Risk Level	Risk	Adaptation Option	Management
Substantial	ST4 - Breakwaters - Reduced harbour tranquillity leading to interruption to service and potential injuries to people and property moored at breakwater		Hazard and Risk assessment addressed potential impacts and provided mitigation strategies to address.

The risk level of impact ST6 was re-assessed during a workshop taking into account the identified adaptation options. The revised risk level was assessed as medium as the likelihood rank decreased to unlikely.

This risk assessment highlighted that some existing standards need to be updated to reflect projected climate changes. The design for the Precinct and the construction studies for this EIS have, accordingly, adopted a Reference Level of 5.5m LAT to accommodate potential climatic impacts.

This assessment has been used to inform relevant areas of this study. Construction levels, as detailed under Section 2.4, have used this information as appropriate in consideration of design levels against 100 year climate change scenarios. Hazard and risk assessments and the Environmental Management Plan for the TMPP have also incorporated this information when undertaking assessment of potential impacts like inundation of pavement areas and mitigation measures against these impacts are identified in Sections 6 and 8 of this document. Under the adopted mitigation strategies it is not anticipated that climatic impacts will negatively effect the TMPP.

3.6 Surface waterways

A description of the existing environment for surface waterways that may be effected by the Precinct, including Ross River, is provided under Section 3.8 – Coastal Environment and Section 3.9 – Water and Sediment Quality. These two sections address in detail the existing environment for surface waterways, which may be affected by the Precinct in the context of environmental values as defined by the EP Act and environmental protection policies.

A description is given in Section 3.8 and Section 3.9 of the waterways associated with the Precinct, their quality and quantity in the area affected by the project and an outline of the significance of these waters to the river catchments system in which they occur. This includes a characterisation of the water quality of the area from a baseline monitoring program.

The Queensland Water Quality Guidelines (2006, QWQG), the Australian and New Zealand Environment and Conservation Council (ANZECC) National Water Quality Management Strategy, the Australian Water Quality Guidelines for Fresh and Marine Waters (November 1992) and the Environmental Protection (Water) Policy 1997 are used as a reference for evaluating the effects of various levels of contamination.

Options for mitigation and the effectiveness of mitigation measures are discussed with particular reference to sediment, acidity, salinity and other emissions of a hazardous or toxic nature to human health, flora or fauna.

Details regarding flooding events are provided, potential impacts and mitigation measures on waterways resulting from the Precinct construction and operation are discussed Section 3.9 provides details of a water quality monitoring program appropriate to predicted impact management.

3.7 Groundwater resources

3.7.1 Overview

This section describes the existing environment for groundwater resources which may be affected by the Precinct in the context of environmental values as defined by the Environmental Protection Act 1994 and environmental protection policies. A review of the quality, quantity and significance of groundwater in the project area has been completed.

3.7.2 Description of environmental values

A Baseline Groundwater Monitoring program was completed as part of the environmental studies for this EIS (refer Appendix P) for a full report on that study component). The monitoring locations, TPA1, TPA3, GW1 and GW2, are shown in Figure 3-14. The assessment occurred during the summer months, capturing flood and heavy rainfall events in the Townsville region.

3.7.2.1 Geology

The bore logs from the baseline assessment (refer to Appendix P) suggest that shallow deposits immediately west of Lot 773 are characterised by layers of sand, silty sand and sandy silty clay of variable thickness and lateral extent, underlain by silty clay. The sandy deposits were encountered to between 3.8 and 5.8 m depth below ground surface and contained some shell material.

The deposits encountered appear to be of a similar composition to those investigated within Lot 773 (i.e. predominantly sandy deposits underlain by silty clay at depth) and similar to material encountered in TPA9.

3.7.2.2 Groundwater Levels

Groundwater levels in GW1, GW2 and TPA3 ranged between around 0.9 and 2.5 m AHD during the period of monitoring and peaked in early February within one day of a significant rainfall event (241.6 mm on February 3, 2009). Groundwater levels at TPA1 were recorded up to 2.6 m AHD, around 1.5 m higher than GW1, GW2 and TPA3 and suggest the presence of a recharge mound in the vicinity of TPA1. This may be associated with recent placement of materials up gradient of TPA1 in the Eastern Reclamation Area however historic data are not available to confirm this. The difference in water level could also be explained if TPA1 monitors a different water bearing horizon to GW1, GW2 and TPA3, however bore construction details are not available for TPA1 or TPA3.

Shallow groundwater levels immediately west and north of Lot 773 are influenced by tidal fluctuations (for example GW1, 8 January to 11 January), which appear to be dependent on tidal range and location, and influenced by significant rainfall events (for example 13 January). Of the monitored bores, the response of groundwater levels to tidal fluctuations is greatest in TPA3 with up to 0.6 m on February 7, 8 and 9, which is 15 - 20% of the tidal range (2.9 to

3.9 m). For the same period, groundwater level fluctuations in GW1 were only up to 0.25 m, or 5 to 7% of the tidal range. Changes in groundwater levels due to tidal fluctuations can be much less than changes in groundwater levels as a result of rainfall recharge.

Interpretation of groundwater levels for 18 December 2008 suggests groundwater flow is predominantly from west to east, towards Lot 773 and Cleveland Bay, with a very shallow hydraulic gradient (around 6.7 $x10^{-4}$). Groundwater flow direction within the Eastern Reclamation Area is not well defined due to the limited number of viable monitoring bores identified in this area (TPA1 and TPA3). The predominant flow direction within this area is likely to be east to south east and north east towards the ocean, however along the southern boundary of the reclamation area groundwater is likely to drain towards Lot 773. Groundwater flow is also likely to be controlled locally by internal bund walls within the Eastern Reclamation Area.

3.7.2.3 Permeability Testing Results

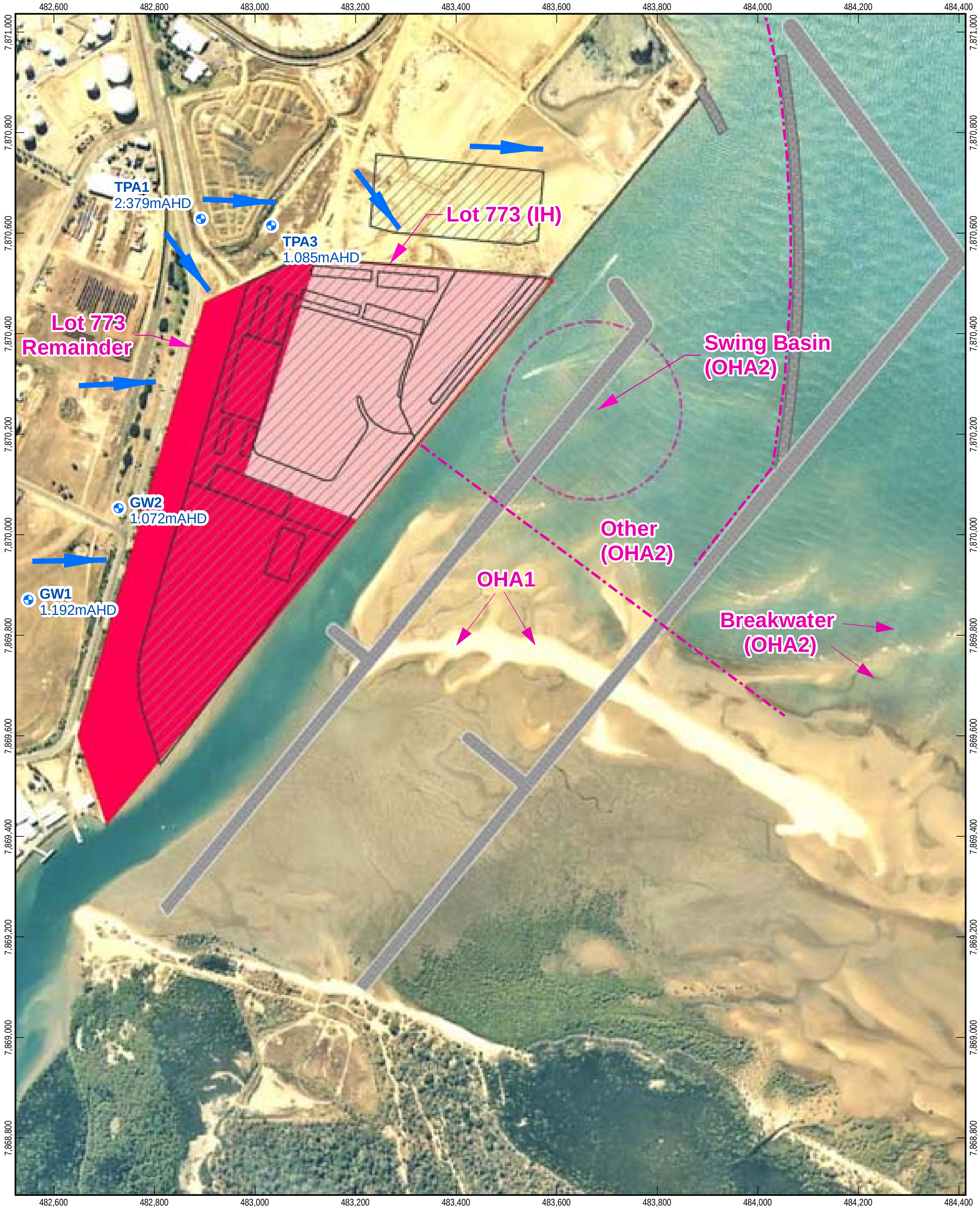
Analysis of the slug test data was carried out using the Bouwer-Rice and Hvorslev analytical solutions, supported by AQTESOLV software (developed by HydroSOLV Incorporated). Hydraulic conductivity values calculated for the screened interval of the monitoring bores are summarised in Table 3-29.

Calculated hydraulic conductivity values range between 13 and 25 m/day. This falls within the range for fine sand (0.02 to 17 m/d) and medium sand (0.08 to 43 m/d) reported in Domenico and Schwartz (1990).

Table 3-29 Permeability Test Results

Bore ID	K[2] (Bouwer-Rice Analytical Solution)	K (Hvolslev Analytical Solution)
GW1 test 1	18 m/d	24 m/d
GW1 test 2	25 m/d	25 m/d
GW2 test 1	13 m/d	13 m/d
GW2 test 2	13 m/d	13 m/d

[2] K – hydraulic conductivity



LEGEND

- Groundwater Bores (with GW elevation)
- Estimated Groundwater Flow Direction
- Proposed Marine Precinct
- Min and Max Options
- Breakwater Option C (Preferred)
- Lot 773 Inner Harbour (IH) and Trawler Basin
- Lot 773 Remainder
- Reporting Areas:
  - OHA1-Outer Harbour Area 1
  - OHA2-Outer Harbour Area 2

1:10,000 (at A4)

0 50 100 150 200 250

Metres

Map Projection: Universal Transverse Mercator
Horizontal Datum: Geocentric Datum of Australia 1994
Grid: Map Grid of Australia, Zone 55

N

Port of Townsville
Marine Precinct EIS

Job Number 42-15399
Revision A
Date 01 July 2009

Baseline Groundwater
Monitoring Locations

Figure 3-14

G:\42\15399\GIS\Projects\EIS\42-15399_410_rev_a.mxd

Level 4 201 Charlotte Street Brisbane QLD 4000 Australia T +61 7 3316 4496 F +61 7 3316 333 E bnemail@ghd.com.au W www.ghd.com.au

Data source: Marine Precinct - ©The State of QLD (Port of Townsville LTD) 2009; Aerial (flown 2004) - ©The State of Queensland (Department of Environment and Resource Management); GW Monitoring Dec 2008 - GHD. Created by: TH

3.7.2.4 Groundwater Quality

Field Observations

During development of GW1 and GW2, GW1 was observed to give off a strong ‘mangrove mud’ odour and GW2 a slight ‘rotten egg’ odour which suggests the presence of hydrogen sulfide in groundwater, one of the by-products of the oxidation of pyrite. Given the environment and ASS mapping for the area this tends to confirm the presence of acid sulfate soils in the vicinity of GW1 and GW2. No similar odours were noted whilst sampling TPA1 or TPA3.

- The recorded field pH is typically neutral to slightly acidic and ranges from 6.03 to 7.71 (GW2), which is below the Queensland Water Quality Guidelines (QWQG 2006) for enclosed coastal water of 8 to 8.4 pH units, in all bores;
- Field electrical conductivity (EC) ranged from 3,850 (GW1) to 58,600 (TPA1) and is comparable to the laboratory analysis of EC. Groundwater at GW1 recorded the lowest values of electrical conductivity (EC), which is in line with its location furthest from the coastline; and
- Dissolved oxygen levels were below the QWQG guideline value for enclosed coastal water of 90-100% saturation at all locations monitored, ranging from 8.3 (GW1) to 65.7% (GW2) saturation.

Laboratory Analysis Results

The following analytes were not detected above laboratory reporting limits:

- Volatile Organic Compounds (VOCs);
- Semi Volatile Organic Compounds (SVOCs);
- BTEX (benzene, toluene, ethyl-benzene, xylene);
- PAHs and phenols; and
- Pesticides.

Major Ions

The cation/anion balance for the major ions was within +/-5% and confirms the accuracy of the major ion analysis. The major ion chemistry characterises the groundwater as sodium-chloride type at GW2, TPA1 and TPA3 and sodium-chloride-bicarbonate type at GW1. This suggests GW1 receives significantly more freshwater than the other three monitoring locations, which is consistent with the EC and TDS values recorded for this location.

Nutrients

- Nutrients (ammonia, nitrite, nitrate and phosphorus) were detected (i.e. above the laboratory limit of reporting) in all monitoring bores, except for nitrite which was not detected in TPA1 or TPA3;
- Concentrations of ammonia range from 0.28 to 5.61 mg/L (both reported for GW2) and exceed the QWQG of 0.008 mg/L at all locations. Concentrations also exceed the ANZECC/ARMCANZ (2000) guideline value for marine ecosystems (95%) of 0.91 mg/L

ammonia in all monitored bores on all occasions except for GW1 and GW2 in the January sampling round (0.68 and 0.28 mg/L respectively);

- Total oxidised nitrogen ranges from 0.02 to 30.9 mg/L and results show that nitrate is the predominant component. Concentrations of total oxidised nitrogen are above the QWQG of 0.003 mg/L; and
- Total phosphorus concentrations range from 0.46 (GW2) to 2.97 Mg/L (TPA1) and exceeded the QWQG for enclosed coastal water of 0.02 mg/L in all bores.

Dissolved Metals

- Dissolved metals concentrations, with the exception of manganese, are typically more elevated in TPA1 than in GW1, GW2 and TPA3;
- Concentrations of dissolved copper (all bores on one or more occasions) and zinc (TPA1 and TPA3 (on one occasion) exceed or equal the ANZECC/ARMCANZ (2000) marine ecosystem guidelines (95%) of 0.0013 mg/L for copper and 0.015 mg/L for zinc;
- Dissolved cadmium concentrations exceed the ANZECC/ARMCANZ (2000) marine ecosystem guideline (99%) of 0.0007 mg/L in TPA1 only, with a measured concentration of 0.0014 mg/L;
- Dissolved aluminium was detected just above the laboratory limit of reporting (0.01 mg/L) in GW1 (up to 0.02 mg/L) and significantly above in TPA1 (0.3 mg/L);
- Dissolved iron concentrations were detected above the laboratory limit of reporting (0.05 mg/L) in GW1, GW2 and TPA3 and ranged from 0.16 to 1.33 mg/L (GW1), however dissolved iron was not recorded above the limit of reporting in TPA1;
- Manganese concentrations range from 0.024 (GW2) to 2.63 mg/L (TPA3); and
- Concentrations of arsenic were detected above the laboratory reporting limit of 0.001 mg/L and range between 0.002 mg/L (GW2) and 0.025 mg/L (TPA3).

Total Petroleum Hydrocarbons

Concentrations of total Petroleum Hydrocarbon (TPH) were detected at all monitoring locations except for TPA1 and ranged from 0.135 to 0.320 mg/L. However, concentrations were typically only just above the laboratory limit of reporting for individual TPH carbon chain fractions (i.e. C10 to C14) and were consistent with the results for BTEX, i.e. no detectable concentrations of the light fraction of TPH (C6 to C9).

TPH was detected in TPA3 as well as GW1 and GW2, which suggests that the presence of TPH is unlikely to be from drilling and bore installation.

3.7.2.5 Preliminary Conceptual Understanding

The following conceptual understanding is based on historic information and data collected as part of this baseline study:

- Infiltration of rainfall to the shallow watertable, through an unsaturated zone approximately 1.0 to 2.5 m thick in existing material, consisting predominantly sandy materials west and north (Eastern Reclamation Area) of Lot 773;

- Potential for dissolution of minerals out of the sediments as rainwater infiltrates the unsaturated zone and as groundwater levels fluctuate as a result of tidal fluctuations. If the material within the unsaturated zone includes ASS material then there is also the potential for the generation of acid as water infiltrates the unsaturated zone and the mobilising of ASS reaction products (heavy metals, acid nutrients) into the shallow groundwater;
- Mixing and dilution of infiltrated water with shallow groundwater at the water table;
- Groundwater flow down hydraulic gradient along more permeable pathways, i.e. layers with sandy material, through pore spaces towards Lot 773 from the west and from the Eastern Reclamation Area to the north;
- Material placed within Lot 773 will develop a shallow water table connected to the existing water table in adjacent materials. The degree of connectivity will depend on construction materials used. Groundwater flow through material placed in Lot 773 is likely to be towards the east or south east towards Cleveland Bay. The rate of groundwater flow from the Eastern Reclamation Area into Lot 773 is likely to be reduced once a water table in Lot 773 is fully established;
- Groundwater within the reclaimed parts of Lot 773 will receive a proportion of freshwater from infiltration of rainfall (where the ground surface is permeable) but the greater proportion of groundwater will migrate onto the site from the existing land adjacent to Lot 773 and from the sea, hence the groundwater might typically range from brackish to saline beneath the site;
- Reduction of tidal influence on the existing materials of the Eastern Reclamation Area that border Lot 773; and
- Shallow groundwater levels in existing material adjacent to Lot 773 (to the west and north) may temporarily increase during placement of material within Lot 773 but are likely to return to within normal ranges once groundwater stabilises and a water table develops within the fill placed in Lot 773. Duration and approach to reclamation works will likely influence this. If significant surcharge of reclaimed material is required additional investigations should examine the potential effects on adjacent ground water quality and flows.

3.7.3 Potential Impacts

The following potential impacts of the development of Lot 773 on groundwater have been identified based on the information presented in this report.

Stage 1 of the Development

- Construction of the Trawler Basin is unlikely to have a significant impact on groundwater levels or on the quality of water bearing horizons within the adjacent existing land (the Eastern Reclamation Area) given that the point of contact of the moorings with the existing land will be relatively small;
- The quality of the water bearing horizon within any reclaimed land as part of the Stage 1 development is not likely to impacted from up gradient sources of groundwater given the limited contact with the adjacent land; and

- Depending on the composition of the fill material(s) used to construct Stage 1 there may be potential for degradation in the quality of groundwater within the fill material as a result of dissolution of minerals, including metals, and leaching of salts from the fill into groundwater. This could occur if the pH of groundwater within the fill material were to become acidic from infiltration of water through oxidised sulfidic materials.

Stages 2 and 3 of the Development

- Potential for a temporary increase in shallow groundwater levels within the existing material adjacent to Lot 773, during placement of fill material within Lot 773. Under extreme circumstances (heavy rainfall combined with a King tide and rapid placement of fill in Lot 773) groundwater levels could potentially rise to ground surface. Given the predominantly sandy nature of the shallow water bearing strata and that the aquifer is unconfined, however, groundwater level increases from loading, although possible, are likely to be insignificant in comparison to increases as a result of rainfall and tidal fluctuations. If significant surcharge is required of reclaimed material detailed investigations of potential impacts should address this potential;
- Potential for degradation in the quality of groundwater within the fill material of Lot 773 as a result of dissolution of minerals, including metals, and leaching of salts from the fill into groundwater however, will depend on the composition of the fill material(s) used to reclaim Lot 773. This could occur if the pH of groundwater established within the fill material were to become acidic from infiltration of water through oxidised sulfidic materials;
- Potential for degradation in the quality of the groundwater that will establish within fill placed in Lot 773 as a result of the migration of existing groundwater onto Lot 773 from up gradient sources containing components including dissolved metals, TPH and nutrients;
- Potential for degradation of the quality of surface water in Cleveland Bay as a result of the discharge of groundwater from within Lot 773 to the ocean;
- Potential for brackish/saline groundwater beneath the site to negatively impact the integrity of foundations and infrastructure within Lot 773, such as through corrosion, if they come into contact with groundwater or the capillary fringe; and
- If acid leachate is generated from ASS materials in the unsaturated zone and/or if foundation materials come into contact with acidic groundwater, for example as a result of acid leachate entering groundwater, then there is potential for a negative impact on foundations and infrastructure above (i.e. in the unsaturated zone) and/or below the water table.

3.7.4 Mitigation Measures

The following is applicable to any groundwater monitoring carried out for the site:

- A suitably qualified and experienced professional will carry out the monitoring in accordance with the AS/NZS 5667.11:1998 Australian/New Zealand Standard for water quality – sampling Part 11; Guidance on sampling of groundwaters;
- Standing water levels are to be recorded prior to purging of all monitoring bores;
- A NATA registered Laboratory is to be used for all analysis; and

- Laboratory Quality Control and Quality Assurance plans and protocols are to be supplied for all samples submitted for QA purposes and field replicate samples and blanks will be collected at a rate of 1 in 10 samples or part thereof.

