

personal information
Deputy Chief Engineer
Scale: As shown
personal information
Date: 19-1-66
Design | Drafting
Prep. M.F.C. | Dr.
Ckd. | Ckd.
ML 2
2-6



Tangent Point
Start of B'water approximately
Crest of Breakwater R.L. 10.0
Existing Natural Surface
R.L.0.0
63° 56' 15"
300' rad. Curve
33° 30' 35"
300' rad. Curve
Continuation Nº. 2-8.
Armour Stone Beach Side
Armour Stone Channel Side
12" Stone 2ft. Thick
18" Stone (3/4') 3ft. Thick
12" Stone 3ft. Thick.
18" Stone (3/4') 3ft. Thick
to 1000' Chainage.
Reduced Levels of Crest
Reduced Levels of Natural Surface
Chainage in feet
For See Drawing
Note:- Stone sizes are nominal. (see Dwg. Nº. 2-9)
LONGITUDINAL SECTION
Scales Vertical 40 feet to an inch
Horizontal not to scale.
True North
approximately 8° 51'
Magnetic North
For complete details of Permanent Survey Traverse, see Dwg. Nº. 4251-22.
To Bailer Tube.
Existing 1.0' Approx.
Toilets
Road
R.L. 9.95' P.S.M.
300' rad.
Magnetic
For continuation see Dwg. Nº. 2-8
Beach
Channel
LEVEL DATUM IS LOW WATER DATUM.
MOOLOOLAH
RIVER
Channel
PLAN
Scale 40 feet to an inch
APERTURE CARD
NO. E-3774
DEPARTMENT OF HARBOURS AND MARINE
QUEENSLAND
PILOT STATION & BOAT HARBOUR – MOOLOOLAH R.
WESTERN BREAKWATER
GENERAL ARRANGEMENT & LONGITUDINAL SECT.
Site Plan 2-1A
West. B'water Cross Sections. 2-9
West. B'water. General Arrang. 2-8.
Amend. Description Date Initl. Associated Drawings Dwg. No.
[redacted]
Scale As Shown.
Design Drafting
Prep. M.F.C.
Chief Engineer
Date 3-6-66.
Ckd. M.F.C.
ML 2
2 - 7



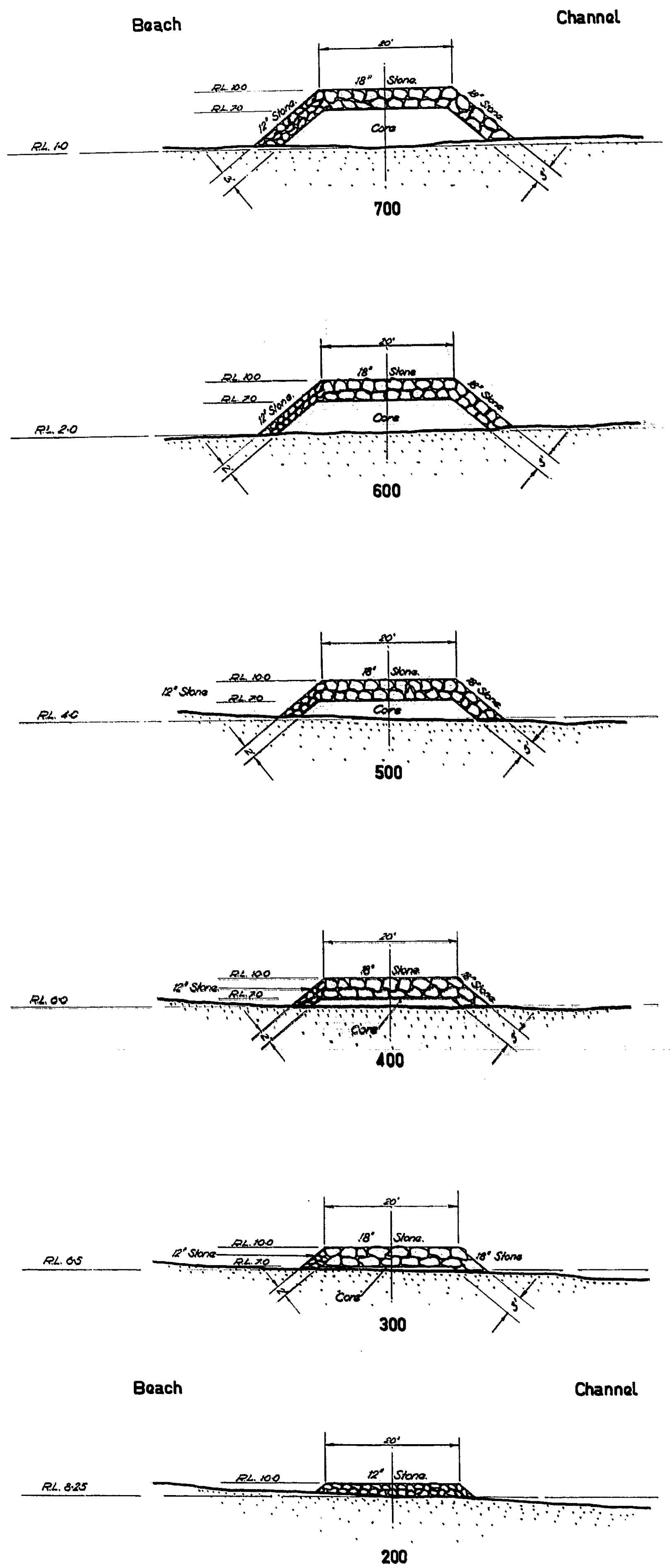
DEPARTMENT OF HARBOURS AND MARINE
QUEENSLAND

