Transport for NSW

Beaches Link and Gore Hill Freeway Connection

Appendix P
Hydrodynamic and dredge plume modelling

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Technical working paper: Hydrodynamic and dredge plume modelling

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Prepared for

Transport for NSW

Prepared by

Royal Haskoning DHV

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Annexures

Glossary

2D	Two-dimensional
3D	Three-dimensional
ADCP	Acoustic Doppler Current Profiler
AHD	Australian Height Datum
BHD	Backhoe Dredge
BoM	Australian Bureau of Meteorology
CBD	Central Business District
C-Map	Commercially available source of digitised Admiralty navigational charts
FM	Flexible Mesh
HW	High Water
LW	Low Water
MIKE 21	Two-dimensional computer modelling software that can simulate flows, waves, sediments, ecology and water quality in rivers, lakes, estuaries, bays and open seas.
MIKE 3	Three-dimensional computer modelling software that can simulate flows, sediments, ecology and water quality in rivers, lakes, estuaries, bays and open seas.
MSL	Mean Sea Level
NTU	Nephelometric Turbidity Unit is a measure of the turbidity of water based on a measure of scattered light.
NSW	New South Wales
OEH	NSW Office of Environment and Heritage (former)
RHDHV	Haskoning Australia Pty Ltd, a company of Royal HaskoningDHV
RMSE	Root mean square error is a measure of difference between observed and predicted values.
Sigma layers	Equidistant depth layer in the hydrodynamic model
SSC	Suspended Sediment Concentration
The project	Beaches Link and Gore Hill Freeway Connection
Z-layers	Depth layers in the hydrodynamic model of constant spacing

Executive summary

This assessment details the findings of numerical modelling to better understand the potential impact of construction activities and operations related to the Beaches Link and Gore Hill Freeway Connection (the project) on the hydrodynamic and water quality of the marine environment.

To inform the assessment, available historical data has been reviewed and additional project specific data has been collected and used to inform a description of the existing environment. The project specific hydrodynamic data was then used to calibrate a three-dimensional (3D) hydrodynamic model that has been established for the project.

The 3D hydrodynamic model was used to assess potential hydrodynamic impacts and water quality impacts. The assessment of hydrodynamic impacts looked at the impacts during construction as well as operational impacts from the project. Water quality impacts are primarily related to the dredging required for the construction of the immersed tube tunnels.

The main outcomes of the hydrodynamic modelling impacts related to the two temporary construction phase cofferdams (Middle Harbour south cofferdam (BL7) and Middle Harbour north cofferdam (BL8)) and associated deep silt curtains, are:

- During the ebb tide current speeds would increase around the Middle Harbour north cofferdam (BL8) at all depths. At the Middle Harbour south cofferdam (BL7), current speeds would increase between the structure and the foreshore at Clive Park but only in the upper water column
- During the flood tide current speeds would decrease in areas surrounding both the Middle Harbour south cofferdam (BL7) and Middle Harbour north cofferdam (BL8), at all depths
- During both ebb and flood tide there would be small increases in current speeds in the middle of the channel
- Overall, these changes in current speeds during construction are unlikely to result in erosion or accretion of the bed of the harbour or adjacent foreshore.

The hydrodynamic impacts of the Spit West Reserve construction support site (BL9) were also assessed. The modelling indicated:

- During the ebb and flood tide currents speeds would be reduced along the foreshore surrounding the Spit West Reserve construction support site (BL9). The reductions in current speed are larger during the flood tide
- The changes in current speeds are not expected to result in erosion or accretion at the bed of the harbour or foreshore.

Modelling of the operational impacts of the immersed tube tunnels on hydrodynamics indicated:

- Changes in current speeds would be minimal. The most pronounced change would be increased current speeds at the northern bank (Seaforth) during the ebb and flood tide
- Changes in tidal water levels, tidal planes, tidal discharge at the project crossing and the tidal prism would be expected to be minimal
- Tidal flushing times would be slightly longer due to the addition of the sill-like feature created by the immersed tube tunnels; however flushing times would remain rapid.

The modelling of dredge plume related water quality impacts during the construction phase has shown the following:

- The extent of the plume of suspended sediment caused by dredging would be relatively small in comparison to the dimensions of the waterway
- Suspended sediment would be transported upstream and downstream of the project crossing, with a slight downstream dominance along the northern bank near Seaforth
- Suspended sediment levels would be higher at the bed of the harbour than at the water surface
- Suspended sediment levels would be highest inside the silt curtains, and generally low in areas outside the silt curtains
- The majority of the deposition due to the dredging activity would occur in the dredging footprint and adjacent to the dredging footprint. Areas of higher deposition would be concentrated in front of the Middle Harbour south cofferdam (BL7) and Middle Harbour north cofferdam (BL8).

A number of environmental management measures are proposed as part of dredging operations. These measures would reduce or avoid the release of suspended sediments during dredging (eg use of appropriate dredging equipment) and manage the suspended sediment that would be released (eg the use of silt curtains as floating barriers, suspended in the water to contain suspended sediment). These measures reflect best environmental practice to reduce the water quality impacts of dredging and would result in an overall reduction in the extent and intensity of the dredge plumes, which is reflected in the modelling results presented in this report.

1 Introduction

This section provides an overview of the Beaches Link and Gore Hill Freeway Connection (the project), including its key features and location. It also outlines the Secretary's environmental assessment requirements addressed in this technical working paper.

1.1 Overview

The Greater Sydney Commission's *Greater Sydney Region Plan – A Metropolis of Three Cities* (Greater Sydney Commission, 2018) proposes a vision of three cities where most residents have convenient and easy access to jobs, education and health facilities and services. In addition to this plan, and to accommodate for Sydney's future growth the NSW Government is implementing the *Future Transport Strategy 2056* (Transport for NSW, 2018), that sets the 40 year vision, directions and outcomes framework for customer mobility in NSW. The Western Harbour Tunnel and Beaches Link program of works is proposed to provide additional road network capacity across Sydney Harbour and Middle Harbour and to improve transport connectivity with Sydney's Northern Beaches. The Western Harbour Tunnel and Beaches Link program of works include:

- The Western Harbour Tunnel and Warringah Freeway Upgrade project which comprises a new tolled motorway tunnel connection across Sydney Harbour, and an upgrade of the Warringah Freeway to integrate the new motorway infrastructure with the existing road network and to connect to the Beaches Link and Gore Hill Freeway Connection project
- The Beaches Link and Gore Hill Freeway Connection project which comprises a new tolled motorway tunnel connection across Middle Harbour from the Warringah Freeway and the Gore Hill Freeway to Balgowlah and Killarney Heights and including the surface upgrade of the Wakehurst Parkway from Seaforth to Frenchs Forest and upgrade and integration works to connect to the Gore Hill Freeway at Artarmon.

A combined delivery of the Western Harbour Tunnel and Beaches Link program of works would unlock a range of benefits for freight, public transport and private vehicle users. It would support faster travel times for journeys between the Northern Beaches and areas south, west and north-west of Sydney Harbour. Delivering the program of works would also improve the resilience of the motorway network, given that each project provides an alternative to heavily congested existing harbour crossings.

1.2 The project

Transport for NSW is seeking approval under Part 5, Division 5.2 of the *Environmental Planning and Assessment Act 1979* to construct and operate the Beaches Link and Gore Hill Freeway Connection project, which would comprise two components:

- Twin tolled motorway tunnels connecting the Warringah Freeway at Cammeray and the Gore Hill Freeway at Artarmon to the Burnt Bridge Creek Deviation at Balgowlah and the Wakehurst Parkway at Killarney Heights, and an upgrade of the Wakehurst Parkway (the Beaches Link)
- Connection and integration works along the existing Gore Hill Freeway and surrounding roads at Artarmon (the Gore Hill Freeway Connection).

A detailed description of the project is provided in Chapter 5 (Project description) and Chapter 6 (Construction work) of the environmental impact statement.

The Gore Hill Freeway Connection component of the project is not relevant to this report and is therefore not discussed further.

1.3 Project location

The project would be located within the North Sydney, Willoughby, Mosman and Northern Beaches local government areas, connecting Cammeray in the south with Killarney Heights, Frenchs Forest and Balgowlah in the north.

Commencing at the Warringah Freeway at Cammeray, the mainline tunnels would pass under Naremburn and Northbridge, then cross Middle Harbour between Northbridge and Seaforth. The mainline tunnels would then split under Seaforth into two ramp tunnels and continue north to the Wakehurst Parkway at Killarney Heights and north-east to Balgowlah, linking directly to the Burnt Bridge Creek Deviation to the south of the existing Kitchener Street bridge.

Surface works would also be carried out at the Gore Hill Freeway in Artarmon, Burnt Bridge Creek Deviation at Balgowlah and along the Wakehurst Parkway between Seaforth and Frenchs Forest to connect the project to the existing arterial and local road networks.

1.4 Key features

Key features of the Beaches Link component of the project are shown in . The key components which are relevant to this report include:

- Twin mainline tunnels about 5.6 kilometres long and each accommodating three lanes of traffic in each direction, together with entry and exit ramp tunnels to connections at the surface. The crossing of Middle Harbour between Northbridge and Seaforth would involve three lane, twin immersed tube tunnels
- Twin two lane ramp tunnels:
  - Eastbound and westbound connections between the mainline tunnel under Seaforth and the surface at the Burnt Bridge Creek Deviation, Balgowlah (about 1.2 kilometres in length)
  - Northbound and southbound connections between the mainline tunnel under Seaforth and the surface at the Wakehurst Parkway, Killarney Heights (about 2.8 kilometres in length)
  - Eastbound and westbound connections between the mainline tunnel under Northbridge and the surface at the Gore Hill Freeway and Reserve Road, Artarmon (about 2.1 kilometres in length).
- Operational facilities, including a motorway control centre at the Gore Hill Freeway in Artarmon and tunnel support facilities at the Gore Hill Freeway in Artarmon and the Wakehurst Parkway in Frenchs Forest
- Other operational infrastructure including groundwater and tunnel drainage management and treatment systems, surface drainage, signage, tolling infrastructure, fire and life safety systems, roadside furniture, lighting, emergency evacuation and emergency smoke extraction infrastructure, Closed Circuit Television (CCTV) and other traffic management systems.

Subject to obtaining planning approval, construction of the project is anticipated to commence in 2023 and is expected to take around five to six years to complete.

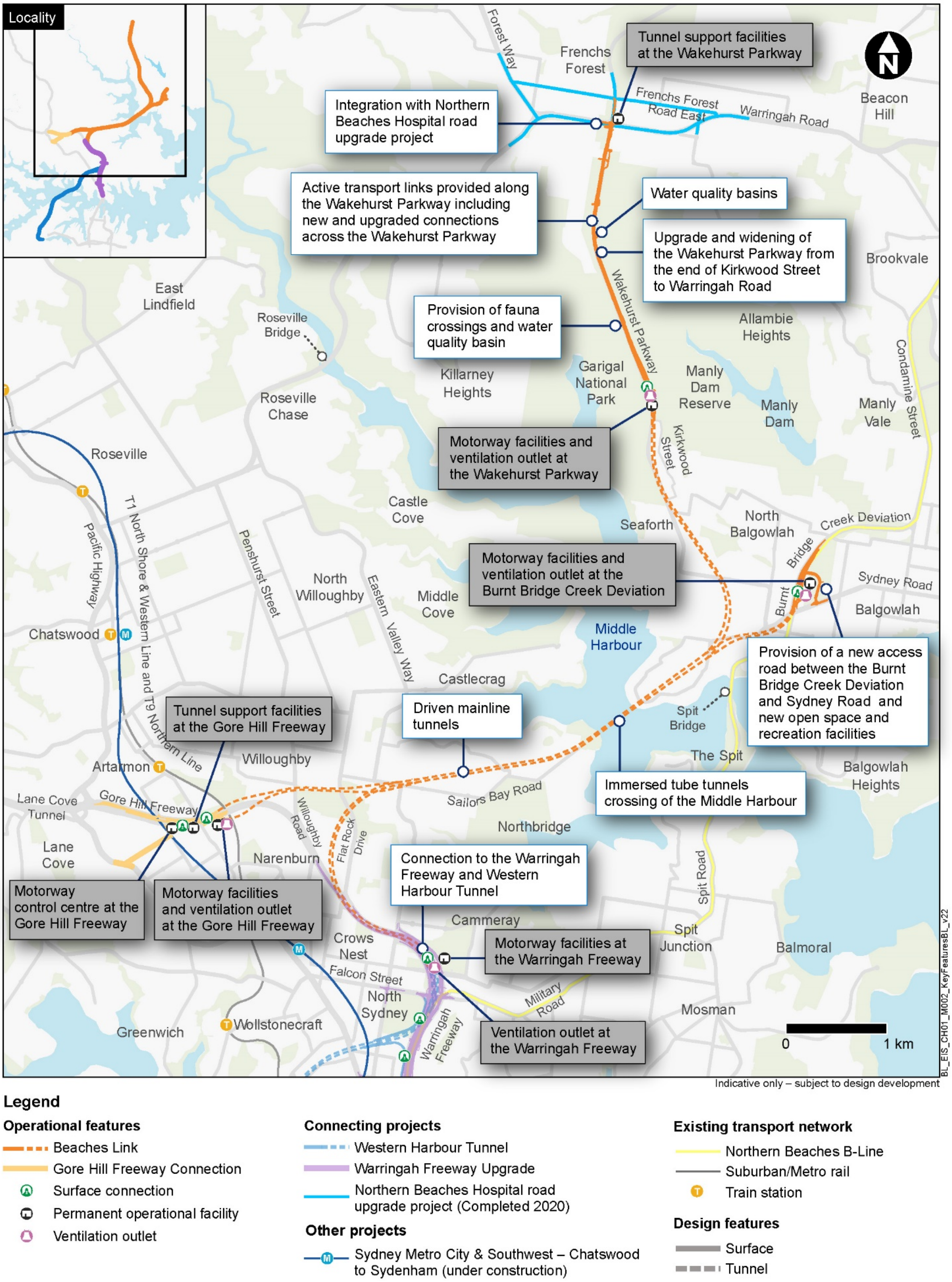

Figure 1-1: Key features of the Beaches Link component of the project

1.4.1 Immersed tube tunnels

The key feature of the Beaches Link component of the project relevant to this report is the crossing of Middle Harbour between Northbridge and Seaforth, which would be constructed as immersed tube tunnels.

The immersed tube tunnels would connect to the driven mainline tunnels in Middle Harbour offshore from Clive Park, Northbridge, and Seaforth Bluff, Seaforth.

The immersed tube tunnels would be installed as a series of pre-cast units. Due to the profile of the harbour bed, the units would sit both partially within in a trench closer to the shore and above the bed of the harbour towards the centre of the harbour crossing. The middle sections would be placed with the tops of the tunnel units being about 9.2 metres above the existing level of the bed of the harbour.

Given the very soft sediments at the bed of Middle Harbour, supporting piles would be required at discrete locations along the immersed tube crossing. A granular locking fill would be placed around the end sections (closer to the shore) of the immersed tube tunnels for stability and protection.

The water depth above the immersed tube tunnels would vary between 16 metres and 22 metres, depending on the distance from the shore.