The following baseline groundwater monitoring is recommended to be carried out for the site:

- Continuation of groundwater level monitoring on a monthly basis for GW1, GW2, TPA1 and TPA3 to obtain a minimum 12 months of data;
- Continuation of groundwater quality monitoring on a quarterly basis of GW1, GW2, TPA1 and TPA3 to obtain a minimum 12 months of data (see Table 3-30); and
- Review of the action criteria proposed for monitoring during construction and after development once 12 months of baseline data have been obtained to determine whether the recommended action criteria and sampling frequencies for the construction and operational phases of development are still appropriate. Update the EMP if necessary.

Monitoring During Development/Construction

Implementation of Stage 1 of the development (Trawler Basin) is not considered to significantly impact existing groundwater levels or groundwater quality and therefore routine monitoring should be conducted during the construction period.

Stages 2 and 3

- Groundwater quality monitoring on a monthly basis (see Table 3-30) in all monitoring bores during construction of Lot 773; and
- Comparison of groundwater level and water quality data against action criteria after every monitoring round and follow up with action if required.

Routine and Post Development/Construction Monitoring

- Establishment of a groundwater monitoring bore network within Lot 773 to monitor the potential impacts on groundwater quality within Lot 773 and potential risk to the receiving environment (Cleveland Bay);
- Quarterly recording of static groundwater levels (see Table 3-30) in all monitoring bores outside of Lot 773;
- Quarterly sampling for selected analytes (see Table 3-30) in all monitoring bores outside of Lot 773; and
- Comparison of groundwater level and water quality data against action criteria after each monitoring round.

Post development monitoring for should be carried out for a minimum of 12 months following completion of construction and the results reviewed by an experienced hydrogeologist to assess future monitoring requirements.

Table 3-30 Baseline groundwater quality sampling frequency and parameters

Parameter	Units	Sampling/ Monitoring Frequency (Baseline)	Sampling/ Monitoring Frequency (Construction/ Development[3])	Sampling/ Monitoring Frequency (Routine & Post Construction[4])
Field Parameters				
Static water level	m AHD	Monthly	Monthly	Quarterly
pH	pH units	Quarterly	Monthly	Quarterly
Temperature	°C	Quarterly	Monthly	Quarterly
Electrical Conductivity (EC)	µS/cm at 25°C	Quarterly	Monthly	Quarterly
Dissolved Oxygen	% saturation	Quarterly	Monthly	Quarterly
Redox potential	mV	Quarterly	Monthly	Quarterly
Laboratory Analysis				
Electrical Conductivity (EC)	µS/cm	Quarterly	Quarterly	Quarterly
Major Ions (Ca, Mg, Na, K, Cl, CO_3, HCO_3, SO_4)	mg/L	Quarterly	Quarterly	Quarterly
Nitrate and Nitrite as N	µg/L	Quarterly	Quarterly	Quarterly
Ammonia as N	µg/L	Quarterly	Quarterly	Quarterly
Nitrogen oxides (NO_3 + NO_2) as N	µg/L	Quarterly	Quarterly	Quarterly
Total Phosphorus	µg/L	Quarterly	Quarterly	Quarterly
Dissolved metals (low level) – Al, As, Cr, Cd, Cu, Fe, Hg, Mn, Ni, Pd, Zn	µg/L	Quarterly	Quarterly	Quarterly
TPH (C6 to C36)	µg/L	Quarterly	Quarterly	Quarterly

Auditing

Auditing shall take place in accordance with the respective Environmental Management Plan (construction or operation) for the site.

[3] Stages 2 and 3

[4] Includes Stage 1

Proposed action criteria (outside of which action should be taken) are identified in Tables 7-1 and 7-2 in Appendix P. Recommended corrective actions are identified in Table 3-31 and reporting actions are identified in Figure 3-32 below.

Table 3-31 Corrective Actions for Potential Impacts Identified

Impact	Response/Action	Corrective Action
During Construction Stages		
Increase in shallow groundwater levels within existing materials adjacent to Lot 773	Review data (levels, rainfall and tides) for increasing trends and compare levels to ground surface to establish cause of increase	Cessation of placement of fill if cause of increase is not considered to be rainfall or tidally related. Continue placement of fill only once levels return to background
Post Construction Stages		
Degradation in the quality of groundwater within the fill material placed within Lot 773 as result of in-situ processes	Increase frequency of sampling of selected water quality parameters within Lot 773 to monthly (including pH, EC and dissolved metals). Conduct a review of site data to determine the cause of degradation and asses the environmental risks to the site	Prepare and implement a remediation program to address the identified risks
Degradation in the quality of the groundwater within fill placed in Lot 773 as a result of on-site migration	Increase frequency of sampling of selected water quality parameters (on-site and off-site) to monthly (including pH, EC and dissolved metals). Identify the reasons(s) for degradation and asses the environmental risks to the site	Implement a strategy to minimise the migration of poor quality groundwater onto the site
Degradation of the quality of surface water in Cleveland Bay as a result of the development from spills/leaks on Lot 773	Documentation of the incident	Application of the correct management options adopted dependent on the level or environmental risk

Table 3-32 Reporting Summary

Report	Content	Timing
Monthly report	The report shall detail the monitoring carried out, any non-compliance events over the monitoring period and general groundwater quality. The report will also detail the action taken to rectify the non-compliance where action is required.	Each construction stage.
Non-compliance report	A brief report will be prepared documenting the non-compliance and any corrective actions.	Where a non-compliance events occurs.
End of construction report	A report summarising groundwater characteristics and trends during each construction Stage.	End of each construction stage.
Annual report	The report will summarise the results of the preceding period including groundwater quality and trends, groundwater levels, current monitoring network and any recommendations for the following 12-month period. This may include recommendation of no further monitoring.	End of the financial year/each 12-month period following completion of construction.
Site specific trigger levels report	The report will review the interim trigger levels and set site-specific trigger levels based on 18 sampling periods over at least a 12-month period.	At the completion of 18 sampling periods

3.8 Coastal environment

3.8.1 Existing wave environment

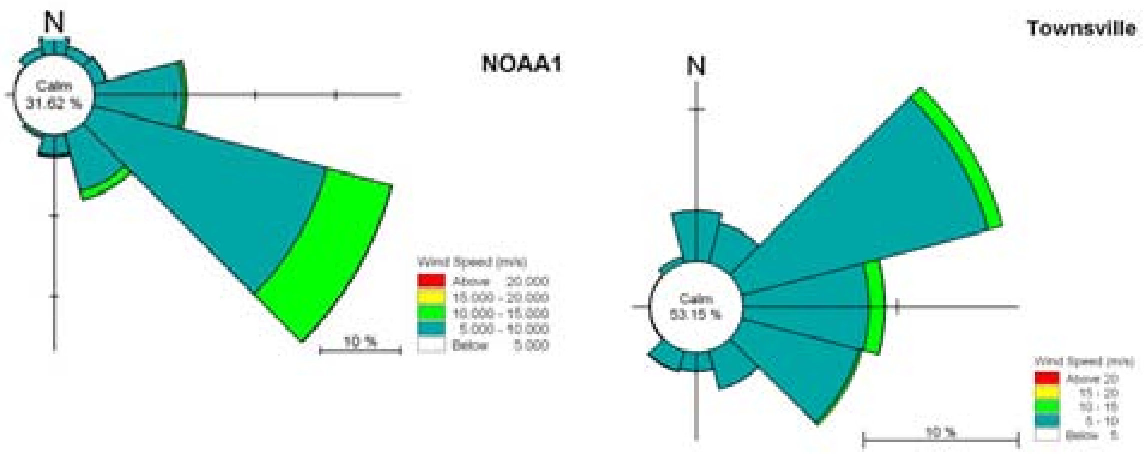
The proposed marina precinct is located at the mouth of the Ross River, on the eastern side of the existing Port of Townsville (refer Figure 2-1). Magnetic Island, situated directly north of the site provides protection from northerly waves. The dominant wind direction is from the trade winds from the south-east to east, however due to Cape Cleveland, waves generated offshore by the easterly wind diffract around Cape Cleveland and become north-east as they propagate into Cleveland Bay.

Offshore waves from other directions can reach the site at reduced heights through refraction around Cape Cleveland and Magnetic Island. The result is that waves reaching the Precinct area and proposed breakwater footprint have a predominately north-east direction.

The Townsville coastline is naturally protected from offshore wave conditions, such as long period ocean swell waves, by the Great Barrier Reef, which sits approximately 70 km from the shoreline. The wave climate in the area is therefore mostly governed by local winds, acting on the area between the reef and the coastline and within Cleveland Bay. As there is a large distance, or fetch, between the reef and the coastline, relatively large waves can still be generated during storms or cyclones.

Analysis of the available wind data for the region from Bureau of Meteorology (BoM) and National Oceanic and Atmospheric Administration (NOAA), extracted from the global WaveWatch III hindcast model, shows a predominant wind direction of north-east close to shore, with a more south-easterly component offshore, at the location of NOAA1, just inside the reef. The respective wave roses are illustrated in Figure 3-15 below.

Figure 3-15 Wave roses offshore and nearshore for Townsville Region



A spectral wave model has been employed to determine the likely 1 in 1 year and 1 in 100 year waves generated near the existing Port of Townsville. This is a wind driven wave model and is based on analysis of the available wave data. A detailed report of all wave modelling for this EIS is available in Appendix Q. The result of this modelling indicates that nearshore wave heights for 1 in 1 year and 1 in 100 year return period conditions are expected to be around 1.0m and 2.8m respectively. The wave heights offshore of Cleveland Bay in these conditions are respectively about 1.6 and 6.1m.

An analysis of different breakwater options was undertaken to determine the optimum breakwater configuration to provide protection to the proposed Precinct as well as allow for future expansion of the port. That assessment is described in detail in Section 1.4.2. Option C was selected from that process as the preferred breakwater configuration for consideration under the EIS studies.

Detailed wave modelling against Option C breakwater has been completed for investigation of performance of this structure under varying incident wave scenarios. A full description of those findings is provided as Appendix Q and effects of the breakwater on wave conditions is discussed below in Section 3.8.3.

3.8.2 Existing coastal processes and sedimentation

The coastal processes that operate in the vicinity of the proposed Precinct at the mouth of the Ross River have been investigated by examining sediment inputs and the processes that effect such including longshore sediment transport and historical sediment movement regime for the area. This has been done in conjunction with an assessment of the influence of waves on sediment movements. From this a description of the existing littoral transport regimes has been developed. The effect of the proposed development on those processes has been assessed and the likely operation issues for the marina precinct in terms of sediment movement have been identified.

The Precinct covers an area to be reclaimed and the proposed breakwater is positioned offshore from the mouth of the Ross River in Cleveland Bay, extending a short distance to the south-east. The sources of sediment that could affect this area of Cleveland Bay are Cleveland Bay itself, the Ross River, and the foreshore areas south-east of the site. Mechanisms for moving sediment are wave action, tidal currents, flood flow currents, wind driven currents, and longshore sediment transport. A detailed study of the coastal influences on sediment movements is provided in Appendix Q and is summarised following.

3.8.2.1 Wave climate

Existing wave climate of Cleveland Bay and the Precinct area is described above under 3.8.1. South-easterly waves under the influence of strong south-easterly trade winds will refract into the bay with a small proportion of the wave energy reaching the proposed site. Less frequent waves from the north-east and north will propagate through the gap between Cape Cleveland and Magnetic Island directly affecting the site.

3.8.2.2 Tidal currents

Tides in Cleveland Bay are mainly semi-diurnal with a spring tide range of around 2.4 metres and a maximum range of 4.0 metres. Ebb and flood tides generate substantial tidal currents especially during the higher range of spring tides (Pringle 1996). However, these are concentrated in the deeper areas of the Bay and have little influence on sediment movement along the shoreline south-east of the Ross River, apart from in the immediate vicinity of the river mouth where tidal currents are aligned with the river channel.

Tidal currents in the Ross River are moderate (refer Section 3.8.4.2) given the depth of the dredged entrance channel and the reduced tidal prism in the river, brought about by the construction of Aplin's weir, approximately 10 kilometres from the river mouth, in 1927.

3.8.2.3 Ross River

The Ross River was originally a primary source of sediments for Cleveland Bay. With the construction of the dam in 1973 and three weirs in the 1900's virtually all bed load transport of sediments to the coast has ceased. Currently 750km^2 of catchment land is located above the dam compared to approximately 45 km^2 located below. The weirs downstream of the dam in addition to altering the river hydraulics also retain sediments depending on their height above the river bed and are occasionally dredged. Sediment input from the catchment to Cleveland Bay is, therefore, unlikely to be reinstated while dredging of the accumulated sediments from behind the weirs continues. For the purposes of evaluating the effect of river flows on the sediment budget at the Precinct site, it can be concluded that the Ross River does not contribute any bed load sediment.

Fine sediment in the form of silts and muds will still be transported down the river as suspended load and a proportion of this could settle out in the Precinct with the majority being carried out into Cleveland Bay. The settlement pattern will depend on the flood flow velocities and the flood volume.

3.8.2.4 Wind driven currents

According to Pringle (1996) sediment on the coast and bed of Cleveland Bay is primarily siliceous and is supplied mainly from terrigenous sources by rivers and creeks, with some of the sediment originating from major floods in the Burdekin River. Wind records show that the prevailing winds are from the south-east which induces surface water currents capable of carrying suspended sediment alongshore. One of the outcomes of this phenomenon is a major current flowing southward along the west, leeward coast of Cape Cleveland, reinforced by the tidal flood current. This current induces sub-tidal bed load movement of sediment by ripple migration, which supplies sediment to the south Cleveland Bay intertidal flats. Further movement of sediment to the west towards the mouth of the Ross River is surmised to be through wave-induced longshore drift.

However, there is no net longshore movement from the bottom of the bay towards the Ross River, so the southern part of the bay is a sediment sink for sediments moving into the bay down the Cape Cleveland coastline.

3.8.2.5 Historical sediment transport regime

Coastal aerial photography has been assessed to determine the historical movement of the coastline and any notable features. The photography obtained was captured on the following dates:

- 14 June 1974;
- 28 November 1978;
- 14 July 1981;
- 14 July 1985;
- 10 September 1991;
- 7 August 1993;
- 17 November 1997; and
- 25 May 2003.

All photography was at a nominal scale of 1:12,000 and was captured within 2 hours of low water. The extent of the coverage was from the Ross River to Sandfly Creek (approximately 3.5 kilometres to the south-east). The photography was rectified and a number of features were mapped for each date. The features mapped are:

- Coastline – defined by the seaward limit of coastal vegetation;
- Beach – defined by the extent of exposed sand along the coastline;
- Exposed Sandbar – defined by areas of exposed sand above water level away from the coastline;
- Submerged Sandbar – defined by areas of sand below water level; and
- Mangroves – defined by the aerial extent of mangroves.

Observations

Coastal migration

There are a number of areas where the coastline has migrated landward by up to 100 metres. However, for all but the area closest to the mouth of the Ross River, the apparent landward movement has been replaced by a growth of the mangrove fringe.

Adjacent to the mouth of the river, there has been a general landward recession of the beach between 150m to 700m from the river mouth, with a maximum recession of 60 metres around 300m from the river mouth. In this area there are two distinct discontinuities in the coastline that appear to be “hard points” against which sand has accumulated. This indicates that there is some longshore transport along this section of beach.

The 100 metre section of coastline immediately adjacent to the river mouth prograded seaward between 1974 and 1978 and since then has shown little movement. It is concluded that the longshore transport movement along this section of beach must drop into the river channel to be distributed along the channel by tidal flows.

Beach migration

In general terms the beach width appears to have narrowed, possibly as a result of increasing vegetation cover. In one particular area at 900 metres from the river mouth, the beach has disappeared having been overtaken by an extensive area of mangroves.

Exposed sand bank

Photographic analysis (refer Appendix R) shows that the presence of the large exposed sand banks near the offshore end of the dredged channel is a relatively recent phenomenon. The first major sand bank appears in 1993, dissipated into a submerged sand bank in 1997, and returns much larger in 2003 and about 40 metres further landward. Since 2003 this sand bank has developed further, providing increased sheltering of the areas landward of it from wave action, thus encouraging the extensive growth of mangroves between the shoreline and the landward edge of the sand bank.

The shape of the sand bank in 2003 and its relative location to the 1993 sand bank indicates a net longshore movement along the seaward face of the sand bank.

Submerged sand bank

In the area adjacent to the offshore end of the dredged channel, the submerged sand banks have moved gradually closer to the channel and closer to the shore with the movement between 1991 and 1993 being predominantly onshore.

Further to the south-east at approximately 1000 metres from the river channel, the onshore movement is demonstrated clearly by a particular sand bank near the offshore limits. The particular sand bank first appeared in 1985 and by 1993 had moved 100 metres shoreward.

Generally the movement of the sand banks is onshore with some longshore movement close to the dredged channel of the Ross River. Away from the river, the lack of any significant longshore movement is demonstrated by the stable location of the channels of Stuart and Sandfly Creeks where they cross the tidal flats.

Mangroves

The main feature to note is the growth of the mangrove fringe between 1974 and 2003. In 1974 the mangroves occupied and area of coastline on either side of Stuart Creek. By 2003 the extent of mangroves had increased threefold with mangroves from about 500 metres from the river mouth to Sandfly Creek. By 2008, the mangrove areas southeast of the Ross River entrance have extended out to the landward edge of the offshore sand bank.

Assessment

Based on the assessment, detailed in Appendix R, the coastal processes in this area comprise:

- Onshore movement of sediment towards the coast under the action of waves and wind driven currents;
- The formation of submerged sand bars at the offshore limits of the tidal flats;
- Progradation of the sand bars across the intertidal flats;
- Establishment of mangroves along the coastal fringe as fine sediment gets pushed up to the shoreline, and
- Possible establishment of new beach line seaward of mangroves.

Adjacent to the dredged river channel movement of sand at the seaward edge of the exposed sand banks and also at the shore face is longshore. In addition the movement of the submerged sand banks in this area is both onshore and towards the dredged channel of Ross River. The dredged channel, therefore, is a sink for this sand movement and it is expected that the channel filling is concentrated adjacent to the outer sand banks and near the mouth of the river.

3.8.2.6 Longshore sediment transport

Wave modelling (described above and in Appendix Q) and data collated to support the breakwater options assessment have been used to support the assessment of the potential longshore sediment transport during normal conditions along the shoreline on the eastern side of the TMPP.

To determine an average nearshore wave climate, a long-term model simulation is required. Given the length of time required to run a wave model with an input data set of multiple years, a representative year was extracted from the NOAA wind time series based on wind speed exceedance distributions. Wind speed exceedance curves were plotted for each year and compared with the exceedance curve for the complete time series (Figure 3-16). 2005 was selected as the representative year, based on the high correlation with the complete 11 year time series.

The digital elevation model (DEM) for this long term simulation was constructed using bathymetric data extracted from C-Map (refer to Figure 3-17). The 2005 predicted Townsville tidal time series was also extracted from C-Map and used as the water level input for the model. As detailed above, the 2005 NOAA wind speed time series was applied over the model area.

Cape Cleveland, to the east of the site, is found to significantly shelter the Precinct area from all directions except north-east (Figure 3-18). Locations closer to the headland have a much higher occurrence of calm conditions (greater than 50% < 0.1m). Wave heights of less than 0.1m are considered to be calm conditions and are expected not to result in significant sediment transport. These conditions were excluded from longshore sediment transport calculations. Locations 1, 2, 3 and 5 depicted on Figure 3-18 were selected for the investigation of sediment transport.