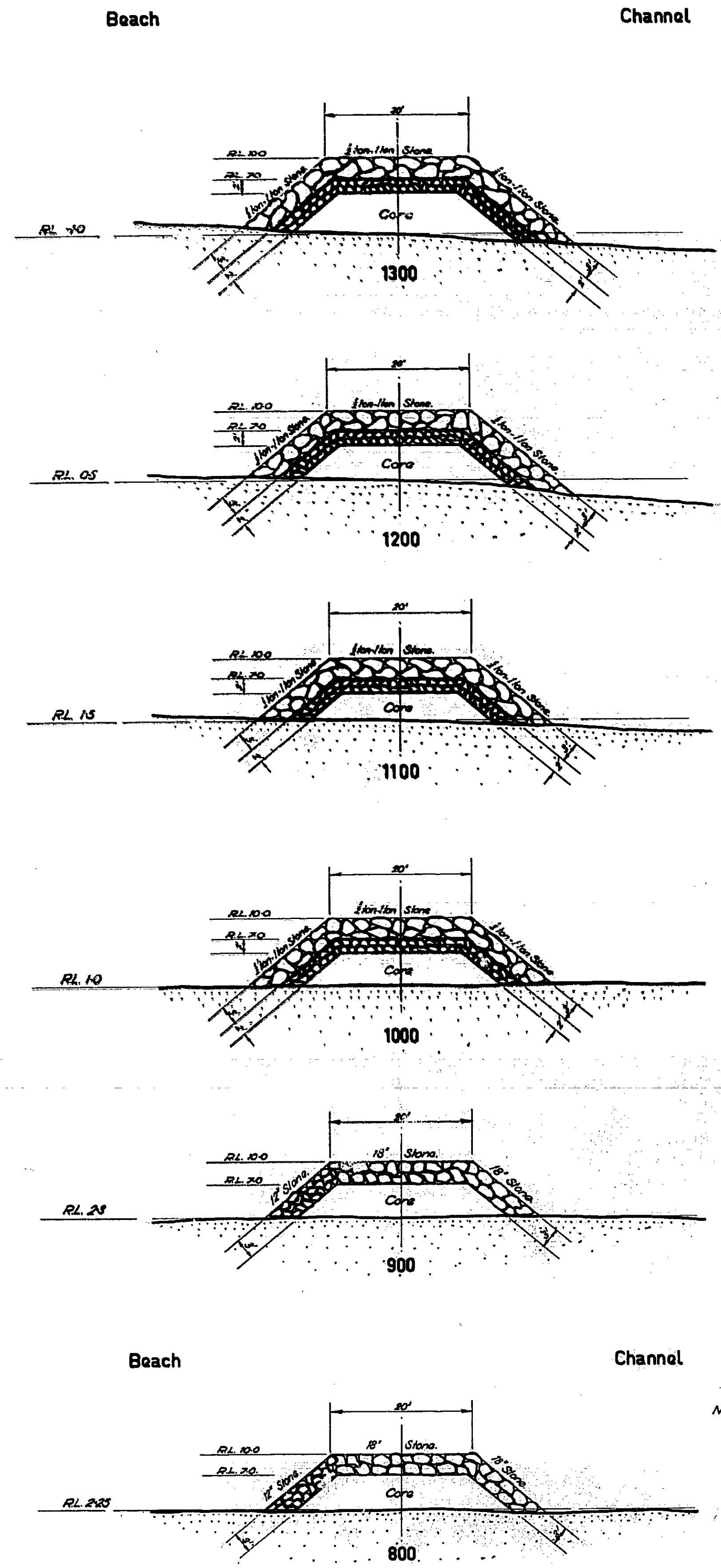
PILOT STATION & BOAT HARBOUR – MOOLOOLAH RIVER
WESTERN BREAKWATER
GENERAL ARRANGEMENT & LONGITUDINAL SECT.