The immersion of the tube tunnel elements would be performed by two immersion pontoons. Temporary anchors would be placed into the bed of the harbour prior to the immersion process to securely position the immersion pontoons and the tunnel elements.

Indicative cross sections of the immersed tube tunnel crossing of Middle Harbour are shown in Figure 1-2 (end sections) and Figure 1-3 (middle sections). An indicative long section of the immersed tube tunnels is shown in Figure 1-4.

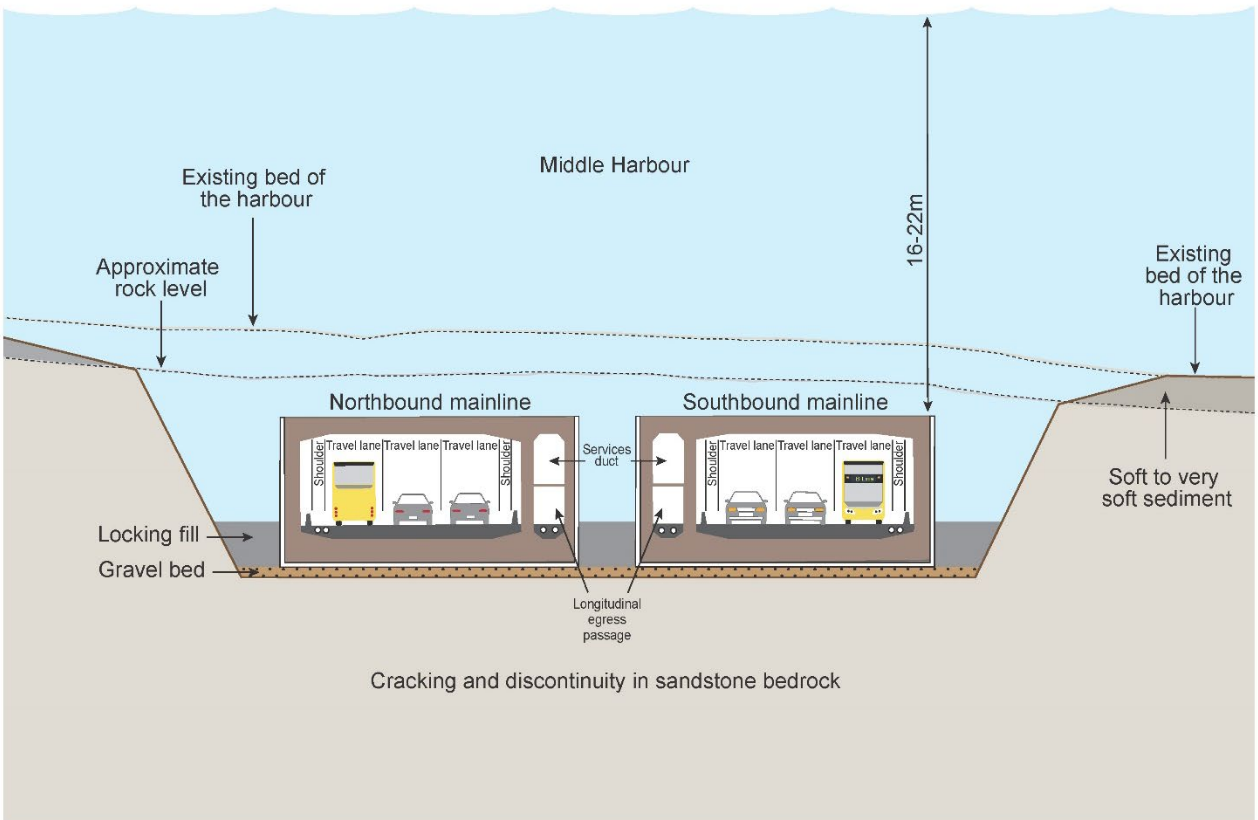

Figure 1-2: Indicative cross-section of the end sections of immersed tube tunnels at Middle Harbour

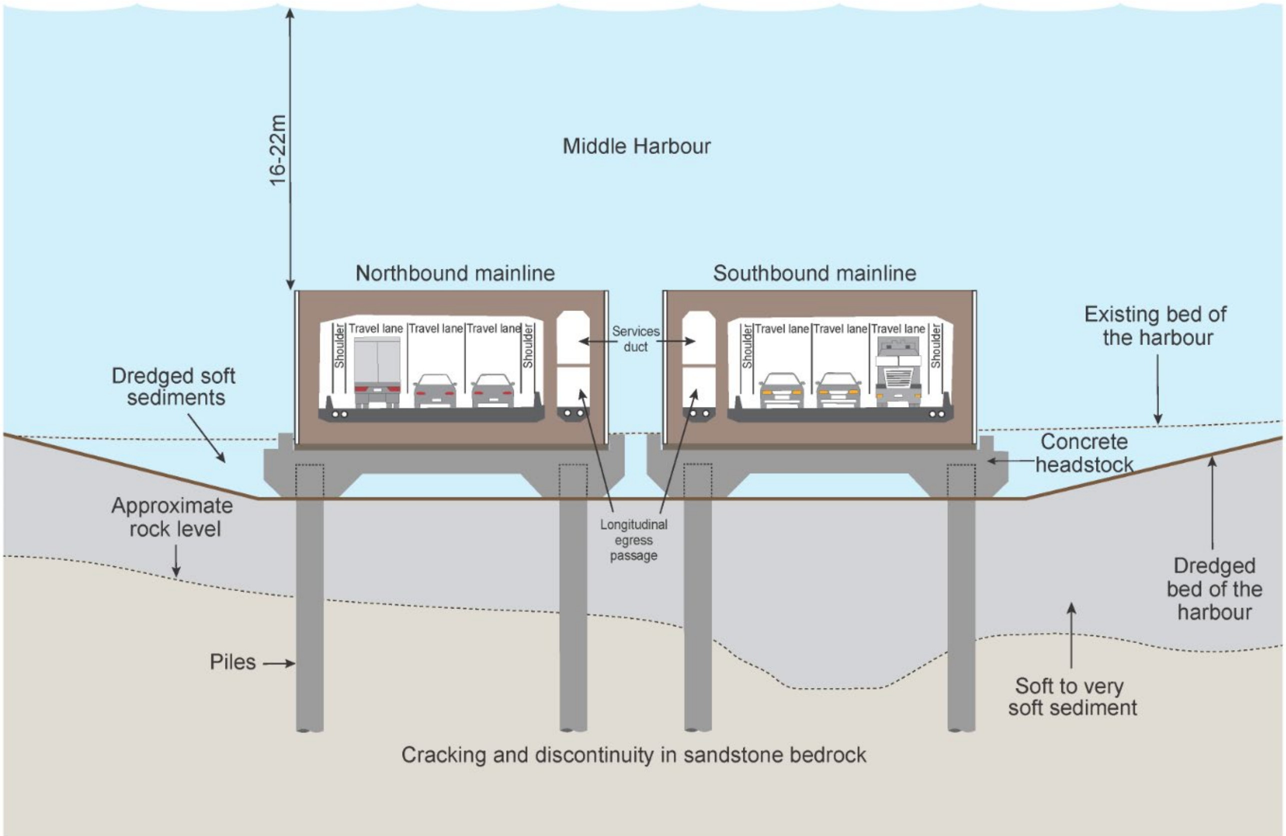

Figure 1-3: Indicative cross section of the middle sections of immersed tube tunnels at Middle Harbour

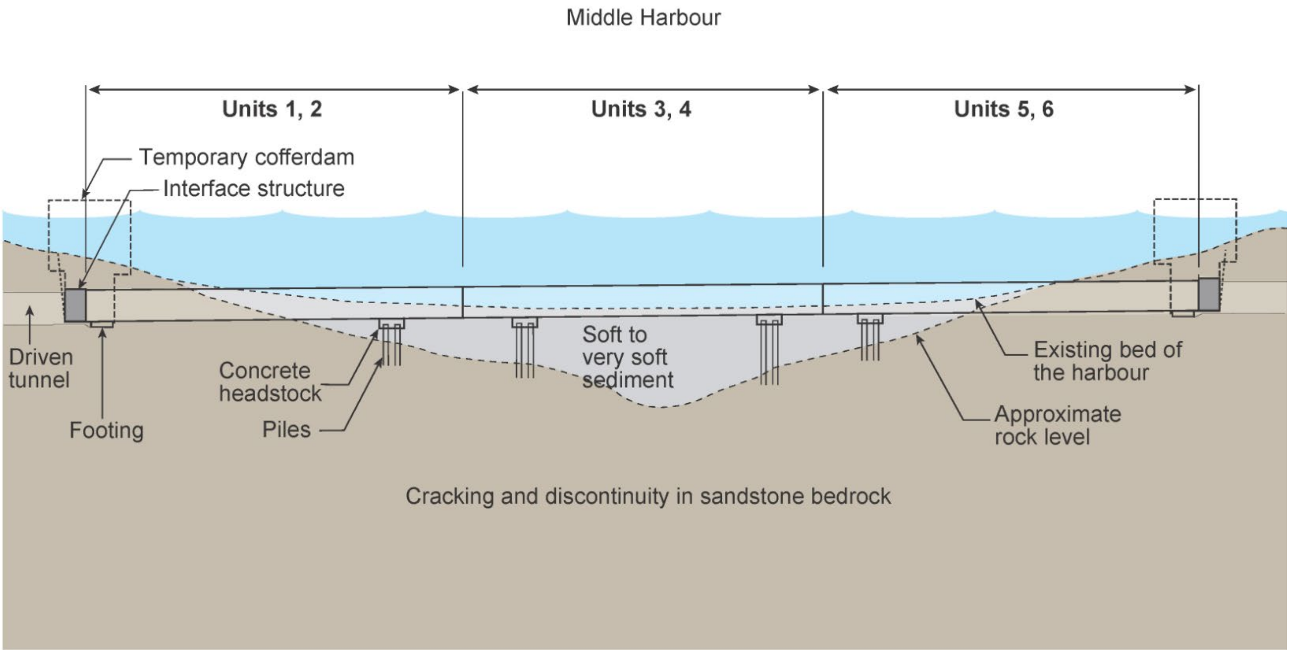

Figure 1-4: Indicative long section of the immersed tube tunnels at Middle Harbour

1.5 Key construction activities

The area required to construct the project is referred to as the construction footprint. The majority of the construction footprint would be located underground within the mainline and ramp tunnels. However, surface areas would also be required to support tunnelling activities and to construct the tunnel connections, tunnel portals, surface road upgrades and operational facilities.

Key construction activities would include:

- Early works and site establishment, with typical activities being property acquisition and condition surveys, utilities installation, protection, adjustments and relocations, installation of site fencing, environmental controls (including noise attenuation and erosion and sediment control), traffic management controls, vegetation clearing, earthworks, demolition of structures, building construction support sites including acoustic sheds and associated access decline acoustic enclosures (where required), construction of minor access roads and the provision of property access, temporary relocation of pedestrian and cycle paths and bus stops, temporary relocation of swing moorings and/or provision of alternative facilities (mooring or marina berth) within Middle Harbour
- Construction of the Beaches Link, with typical activities being excavation of tunnel construction access declines, construction of driven tunnels, cut and cover and trough structures, construction of surface upgrade works, construction of cofferdams, dredging and immersed tube tunnel piled support activities in preparation for the installation of immersed tube tunnels, casting and installation of immersed tube tunnels and civil finishing and tunnel fitout
- Construction of operational facilities comprising:
  - A motorway control centre at the Gore Hill Freeway in Artarmon
  - Tunnel support facilities at the Gore Hill Freeway in Artarmon and at the Wakehurst Parkway in Frenchs Forest
  - Motorway facilities and ventilation outlets at the Warringah Freeway in Cammeray (fitout only of the Beaches Link ventilation outlet at the Warringah Freeway (being constructed by the Western Harbour Tunnel and Warringah Freeway Upgrade project), the Gore Hill Freeway in Artarmon, the Burnt Bridge Creek Deviation in Balgowlah and the Wakehurst Parkway in Killarney Heights
  - A wastewater treatment plant at the Gore Hill Freeway in Artarmon
  - Installation of motorway tolling infrastructure
- Upgrade and integration works at Balgowlah and along the Wakehurst Parkway with typical activities being earthworks, bridgeworks, construction of retaining walls, stormwater drainage, pavement works and linemarking and the installation of roadside furniture, lighting, signage and noise barriers
- Testing of plant and equipment and commissioning of the project, backfill of access declines, removal of construction support sites, landscaping and rehabilitation of disturbed areas and removal of environmental and traffic controls.

Temporary construction support sites would be required as part of the project (refer to), and would include tunnelling and tunnel support sites, civil surface sites, cofferdams, mooring sites, wharf and berthing facilities, laydown areas, parking and workforce amenities.

Only three construction support sites are relevant to this report. These are:

- Middle Harbour south cofferdam (BL7)
- Middle Harbour north cofferdam (BL8)
- Spit West Reserve (BL9).

A detailed description of construction works for the project is provided in Chapter 6 (Construction work) of the environmental impact statement.

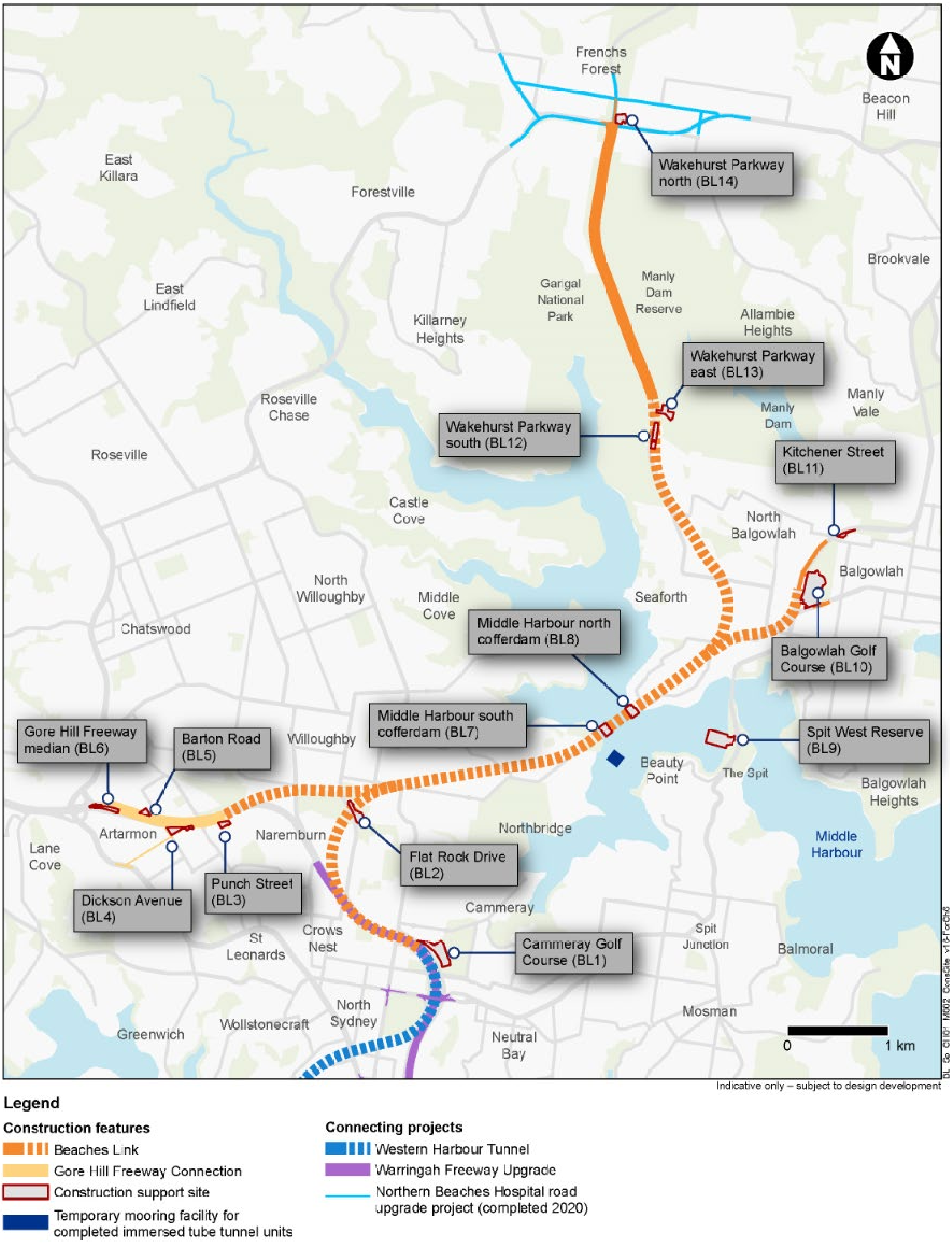

Figure 1-5: Overview of the construction support sites

1.6 Purpose of this report

This report has been prepared to support the environmental impact statement for the project and to address the environmental assessment requirements of the Secretary of the NSW Department of Planning, Industry and Environment ('the Secretary's environmental assessment requirements').