Figure 3-16 NOAA Wind Exceedance Plot (Offshore Townsville

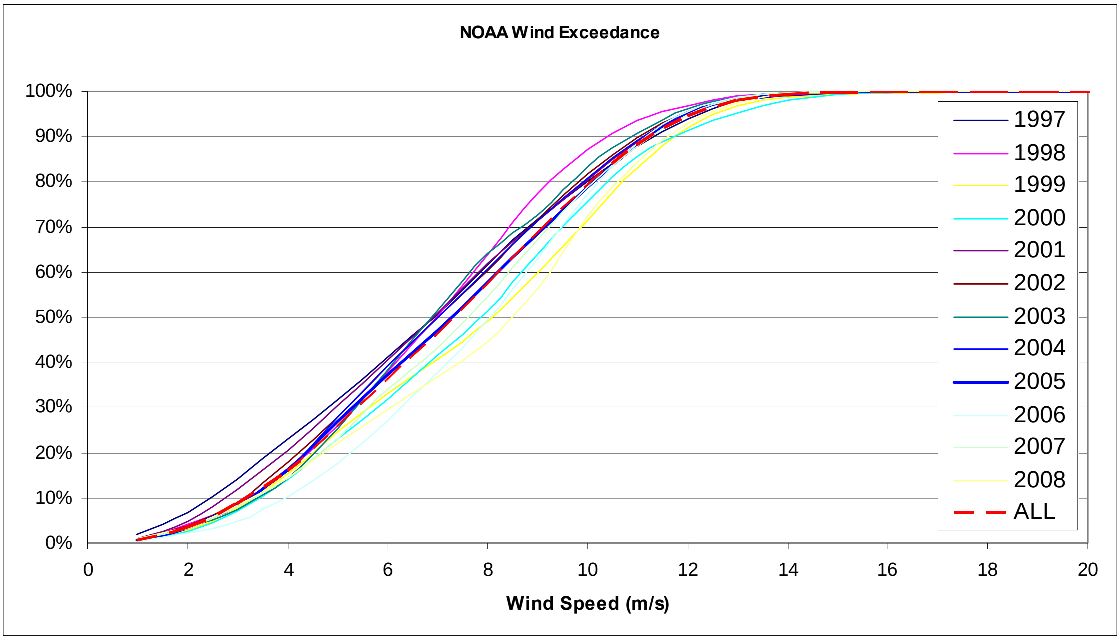


Figure 3-17 DEM of Townsville and the Surrounding Area

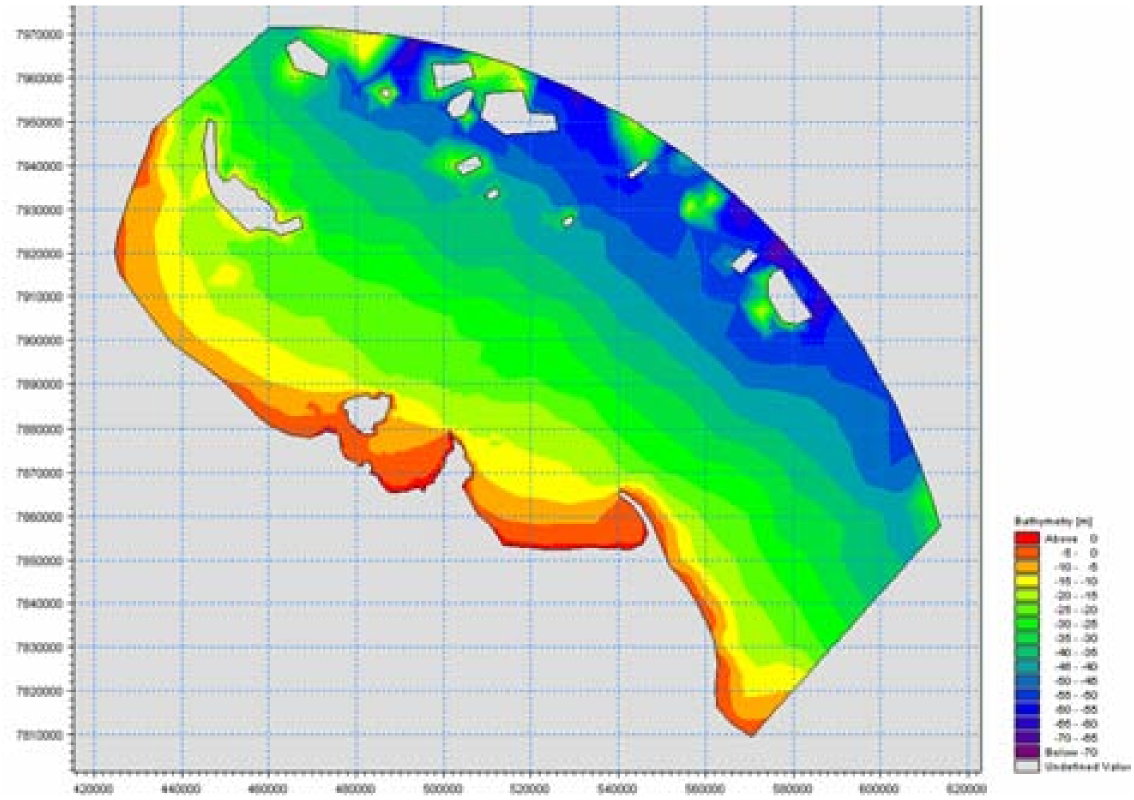
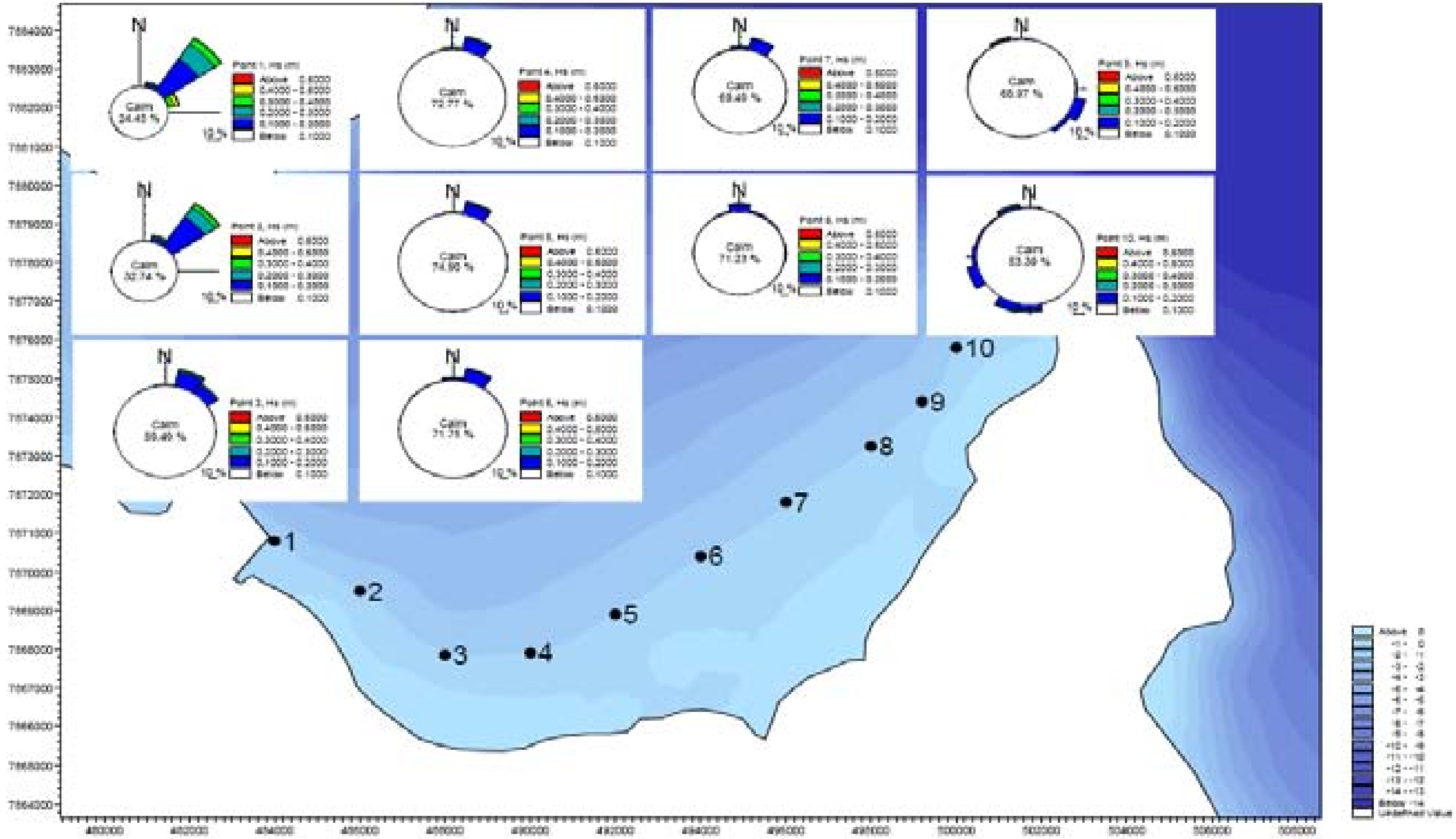


Figure 3-18 Wave Roses and Nearshore Reporting Locations



3.8.2.7 Potential Longshore Transport

Longshore sediment transport volumes of approximately 4000 m3/year were calculated at the more sheltered locations, however there is a potential for up to 25,000 m3/year closer to the Precinct (location 1, Figure 3-18), given the higher degree of exposure to the dominant wave direction. Refer to Figure 3-19 for the positive and negative sediment transport rates at the four locations.

Some of this transported material has contributed to the sand bank that has formed south east of the entrance channel, with the remainder falling into the entrance channel and being distributed along the channel by tidal currents and flood flows from the Ross River. Therefore under existing conditions there is potential for this longshore transport to result in silting of the channel and this is borne out by the maintenance dredging of the channel that is currently carried by the Port.

In addition, the negative transport rate at point 2 may transport a portion of any sediment plume from the Ross River under extreme runoff conditions, to the nearshore sandbar evident in aerial photography. Additionally, the positive rate at point 3, combined with the negative rate at point 2, may create a sediment transport null point, which will contribute to the stability of the sandbar at this location.

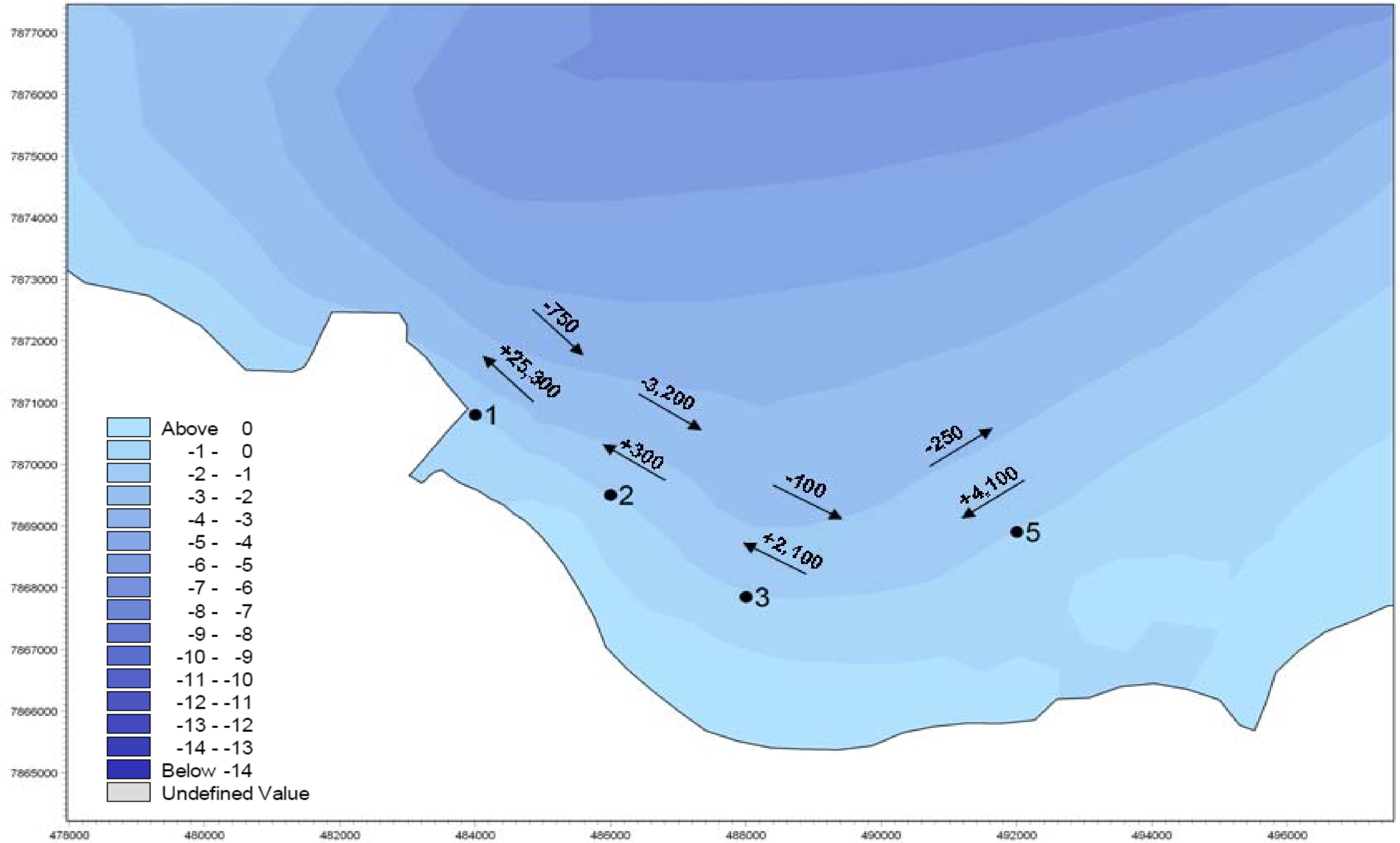
The above values apply to a grain size (D50) of 0.4mm; however final transport volumes are very sensitive to sediment size. The chosen grain size range results in a sediment transport range of 2,000 to 35,000 m^3/year over the area between the marina precinct and Cape Cleveland.

Of interest, in the context of the coastal processes described above, are the dredging records for the river channel, summarised in Appendix R. The average extraction rate from the river is 37,600m3/yr between 1971 and 2006. The maximum siltation rate from longshore transport at the river mouth and adjacent to the outer sand bar is 24,550 m3/yr (refer Figure 3-19). Given that there are other sources of siltation, this indicates that the longshore transport calculations are of the right order.

3.8.2.8 Conclusions

It is expected that longshore sediment transport rate in the vicinity of the south eastern extremity of the Option C breakwater is towards the Precinct (north westerly) and of the order of 15,000 m3/year taking into account the rate of change of transport potential towards point 1. A potential sediment transport null point is located between points 2 and 3 (refer Figure 3-18), resulting in the build up of the nearshore sand bank evident in aerial photography. The larger transport potential near the dredged entrance channel of the Ross River has resulted in the formation of the prominent sand bank abutting the dredged channel, and has contributed to silting of the channel.

Figure 3-19 Calculated sediment transport rates (m^3/yr) for median grain size of 0.4mm.



3.8.3 Effects of breakwaters on coastal processes

3.8.3.1 Wave environment

Figure 3-20 and Figure 3-21 below show the significant wave contours for the Precinct and port area with no breakwater and with Option C breakwater configuration.

Model results for the existing, no breakwater scenario, reveal that the average wave heights at the location in yearly conditions can be as high as 1.0m. This suggests that, for the no breakwater scenario, smaller vessels (<25m) will have difficulty in navigation and berthing will be also challenging, even for larger size recreational vessels, without protection from ambient wave conditions. The extreme events will also expose vessels to large waves of 1.5m or greater.

Option C provides a high level of protection to the Precinct and in the lee of the breakwater, as required for boat mooring, while allowing for future expansion of the port.

This improvement is further illustrated by analysis of the wave parameters, extracted adjacent to the reclaim area for the proposed marina precinct and reported in Table 3-33 for the two analysed return periods.

Table 3-33 Wave parameters comparison

Option	Return Period (yr)	Hs	Tp	MWD
No Breakwater	100	1.2 – 1.4	9.0	45
	1	0.6 – 0.8	6.0	40
Option C	100	0.4 – 0.6	9.0	40 – 50
	1	0.0 – 0.2	6.0	35 – 40

It should be noted that due to the model limitations with respect to diffraction and reflection, the wave conditions inside the breakwaters reported above should be considered indicative only and were used for comparison purposes. A more specific model with the ability to take into account diffraction and reflection interactions with structures was employed to further evaluate the impact of the breakwater on the Precinct and to ensure that the Precinct would comply with AS3962, the Australian Standard for the Design of Marinas. Based on the standard, a limiting wave height of 0.3m in 1 in 1 year would be considered acceptable and 0.25 m excellent.

The model results evaluating tranquillity behind the main breakwater and inside the proposed harbour basin are presented below in Figure 3-22, for 1 in 1 year and 1 in 100 year return period cases.

Figure 3-20 No breakwater significant wave height contours. 1 in 1 year return period (left) and 1 in 100 year return period (right)

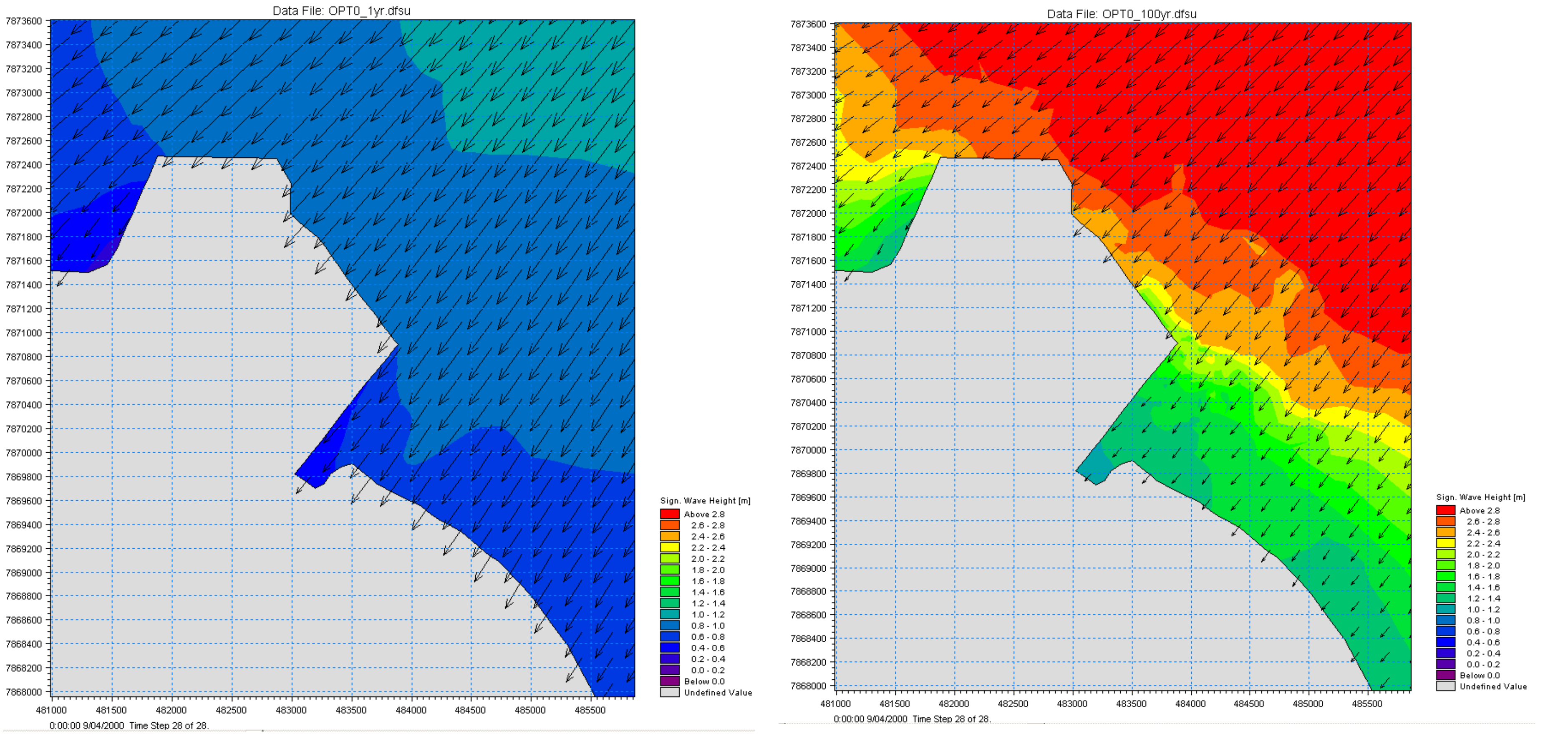
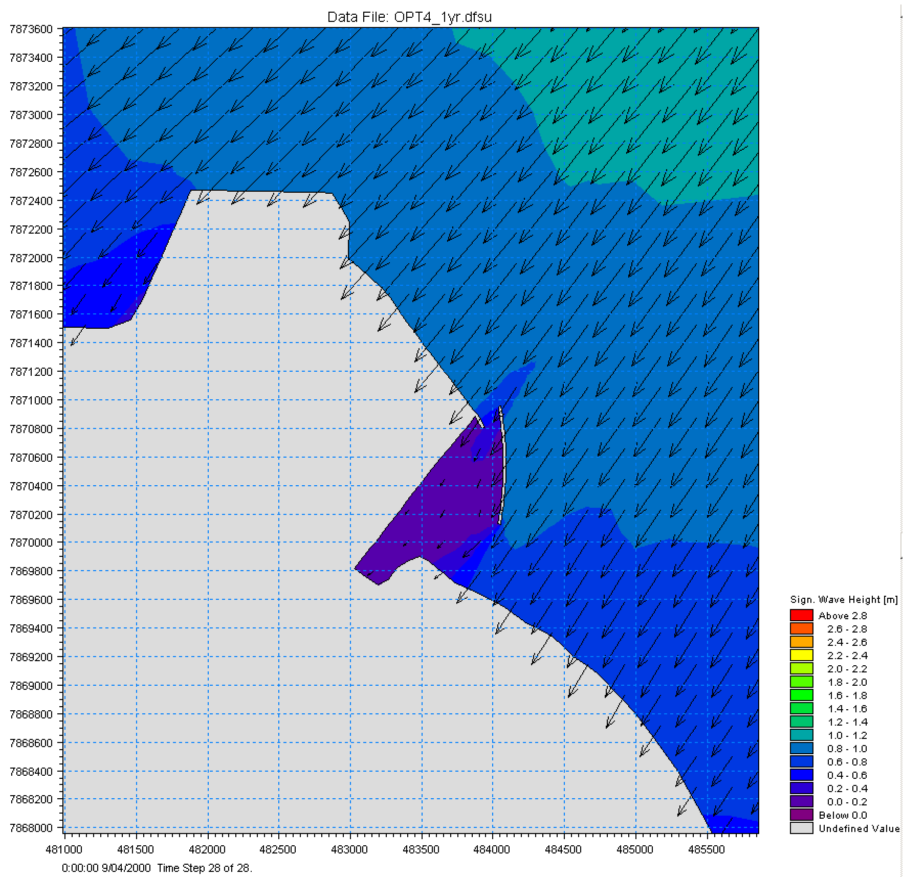


Figure 3-21 Option C breakwater configuration significant wave height contours. 1 in 1 year return period (left) and 1 in 100 year return period (right).