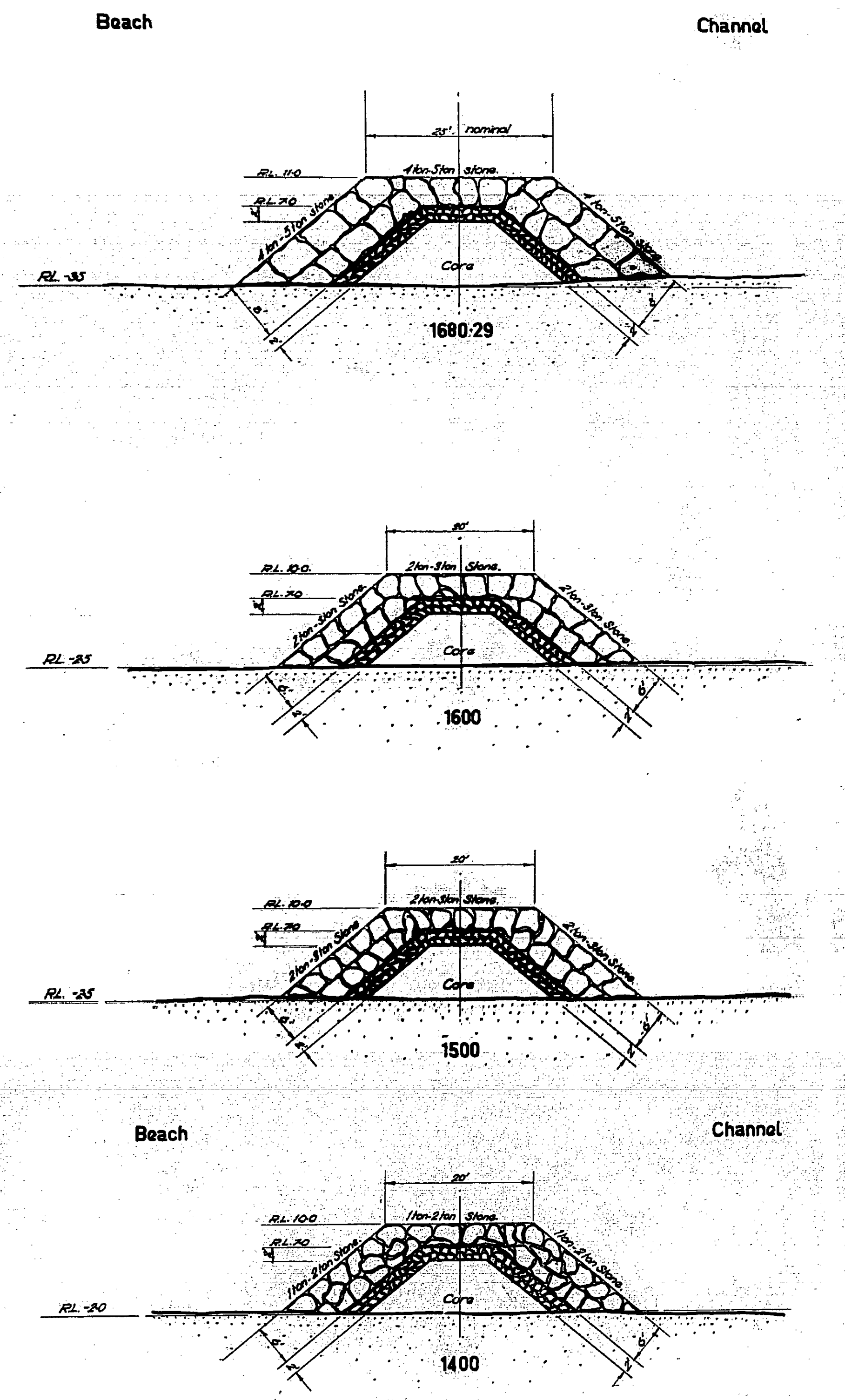
Amend.	Description	Date	Initl.	Associated Drawings	Dwg. No.
				Site Plan	2-1A
				West. B'water Cross Sections	2-8
				West. B'water General Arrange.	2-7