This report documents a hydrodynamic and water quality impact assessment for the crossing at Middle Harbour. It provides:

- A description of the existing marine environment based on available information and recently collected, project specific, hydrodynamic and water quality data
- A summary of the establishment and calibration of the three-dimensional (3D) numerical models used in the impact assessment
- A summary of the dredging methodology and assumptions as they relate to the dredge plume modelling
- Results of the predictive modelling carried out to assess potential construction impacts for the following items:
    - Impacts of the temporary cofferdams and 12 metre deep silt curtains on the hydrodynamics
    - Impacts of the Spit West Reserve construction support site (BL9) on the hydrodynamics
    - Impacts of the dredging on water quality for the various stages of dredging.
- Results of the predictive modelling carried out to assess the potential operational impacts for the following items:
    - Impacts of the immersed tube tunnels on the hydrodynamics
    - Impacts of the immersed tube tunnels on the flushing characteristics of Middle Harbour upstream of the immersed tube tunnels.

The hydrodynamic and water quality modelling has been carried out by Haskoning Australia Pty Ltd, a company of Royal HaskoningDHV (RHDHV), on behalf of Transport for NSW. The hydrodynamic and water quality impact assessment work is part of the technical and environmental advisory services RHDHV has carried out in relation to the Western Harbour Tunnel and Beaches Link program of works.

1.7 Secretary's environmental assessment requirements

The Secretary's environmental assessment requirements relating to hydrodynamic and dredge plume modelling, and where these requirements are addressed in this report are outlined in Table 1–1.

Table 1–1: Secretary's environmental assessment requirements – hydrodynamic and dredge plume modelling

Secretary's environmental assessment requirements	Where addressed
9. Water - Hydrology	
1. The Proponent must describe (and map) the existing hydrological regime for any surface and groundwater resource (including reliance by users and for ecological purposes) and groundwater dependent ecosystems likely to be impacted by the project, including rivers, streams, wetlands and estuaries as described in Appendix 2 of the Framework for Biodiversity Assessment – NSW Biodiversity Offsets Policy for Major Projects (OEH, 2014).	Section 3

Secretary's environmental assessment requirements	Where addressed
3. The Proponent must assess (and model if appropriate) the impact of the construction and operation of the project and any ancillary facilities (both built elements and discharges) on surface and groundwater hydrology in accordance with the current guidelines, including:	
(a) natural processes within rivers, wetlands, estuaries, marine waters and floodplains that affect the health of the fluvial, riparian, estuarine or marine system and landscape health (such as modified discharge volumes, durations and velocities), aquatic connectivity water-dependent fauna and flora and access to habitat for spawning and refuge;	Section 6 and Section 8
(d) direct or indirect increases in erosion, siltation, destruction of riparian vegetation or a reduction in the stability of river banks or watercourses;	Section 6 and Section 8
(f) measures to mitigate the impacts of the proposal and manage the disposal of produced and incidental water	Section 6 and Section 8
10. Water Quality	
The Proponent must:	
(a) describe the background conditions for any surface or groundwater resource likely to be affected by the development;	Section 3
(c) identify and estimate the quality and quantity of all pollutants that may be introduced into the water cycle by source and discharge point and describe the nature and degree of impact that any discharge(s) may have on the receiving environment, including consideration of all pollutants that pose a risk of non-trivial harm to human health and the environment;	Section 7 and Section 8
(e) assess the significance of any identified impacts including consideration of the relevant ambient water quality outcomes;	Section 7 and Section 8
(h) demonstrate that all practical measures to avoid or minimise water pollution and protect human health and the environment from harm are investigated and implemented;	Section 7 and Section 8
(i) identify sensitive receiving environments (which may include estuarine and marine waters downstream including Burnt Bridge Creek, Quarry Creek and Flat Rock Creek) and develop a strategy to avoid or minimise impacts on these environments.	Section 7 and Section 8

2 Available data

2.1 Introduction

This section provides an overview of both the historical data available for the Port Jackson region and the project specific hydrodynamic data collected as part of this project. The project specific data collection exercise was designed to ensure that any significant data gaps, based on a review of the historical data available, were filled so that there was sufficient data available to describe the existing environment and calibrate the hydrodynamic models.

2.2 Historical data

This study has utilised the historical water level, wind, and water quality data for the sites in Port Jackson region as shown in Figure 2-1. Table 2–1 provides additional details of monitored sites shown in Figure 2-1.

Bathymetric data is made up of the latest available data provided by Transport for NSW for areas around the crossing of Middle Harbour. For other areas, the detailed bathymetric data was augmented with digitised navigation charts available from C-Map. C-Map is a commercially available source of digitised Admiralty navigational charts.

Table 2–1: Review of available data at the study site

Data Type	Location	Description
Water Level	Sydney (Live) Silverwater Bridge	Data from two tide gauges managed by Manly Hydraulics Laboratory were used in this study. Sydney (live) is located close to the entrance of Port Jackson (1987 – 2017). Silverwater Bridge is located upstream in the Parramatta River (2012 – 2017).
Wind	Fort Denison West Wedding Cake	This study predominately utilised Australian Bureau of Meteorology (BoM) weather stations at Fort Denison (1990– 2017). While additional BoM meteorological stations in the Port Jackson region were analysed, Fort Denison was considered to be the most representative of overwater wind at the crossing of Middle Harbour with a sufficiently long record.
Water quality	Barangaroo	RHDHV has made reference to water quality data (principally turbidity data) that has been collected at Barangaroo. This data set covers a few nearshore sites with the monitoring periods spanning more than a year (since early 2016). However, it should be recognised that Barangaroo is located in the main arm of Port Jackson, some distance from the site of the crossing of Middle Harbour.

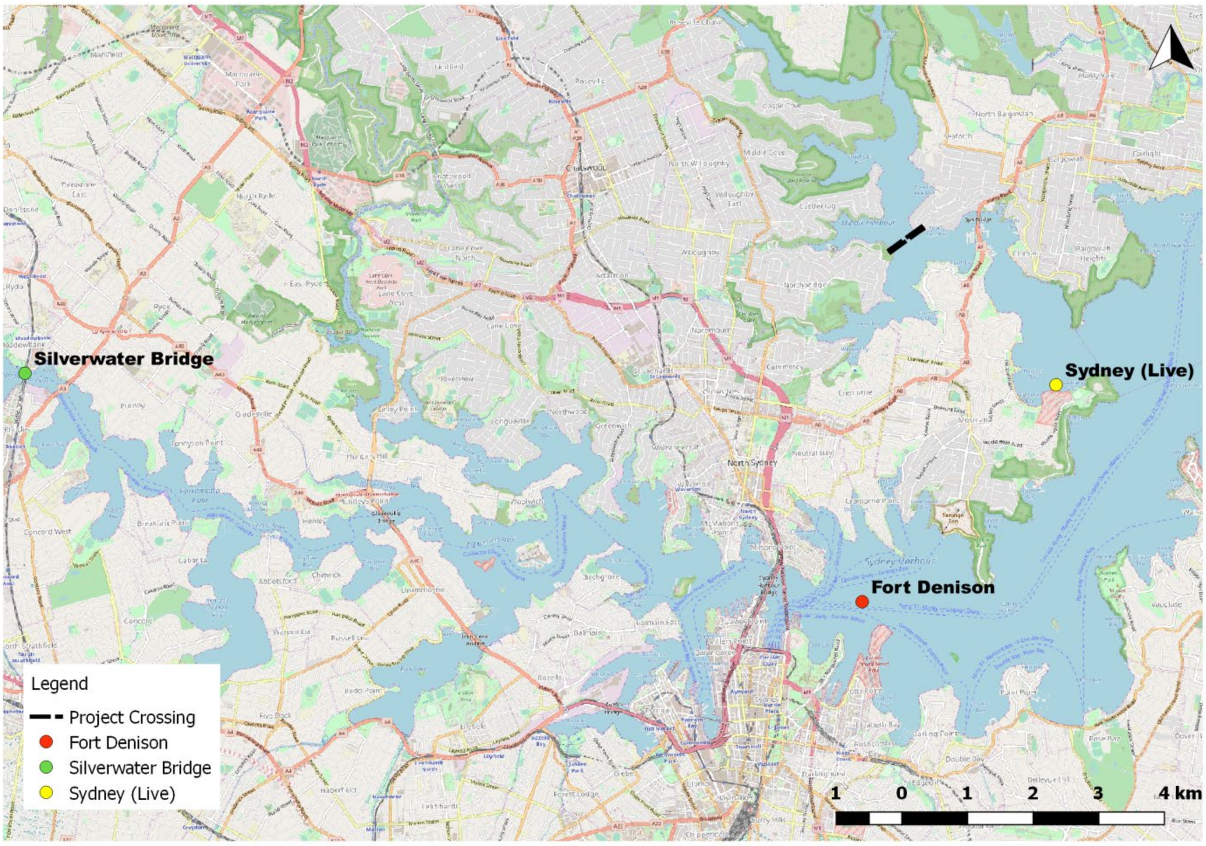

Figure 2-1: Existing data available in the Port Jackson and Parramatta River area

2.3 Project specific data collection

Project specific hydrodynamic and water quality data was collected at the proposed immersed tube tunnel crossing of Middle Harbour. Details regarding the monitoring campaign, including a factual presentation of the data, are found in **Annexure A**. The aim of the monitoring was to ensure the hydrodynamic modelling, assessment of environmental impacts, and dredging advice is supported by site specific measurements. The locations where project specific data was collected at Middle Harbour are shown Figure 2-2. The measured data collected is summarised as:

- Two in-situ monitoring sites (MH1 and MH2) located near the project crossing were used to measure temporal variability in hydrodynamic and water quality conditions due to tidal and non-tidal influences. Each site provided continuous measurements of water level, current velocity and acoustic backscatter using an ADCP (Acoustic Doppler Current Profiler) type instrument. At any one time, one of the in-situ monitoring sites also measured water quality parameters (primarily turbidity). Water quality monitoring for the project was primarily carried out by Cardno (2020) on behalf of Transport for NSW and reported separately (refer to Appendix Q (Technical working paper: Marine water quality)). The water quality data collected as part of RHDHV's monitoring was carried out to inform an understanding of the concurrent turbidity at the in-situ monitoring locations. The monitoring period for which data was available for this report was between 17 August 2017 and 1 November 2017, totalling a monitoring period of 76 days

- Vessel mounted ADCP transects were carried out along two transects in Middle Harbour as shown in Figure 2-2. Vessel mounted ADCP transects were carried out during spring tidal conditions on 22 August 2017 to determine spatial variability in currents and discharge throughout a tidal cycle Spring tide conditions were selected as they correspond to higher tidal current speeds, ensuring good records were obtained (noting that typical current speeds in parts of Middle Harbour are low in magnitude)
- Opportunistic surface sediment samples from the bed of the harbour were collected at each crossing location and analysed for particle size distribution.

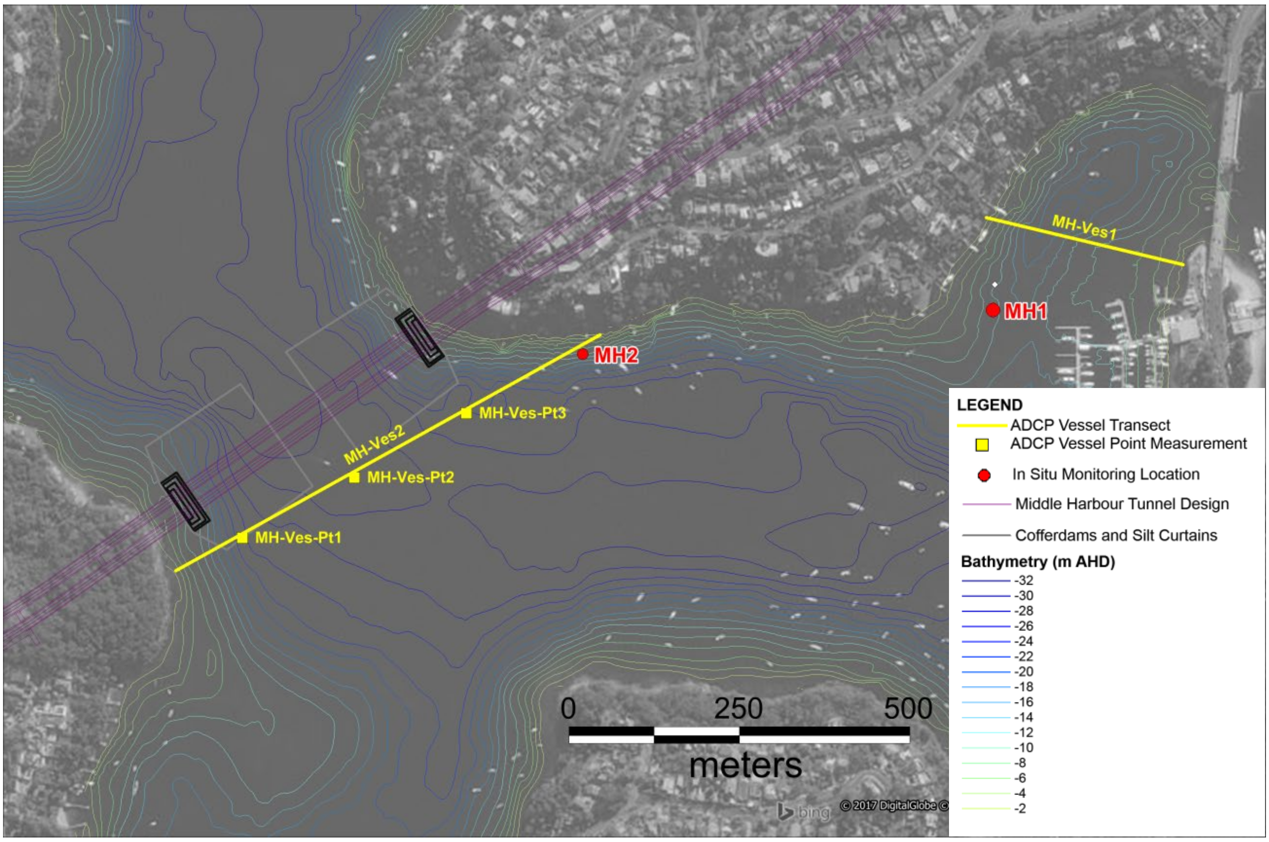

Figure 2-2: Map showing the hydrodynamic and water quality monitoring locations at Middle Harbour

3 Description of the existing environment

3.1 Site description

The crossing of Middle Harbour is located within Port Jackson about seven kilometres to the north-east of the Sydney Central Business District (CBD).