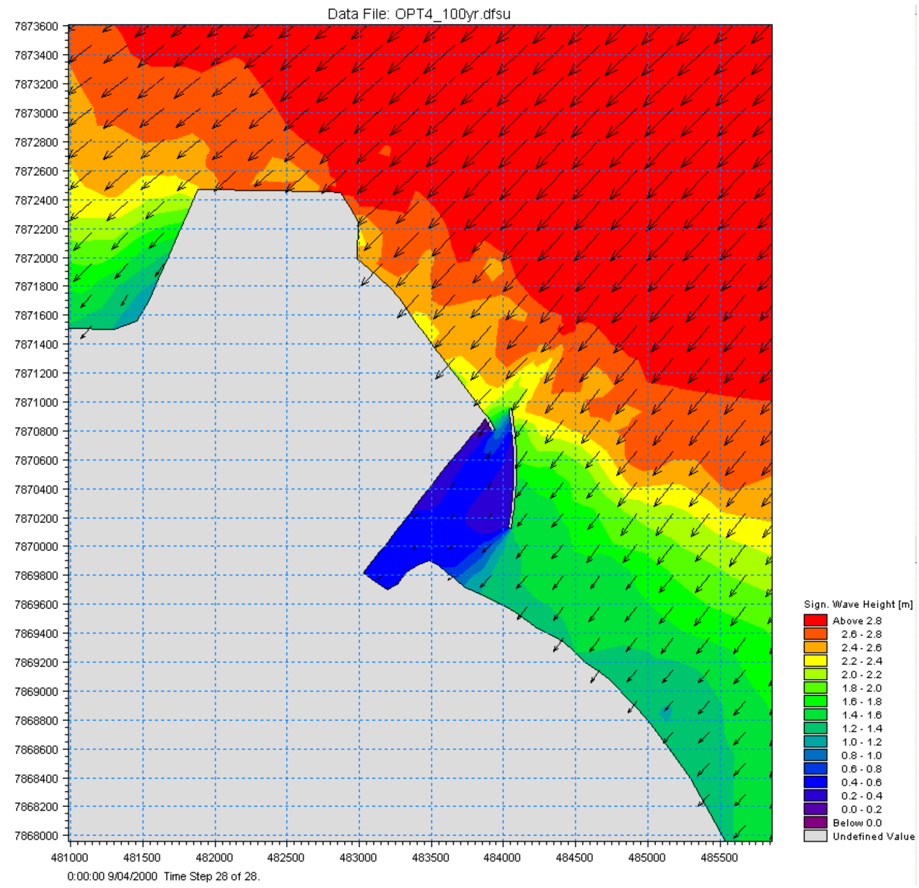
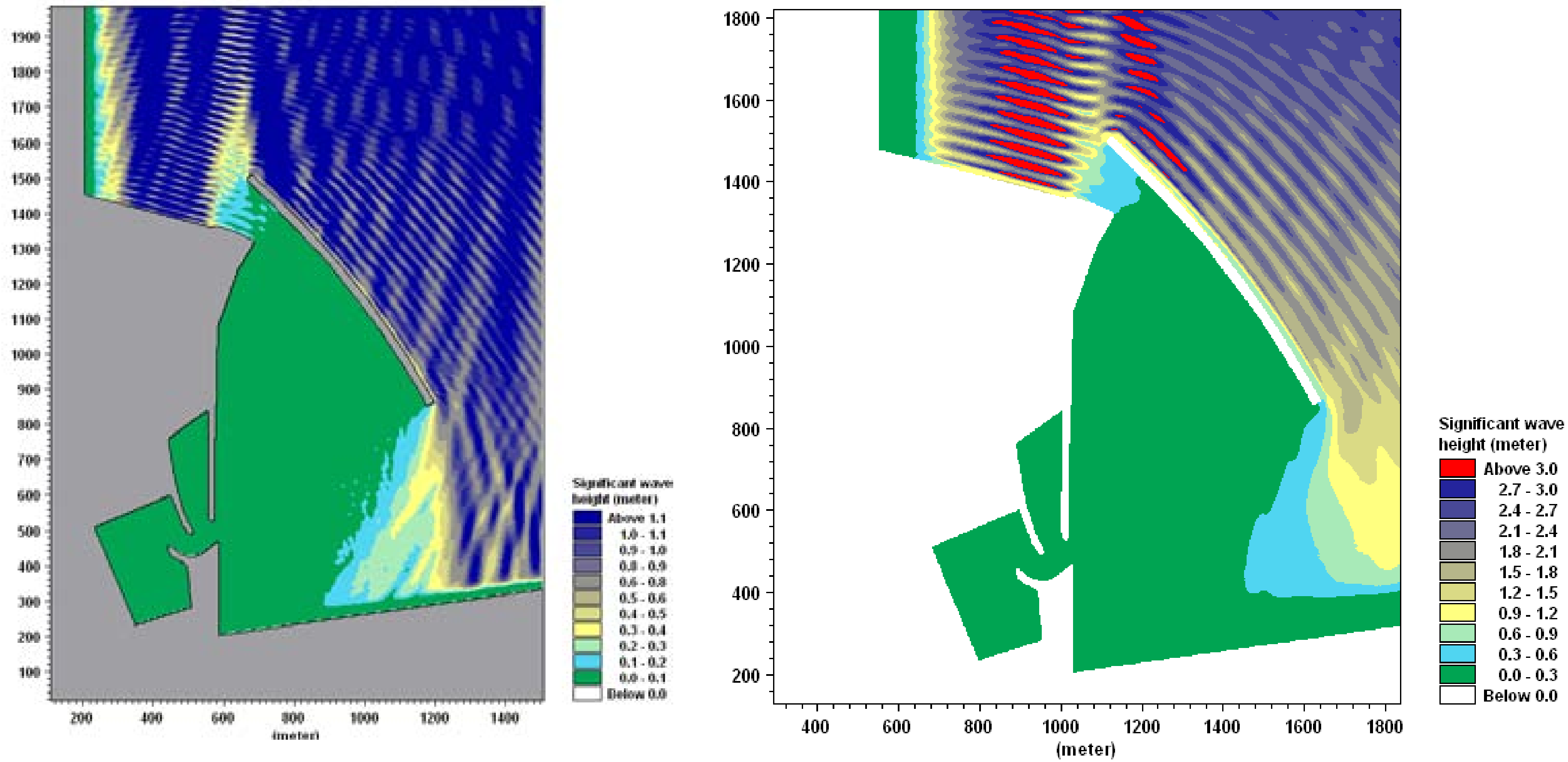


Figure 3-22 Option C significant wave height contour plots - refined model. 40°, Uni-directional, Monochromatic. 1 in 1 year (left) and 1 in 100 year (right).



The detailed modelling results reveal that in 1 in 1 year conditions, the significant wave height behind the breakwater and in the harbour is less than 0.2m in all cases. However, it can be seen that during 1 in 1 year events, reflection from the port structures at the entrance of the main breakwater increases the wave heights to as high as 1.5m. This may cause some navigation issues for smaller vessels coming through the channel into the harbour under storm conditions.

Evaluation of wave heights inside the harbour for 1 in 100 years has also revealed that the breakwater structure provides acceptable level of protection during storms. Wave heights inside the breakwater are generally very small, less than 0.3m. However, reflection from the breakwater has a similar effect to 1 in 1 year and can create large waves at the entrance, up to 3m in extreme events, potentially causing navigation difficulties.

The modelling results in general confirm that the layout of the breakwater is adequate to provide high level of protection against waves in all conditions to AS 3962 standards.

3.8.3.2 Longshore sediment transport - effects to the west

The Port development blocks any influence of coastal processes in the vicinity of the Precinct on the coastal areas north-west of the Port. The establishment of a Precinct will not influence this fact. The Port development (including the Port areas beyond the original coastline, breakwaters, other reclaimed areas, and the dredged entrance channel) effectively isolates the processes that occur south-east of the Port from the areas to the north-west.

There is no doubt that the Port and ancillary development have had a profound effect on the Strand beach immediately to the west. In a report to the Townsville City Council Mabin (1996) stated that since 1874 the Port has blocked the supply of sand to the beach from the Ross River mouth and the breakwaters that extend nearly 2km out into Cleveland Bay have shielded the beach from much of its normal wave energy.

However, there is another factor that needs to be considered in relation to the state of the coastline and that is the changes to the sand supply in this region. The Sinclair Knight Merz report (SKM 1996) highlighted that the loss of the sand supply to the coast is a more fundamental reason for the degradation of the coastline. The two principal causes of lost sand supply to the beaches to the west of the Ross River are changes to the river hydraulics (through the construction of weirs and dams affecting both the supply of sediments to the river and the flushing of these from the river) and sand mining of existing river resources.

Notwithstanding the reasons for the degradation of the coastline west of the Port area, the proposed Precinct will have no additional contributory effect on either of the causes of the degradation outlined above and hence will have no influence on the state of the beaches to the west in either the short or long term.

3.8.3.3 Longshore sediment transport - effects to the east

The Ross River and its current dredged channel form the boundary of longshore sediment movement from the beach and tidal flats to the south-east of the marina precinct. The sediment movement in this area is a mixture of onshore and alongshore at the outer margins of the tidal flat and predominantly along the beach towards the Ross River close to the river entrance. Further to the south-east away from the river, sediment movement is predominantly onshore.

Breakwaters proposed to be parallel to the existing dredged channel will affect sediment movement into the channel near the outer sand banks. Where the breakwater crosses the active littoral zone, it can be expected that there will be a slow build-up against the breakwater extending away to the south-east. The rate of build-up will be commensurate with the prevailing longshore transport rate. Similarly, a breakwater connected to the shore south-east of the river mouth will stop the small north-westerly flow of sand along the shore face in this area and lead to a slow build-up against the breakwater on the south-eastern side.

The breakwater proposed as Option C is located offshore from the currently prominent sand banks adjacent to the Ross River entrance channel and will directly affect sand transport that occurs in the shallow waters seaward of these banks. It will have an indirect effect on the longshore transport by "shadowing" the area closest to the Ross River channel effectively reducing the longshore transport to zero adjacent to the channel.

The principal beneficial effect of the breakwater in terms of coastal process is that they will provide some control over the longshore movement of sediment into the Precinct inner harbour and the existing dredged channel and reduce the maintenance dredging requirements in the short to medium term. In the long term, at the point where the longshore transport has effectively "filled" behind the breakwaters and the sediment paths have re-established around the breakwater structures and into the Precinct, increased maintenance dredging may be required.

An additional effect of breakwaters on the adjacent coastline is that generated by wave reflection and is dependent on the slope and nature of the seaward slope of the breakwater and the orientation of the breakwater to the coastline. For Option C, reflected waves could propagate parallel to the coastline to the south east and influence the longshore transport volume and direction in this area.

The coastal processes in the vicinity of the Precinct comprise both onshore/offshore and longshore components and are influenced by the proposed Option C breakwater structures in a number of ways. However, the processes are capable of moving sediment at only relatively slow rates due to the low wave climate and hence any changes will take time to develop and will be restricted to the local area. It is concluded that it is unlikely that there will be any significant affects on coastal processes from the Option C breakwater structures forming the Precinct on the coastal areas beyond around 500m south-east of the breakwater structures.

3.8.3.4 Longshore sediment transport changes resulting from Option C

The main breakwater in Option C extends from near the corner of the POTL eastern reclamation south east in a curve finishing offshore from the large prominent sand bank located at the seaward edge of the tidal flats. The likely changes to longshore sediment transport that may occur in the vicinity of the breakwater have been examined and are discussed below.

It is unlikely that sedimentation will cause major changes at the main entrance to the marina, due to the depth of the dredged channel reducing the ability of the currents to mobilise the bed sediments and the very limited sediment transport around the outside of the breakwater.

However, at the south eastern end of the breakwater, the water depths are much less and any currents generated by flood flows or tidal flows will have a much greater influence on sediment movement. In addition, it is here that the longshore sediment transport potential is the greatest.

Due to the “shadowing” effect of the breakwater, the longshore sediment transport will tend to accumulate in the lee of the breakwater and extend out to the south east over time. The growth of this sedimentation towards the end of the breakwater will be limited by the flood and tidal flows between the end of the breakwater and the sand bank. Flood flows and ebb tide flows will push sediment from shallow areas inside the marina and from the accumulated longshore transport deposition area, out of the marina onto the outer margins of the tidal flats to the south east. Flood tidal flows will cause sediment to move into the marina depositing sediment in the dredged areas adjacent to the end of the breakwater and areas where the current velocities are low. The above scenarios are summarised in Figure 3-23.

The conclusions in relation to dredged areas that may be affected by sedimentation are as follows:

- Sediment from longshore transport along the seaward edge of the outer sand banks will accumulate in the lee of the south eastern end of the Option C breakwater with most of the sediment accumulating along the outer sand bank to the south east. This sediment is not expected to settle far enough into the marina precinct to affect the dredged access channel or the mooring area immediately behind the breakwater.
- Flood flows from the Ross River and ebb tide flows will generally push sediment out of the marina precinct through the marina entrance and at the south eastern end of the breakwater and therefore not affect the dredged areas. However, large floods may move sediment out of the river into the Precinct and may also cause a general redistribution of sediments in the area, some of which may be deposited into the dredged areas.
- Flood tidal flows between the south eastern end of the breakwater and the outer sand banks may move sediment from the edge of the longshore accumulation and the ebb tide delta to the south east into the Precinct and this sediment is likely to accumulate in the dredged channel and the mooring area behind the breakwater. The rate of accumulation is governed by the strength of the currents and the availability of sediment. Lower sediment availability due to the trapping of the longshore transport, compared with existing conditions indicates that the rate of sedimentation will to be low.

1:15,000 (at A4)

0 100 200 300 400 500

Metres

Map Projection: Universal Transverse Mercator
Horizontal Datum: Geocentric Datum of Australia
Grid: Map Grid of Australia 1994, Zone 55

N

LEGEND

- Sediment Transport Calculation Points
- Breakwater Option C (Preferred)
- Proposed Marine Precinct

Sediment Contours (1m)

- 0
- -1
- -2

GHD

PORT of TOWNSVILLE

Port of Townsville
Marine Precinct EIS

Job Number	42-15399
Revision	A
Date	01 July 2009

Predicted Sediment Distribution Patterns from Longshore Transport and Tidal Currents

Figure 3-23

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Level 4 201 Charlotte Street Brisbane QLD 4000 Australia **T** +61 7 3316 4496 **F** +61 7 3316 333 **E** bnemail@ghd.com.au **W** www.ghd.com.au

Data source: Sediment Deposition - GHD 2009; Marine Precinct - ©The State of QLD (Port of Townsville LTD) 2009; Aerial (flown 2004) - ©The State of Queensland (Department of Environment and Resource Management). Created by: TH

3.8.4 Hydrodynamics and sedimentation

3.8.4.1 Background

Hydrodynamic and sediment transport models were originally developed by GHD for the then Townsville Port Authority from 2001 to 2003. These models, which had previously been calibrated to tide levels and sediment transport trends for the outer harbour, were also applied to the marine precinct area. For the current study, a number of improvements have been made, with the introduction of an additional level of high resolution nesting allowing detailed visualisation of results in the area of interest. Additional model calibration to measured currents has also been undertaken, as described further below.

Given the origin of the model, much of the data that has been utilised relates to the earlier work. Data sources included reports relating to previous capital works (TPA Capital Dredging Works 1993), research publications (Pringle (1989), Kettle *et al*. (2001)), data collection (wave data recording program Townsville Region 1975-1997) and operational numerical models (GHD 2001). This has been augmented by additional datasets with respect to localised bathymetry, tidal boundary conditions, and measurements of turbidity.

3.8.4.2 Tidal characteristics

Tidal constituent data has been obtained from Queensland Transport for the Port of Townsville. Tidal planes are given in the Tide Tables (QDOT 2009) as follows:

Table 3-34 Semidiurnal tidal planes for the Port of Townsville

Tidal Plane	Abbreviation	m AHD
Highest Astronomical Tide	HAT	+2.15
Mean High Water Springs	MHWS	+1.21
Mean High Water Neap	MHWN	+0.36
Mean Sea Level	MSL	+0.10
Mean Low Water Neap	MLWN	-0.27
Mean Low Water Springs	MLWS	-1.13
Lowest Astronomical Tide	LAT	-1.86

Tidal ebb and flood generates important tidal currents especially during the higher range of spring tides (Pringle A. 1989). Flood tide currents entering Cleveland Bay from the east, swing round Cape Cleveland and move across the Bay south-westwards with speeds of up to 0.5 m/s. Flood tide currents entering Cleveland Bay from the north, swing closer to Magnetic Island, reaching speeds of 0.2 to 0.3 m/s. A third flood stream, entering the Bay through the West Channel between Magnetic Island and Cape Pallarenda, reaches a speed of 0.7 m/s.

According to Mason *et al.* (1991), during neap tides (range 0.5-0.8 m) currents are of irregular direction and are generally less than 0.05 m/s velocity; during spring tides (2.3-3.6 m) currents vary between 0.15-0.30 m/s with minor asymmetry (flood slightly stronger). During extreme spring tides, currents may exceed 0.70 m/s. The measured tidal asymmetry indicated that net sediment transport should be into the Bay. The above general transport patterns were originally replicated in the GHD (2003) report.

3.8.4.3 Sediment data

Information pertaining to the distribution of sediments and turbidity within the study area is available from a variety of sources. This information is a key input to the sediment modelling when considering the potential impacts of the dredging process.

- Larcombe and Ridd (TPA EMP, 1993) report that sea-bed sediments with bimodal grain size distribution are common in Cleveland Bay. Given that 7% to 40% of material is finer than coarse silt, there is ample opportunity for the resuspension of sediment within Cleveland Bay.
- Peak near-bed suspended sediment concentrations (SSC) of 300 mg/l have previously been measured in water depths of 3 to 15 m,
- Mean near-bed suspended sediment concentrations of the order of 100 mg/l have been measured;
- Typical threshold shear stresses for sediment re-suspension are estimated at 1 N/m2.

More recent water quality measurements from work conducted for the current EIS (refer Section 3.9) have shown that:

- The turbidity level is fairly uniform across the water column;
- The calibration work concluded that a 1:3.5 relationship exists between the total suspended solids concentration (TSS) and turbidity NTU, i.e. TSS (mg/l) = 3.5 Turbidity (NTU).
- A median background concentration of 80 mg/L has been adopted based on measurements in the study area.

3.8.4.4 Sediment transport

A detailed explanation relating to the key driving forces affecting sedimentation patterns in this area was provided in earlier reports (GHD 2001). Further consideration has now been given with respect to littoral transport processes in the marine precinct. This report (Coastal Processes Study, GHD 2009) is presented as a separate appendix (Appendix R) to the EIS. An overview of key findings is reproduced below.

In this area the coastline configuration comprises major sand banks offshore near low water mark, shallow mud flats between the sand banks and the shore face, and a narrow sandy beach at the shore face. There are therefore two potential longshore transport pathways, one along the seaward edge of the sand banks and a second along the beach near the mouth of the river. The transport along the offshore sand bank will be the dominant mechanism as the sand bank is exposed to the limited wave climate that can mobilise the sediments. Transport along the beach is much less significant as the sand bank protects the beach from all waves except those that propagate across the sand bank at the highest of high tides.

3.8.4.5 Bathymetry

The bathymetry of the study area, a key input to the modelling process, has been based on a range of sources, including survey data provided by the Port of Townsville, DHI's CMAP database, the Australian Admiralty Charts, Australian geological Survey Organisation data and ETOPO2 datasets as listed below:

- Australian Hydrographic Chart 257 (Townsville Harbour and Ross River Entrance), Scale 1:7,500
- Australian Hydrographic Chart 256 (Cleveland Bay and Approaches), Scale 1:50,000
- Australian Geological Survey Organisation (AGSO) bathymetric 30 arc second grid
- ETOPO2 - The "Smith/Sandwell" data base, a set of 2-minute gridded ocean bathymetry derived from 1978 satellite radar altimetry of the sea surface that was interpreted as gravity anomalies and extrapolated to depth equivalents.

Recent aerial observations from commercial aircraft have shown that the sand bar to the west of Ross River mouth (refer Appendix I and R) has grown further. It is evident that there is no water transport across the sand bar under prevailing oceanic conditions. In the absence of detailed bathymetry in this very shallow mudflat, bathymetric data in the model has been manually adjusted to represent the sand bar. It will be shown later that impacts relating to the proposed development do not extend to this region.

3.8.4.6 Measured currents

Two acoustic doppler current profilers (ADCPs) were deployed at locations close to the site, in order to collect measurements of tidal currents and wave heights. The data from these ADCP units has subsequently been utilised in the calibration of both the wave and hydrodynamic models. The location of the ADCP deployments is shown in Figure 3-24. Data was collected for more than 1 month at each location, providing an enhanced data set for the purposes of calibrating the model for tidal currents. Previous calibrations had been primarily reliant on tidal water levels, with limited available current measurements.

LEGEND

- ADCP Locations
- Project Area of Interest
- Proposed Marine Precinct
- Proposed Breakwater

1:150,000(at A4)

0 1 2 3 4 5

Kilometres

Map Projection: Universal Transverse Mercator
Horizontal Datum: Geocentric Datum of Australia 1994
Grid: Map Grid of Australia, Zone 55

N

Port of Townsville
Marine Precinct EIS

ADCP Deployment
Locations

Job Number	42-15399
Revision	A
Date	01 July 2009

Figure 3-24

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Data source: Marine Precinct ©The State of QLD (Port of Townsville LTD) 2009; ADCP locations - GHD; 250K Topo Data - ©Commonwealth of Australia (Geoscience Australia) 2009;. Created by: TH

3.8.4.7 Numerical model establishment

Utilising the Delft software, existing models were enhanced in order to allow the simulation of water levels, tidal currents, bed shear stresses, waves, flushing characteristics and sediment transport.