[redacted]

Scale: 40 feet to an inch
Design: Prep. M.F.C.
Drafting: Dr. [illegible]
Date: 3-6-66
Ckd. [illegible]
Ckd. M.F.C.
ML 2
2 - 8







Notes:-

1. Primary Armour Stone sizes shown are nominal. Up to 50% oversize may be tolerated, but the smaller stones should be placed in the layer adjacent to the secondary armour.
2. Secondary Armour Stone should be composed of 12"-18" nominal sizes.
3. Core Material is to be composed of 6"-12" nominal sizes, (free from binders,) as run of quarry material. Any larger sizes should be selectively dozed to the outer edges of the core.
4. Core and Armour should be layer placed as a protection against tidal erosion, and each layer placed successively not more than one foot higher than the tide.

All side slopes as for natural angle of repose. (Assumed as $1\frac{1}{4}$ horizontal to 1 vertical.)

All layer thicknesses are minimum.

DEPARTMENT OF HARBOURS AND MARINE
QUEENSLAND

PILOT STATION & BOAT HARBOUR - MOOLOOLAH RIVER

WESTERN BREAKWATER CROSS SECTIONS

Amend.	Description	Date	Initl.	Associated Drawings	Dwg. No.
				West. B'water & General Arrang.	2-7
				West. B'water & General Arrang.	2-8
				Site Plan	2-1