Port Jackson is a drowned river valley that was formed during a period of natural sea level rise about 10,000 years ago. Port Jackson is comprised of three harbours: North Harbour, Middle Harbour and Sydney Harbour (the main branch of the estuary). The Middle Harbour region of Sydney Harbour is the north-western branch of the estuary, and is one of the three main tributaries; the other two being Parramatta River (Western Harbour) and Lane Cove River.

The waters of Port Jackson are typically well mixed due to low fresh water discharges and turbulent tidal mixing. The rainfall pattern is typically erratic and spatially variable, being characterised by generally dry conditions, with infrequent high rainfall events of greater than 50 millimetres per day.

The hydrodynamic conditions at the crossing of Middle Harbour location are primarily driven by astronomical tides. To a lesser extent wind also contributes to the overall circulation, however, other influences from barometric effects and freshwater flows from local creeks and rivers are comparatively small. The wave climate is limited to locally generated wind waves and waves from boat wakes (predominately recreational craft).

Two popular public swimming areas are located within Middle Harbour near the crossing; Northbridge Baths located about one kilometre upstream and west of the crossing in Sailors Bay, and Clontarf Baths located about 1.8 kilometres downstream and east of the crossing opposite The Spit.

3.2 Bathymetry

3.2.1 Level datum

All levels in this section refer to Australian Height Datum (AHD) unless noted otherwise. Chart Datum lies 0.925 metres below AHD. Chart Datum is equal to the zero marker at the Fort Denison gauge.

3.2.2 Bathymetry

The adopted bathymetry for the study site was based on the latest available bathymetric soundings, provided by Transport for NSW. Bathymetry data is presented in Figure 3-1.

The bed of Port Jackson is comprised of many deep holes, shoals, basins, rocky islands and reefs. The bathymetry in the main channel of Middle Harbour upstream of the Spit Bridge is relatively deep, having formed from a drowned 'V' shaped valley which then slowly infilled with sediment to presently resemble a 'U' shaped channel. The shape of the waterway area is complex with a number of off-channel embayments.

The location of the crossing of Middle Harbour stretches from Clive Park, Northbridge in the south to Seaforth Bluff, Seaforth in the north and crosses the main channel of Middle Harbour. Figure 3-1 provides a cross section of the channel at the proposed crossing location. The bathymetry at the proposed crossing location is best described as a relatively deep defined symmetrical channel. The depth of the channel at the proposed crossing location is particularly deep being up to 32 metres at its deepest point.

An important feature of the bathymetry near to the proposed crossing site is the constriction in the main channel of Middle Harbour, between The Spit and Seaforth. The water depths at this section of the main channel are shallow (five to 10 metres) in comparison to the main reaches of Middle Harbour directly upstream. The Spit along with the Spit Bridge and its associated piers act as a constriction to tidal flows. The channel also goes through a near 180 degree bend as it passes through this constriction. From a hydrodynamics perspective this feature acts to control the volume of tidal waters that propagate upstream beyond The Spit.

A number of bays are also located in close proximity to the location of the proposed crossing. These bays include Quakers Hat Bay directly to the south, Sailors Bay to the west and Fig Tree Cove to the north. These off-channel embayments adjoin the main channel and from a hydrodynamics perspective act as large reservoirs for tidal waters.

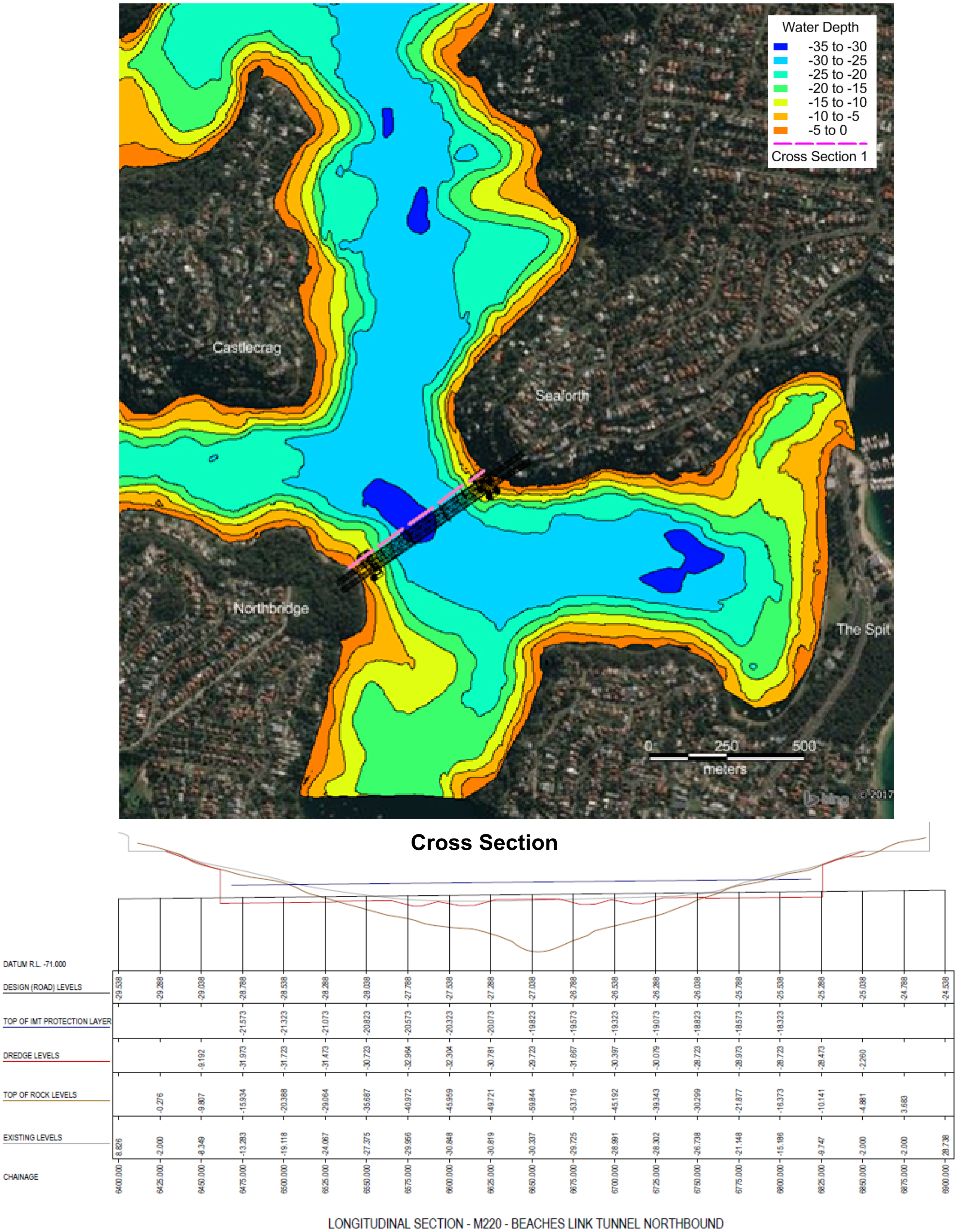

Figure 3-1: Local bathymetry at the project crossing (data source: Transport for NSW)

3.3 Metocean

Following standard metocean conventions, wind and wave direction are reported as the direction the wind/wave is coming from in degrees clockwise from True North. Current direction is reported as the direction the current is going to in degrees clockwise from True North.

3.3.1 Tides

Port Jackson is a semi-diurnal, micro-tidal (with approximate range: one metre for neap tides and 1.3 metres for spring tides) estuary. A number of large, shallow, muddy bays adjoin the main channel and represent large reservoirs for tidal water. Despite the low tidal range, in the absence of any constant source of freshwater discharge, ebb and flood tidal discharges are the dominant cause of water movement in the harbour.

The relatively deep channel throughout Middle Harbour is a key feature that would influence the natural residence time of waters in the region. The channel constriction at Spit Bridge limits the volume of tidal waters able to propagate upstream. The constriction results in high current speeds at Spit Bridge, but low current speeds further upstream.

The tidal plane values near the entrance to Port Jackson (Camp Cove) are shown in Table 3–1. Figure 3-2 displays example water level data from May 2016 to July 2016.

Table 3–1: Tidal planes Sydney Harbour (Manly Hydraulics Laboratory, 2016)

Tidal plane	Camp Cove (33˚50’, 151˚17’) (m AHD)
Highest Astronomical Tide (HAT)	1.15
Mean High Water Springs (MHWS)	0.65
Mean High Water Neaps (MHWN)	0.40
Mean Sea Level (MSL)	0.03
Mean Low Water Neaps (MLWN)	-0.36
Mean Low Water Springs (MLWS)	-0.61
Lowest Astronomical Tide (LAT)	-0.90

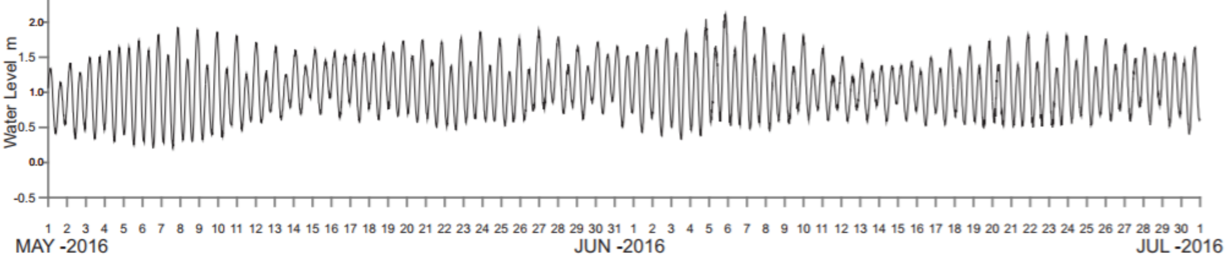

Figure 3-2: Sydney Harbour water level referenced to zero metres (ie Chart Datum) at Fort Denison (Manly Hydraulics Laboratory, 2016)

3.3.2 Currents

A summary of current speed statistics taken from the MH1 and MH2 in-situ project monitoring sites is provided in Table 3–2. Additional information, including time series plots, current roses, current velocity scatter plots and depth profile plots of the currents recorded at MH1 and MH2 are provided in **Annexure A**. Analysis of the in-situ current data found that uniform flow typically occurs throughout the water column during peak flood and ebb flows (ie low variation in currents with depth), however, vertical flow separation can occasionally occur during periods when low tidal currents coincide with high wind speeds.

Table 3–2: Summary of current speed statistics at the project monitoring sites

Parameter	Statistic	MH1	MH2
Flood Current Speed (m/s)	Maximum	0.72	0.15
	95th percentile	0.42	0.07
	Mean	0.17	0.03
Ebb Current Speed (m/s)	Maximum	0.37	0.21
	95th percentile	0.21	0.08
	Mean	0.09	0.04

The spatial (two-dimensional (2D) depth averaged) and vertical patterns in tidal currents are shown in Figure 3-3 and Figure 3-4 for peak flood and ebb tidal stages, respectively. It is observed that spatial current patterns in Middle Harbour are influenced by the complex shape of the harbour, the relatively deep U-shaped channel and the constriction at Spit Bridge. Tidal current speeds at the deeper crossing site were less than 0.2 metres per second (m/s), while nearer the Spit Bridge constriction, current speeds of up to 0.7 m/s were recorded. As seen in Table 3–2 there is substantial tidal asymmetry in current speeds near the Spit Bridge with peak flood current speeds at MH1 faster than ebb currents. It is further observed that spatial measurements, similar to the in-situ monitoring, showed little change in speed with depth.

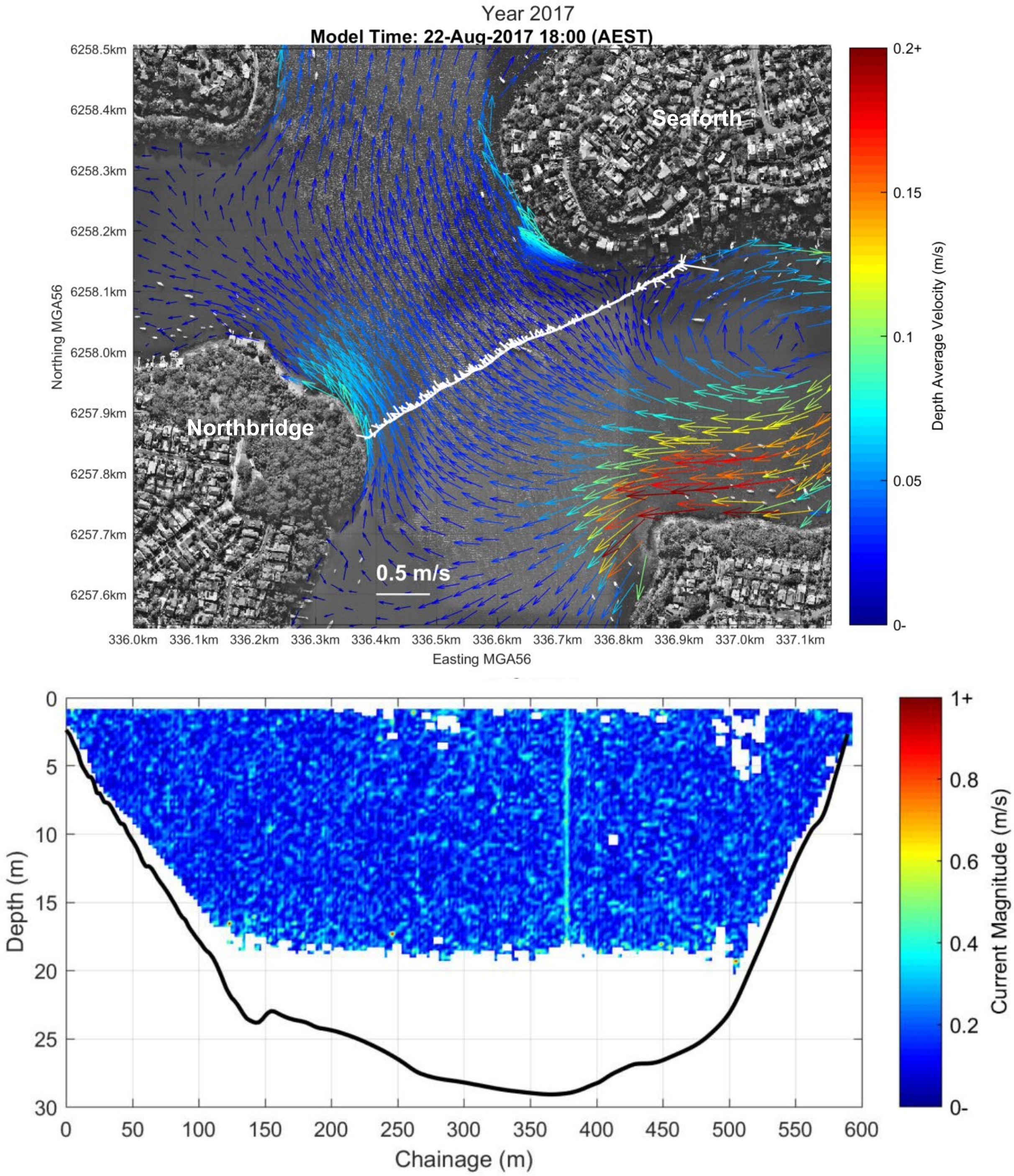

Figure 3-3: Spatial flood tidal current patterns based on measured and modelling data (top) and measured flood current speeds with depth along transect MH-Ves2 (bottom)