- Numerical models of waves and currents cover areas from the mouth of the Ross River and encompass all of Cleveland Bay,
- The flushing characteristics of the mouth of the Ross River and the proposed marina precinct are defined in terms of the elapsed time to reach the e-folding time of flushing The latter analysis has been undertaken using a conservative, non-decaying substance (tracer) with the entire modelling area initialised with a constant concentration of the substance of 1 kg/m3.
- The e-folding time of flushing is a classical estimate of the flushing potential of a water body and is encountered when the concentration in the water column at a specific location is reduced to 1/e (approximately 37%) of the initial concentration.
- Wave-current interaction has been simulated by iteratively coupling the depth integrated hydrodynamic model (Delft3D FLOW) to the 2D phase-averaged spectral wave model SWAN.

Listed below are the main modelling assumptions relating to this study, as initially established for the Commercial Marina study (GHD 2003).

- Local winds are spatially uniform and varying in time. They are represented by a dataset collected at Cape Cleveland;
- A uniform value of Manning's number (0.023), a hydraulic parameter that describes bed roughness, is adequate to force the model to replicate the tidal flows in Cleveland Bay and the mouth of the Ross River.
- Sediment transport, as for the underlying hydrodynamics, has been modelled as two-dimensional (vertically-integrated);
- Following the outcome of a study by Larcombe *et al*. (2000), it is assumed that Cleveland Bay turbidity is not limited by sediment availability for re-suspension from the sea-bed;
- Swell from the east and south-east is the key driver for re-suspension of bed material. Lou and Ridd (1996) analysed two high turbidity events, recorded in the Bay in 1993, where suspended sediment concentrations (SSC) reached over 100 mg/l. The analysis revealed that these events were the result of strong swell events.
- For most of the modelled scenarios (i.e. other than the flood event scenarios), freshwater inflow from the Ross River has been ignored. This is consistent with the report of Kettle *et al*. (2001), which states that the regulation of Ross River has reduced fluvial discharge into the Bay, in turn increasing the influence of tidal processes. Peak annual flows of 500 to 1000 m3/s were common before the Ross River Dam was constructed with zero flows recorded only once every 25 years. By contrast, zero flows occurred for 48% of all years after the dam was constructed. Freshwater inflow, and hence sediment load from the Ross River, has been assumed nil for three of the four modelled scenarios (refer Table 6 of Appendix I). In

the case of the fourth scenario (adopted Precinct layout under the effect of a flood tide event), a flood event with maximum flood discharge of 1090 m3/s and a sediment load of 500 mg/l has been modelled. The selected discharge is nominally representative of a 100 year average recurrence interval (ARI) event.

3.8.4.8 Model grids

The local scale hydrodynamic model of the Marine Precinct (referred henceforth as model “D”) shown in Figure 3-25 has been established on a curvilinear orthogonal grid. The grid has highest resolution (10m x 10m) in the Marine Precinct, extending from the proposed breakwater to the upper extent of the Ross River. The “D” model was nested into a “C” model (Figure 3-26) which has cells 40 by 60 m near the mouth of the river while cells at the seaward extent lie in the range of 300 m by 600 m (at the seaward entrance of Platypus Channel) and up to 600 m by 700 m (at the tidal flats in Southern Cleveland Bay).

In turn, the “C” hydrodynamic model was nested within two large-scale regional models (referred henceforth as models “A” and “B”), which provided tidal elevation, salinity and temperature forcing for model “C” at the seaward open boundaries. Models “A” and “B”, which are part of the modelling system developed in 2001 for the investigation and mitigation of siltation in Platypus Channel (GHD 2001).

Existing hydrodynamic conditions are modelled first to provide a basis for comparison with hydrodynamic results following the establishment of the proposed marina and breakwater. The “A” model was first run using tidal constituents for August 2008 with three subsequent levels of nesting leading to the above mentioned “D” model.

Figure 3-25 “D” model grid showing increasing resolution towards the Precinct

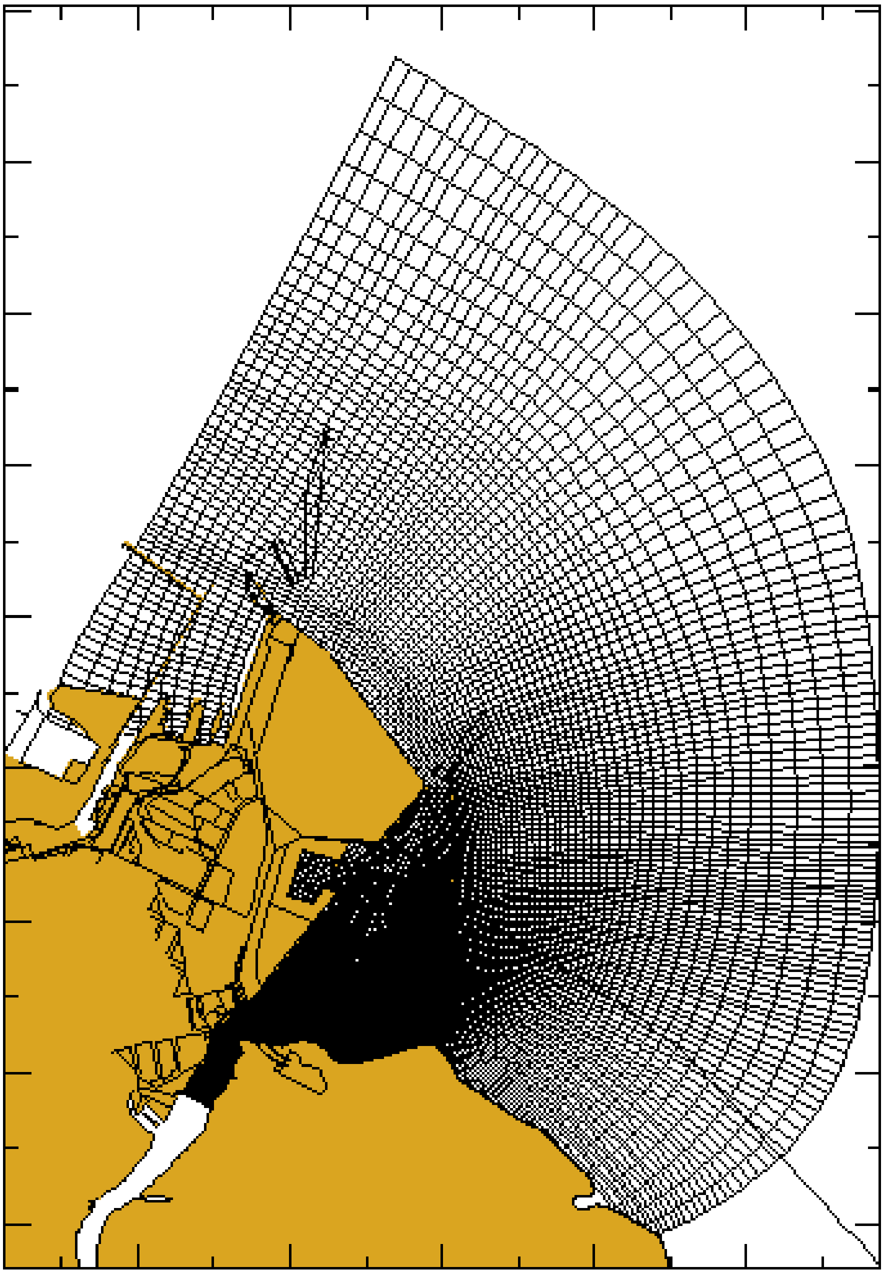
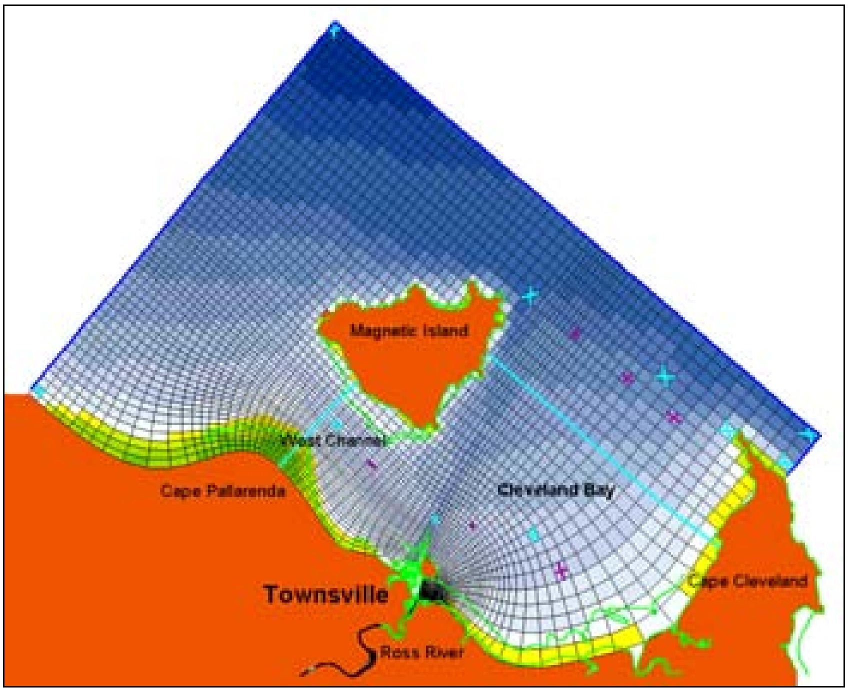


Figure 3-26 Grid for Model "C"



Wave modelling in Cleveland Bay and at the entrance of the Ross River has been undertaken using the two-dimensional, phase-averaged spectral wave model SWAN integrated into the Delft3D suite of models. In the present study, the domain of the SWAN model was slightly larger than the "C" class model.

3.8.4.9 Sediment transport model

Sediment transport modelling has been carried out using the online sediment transport module of Delft3D. The sediment transport model is capable of simulating cohesive and non-cohesive transport under wave and tide action, deposition, hindered settling and flocculation of suspended sediment and re-suspension of seabed material subject to consolidation. The sediment transport model simulates short-term transport of suspended (cohesive) sediment generated in the process of dredging, and has been operated for a period of two months.

3.8.4.10 Modelling scenarios

A range of modelling scenarios were investigated in order to provide an assessment of the combined impacts of tides, waves and winds and a 100 year average return interval flood event in the vicinity of the proposed marina and channel dredging works. These scenarios, described in Table 3-35 have been built around a variety of forcing conditions (tide, tide with prevailing waves, tide with storm waves, and tide with flood), which have generally been run for both the existing and developed (with breakwaters and marina constructed) conditions. A dredge plume scenario has also been investigated.

Table 3-35 Scenarios and modelled processes.

#	Modelled scenarios	Tide	Tide & Wave Interaction	Sediment Transport	Ross River Inflow
1	Existing conditions	✓	✓	X	X
2	Developed conditions - proposed marina dredged to –4.5 m LAT while approach channel dredged to –3.0 m LAT and breakwater Option C.	✓	✓	X	X
3	Construction dredge scenario (existing conditions / no breakwater in place)	X	✓	✓	X
4	100 year ARI flood event characterised by a maximum discharge of 1090 m^3/s	X	X	✓	✓

3.8.4.11 Hydrodynamic model calibration

Whilst calibration of the Port of Townsville model and Marine Precinct models were initially completed in 2001 and 2003, a second level of calibration of the model to tidal currents has been undertaken for the current study. This takes into consideration the newly acquired acoustic doppler current profiler (ADCP) measurements of tidal currents within Cleveland Bay. The “C” grid model was used for this calibration with measurement of currents and waves completed between 15 August 2008 and 22 September 2008 within Cleveland. Data from the offshore ADCP has been used in the calibration.

Plots of current magnitude (Figure 3-27) and water level (Figure 3-28) over a one month period show very good correlation between measured and modelled values. In addition to the fit by eye, the use of statistical methods allows a quantitative assessment of the standard of calibration. In this case, the method of measuring correlation between two data sets is to calculate the root mean square (RMS) error. The RMS error for the offshore current magnitude time series was 6%, which indicates a very good level of correlation. Figure 3-29 shows measured and modelled currents for a shorter time frame (3 days) and indicates that while the phase and relative magnitudes in currents match quite well (an RMS error of only 3%), the modelled currents are slightly underestimated. One reason for this discrepancy is that the calibration model does not include the effect of waves.

Figure 3-27 Measured (ADCP) versus modelled current magnitude time series for offshore site.

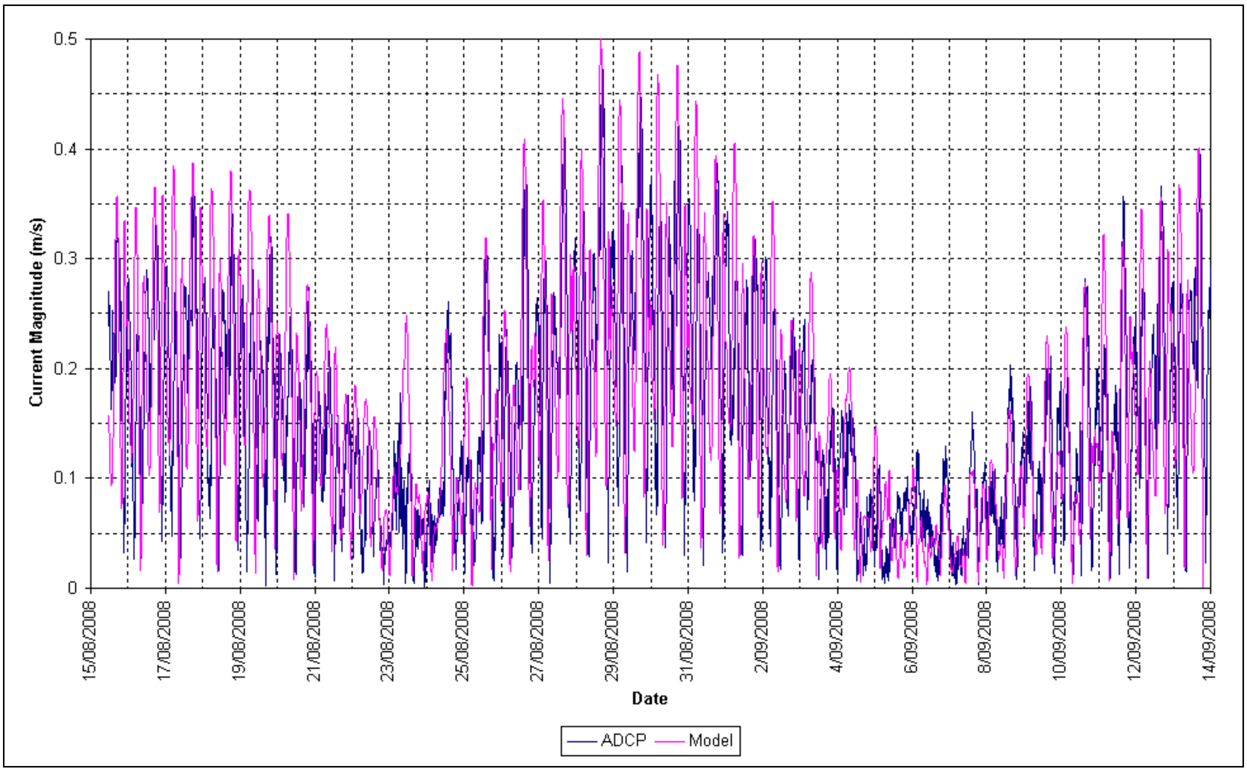


Figure 3-28 Measured (ADCP) vs modelled water level time series for offshore site

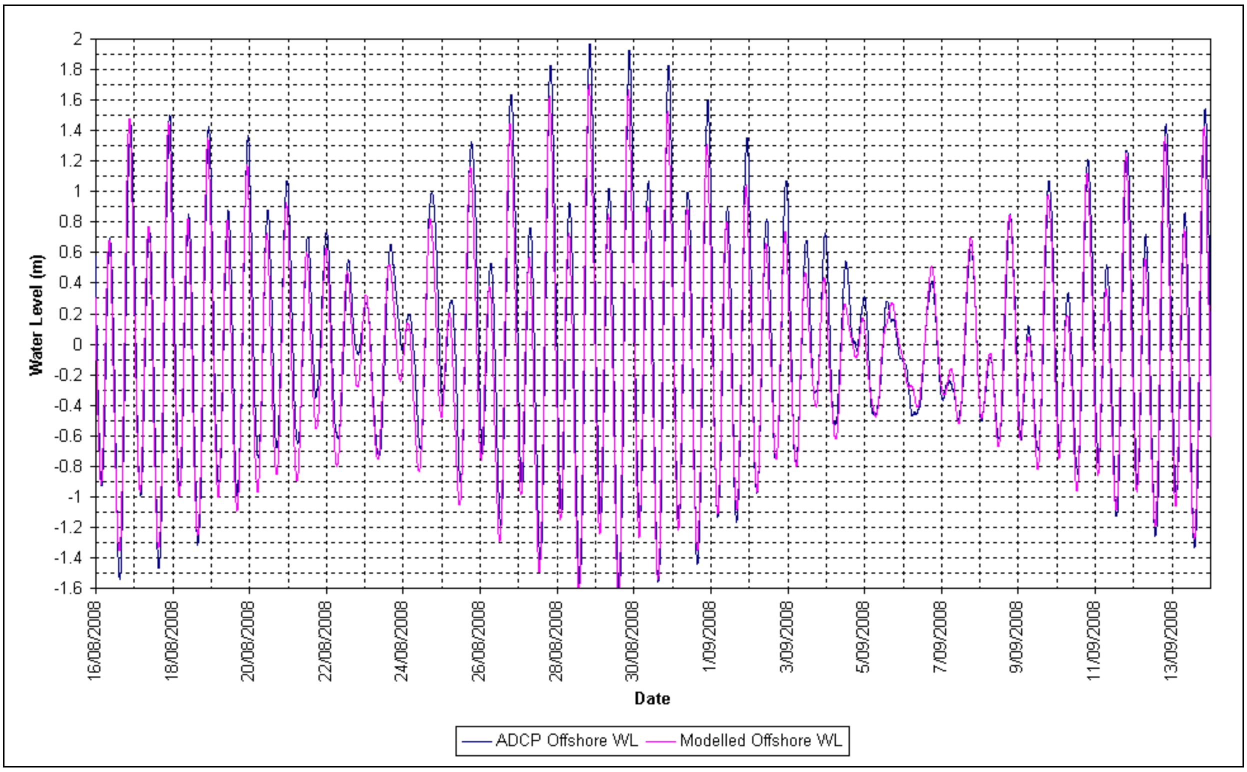
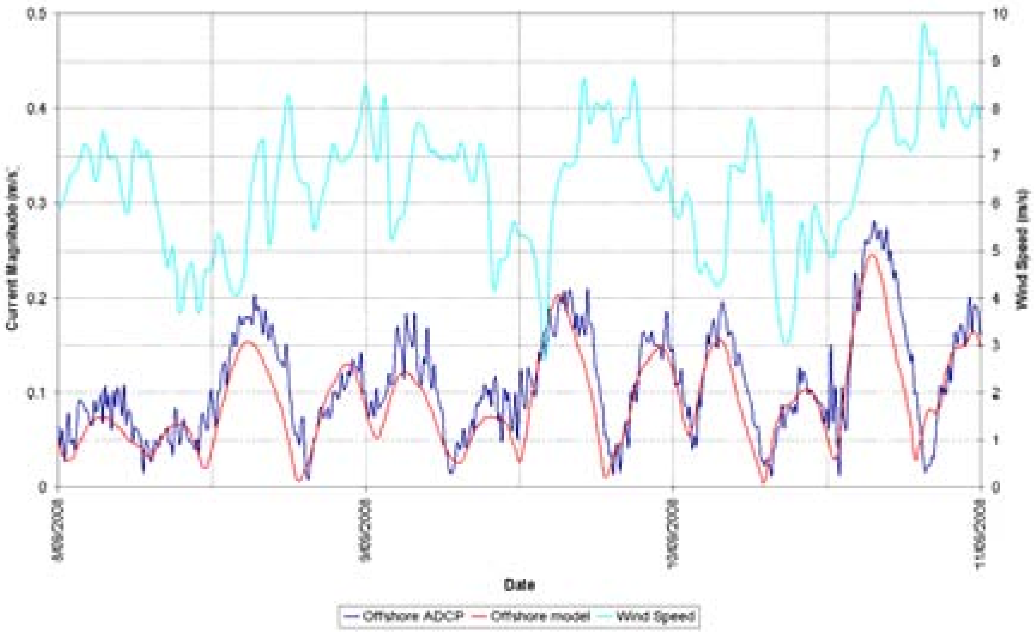


Figure 3-29 Presentation of ADCP measured and modelled current magnitudes (offshore site) for a shorter time frame, showing the standard of correlation between measured and modelled values.



3.8.4.12 Modelling Results – tidal circulation

Modelling results are presented in several formats, in accordance with the key items of interest. These consist of hydrodynamics (water levels and currents), bed shear stresses and flushing potentials, each of which is influenced by the hydrodynamic forces of tides and waves during various meteorological conditions.

Net current circulation patterns are influenced mostly by a combination of tidal and wave action. Consequently, greater emphasis has been placed on “tide and day to day wave” model results over “tide only” results, though both provide very similar results.

Large scale circulation results in Cleveland Bay have been extracted from the “C” class model, which offers the highest resolution model covering the whole area. With reference to drogue studies, previous reports (including GHD 2003) reported a distinct tidal circulation/oscillation pattern in Cleveland Bay which presents itself with an anticlockwise rotation on a local (drogue) scale. This is distinct from the large scale clockwise rotation, as indicated by residual current plots. Current magnitudes vary between 0 – 0.5 m/s with the largest currents produced during peak flood and ebb flows, particularly along Magnetic Island (maps of current vectors provided in Appendix I, with current patterns in the marine precinct presented in the same report.