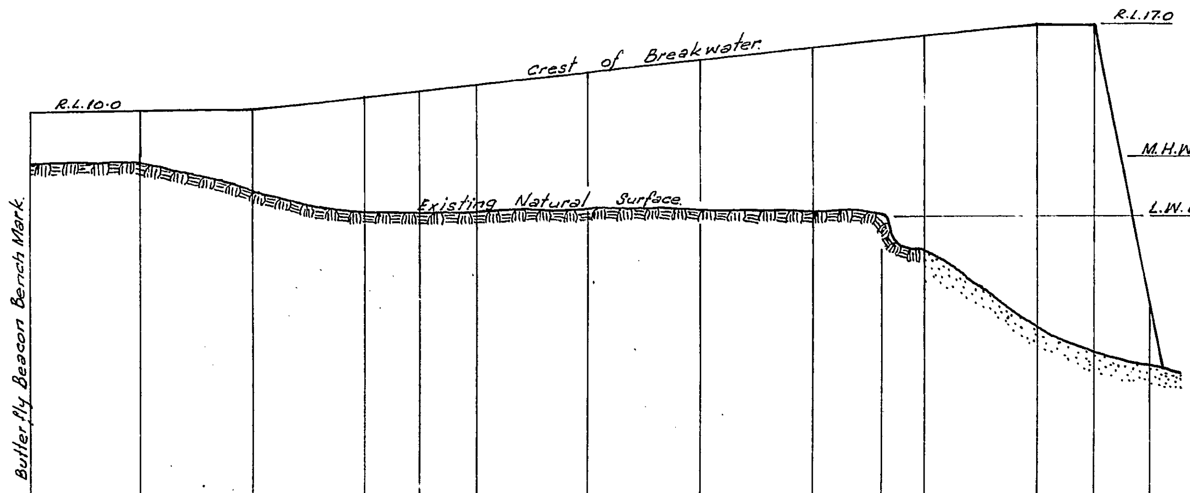
[redacted]

Scale: 10 feet to an inch
Date: 3-6-66
Design: Prep. M.F.C. Ckd.
Drafting: Dr. Ckd. M.F.C.

ML 2 | 2 - 9

APERTURE CARD
NO. E-3776

1/20

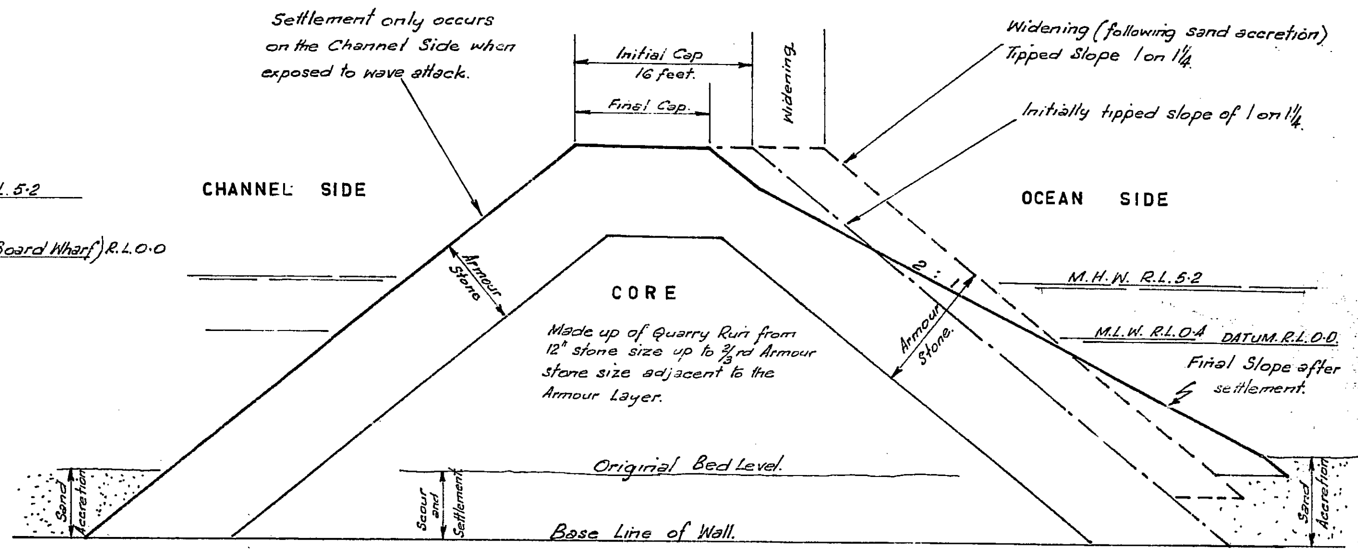


ARMOUR STONE OCEAN SIDE	½ ton to ¾ ton 4 ft. thick — ¾ ton to 1 ton 6 ft thick — 1 ton to 2 ton 6 ft. thick — 2 ton to 4 ton. 8 ft thick — 4 ton to 5 ton. 8 ft thick
ARMOUR STONE CHANNEL SIDE	½ ton to ¾ ton 4 ft thick — ¾ to 1 ton 6 ft. thick — 1 ton to 2 ton 6 ft. thick — 2 ton to 4 ton 8 ft thick — 4 ton to 5 ton. 8 ft thick