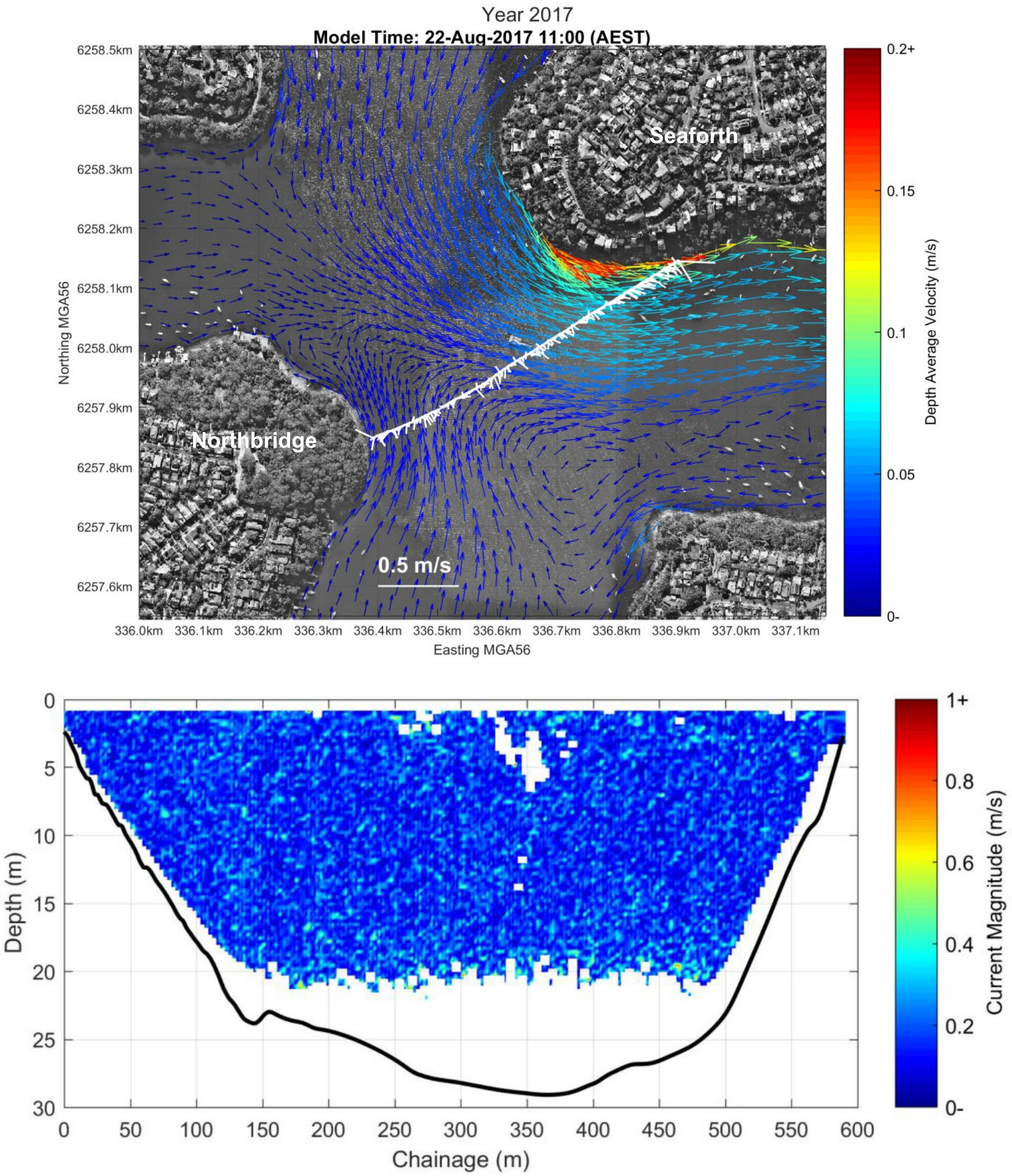

Figure 3-4: Spatial ebb tidal current patterns based on measured and modelling data (top) and measured ebb current speeds with depth along transect MH-Ves2 (bottom)

3.3.3 Wind

A wind analysis was carried out at the Fort Denison BoM weather station. This station was selected over the Sydney Harbour Station (West Wedding Cake Island) due to its location, which is further into Port Jackson sheltering it from the stronger coastal winds. Consequently, the Fort Denison weather station was considered the most representative of overwater wind conditions at the crossing of Middle Harbour compared to other available data sources.

Table 3–3 details wind statistics based on monthly percentiles and Figure 3-5 displays seasonal wind roses. Summer is dominated by onshore (easterly and north-easterly) winds which are occasionally interrupted with southerly winds (ie southerly change). During winter and autumn westerly winds are prevalent. Stronger wind speeds were observed throughout spring and summer, while the autumn and winter tended to have slower wind speeds.

Table 3–3: Monthly wind statistics derived from the Fort Denison weather station (1990-2017)

Season	Month	50th Percentile wind speed (m/s)	90th Percentile wind speed (m/s)	Predominant wind direction (from)
Summer	January	4.7	7.8	East
	February	4.2	7.8	East
Autumn	March	4.2	7.8	East-West
	April	4.2	6.7	West
	May	4.2	6.7	West
Winter	June	4.2	7.2	West
	July	4.2	7.2	West
	August	4.7	7.8	West
Spring	September	4.7	8.3	West
	October	4.7	8.3	East-West
	November	4.7	8.3	East
Summer	December	4.7	8.3	East

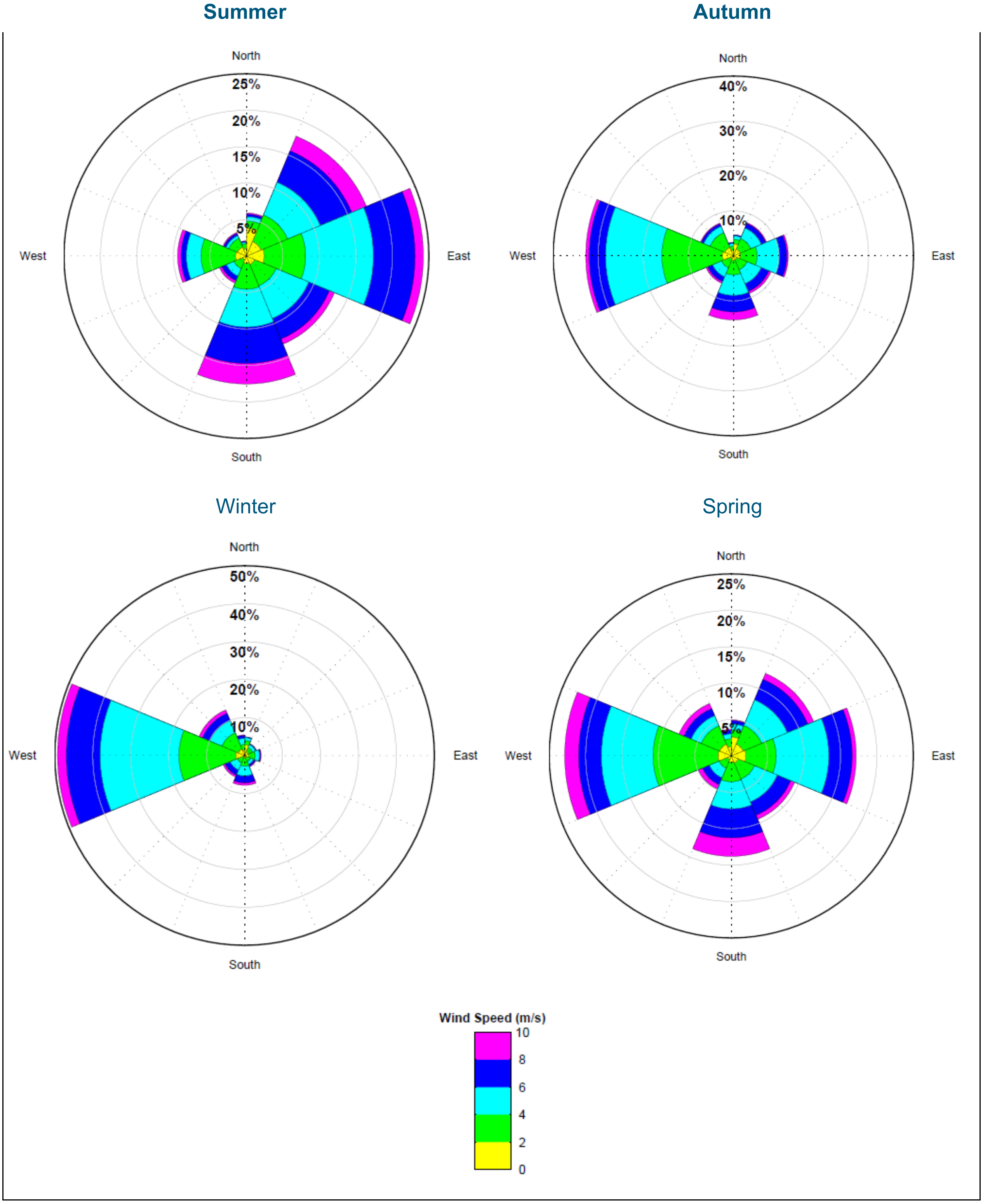

Figure 3-5: Seasonal wind roses derived from data at the Fort Denison weather station

3.3.4 Waves

Ocean swells that enter the harbour are diffracted by the complex bathymetry and shoreline configuration such that most of Port Jackson is affected only by locally derived wind and ship-generated waves. Within Middle Harbour vessel traffic is primarily composed of recreational craft and vessel wakes are generally weak. The wave climate at the crossing of Middle Harbour is a low energy or mild wave climate with wave heights typically less than 0.3 metres and wave periods of less than four seconds.

The bathymetry within the vicinity of the Middle Harbour crossing is relatively deep meaning that the potential effect of waves (either wind waves or boat wakes) on hydrodynamic or sediment plumes at the bed of the harbour is considerably reduced.

3.4 Rainfall and freshwater inputs

The mean annual rainfall observed at Observatory Hill, Sydney is 1215 millimetres. Figure 3-6 illustrates the mean monthly precipitation observed at Observatory Hill. It is evident from the graph that rainfall is evenly spread throughout the year with low to moderate variability between seasons. Average mean monthly rainfall between the years 1859 and 2017 ranged from a minimum of 67.9 millimetres in September to a maximum of 133.2 millimetres in June. The mean number of days per month where rainfall exceeded one millimetre ranged from 7.2 days in August to 9.8 days in March over this period (BoM, 2017).

It is important to note that rainfall in Sydney is highly variable both year to year and month to month. Much of the variability in precipitation is due to large-scale climate variations, with El Niño Southern Oscillation being the most important (BoM, 2015).

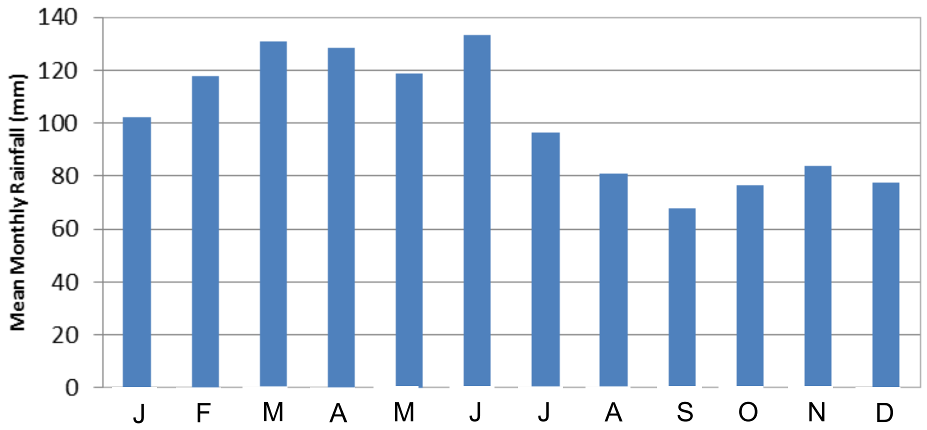

Figure 3-6: Mean monthly rainfall observed at Sydney's Observatory Hill (1859-2017)

Freshwater input into Port Jackson is entirely dependent upon runoff from rainfall in the local catchment. There are no permanent rivers or streams which discharge into the system. The Parramatta and Lane Cove Rivers are merely arms of the estuary and provide limited to no freshwater flux into the system, except during major freshwater events. The main tributary to Middle Harbour is Middle Harbour Creek, which also provides limited freshwater flux into the system.

Due to a low freshwater input into the systems, the Port Jackson estuary is considered to be generally well mixed with tidal currents being the primary mechanism for water movement. It is considered that, for a freshwater event to be of sufficient magnitude to transport dredging related sediment beyond the influence of tidal currents, the turbidity plumes generated from such an extreme rainfall and runoff event would itself be significant, such that it would be difficult to determine the impact of the project above the natural environment.

3.5 Suspended sediments

The ambient suspended sediment concentrations (SSC) for the water of Port Jackson is of particular relevance to this project due to the project's dredging requirements and the potential for influence on sensitive ecological habitats.

Turbidity (which is typically used as an indicator of SSC) of the waters within Port Jackson displays a noticeable gradient from high turbidity in the shallower upper reaches of the Parramatta River and longer embayments, to low turbidity in the lower reaches of the harbour where tidally-driven ocean exchange influences water quality (Cardno, 2020). Turbidity data for the greater Port Jackson estuary is available from various sources and was reviewed in Cardno (2020). A summary of measured turbidity for the waters around Balls Head Bay in Western Harbour is provided in Table 3–4 below. There is limited existing ambient turbidity data for Middle Harbour.

Table 3–4: Ambient turbidity characteristics near Balls Head (Cardno, 2017)

Condition	Ambient Turbidity Range
Dry weather	<1 to 4 NTU
Wet weather	4 to 20 NTU - short-lived events ~<2 days with higher values on ebbing tide

The turbidity values noted in Table 3–4 are consistent with turbidity data measured in Darling Harbour. Generally, ambient turbidity in Darling Harbour is low (less than five Nephelometric Turbidity Unit (NTU)) with higher turbidity only observed during notable catchment rainfall events, as a result of suspended solids entering the harbour via stormwater outlets and sewer overflows (Robinson et al., 2014).