Figure 3-30 illustrates the predicted residual currents (currents averaged for the semidiurnal M2 tidal constituent over the entire model simulation time) for existing conditions (i.e. no breakwater or marina), though the proposed marine precinct and breakwater layout are indicated as a background layer. A well defined eddy adjacent to the existing port reclamation is the most prominent feature. This eddy effectively covers the area that would be protected by the proposed breakwater. There are also two smaller clockwise eddies within the Marine Precinct /

Ross River mouth area, which may have arisen owing to the existence of the main channel between the Ross River mouth and Cleveland Bay. The channel is flanked by shallow regions, resulting in the residual patterns observed.

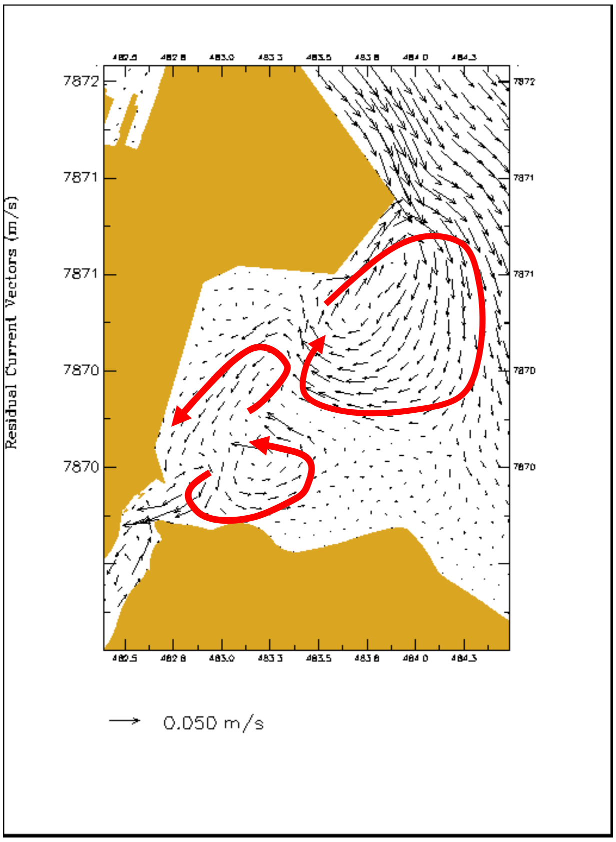


Figure 3-30 Residual currents resulting from tide only forcing for existing conditions.

3.8.4.13 Existing conditions – bed shear stresses

In order to better present the variation of bed shear stress that occurs during different tidal cycles, a series of time histories (i.e. bed shear stress v time) are presented for the “tide and prevailing waves” scenario. As previously indicated, the significant wave height applied at the model boundary for this scenario is 0.7 m. Given the relatively low wave height, it will be seen that the results for the tide plus waves scenario are similar to those for the tide only case.

Result plots have been generated in two primary formats: x-y plots of shear stress vs time at selected locations, and spatial plots of shear stress over the entire area of interest at specific times. Time history format results have been generated for nine sites, the locations of which are illustrated in

Figure 3-31. These sites were selected in order to cover a range of different exposures, and include the channel, inside and outside of the breakwater and the shallower area towards the inter-tidal flats. The full set of results is provided in Appendix I.

Figure 3-31 Location of numerical monitoring stations

Results have also been prepared in spatial format, as illustrated in

Figure 3-32. The plot illustrates bed shear stresses generated during the spring tide of 28th August 2008, during the peak flood stage of the tide. It is evident from

Figure 3-32 that maximum shear stress values of 1.50 to 1.7 N/m^2 are seen at the Ross River entrance while values of up to 2.2 N/m^2 are indicated in the sand shoal to the east of the river entrance. However, this localised peak, which is also seen for the developed condition case, is regarded as a function of shallow bathymetry as represented in the models, and is unlikely to be this high in reality. Elsewhere within the Precinct, BSS values are typically less than 1.0 N/m^2.

Figure 3-32 Spatial Plot of Bed Shear Stresses during Spring Tide

3.8.4.14 Flushing characteristics

The consideration of flushing time (potential) is undertaken in order to consider whether a water body is at risk of poor water quality. The technique involves the simulation of a passive tracer, with flushing time calculated using the e-folding technique. The e-folding time is the time required for the tracer to reach a concentration of $(1/e)\times C_o$, where C_o is the initial concentration of the tracer. When considering flushing time assessments, it is important to understand that the bigger the body of water, the longer the flushing time will be. Hence, the definition of whether flushing characteristics are good or bad must be determined in conjunction with the consideration of the size of the water body, and an appreciation of water quality measurements.

For the existing (undeveloped) conditions, flushing times were modelled as 1 day throughout the area of interest. A comparison of flushing times between existing and developed conditions is provided in the following section.

3.8.4.15 Ross River flood events

The Ross River is highly regulated, with the Ross River Dam and several weirs constructed. This provides a mitigated pattern of flood flows, with the river discharging into Cleveland Bay in the general vicinity of the proposed marine precinct. It is noted that for flood events occurring at low tide, the flood will tend to be contained largely within the existing channel, with shallow sandbanks to the north-east of the river mouth acting as a constraint.

3.8.5 Potential impacts and mitigation measures

3.8.5.1 Hydrodynamic circulation

In the developed case, the predominant flow exchange occurs through the breakwater entrance with the remaining exchange happening at the tail (southern end) of the eastern breakwater. Tidal current magnitudes are of the order of 0 to 0.35 m/s at the site of the breakwater entrance for the existing case with 0 to 0.60 m/s currents predicted for the developed case. At the tail of the eastern breakwater, existing currents vary between 0 and 0.35 m/s compared to the developed case which shows current magnitudes in the range of 0 to 0.55 m/s. Flows into the Ross River mouth and adjacent channel appear slightly weaker than for existing conditions.

A two day current magnitude time series is presented in Figure 3-33 for the two breakwater related locations discussed above. The figure shows that whilst predicted (modelled) shear stresses increase at both ends of the breakwater, these increases are not large for normal (i.e. tide only, or tide plus prevailing waves) conditions.

Figure 3-34 shows residual currents (currents averaged for the semidiurnal M2 tidal constituent over the entire model simulation time) for developed conditions. When breakwaters and Precinct structures are introduced to create the developed case, the eddy previously seen in the existing case (refer Figure 3-34) is enhanced due to "funnelling" effects while the smaller eddies in the Precinct are reduced.

Time series of water levels at three different locations within the study area reveal that for prevailing conditions, there is no change to water levels when comparing the existing and developed case results. These results (presented as Figure 19 in Appendix I) relate to three sites; namely (a) the mouth of the Ross River, (b) the entrance to the marina, and (c) the channel entrance to the breakwater.

Figure 3-33 Current magnitude (tide only case) at three locations (proposed marina entrance, tail of eastern breakwater and in navigation channel at breakwater entrance)

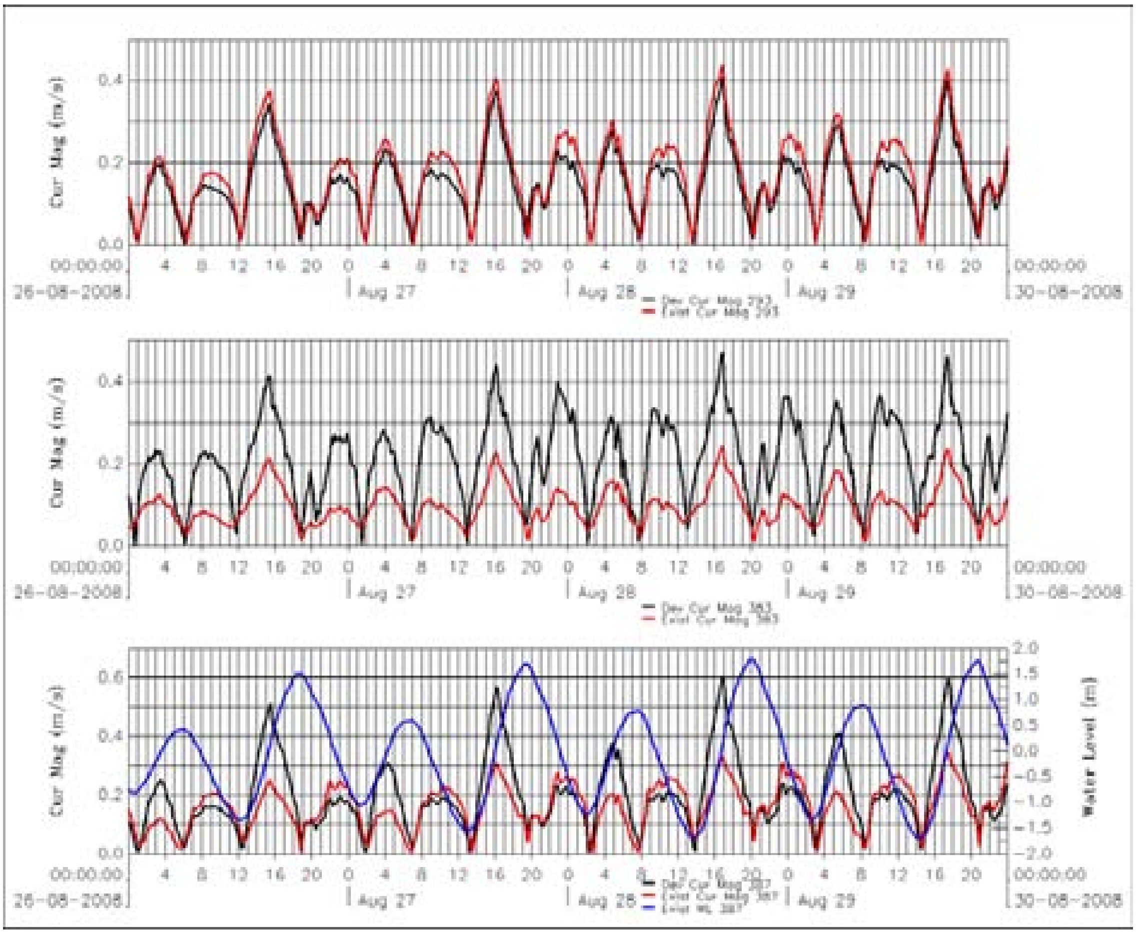
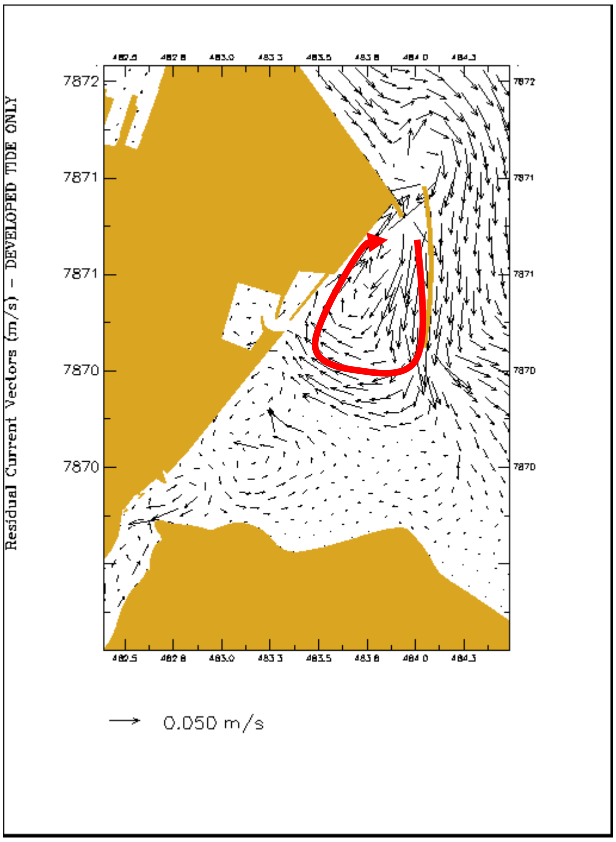


Figure 3-34 Residual currents as a result of tidal forcing on the developed bathymetry and structures



3.8.5.2 Bed shear stresses

Bed shear stress plots can be used to assess the potential for sediment erosion or deposition, with differential plots (i.e. the difference between existing and developed condition bed shear stresses) used to assess potential impact. Differentials in this context are values of a parameter in developed conditions minus the values of the same parameter in existing conditions and in applications concerning bed shear stresses (BSS), areas of potentially increased BSS are presented as positive values while potentially reduced BSS are indicated by negative values.

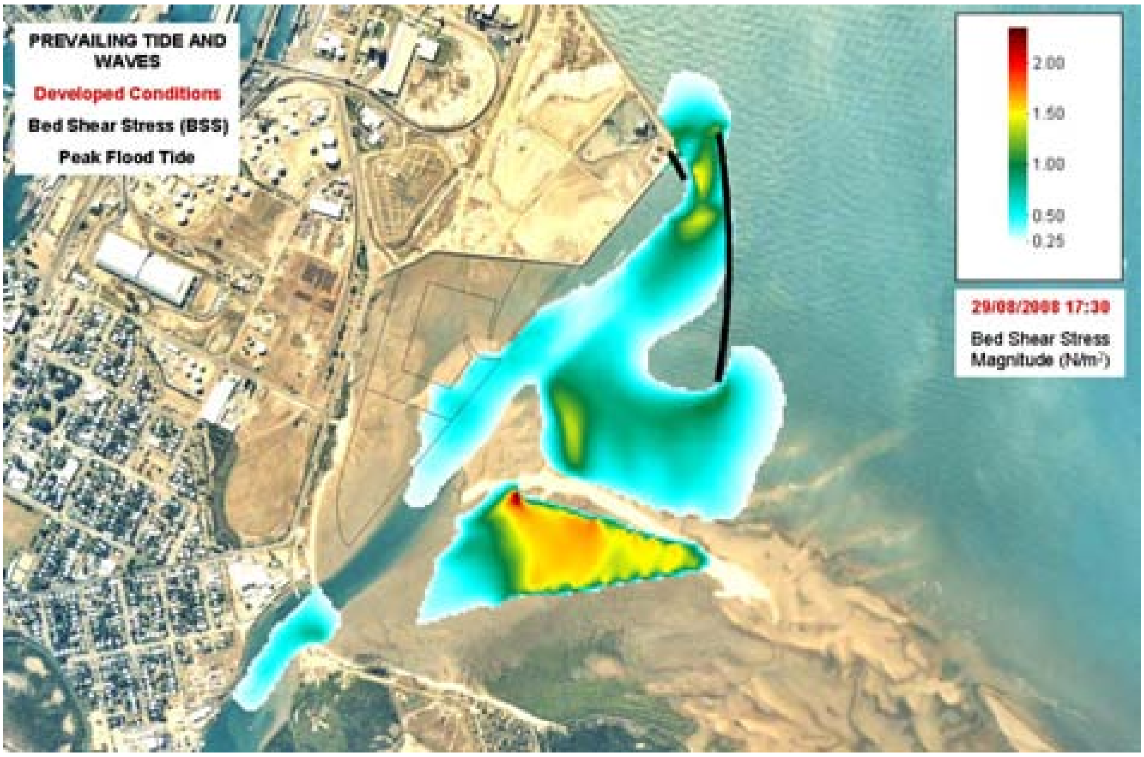
Whether erosion or deposition occurs is dependent on the threshold value of shear stress that applies. Based on GHD's previous work (GHD 2001, GHD 2003) in this area, a threshold for erosion of 1 N/m^2 was identified. Hence, values exceeding 1 N/m^2 may see either erosion or the resuspension of previously deposited material.

Values falling below the above thresholds can indicate a potential for deposition. Where a maximum value changes, but remains on the same side of the threshold (i.e. above or below the threshold) then the impact is related more to the time for which a threshold is exceeded.

Predicted bed shear stresses for peak flood flow during a spring tide on the 29th August 2008 is presented following. The results relate to the tide plus prevailing waves scenario.

In Figure 3-35, it can be seen that the breakwater causes two key changes. Shear stresses increase at the tail of the eastern breakwater to values of about 1 N/m^2 and also in the breakwater entrance with values as high as 1.7 N/m^2. The implications are limited to minor changes in erosional and depositional characteristics for a short period of time. Bed shear stress over the shallow flats are similar to the existing case.

Figure 3-35 Bed shear stresses during flood phase of spring tide (prevailing scenario) for developed conditions.



Additional results are provided in

Figure 3-36 and Figure 3-37.

Figure 3-36 provides a map of shear stress differentials (i.e. developed stresses less existing stresses for a given instant in time). These results, which are very similar to those for the tide only scenario, support the findings stated above. That is, changes in shear stresses are relatively small, being typically less than 1.25 N/m^2. The main changes occur at the ends of the proposed breakwaters, which will have minor implications for breakwater design. From an environmental impact perspective, there will only be minor changes in erosional and depositional characteristics for these conditions.

Figure 3-36 Differential of bed shear stresses during flood phase of spring tide (prevailing scenario) for developed conditions.

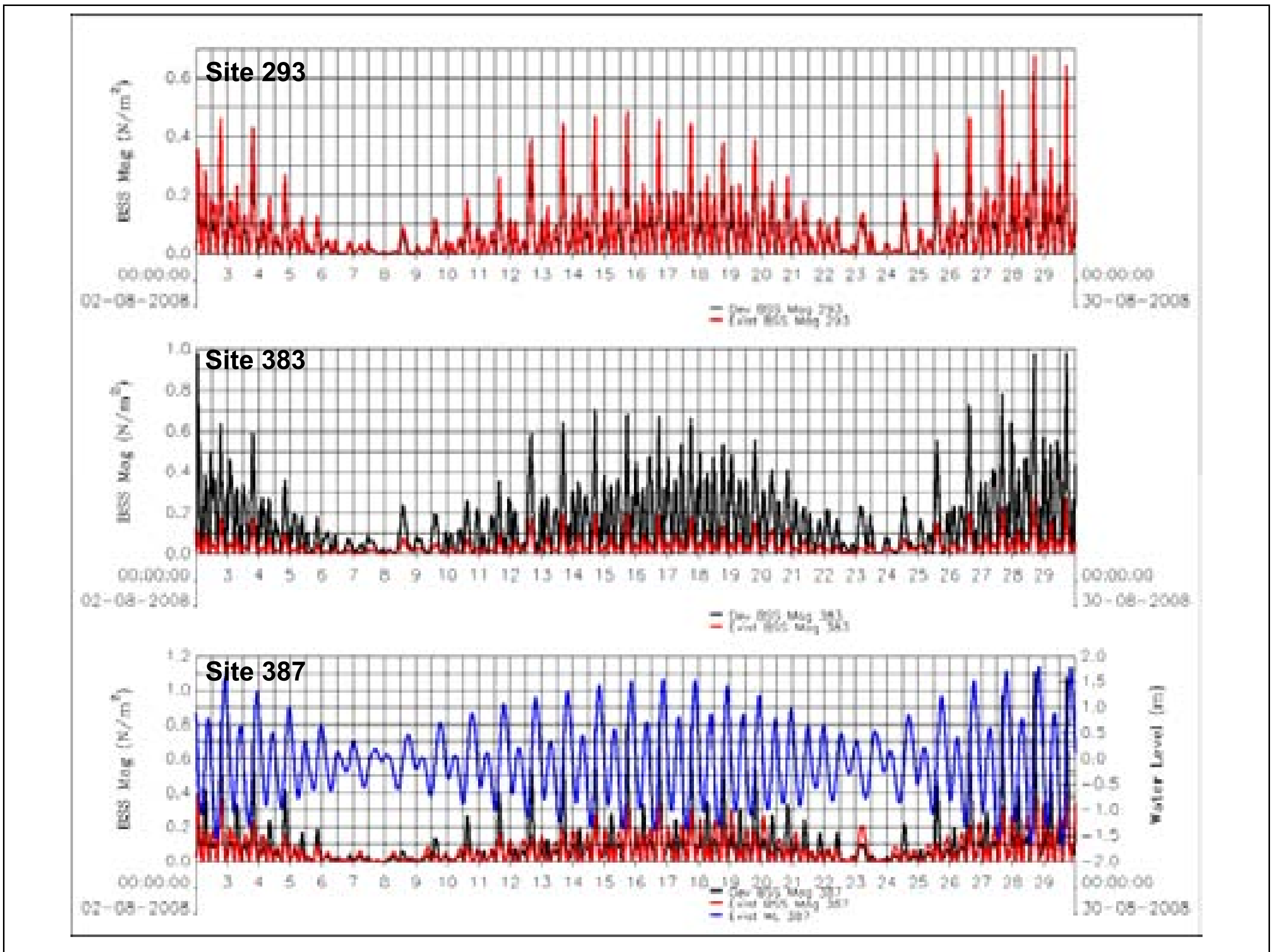
The spatial plots presented above provide information for single instants in time only. Time history plots therefore also of value, providing an enhanced understanding of the variable nature of shear stresses. Figure 3-37 provides time history results for three locations, station 293 (entrance of the proposed marina), station 383 (immediately south of the main breakwater), and station 387, located at the entrance of the two breakwaters (refer

Figure 3-31 for locations).

The generation of higher shear stress values at station 383 and 387 is evident, pushing peak values to 1.1 N/m^2 during spring tides. However, during neap tide periods (6th to 10th and 22nd to 26th August), shear stress values remain very low.

Bed shear stress values at station 417 are reduced for developed conditions with values reduced to half (from 0.70 N/m^2). Results for other locations are presented in Section 6.3 of Appendix I.