CHAINAGE	00'	100'	200'	300'	350'	400'	500'	600'	700'	760'	800'	900'	950'	1000'
R.L. OF CREST.	10.0	10.0	10.0	11.0		12.0	13.0	14.0	15.0		16.0	17.0	17.0	
R.L. NATURAL SURFACE	5.4	5.3	2.6	0.7		0.6	0.9	0.6	0.3		-3.0	-10.0	-12.0	-13.0

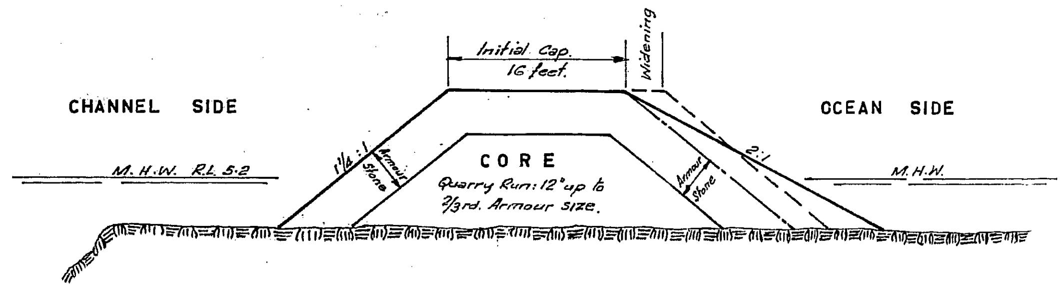
EASTERN BREAKWATER

Scales: Hor: 100 feet to an inch.
Vert: 10 feet to an inch.



TYPICAL PROFILE - OUTER SECTION.

Scale: 10 feet to an inch



TYPICAL PROFILE - ROCK SHELF SECTION

Scale: 10 feet to an inch.

NOTES ON CONSTRUCTION.

1. *The breakwater is to be constructed initially to the cap width shown.*
2. *The core is to be formed by direct end tipping of the smaller stone.*
3. *In the core, segregation of sizes is important, to produce as permeable a mass as possible. Smaller sizes to the base, larger sizes adjacent to the Armour Layer.*
4. *The large armour stone is to be skew tipped from the end of the advancing core.*
5. *It is important that the full thickness of armour stone be placed on the crown of the inital section, rather than the smaller sizes which would give a better surface for trucks.*
6. *The breakwater is to be later widened using only armour stone sizes.*

INDIVIDUAL STONE WEIGHT			STONE SIZES BASED ON A CUBE
lb.	cwt.	ton.	3 equal sides of :-
170	1·52		12 inchs
579	12·1	0·26	18 inchs
1360		0·6	2'- 0"
2,650		1·2	2'- 6"
4,590		2·1	3'- 0"
7,320		3·3	3'- 6"
10,900		4·9	4'- 0"

APERTURE CARD
NO. E-3814

DEPARTMENT OF HARBOURS AND MARINE
QUEENSLAND

PILOT STATION ON THE MOOLOOLAH RIVER.
EASTERN BREAKWATER
TYPICAL DETAILS.

Amend.	Description	Date	Initl.	Associated Drawings	Dwg. No.

Senior Engineer Design
Chief Engineer
Deputy Chief Engineer
Date
Scale: As Shown.
Design: Prep. M.F.C. 24/8
Drafting: Dr. M.F.C.
Ckd.
Ckd.

BRW CLIENT CHOICE AWARDS 2013 WINNER

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