Figure 3-7 presents turbidity data measured at Darling Harbour during a notable catchment rainfall event. As illustrated, brief periods of elevated turbidity up to about 30 NTU occur in response to the significant catchment rainfall events (June 2016 storm event). However, due to the deep water and efficient tidal flushing, these events generally dissipate within a few days.

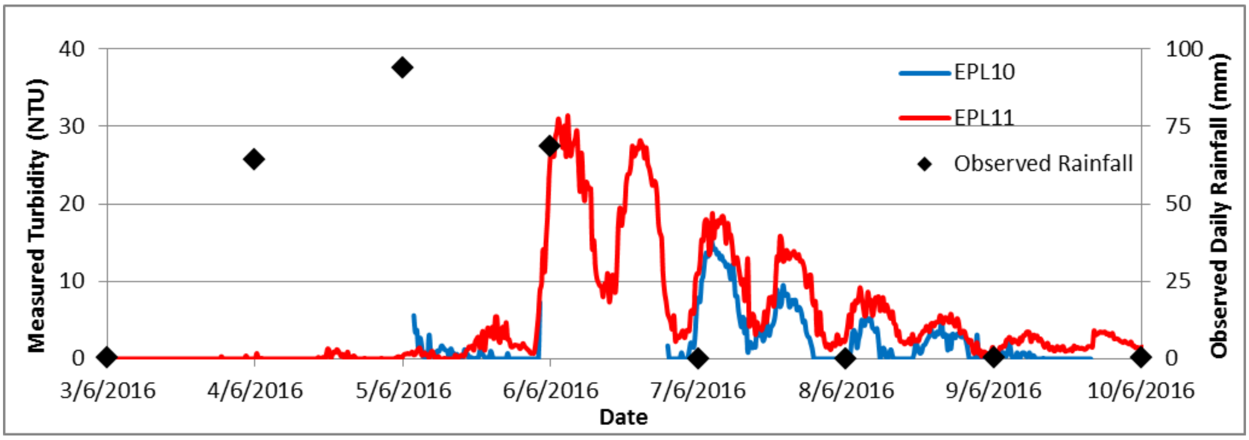

Figure 3-7: Turbidity observed at Darling Harbour during notable catchment rainfall event (data source: water quality data collected by RHDHV at Barangaroo)

An example of the naturally high turbidity which occurs within Middle Harbour following heavy rainfall is shown in Figure 3-8 (Peach Tree Bay) and Figure 3-9 (near Clive Park) for an event in early February 2020. The photos were taken on 11 February following rainfall of 78 millimetres, 69 millimetres, 65 millimetres and 165 millimetres over the previous four days, as measured at Observatory Hill. Peach Tree Bay is located just upstream of the proposed project crossing location adjacent to Seaforth; Clive Park is located near the proposed project crossing location in Northbridge.

Figure 3-8: Natural high turbidity following heavy rainfall (11 February 2020, Peach Tree Bay)

Figure 3-9: Natural high turbidity following heavy rainfall (11 February 2020, near Clive Park)

In this report, model results are reported in SSC (milligrams per litre (mg/L)). While SSC to NTU relationships have been developed at other nearby sites there is no project specific relationship available for this report.

As noted in **Section 3.4**, the typical pattern of catchment discharge into Port Jackson estuary is one of low-flow conditions, with the occasional medium/high-flow events associated with rainfall events in the catchment. Under the typical low flow conditions, the estuary is almost fully saline and considered to be in a well-mixed state (Hatje et al., 2001). It is noted that during medium or high flow conditions, the estuary becomes stratified, with vertical stratification occurring due to buoyant freshwater runoff overlying the more dense saline water of the estuary. The freshwater runoff produces a surface turbid layer which is known to thicken as it progresses downstream (Cardno, 2020).

4 Model setup and configuration

4.1 Overview

The existing RHDHV MIKE 21 Flexible Mesh hydrodynamic model of Sydney Harbour and the Parramatta River was updated and refined for this study (refer Figure 4-1). This MIKE 21 model has been used on a number of previous projects.

To support the environmental impact statement, the MIKE 21 model was developed further. It was calibrated to the project specific water level, current and flow measurements and upgraded to a 3D hydrodynamic model (ie MIKE 3).

The MIKE software suite was developed by the Danish Hydraulic Institute. It is internationally recognised as state-of-the-art and has been adopted by RHDHV and others globally in similar environments. It has a track record of providing a realistic representation of the natural marine environment. The flexible mesh allows the spatial resolution of the computational grid to be locally increased in areas of interest (ie the crossing of Middle Harbour) while the resolution in other areas can be coarser to help maintain acceptable model run times.

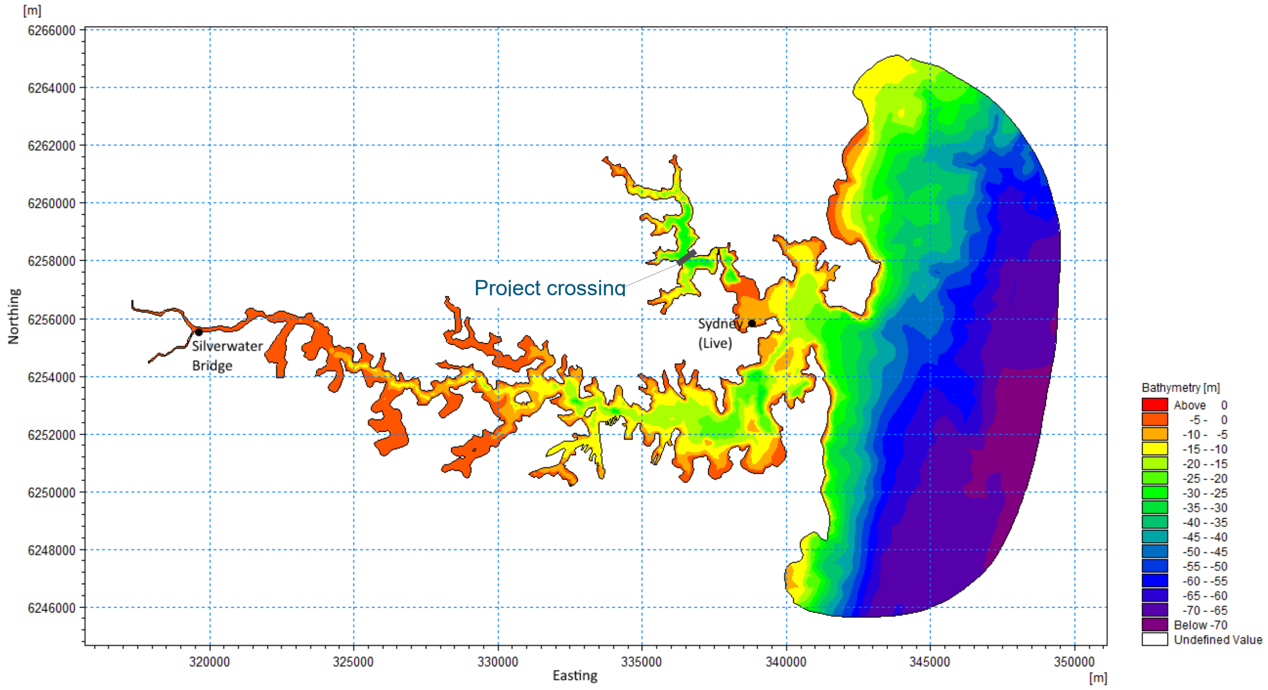

Figure 4-1: Extent of the RHDHV Sydney Harbour and Parramatta River MIKE 21 model domain including bathymetry and location of water level calibration sites

4.2 Hydrodynamic model

4.2.1 Model bathymetry

The model bathymetry was defined based on measured data supplied by Transport for NSW along with digitised navigation charts from C-Map. A compilation of hydrographic soundings from the Port Jackson area was supplied for the areas around the project crossing. The soundings covered the waterway area for a distance of about 3000 metres upstream and 1500 metres downstream from the crossing. The remaining model bathymetry was defined based on the digitised chart data extracted from C-Map. The model bathymetry can be seen in Figure 4-1 to Figure 4-3.

4.2.2 Model domain

The model mesh was refined to include additional spatial resolution in the model at the project crossing location (refer Figure 4-3). In addition, the model mesh was refined to ensure adequate representation of the project design including temporary cofferdams and other existing structures such as the main piers of the Spit Bridge. The average model mesh resolution is detailed in Table 4–1.

Table 4–1: Average model mesh resolution per area

Area	Average element arc length (metres)
Offshore	450
Sydney Harbour entrance	250
Sydney Harbour	140
Middle Harbour	70
Crossing of Middle Harbour	35

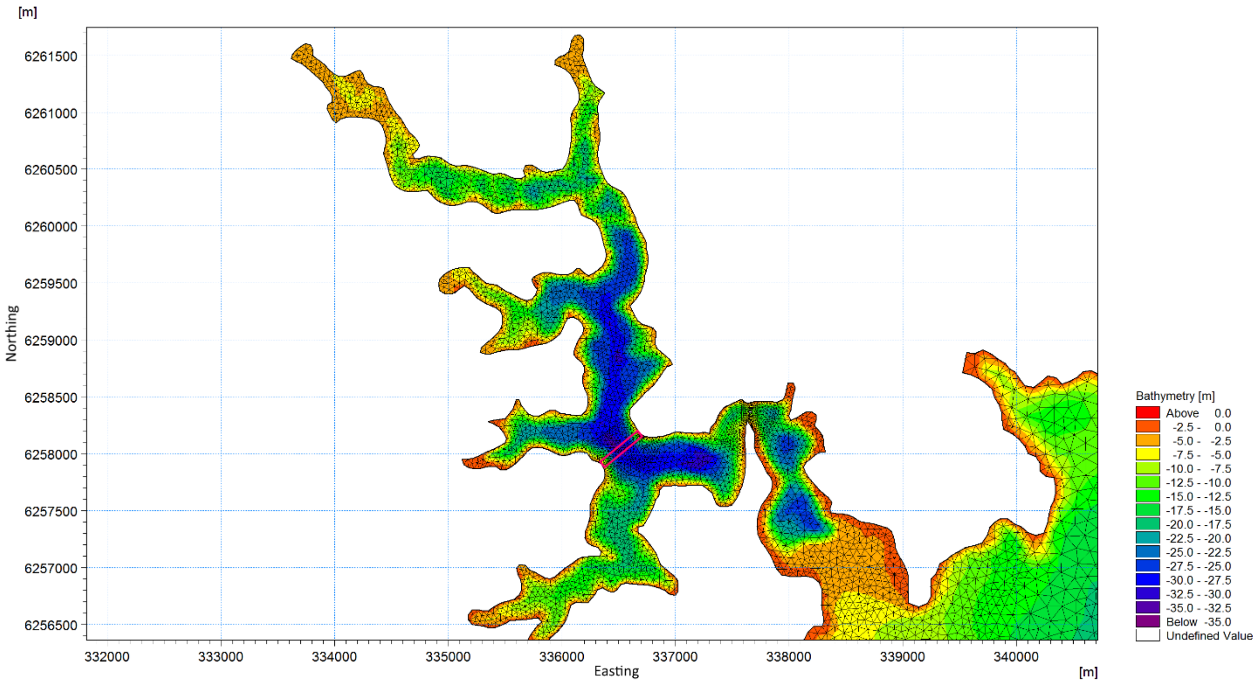

Figure 4-2: Model mesh and bathymetry at the proposed project crossing location (red polygon)

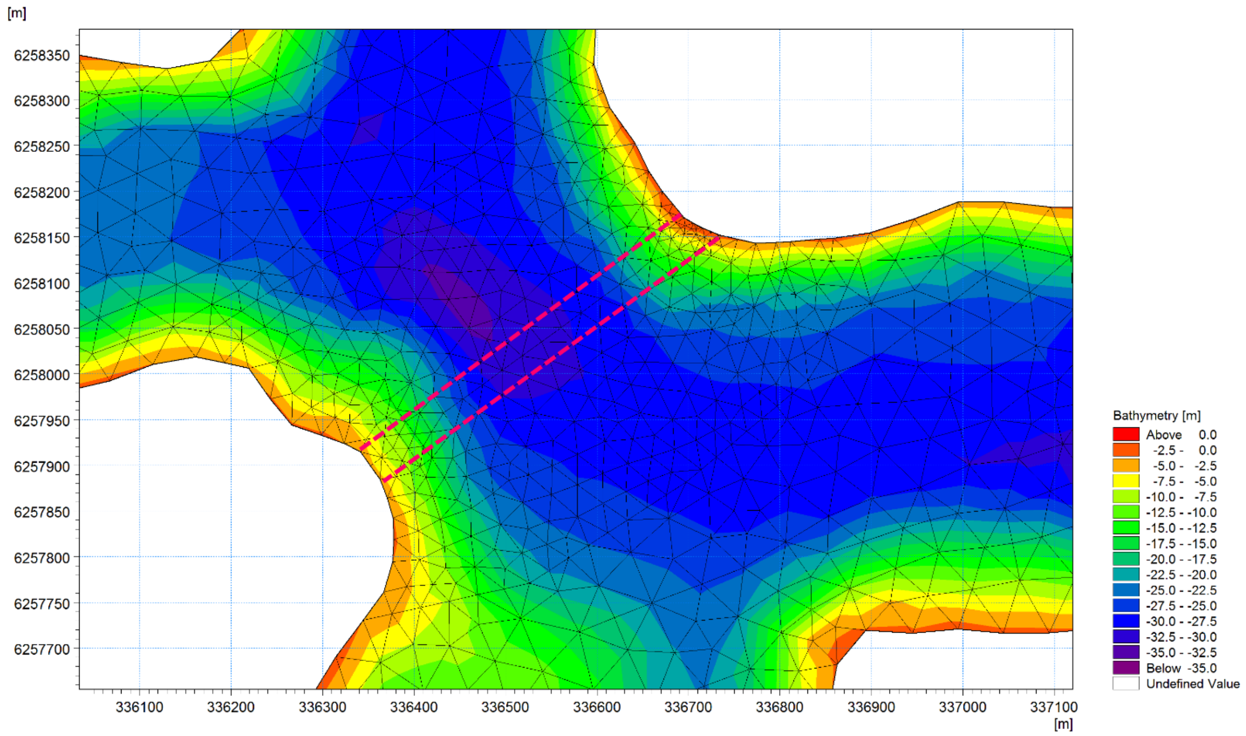

Figure 4-3: Model mesh at the proposed project crossing location (the dashed red line represents the proposed tunnel alignment)

4.2.3 Model boundaries

To define suitable hydrodynamic model boundary conditions, the measured water level data from the Sydney (Live) tide gauge was adopted. A harmonic analysis of a sufficiently long record of the Sydney (Live) measured water level was performed. Subsequently, the resulting tidal constituents were utilised to predict water levels for the required model simulation period. A phase shift was then applied to account for the distance between the model's offshore boundary and the location of the Sydney (Live) tide gauge (about 11 kilometres).