Figure 3-37 Time histories of bed shear stress at numerical monitoring stations 293, 383 and 387 for existing (red) and developed (black) conditions under the combined effects of tide and prevailing wave conditions



3.8.5.3 Bed Shear stresses associated with 1yr ARI Storm Event

The impact on bed shear stresses for the tide and 1yr ARI storm case follows a similar pattern to that determined for the “tide plus prevailing waves” case. As indicated in Figure 3-38, one year ARI storm waves result in bed shear stresses in excess of 2 N/m^2 at the mouth of Ross River for existing conditions. Shear stresses of about 0.7 N/m^2 are also observed eastern end of the sand shoal, a region where stresses were insignificant for prevailing conditions. With reference to Appendix I, it is also noted that:

- With the addition of the proposed breakwaters, current magnitudes are enhanced, with values of 0.45 m/s occurring at the tail of the eastern breakwater, compared to existing condition values of 0.25 m/s; and
- Current magnitudes at the tail of the eastern breakwater are increased from 0.3 to 0.6 m/s.

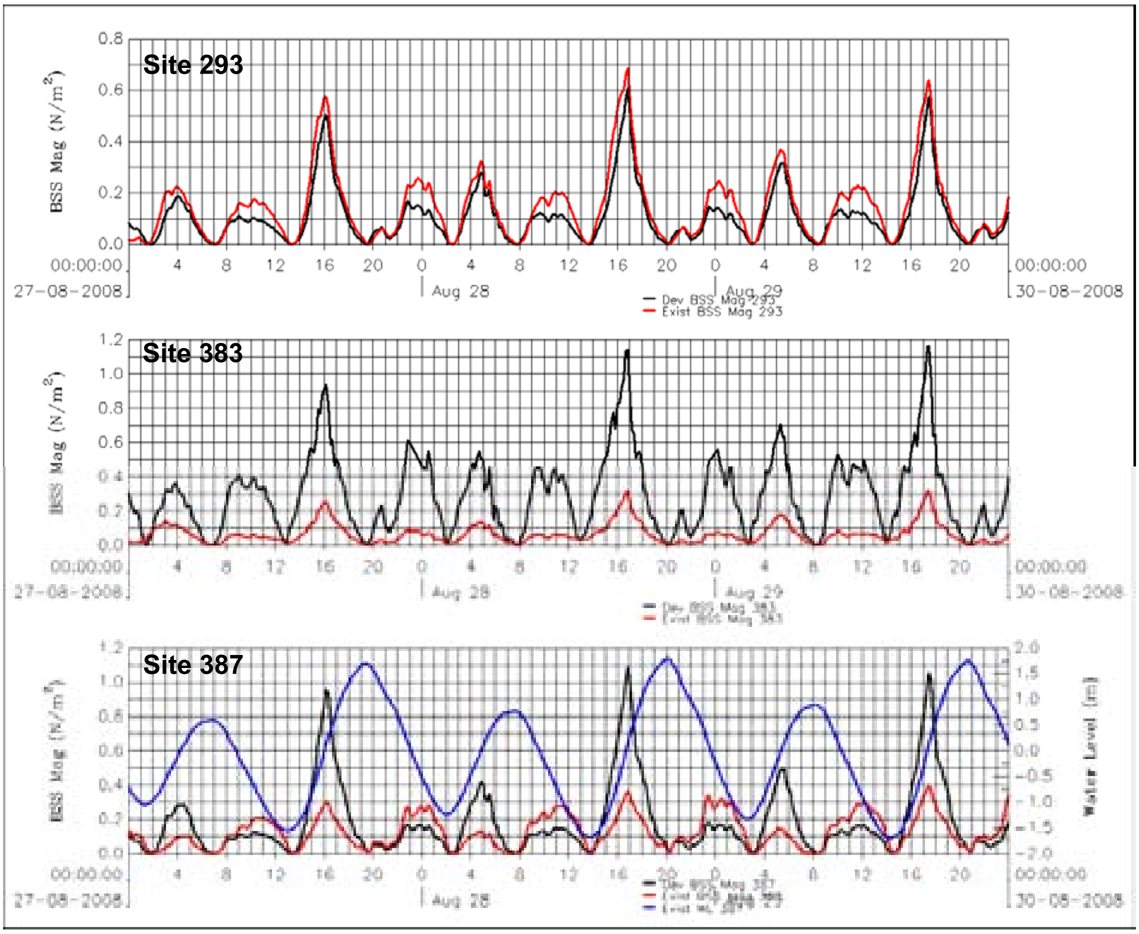
Figure 3-38 Existing bed shear stresses for storm wave (1 year ARI) conditions.

Figure 3-39 Differentials of BSS for storm wave (1 year ARI) conditions.

Figure 3-39 provides a differential plot for the storm wave scenario. The simulation was run for a spring tide period at the end of August 2008. Note that values in this figure are slightly lower than those for tide plus prevailing wave conditions as the map represents a time 30 minutes after flood tide has peaked (since results for this model run were written in two hour intervals). In reality, BSS values would be slightly higher than that observed for tide and prevailing wave conditions, as reflected in time histories. The breakwater also provides protection, and hence both the marina and navigation channels show little change in BSS with limited potential for any significant risk for erosion or siltation in this region during the simulated storm wave condition.

Results are also presented in time history format. Figure 3-40 provides a comparison of bed shear stresses for existing and developed conditions at three stations, one of which (station 383) is located at the southern end of the proposed breakwater. Monitoring station locations were previously provided in Figure 3-31.

Figure 3-40 Time histories of bed shear stress at numerical monitoring stations 314, 383 and 417 under the combined effects of tide and 1 year ARI storm event: Developed conditions (black solid line) versus existing conditions (red solid line)



Bed stresses for station 387 (located between the two breakwaters) have increased in comparison to the prevailing wave conditions (now exceeding 1.0 N/m^2 for the developed case compared to 0.9 N/m^2 previously), whist the difference between existing and developed is only of the order of 0.7 N/m^2 at peak flood tide.

Shear stresses for station 383 (southern end of breakwater) indicate a significant (percentage wise) increase associated with the developed scenario (1.2 N/m^2 compared to 0.3 N/m^2 for existing case). Station 383 shows the largest post-development increase in bed shear stress compared to the other two numerical stations as it is relatively more exposed to wave climate than station 293 or 387.

Values of bed shear stress at station 293 remain relatively unchanged between existing and developed cases.

3.8.5.4 Flushing characteristics

Table 3-36 provides a summary of the e-folding or flushing time at six locations (as indicated in Figure 12 of the Appendix I) for both existing and developed conditions. Results are provided for both the "tide only" and "tide plus prevailing wave" scenarios. Results indicate that once the marina and breakwater are constructed, it could take up to 50% longer for pollutants to leave the Precinct. However, while all the locations studied showed an increase in flushing times, the differences are not significantly high, given that the marina would tend to flush approximately 63% of the contaminant(s) in approximately 1.6 days under normal tidal and wave driven circulation. There is minimal difference for flushing times when comparing the tide only case with the tide plus waves case.

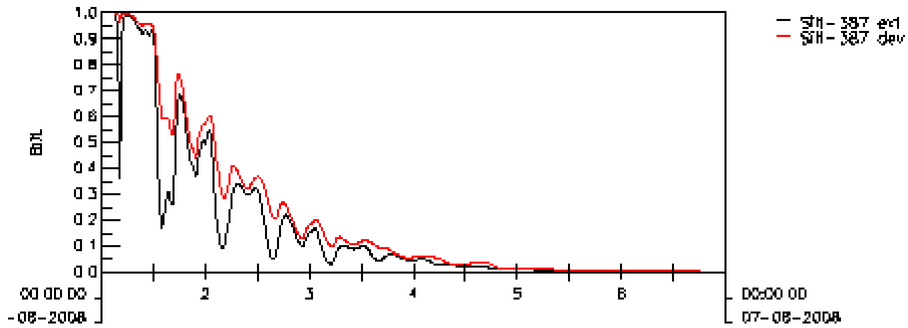
Table 3-36 Flushing Potential for a Passive Tracer Using E-Folding Technique

Location	Flushing Potential (days)			
	Tide and Wave Conditions		Tide Only Conditions	
	Existing	Developed	Existing	Developed
1 – inside marina	1.09	1.53	1.09	1.60
2 – inside marina	0.99	1.44	1.06	1.55
3 – channel adjacent to marina entrance	0.85	1.28	0.85	1.28
4 – channel between breakwaters	1.03	1.38	1.03	1.37
5 – river mouth at site of proposed access road	0.92	0.93	0.92	0.93
6 – N of proposed breakwater outside Marina Precinct.	1.03	1.22	1.03	1.22

The means of determining the flushing time is illustrated in Figure 3-41. The plot indicates the decay of dye over time, commencing with a concentration of 1 kg/m^3. As flushing with “clean” water occurs (i.e. water with no dye), the concentration at the point of interest decreases. The above plot provides results for Station 387, the location of which is described earlier. Reference can also be made to

Figure 3-31.

Figure 3-41 Passive tracer concentration time series at Ross River mouth/entrance (top panel), breakwater entrance/channel (middle panel) and proposed marina site (bottom panel)



3.8.5.5 Ross River Flooding

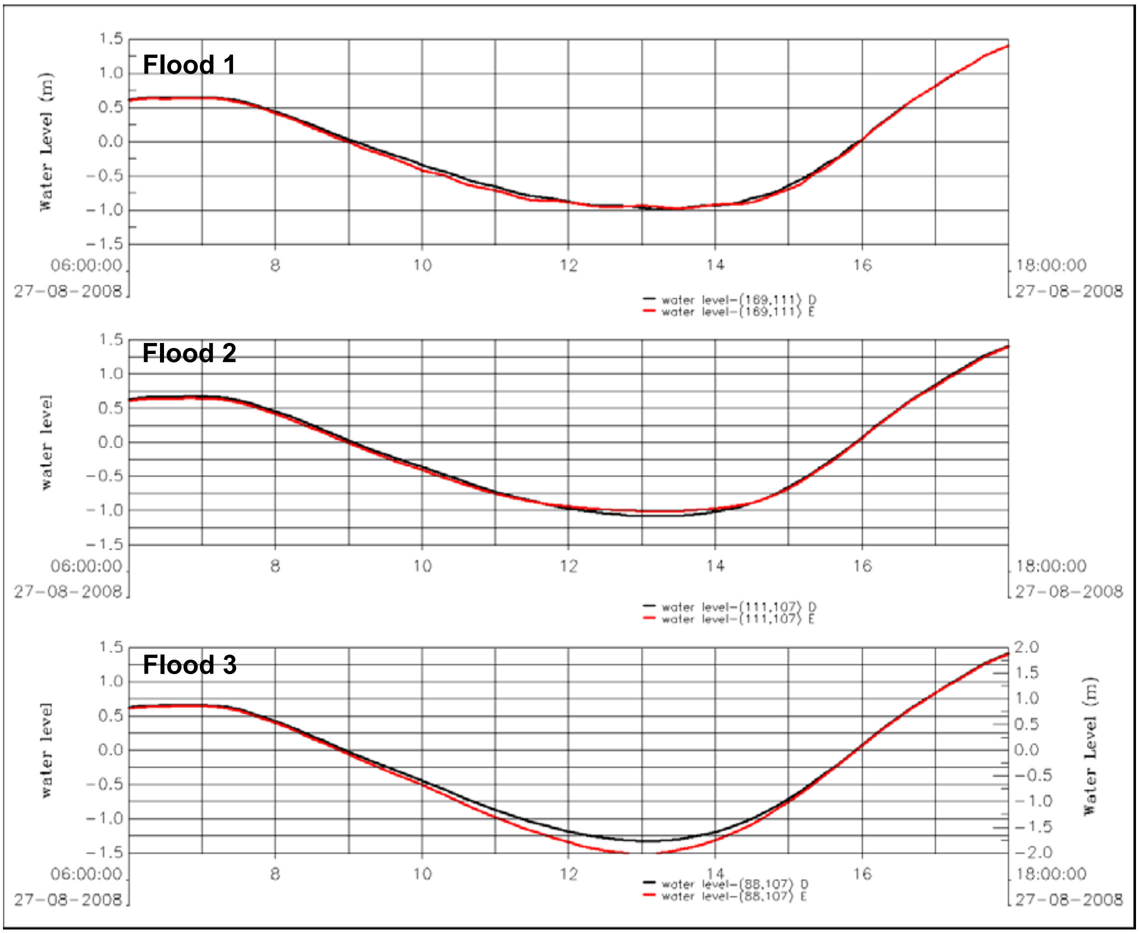
With the Ross River discharging directly into the Precinct area, it is necessary to consider whether there are any potential implications for flooding, and in particular, to assess the potential for impacts on upstream flood levels. Modelling studies to investigate the cumulative impacts of the TPAR and Precinct are being conducted under TPAR studies. Information available at the time of reporting for this EIS has been assessed and potential influence of the Precinct on flooding is considered following. Additional comments are also provided in the Cumulative Impacts section of this document (refer Section 3.17).

A flood event with a peak discharge of 1090 m^3/s, has been selected for the assessment. This is nominally equivalent to a major (e.g. 1 in 100 yr ARI) event. Results are presented in terms of changes to water levels, both within the Ross River and throughout the marine precinct area, and also with respect to changes in predicted bed shear stresses (representing erosion and sedimentation characteristics).

Results relating to current magnitude and direction are also presented in Appendix I, with a comparison of currents for the existing and developed cases over a period spanning peak river flood. For the existing case, flow patterns tend to follow the channel leading from the Ross River into the Cleveland Bay. For the developed case, there is a strong branching (separating) flow between the breakwaters and also along the tail of the eastern breakwater. The flow between the two breakwaters produces large currents between 2 to 3 m/s.

Figure 3-42 provides an indication of predicted water levels as a result of the proposed development at three locations.

Figure 3-42 Water level (m) at Ross River mouth/entrance (top panel), proposed marina site (middle panel) and breakwater entrance/channel (bottom panel).

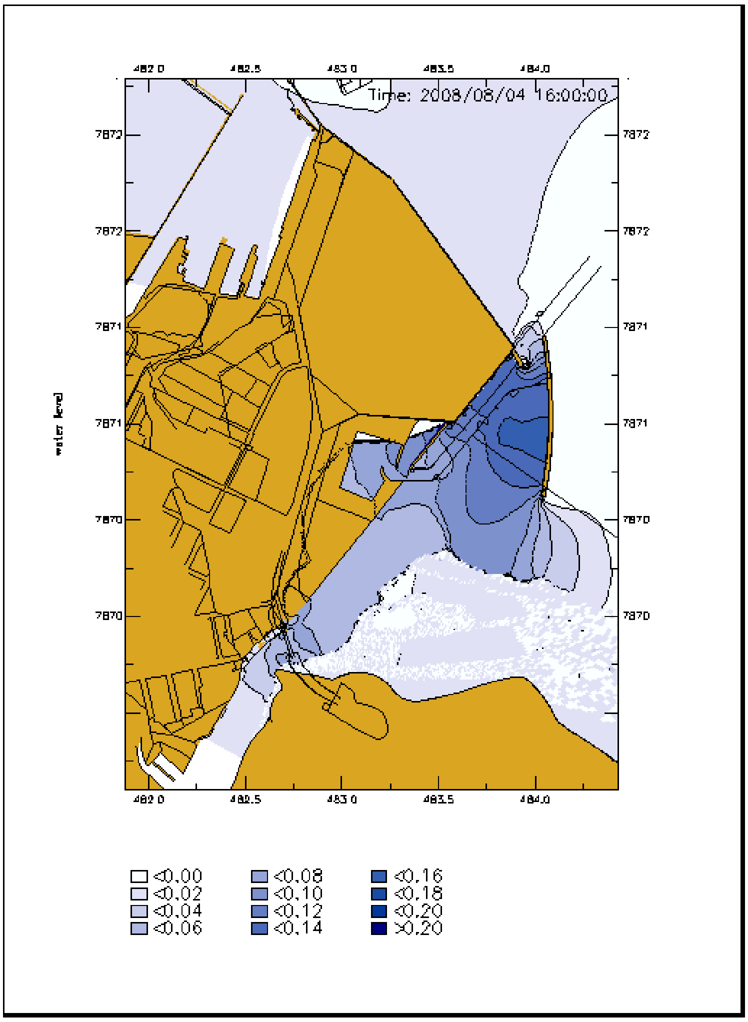


The top panel in the figure indicates that the water level is not affected in the Ross River entrance, with Station Flood 1 (flood numerical monitoring stations shown in

Figure 3-31) located mid channel under the proposed access road. Similarly, only small increases (0.10 m) are seen at the proposed Marina site (refer middle panel of Figure 3-42). Larger differences are observed in the region between the marina and the breakwater, with water level elevation differences of up to 0.25 m indicated adjacent to the breakwater entrance, though it is important to note that these occur at low tide, and hence do not affect the peak flood level.

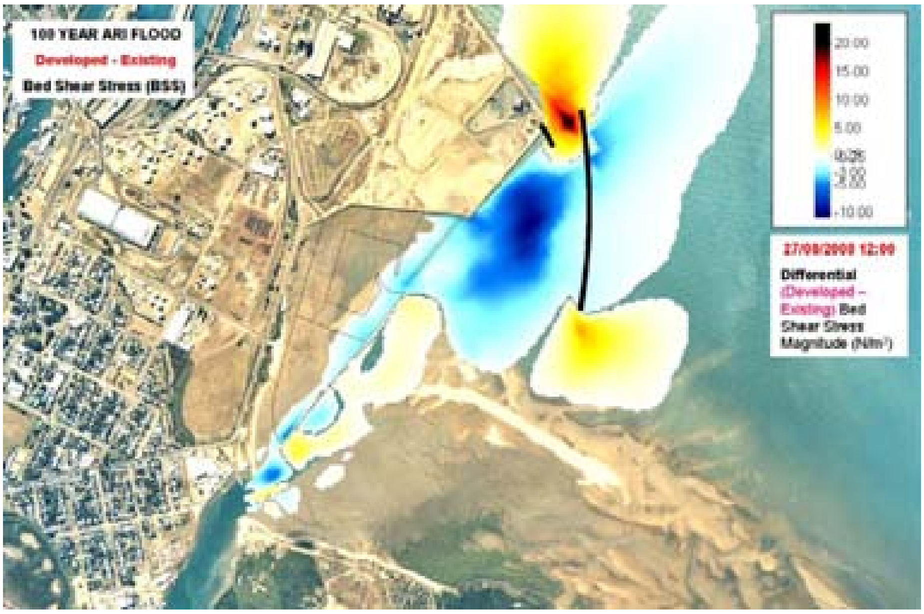
Maximum differences of up to 0.25 m are observed behind the breakwater as indicated in Figure 3-43. This map provides results for the time when differentials are a maximum, though again it is noted this would occur at low tide, and not at the time of maximum water level.

Figure 3-43 Map of differential water level for 100 year flood event in Ross River.



The potential impact on shear stresses (Figure 3-44) is that the channel between the two breakwaters is likely to scour during a flood event for both existing and developed conditions. The impact of the development is observed to be confined largely to three locations: large increases at the entrance of the breakwaters and at the tail of the eastern breakwater (values in excess of 15 N/m^2 for the first location and 5 N/m^2 for the latter) and decreases behind the breakwater (5 – 10 N/m^2). These locations will need careful consideration during design.

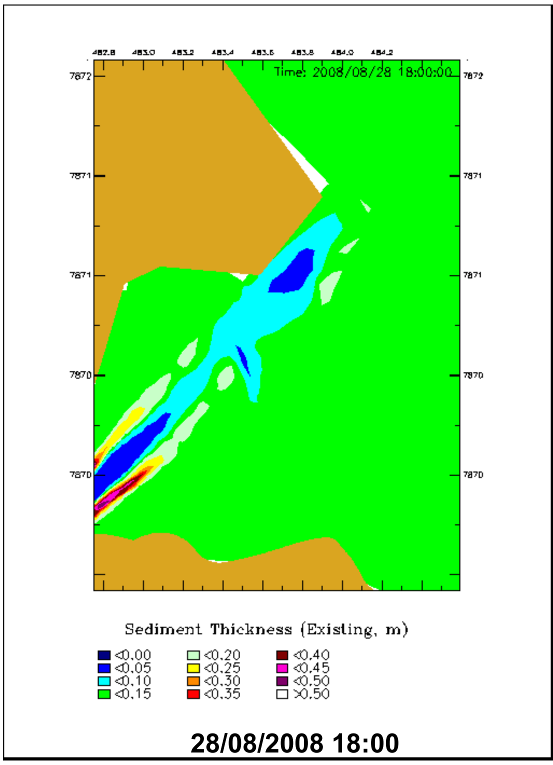
Figure 3-44 BSS differential for peak flood flow through the entrance of the Ross River.



Sedimentation and Erosion Potential for River Flooding

Sediment deposition and erosion potential has been assessed for 100 year ARI flood conditions. With a sediment load of 500 mg/L and erosion threshold of 1 N/m^2 and deposition of 0.25 N/m^2, sediment deposition of the order of 0.5 m occurs on either side of the navigation channel while erosion is seen in the channel itself. A similar pattern is seen for developed conditions at the mouth of the river. In this case, sediment appears to be completely eroded within the breakwater entrance while sediment deposition of 0.35 m is observed at the entrance of the marina (Figure 3-45). A sediment thickness differential plot (Figure 3-46) shows that there is potential for slight scouring at the tail end of the eastern breakwater and in the channel between the breakwaters. Indication of sediment deposition at the mouth of the marina is evident, suggesting maintenance issue needs to be addressed in detail design. The sand shoal and mangrove flats appear to be effectively unaffected in terms of sediment deposition or erosion.

Figure 3-45 Existing (left) and developed (right) sediment thickness following flood event in Ross River.