4.2.4 Vertical structure

As outlined in **Section 4.1**, the MIKE 21 (2D) model was upgraded to a 3D model (ie MIKE 3). The vertical structure of the model was varied depending on the modelling application. For model calibration and the main component of the modelling (ie dredge plume modelling) the 3D model used a vertical mesh comprising five sigma layers. Each sigma layer is 20 per cent of the water depth, however layer thickness varies with water level (ie tidal fluctuation). The vertical mesh has a fixed number of layers over the entire model domain. For modelling of operational impacts of the immersed tube related to the creation of a sill-like feature in Middle Harbour the 3D model used a vertical mesh comprising both sigma and z-level layers, as described further in **Section 6.2**.

5 Model calibration

5.1 Introduction

Model calibration is the process of setting physically realistic values for model parameters so that the model reproduces observed values to the desired level of accuracy. The process provides confidence in the model results and is essential for the accurate representation of the coastal hydrodynamics. A calibration exercise is required to demonstrate that the performance of the hydrodynamic model is considered to be representative of the natural environment and is of suitable accuracy to quantify potential impacts due to the project.

Ideally hydrodynamic models should be calibrated against measured water level, discharge and current measurements at a number of locations throughout the model domain. An assessment of the differences between the measured and modelled values should then be carried out to enable the level of calibration achieved to be quantified. The calibration of a hydrodynamic model when tidal forcing dominates should be carried out over a full lunar cycle (about 29 days).

As described in **Section 2.3**, two project specific in-situ monitoring sites were established at the project crossing. Each monitoring site provided continuous measurements of water level and current velocity which were used for model calibration. In addition to the two project specific in-situ monitoring sites, vessel mounted ADCP transects were carried out at two locations (ie MH_Ves1, and WH_Ves2) to measure spatial patterns in current velocities and subsequently calculate discharge. Figure 2-2 displays the in-situ monitoring sites and ADCP transect locations for Middle Harbour.

Tidal variation is the governing physical process for the hydrodynamics of Port Jackson. The model calibration therefore focused on astronomical tide. The approach used to calibrate the hydrodynamic model can be divided into two stages:

- 2D Calibration: The MIKE 21 model was calibrated to measured water level, current and discharge data
- 3D Calibration: The calibrated MIKE 21 model was then converted to a MIKE 3 model with five vertical layers. The calibration was verified by comparing the current speed at in-situ ADCP locations.

The 2D and 3D model calibration is presented below.

5.1.1 Calibration standards

The calibration standards presented in Table 5–1 were adopted for this study based on the recommendation from Williams and Esteves (2017). These standards, which are applicable to estuarine waters, have been used to demonstrate that the model is capable of accurately representing the natural processes.

Table 5–1: Calibration standards adopted for the hydrodynamic model

Model predictions	Root Mean Square Error (RMSE)
Water level	±10% of measured level (spring tide), ±15% of measured level (neap tide)
Water level phase	Timing of high/low water to within ±15 minutes at the mouth of the estuary or ±25 minutes at the head of the estuary
Average current speed	±20% of measured speed
Peak current speed	Within <0.05 m/s (very good), <0.1 m/s (good), <0.2 m/s (moderate) or <0.3 m/s (poor) of the measured peak speed
Current direction	±15 degrees of measured direction
Discharge (Q m^3/s)	±5% (very good), ±10% (good), ±15% (moderate) or >15% (poor) of measured flows

The statistical standards provided in Table 5–1 are a good basis for assessing model performance, however experience has shown that sometimes they can be too prescriptive and it is also necessary for visual checks to be carried out. Under certain conditions, models can meet statistical calibration standards but appear to perform poorly. Conversely, seemingly accurate models can fall short of the guidelines. Accordingly, a combination of both statistical calibration standards and visual checks has been used to ensure that the model is reliably representing the natural processes.

Calibration also included comparison to spatial current patterns which are not specifically mentioned in Table 5–3. Typically, similar levels of agreement (in terms of the relevant percentage and magnitude of the differences presented in Table 5–1) would be expected for these spatial comparisons.

5.2 2D calibration

The MIKE 21 hydrodynamic model was calibrated against measured water level, current speed and direction as well as ADCP transect and discharge data. The model calibration was carried out over a 35 day period from 18 August 2017 to 21 September 2017.

5.2.1 Water levels (tide gauge sites)

The measured water level data at Sydney (Live) and Silverwater Bridge (Parramatta River) underwent post-processing in order to estimate water level variation based on tides only. The modelled and measured tidal water levels over the calibration period are shown in Figure 5-1 and Figure 5-2 and a statistical summary of the comparison is provided in Table 5–2. Given the calibration standards in Table 5–1, in this case the model can be considered to be providing an accurate representation of tidal water levels throughout Port Jackson.

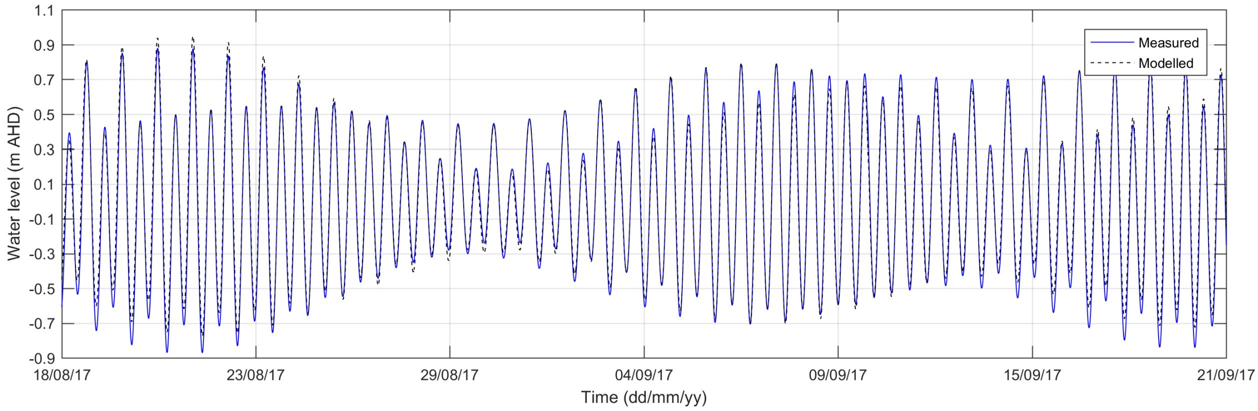

Figure 5-1: Measured and modelled water levels at the Sydney (Live) tide gauge

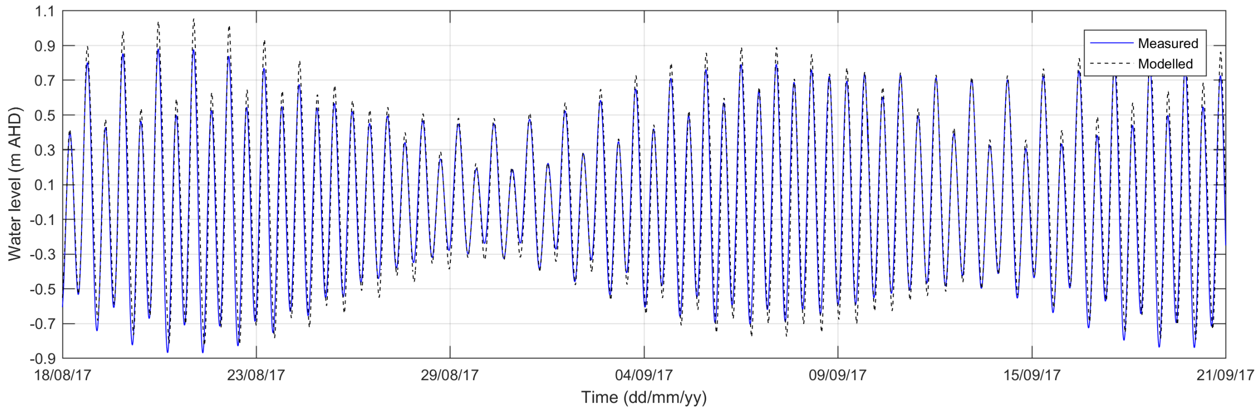

Figure 5-2: Measured and modelled water levels at the Silverwater Bridge (Parramatta River) tide gauge

Table 5–2: Water level calibration statistics at tide gauge locations

Statistical description	Sydney (Live)	Silverwater Bridge
Mean HW difference (metres)	0.01	-0.02
Mean HW difference relative to tidal range (per cent)	1.0	-1.4
Mean LW difference (metres)	-0.02	-0.0
Mean LW difference relative to tidal range (per cent)	-1.7	-0.3
RMSE for HW (metres)	0.02	0.04
RMSE for LW (metres)	0.02	0.03
Mean HW phase lag (minutes)	1.3	3.6
Mean LW phase lag (minutes)	3.1	0.4

Note: HW = High Water LW = Low Water

5.2.2 Water levels (ADCP monitoring sites)

Similar to the calibration analysis performed for the Sydney (Live) and Silverwater Bridge tide gauges, water level data collected by the two ADCP instruments (deployed at MH1 and MH2) underwent post-processing to determine the tide only water level variations, which were then compared to the modelled data. The modelled and measured tidal water levels over the calibration period are shown in Figure 5-3 and Figure 5-4 and a statistical summary of the comparison is provided in Table 5–3.

The MH2 in-situ measurements did not include water level[1], instead the water level at MH1 was used. The MH1 and MH2 sites are located in close proximity and therefore only very minor amplitude or phase differences in water levels are expected.

Table 5–3: Water level calibration statistics at the MH1 and MH2 ADCP in-situ monitoring sites

Statistical description	MH1	MH2
Mean HW difference (metres)	0.0006	0.0007
Mean HW difference relative to tidal range (per cent)	-0.06	-0.05
Mean LW difference (metres)	0.04	0.04
Mean LW difference relative to tidal range (per cent)	3.2	3.1
RMSE for HW (metres)	0.04	0.04
RMSE for LW (metres)	0.06	0.06
Mean HW phase lag (minutes)	-0.4	-0.4
Mean LW phase lag (minutes)	-4.6	-4.6

Note: HW = High Water LW = Low Water
The differences in phase of the high and low waters were derived by subtracting the time of the measured value from the time of the model value. A negative value therefore indicates that the model is early compared to the measured data

With reference to the calibration standards provided in Table 5–1, the following observations can be made with respect to the measured and modelled data.

- The modelled water level accurately represents the measurements at MH1 with phasing less than five minutes and a 3.2 per cent difference in tidal range
- The modelled water level at MH2 performed well compared to the measured water level at MH1, with phasing less than five minutes and 3.1 per cent difference in tidal range.

5.2.3 Current speed and direction (ADCP monitoring sites)

In order to statistically compare the measured ADCP current data against the modelled currents, the collected ADCP data underwent harmonic analysis. The harmonic analysis allowed for the generation of a time series of predicted tidal current velocities which are solely dependent on tidal influences. This process allowed for a direct comparison against the modelled data, whereby the model was driven through tidal forcing only. Due to the weak tidal currents at MH2, harmonic analysis did not result in a good fit and for this site non-tidal influences were removed using a low pass filter. The modelled and measured current speeds and directions at the MH1 and MH2 locations are shown in Figure 5-3 and Figure 5-4 respectively. A statistical summary of the comparison is also provided in Table 5–4.

With reference to the calibration standards provided in Table 5–1, the following observations can be made with respect to the measured and modelled current data.

[1] *This is because of the instrument configuration used at this site. At MH2 the ADCP type instrument was mounted on a buoy looking downward and is therefore not able to measure water level variations (for more details see RHDHV, 2017a).*

- The average difference between modelled and measured peak current speed at MH1 indicated very good model performance (less than 0.01 m/s). It is noted that the model is able to accurately reproduce the significant tidal velocity asymmetry observed at this location. The observed tidal asymmetry occurs due to the flood tide jet that exits the constriction under the Spit Bridge and flows adjacent to the Seaforth shoreline, resulting in higher flood tide current speeds when compared to ebb currents. The root mean square error (RMSE) for peak speed during the ebb and flood tide was 0.03 m/s and 0.02 m/s denoting very good model performance. The model replicated current directions well, with the RMSE for current direction also falling within the calibration standards
- MH2 is located in an area of low current speeds (generally less than 0.1 m/s) and is likely to be susceptible to wind (which was not applied as a model boundary). In addition, when tidal currents are weak they can be variable in direction due to the general lack of momentum in the flow. This is particularly apparent in the measured direction which does not have a distinct tidal character (see Figure 5-4). The mean difference between modelled and measured peak current speed at MH2 indicates good model performance (less than 0.011 m/s). The RMSE for current speed during the ebb and flood tide is 0.03 m/s and 0.02 m/s respectively, which is within the calibration standards. Additionally, RMSE for current direction during the ebb tide was within the calibration standards, with 13.7 degrees. However, due to the very low currents during the flood tide (and potentially the effects of wind) the measured direction was too variable to allow a meaningful comparison (see bottom panel of Figure 5-4).

In summary, the modelled currents for the two calibration sites closely matched the measured ADCP data. The differences in the measured and modelled data fell within the calibration standards for both speed and direction, as noted in Table 5–1.

Table 5–4: Current speed and direction calibration statistics at the MH1 and MH2 ADCP in-situ monitoring sites

Statistical description	MH1	MH2
Mean difference in speed of flood (m/s)	-0.008	-0.009
Mean difference in flood speed relative to maximum observed speed (per cent)	-1.8	-9.2
Mean difference in speed of ebb (m/s)	0.014	0.011
Mean difference in ebb speed relative to maximum observed speed (per cent)	6.6	7.6
RMSE for flood speed (metres)	0.03	0.02
RMSE for ebb speed (metres)	0.02	0.03
RMSE for direction of flood (degrees)	6.0	variable
RMSE for direction of ebb (degrees)	1.3	13.7