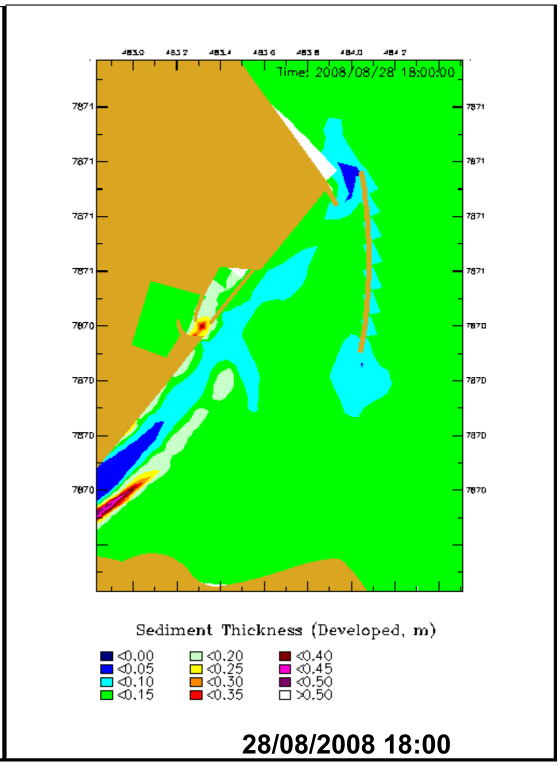
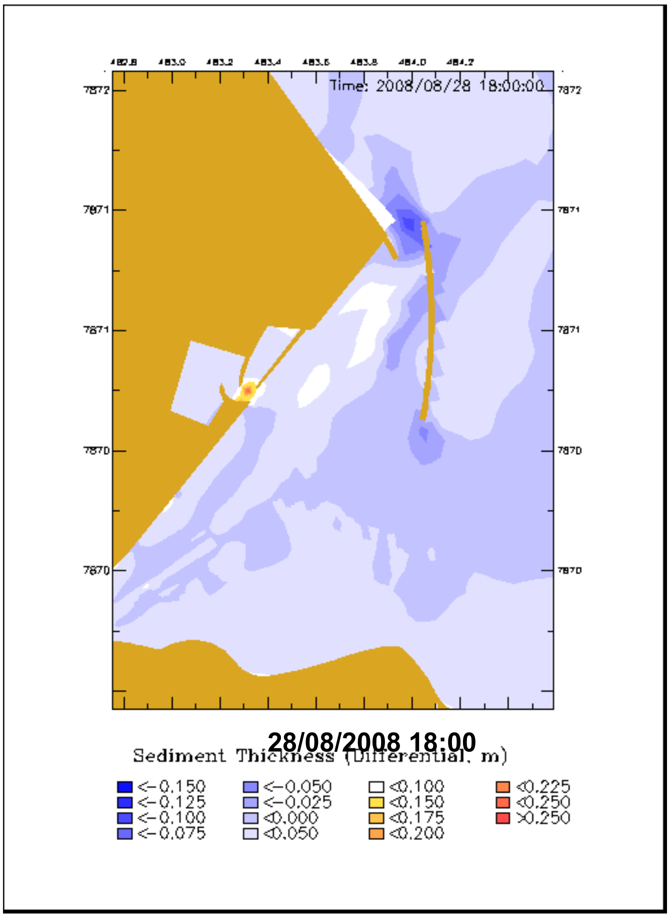
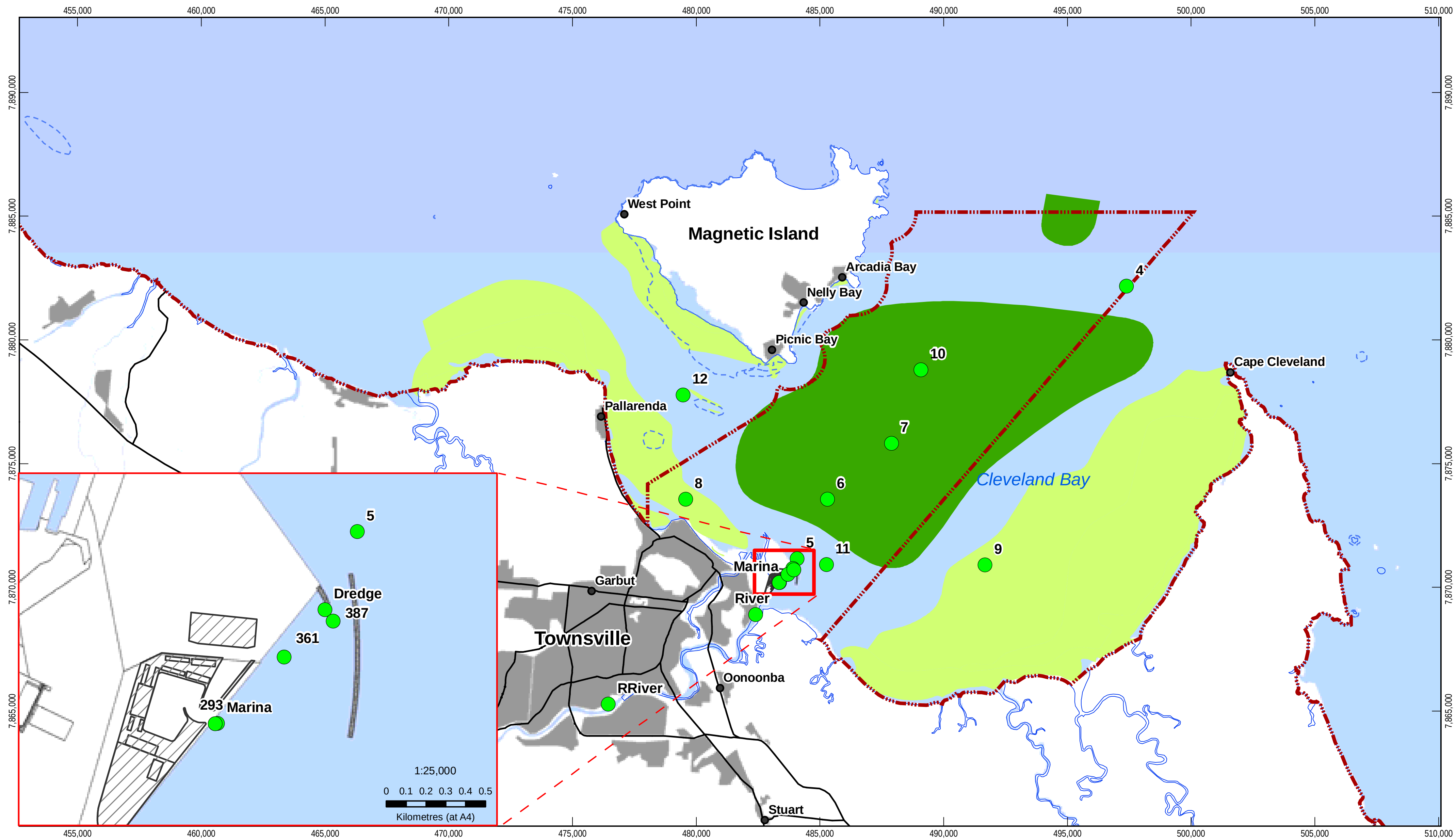


Figure 3-46 Comparison of sediment deposition from flood events for existing and developed conditions.



3.8.5.6 Dredge plumes

Sediment transport modelling has been undertaken, coupled with tides and prevailing wave conditions for a period of two months. Monitoring/observation stations have been set up in the model to cover the entire area of interest, particularly areas where coastal and deep water seagrasses grow (refer Figure 3-47). Details pertaining to the establishment of the sediment model are provided in Appendix I. It is important to note that all results represent a plume with no background concentrations, as this allows the shape and concentration of the plume to be easily identified. When considering potential impacts, the nominated background (median) concentration of 80 mg/L should be added.



1:200,000

0 1 2 3 4 5

Kilometres (at A4)

Map Projection: Transverse Mercator
Horizontal Datum: Geocentric Datum of Australia 1994
Grid: Map Grid of Australia, Zone 55

N

LEGEND

- Key Numerical Monitoring Stations
- Major Road
- Marine Precinct Stages 20081201
- GBRMP Boundary
- Coastal Seagrass Meadows 2007
- Deepwater Seagrass Meadows 2007

Port of Townsville
Marine Precinct EIS

Job Number	42-15399
Revision	B
Date	01 July 2009

Environmentally Sensitive
Areas for Seagrass.

Figure 3-47

G:\42\15399\GIS\Projects\PDFs\42-15399_806_rev_b.pdf

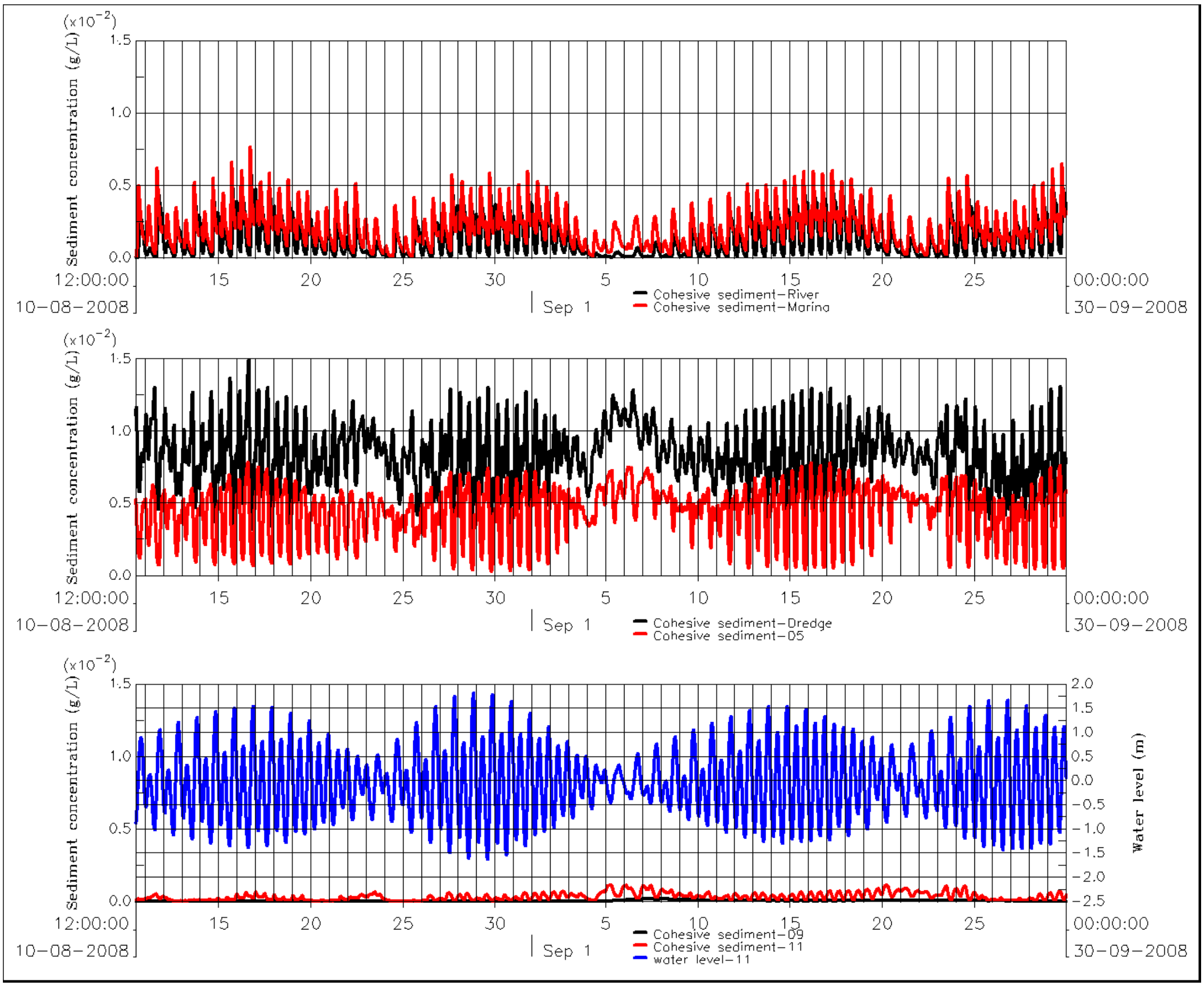
201 Charlotte Street Brisbane QLD 4000 Australia 61 7 3316 3000 61 7 3316 3333 bnemail@ghd.com.au www.ghd.com.au

Time series at several locations (defined in Figure 3-47) are plotted in Figure 3-48, with the spatial variation of the dredge plume during a spring tide presented in Appendix I. The time histories provide an understanding of peak suspended sediment concentrations, and of the variable nature of plumes, whilst the spatial plots demonstrate the full extent of the plume, for the modelled conditions.

Time history plots are presented with units of g/L x 10^{-2}. Hence, a value of 0.5 on the left axis is equivalent to 5 mg/L, 1 equates to 10 mg/L and so on. Where units are presented as g/L x 10^{-3}, the conversion is linear (i.e. a value of 10×10^{-3} g/L on the left axis equates to 10 mg/L.

As with all sediment modelling, values should be regarded as indicative rather than absolute. Actual values can change subject to type of equipment used, variable conditions, and in particular, significant wind and wave events. The plotted extent is similarly indicative.

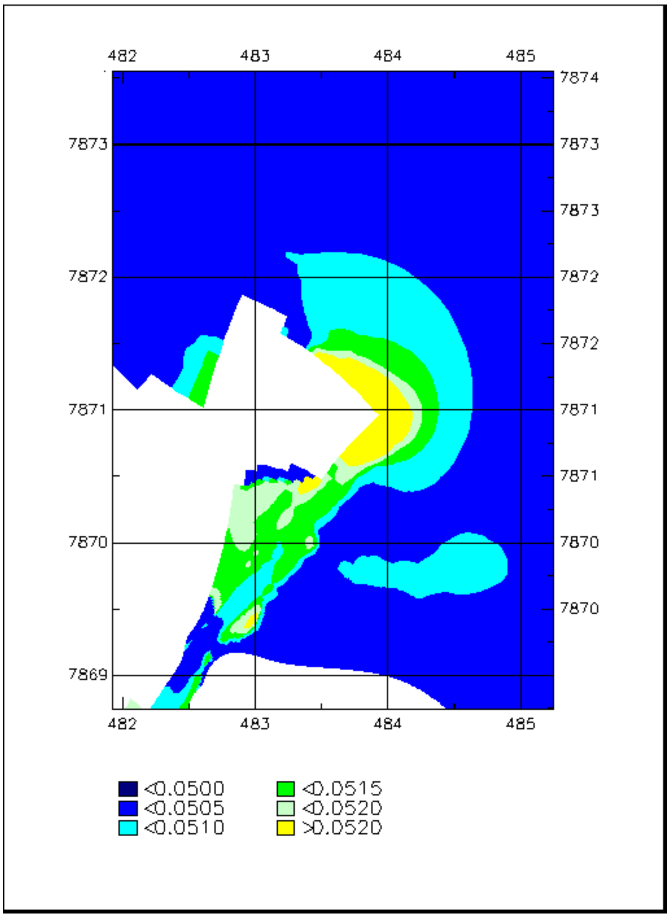
Figure 3-48 Suspended sediment concentration time series at key monitoring stations



With reference to the above figure, it is evident that suspended sediment concentrations associated with the plume are predicted to be relatively low, with a peak of 15 to 20 mg/L indicated in the middle panel.

With a median background concentration of 80 mg/L indicated in Section 3.9, this value is low. Even if doubled, there would be little impact evident in the form of increased turbidity.

Figure 3-49 Predicted sediment deposition for 2 months of dredging



An indication of the areas potentially subjected to sediment deposition is provided in Figure 3-49. The results relate to the 2 month period of dredging, and hence would need to be factored in accordance with the actual time of dredging. For example, if dredging is considered likely for a 6 month period, then the predicted sediment depths would need to be multiplied by a factor of 3.

The starting depth in the model is 0.05m (5cm), in that a layer of material 5cm in depth is assumed to exist prior to dredging. Hence, the main point of interest is to identify the depth of dredged material exceeding 0.05m.

By way of example, the green colour in Figure 3-49 denotes <0.0515m. That is, the estimated depth of deposited material for the 2 month period is 0.0015m, or less than 1.5mm. Yellow is therefore >2mm (and potentially up to 3mm). On this basis, and recognising that the period of simulation is 2 months, the total sediment depth could be 4 to 6 mm in the yellow area if dredging were to occur for a 4 month period, or 6 to 9mm for dredging over a 6 month period. This estimate is conservative, in that it does not allow for the resuspension and transport of deposited material during storm events that might occur within the 6 month period.

An additional observation relates to the boundary conditions of the model, and their affect on results. The dredge plume run is driven by a combination of tide plus 0.71m waves coming from just north of east. Hence, deposition patterns are more likely to be pushed to the west. Under different wave conditions, it is therefore possible that some of the material might stay (deposit) more to the east of where indicated by Figure 3-49.

The results of sediment transport modelling illustrate that the relatively low sediment loading from the proposed dredging works is unlikely to generate a plume of either significant concentration or extent. Modelling undertaken (driven by tide with 0.7m waves) indicates the spatial scale of the sediment plume is confined to a local scale of a few hundred meters, with maximum concentrations of the order of 20 mg/l close to the sediment source. The plume is not predicted to extend over any environmentally sensitive areas, other than at low concentrations, which lie well within the natural variation in turbidity.

The spatial plot does suggest a net transport to the northwest, though this is due in part to the wave conditions that drive the model. This is confirmed by consideration of time series data at sites 8 and 12, both of which show an increase in sediment concentration over the model duration. It would therefore be reasonable to expect that a differing wave conditions might result in a plume extending further to the east (i.e. into Cleveland Bay), but the concentrations would remain low in comparison to naturally occurring levels.

With median background turbidity measured at 80 mg/l, there does not appear to be any significant potential impacts associated with the dredge plume. Furthermore, it is noted that the 95^{th} percentile value is over 100 NTU (or over 350 mg/L), and hence the addition of the background value of 80 mg/L to the predicted concentrations arising from the plume is not likely to lead to the 95^{th} percentile value being reached. This conclusion is unlikely to change unless a completely different dredging operation to that proposed occurs.

3.8.6 Mitigation of impacts

Coupled hydrodynamic, wave and sediment transport modelling was undertaken in order to describe the existing hydrodynamic characteristics of Cleveland Bay, and in order to assess potential impacts associated with the construction of the proposed marine precinct and associated breakwaters. The modelling exercise provides an understanding of general circulation patterns in Cleveland Bay (as driven by tide and waves) as well as informing details of circulation, sedimentation and flushing patterns in the vicinity of the proposed marina and breakwater development within the Precinct.

Predicted impacts are low, leading to a limited need for formal mitigation measures.

The following conclusions can be derived from this study.

- There is no significant impact on water levels as a result of the proposed development under the driving forces of tide and wave (both prevailing and 1 year storm wave) conditions. However an increase in water level of up to 0.25 m is observed behind the proposed eastern breakwater during 100 year floods in the Ross River, albeit that this increase occurs at low tide;
- Tidal current magnitudes are expected to be reduced significantly at the proposed Marina site while an increase in current between the breakwaters is predicted. This will lead to an

increased potential for sedimentation within the marina, which will need to be catered for in estimating ongoing maintenance requirements.

- Absolute values of shear stress appear to remain relatively low (i.e. less than the 1 N/m2 threshold for erosion) under the majority of conditions, with increases in bed shear typically less than 0.5 N/m2. However, during spring tide flood flows, bed shear values exceed 1.25 N/m2 with differentials as high as 1.0 N/m2.
- Under major river flood conditions, bed shear stresses could potentially increase by 5 – 20 N/m2 in the entrance and at the tail of the eastern breakwater. This imposes a risk of scour, which will need to be addressed during design.
- The flushing time for contaminants increases by approximately 12 hours (i.e. an increase of 35%) over the existing conditions for most sites within the Marine Precinct, including the proposed marina. This potential increase in flushing time is not like to have a high impact as most passive contaminants are flushed within 1.6 days, which is a relatively short time. No mitigation measures are recommended, other than ongoing monitoring of water quality.
- Dredge plume modelling was undertaken for a period of one month to assess the potential impacts of dredging in the navigation channel closest to the breakwater entrance. The sediment plume has maximum concentration of approximately 20 mg/l in the vicinity of the dredge source and extends a few hundred meters radially outwards. Management of the dredge program will require monitoring, as undertaken for similar programs. Given the low magnitude of predicted turbidity, the modelling suggests that measures such as silt curtains are unlikely, though use of one near the mouth of the Ross River should be considered.
- Depths of sediment deposition are estimated to be of the order of 2 to 3mm per 2 month period. Actual values will depend on ambient wind and wave conditions, the dredge used, and the amount of material in suspension during natural turbidity events, which have been measured at an order of magnitude higher than those predicted for the dredging activity. If dredging were to continue for a period of 6 months, then 6 to 9mm of material is predicted to settle.

3.9 Water and sediment quality

3.9.1 Description of environmental values

3.9.1.1 Overview

The TMPP is located in the tidally influenced river mouth of the Ross River. The mouth of the Ross River has been highly modified over the past 100 years, particularly with the development of urban areas and Port of Townsville facilities on the northern bank. Potential influences on water and sediment quality from the urban areas and Port operations include stormwater run off, accidental spills of hydrocarbons and other products and dust and spillage of bulk commodities that are imported and exported through the Port. Other impacts on water and sediment quality within the Project Area include inputs of heavy metals, hydrocarbons, pesticides and herbicides from catchment activities such as urbanisation, agriculture, Ross River Dam and the presence of light industry. The Ross River discharges into Cleveland Bay, which forms part of the Great Barrier Reef World Heritage Area. The Ross River is located