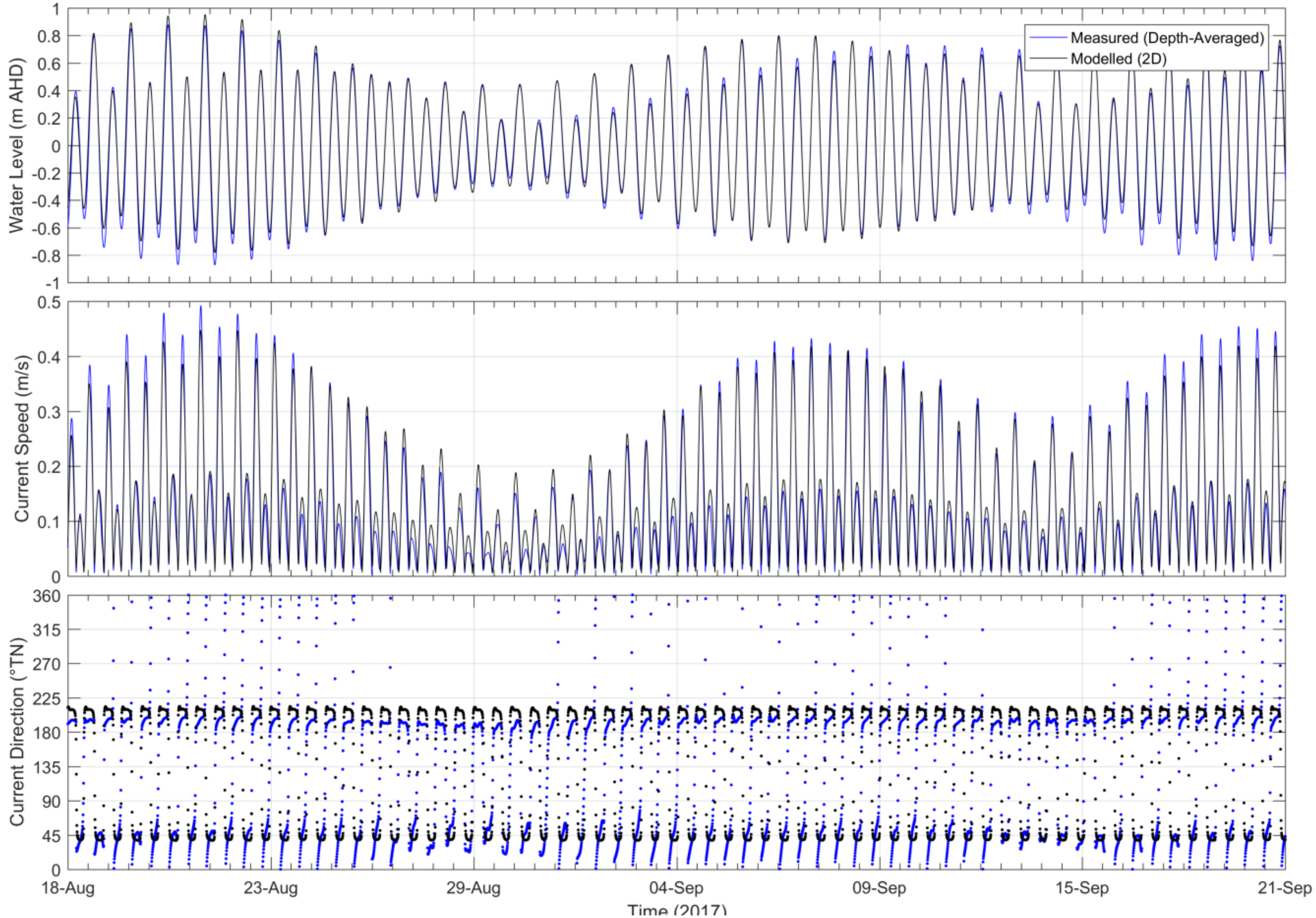

Figure 5-3: Measured and modelled water levels, current speed and current direction at MH1

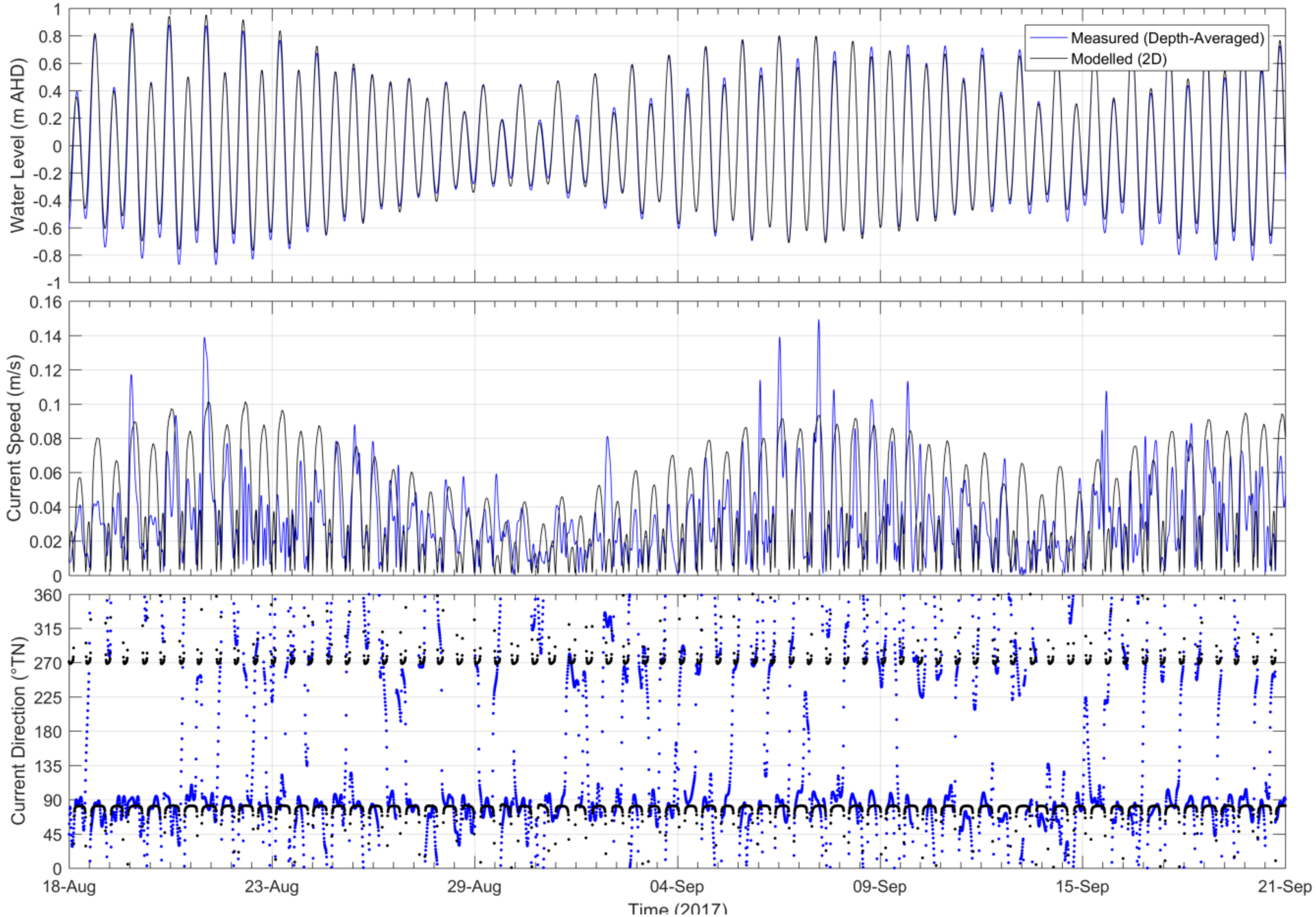

Figure 5-4: Measured and modelled water levels, current speed and current direction at MH2

5.2.4 Discharges and velocity transects (ADCP vessel transects)

In addition to in-situ monitoring, two predefined ADCP vessel transects were carried out between nine and 11 times throughout the tidal cycle on 22 August 2017. This exercise provided measurements of current velocities along the predefined transects shown in Figure 2-2. From these velocity transects, tidal discharge (m^3/s) was calculated. The modelled and measured discharges throughout the tidal cycle at MH_Ves1 and MH_Ves2 on 22 August 2017 are shown in Figure 5-5 and Figure 5-6. The measured point discharges (blue dots) align well with the modelled discharge at MH_Ves1 and MH_Ves2.

Figure 5-7 to Figure 5-10 display the measured and modelled current speed and direction for transects at MH_Ves1 and MH_Ves2 during the peak ebb and flood stages of the tide.

There is good agreement between modelled and measured for both current speed and direction across these transects indicating that the model performs well at representing the observed spatial current patterns at the crossing locations. Of particular note is that the model flood current speeds at MH_Ves1 do identify the strong flood flows in the main channel and the return eddy east of the main channel (with lower speeds than in the main channel). However, current speeds in the main channel are underestimated (refer Figure 5-8). Some variability exists at MH_Ves2, for example, the difference between modelled and measured current direction observed next to Northbridge during the ebb tide. However, it is noted that current speeds at MH_Ves2 are very low (about 0.05 m/s).

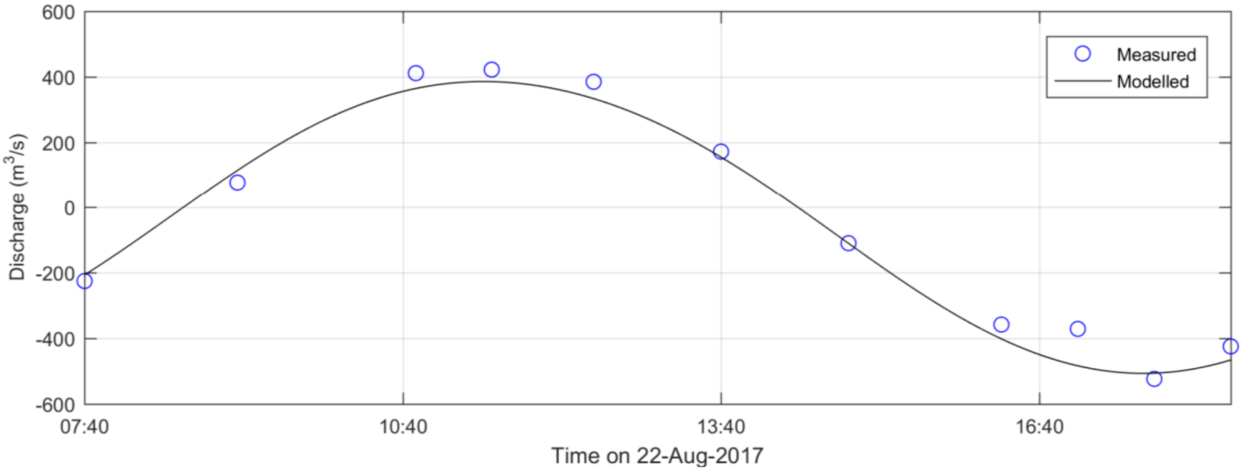

Figure 5-5: Measured and modelled discharge volumes at transect MH1

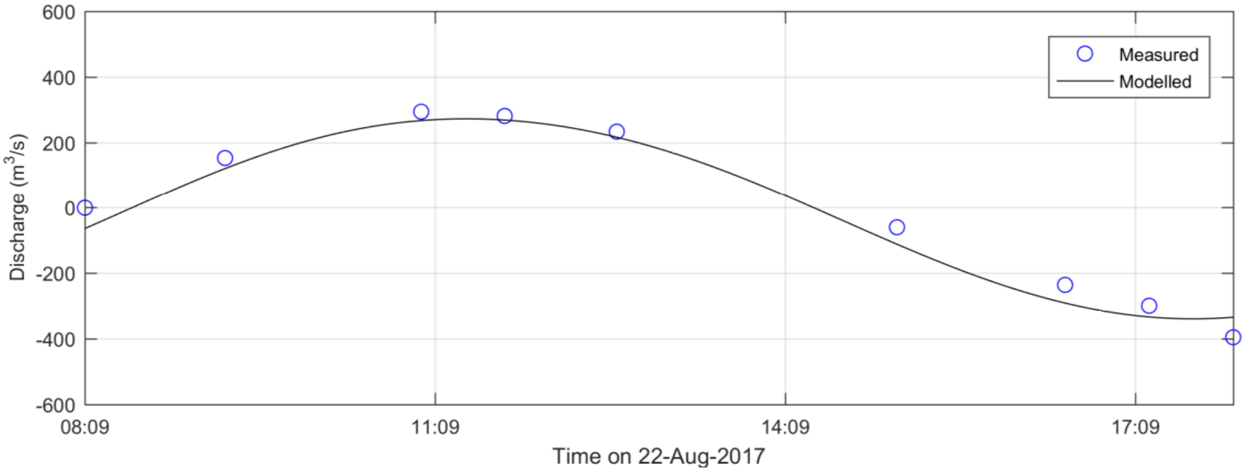

Figure 5-6: Measured and modelled discharge volumes at transect MH2

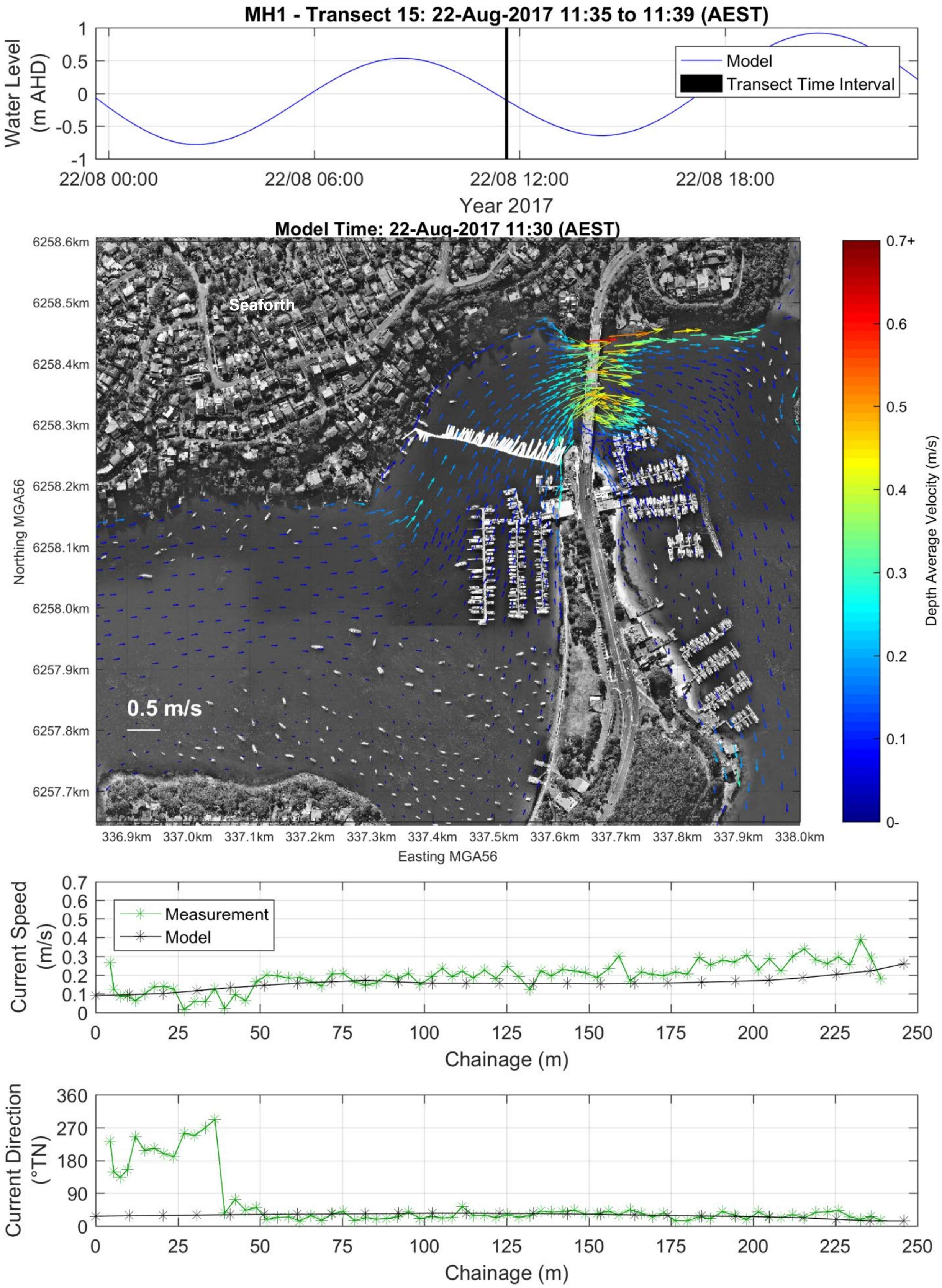

Figure 5-7: Measured and modelled current speed and direction at transect MH1 (ebb)

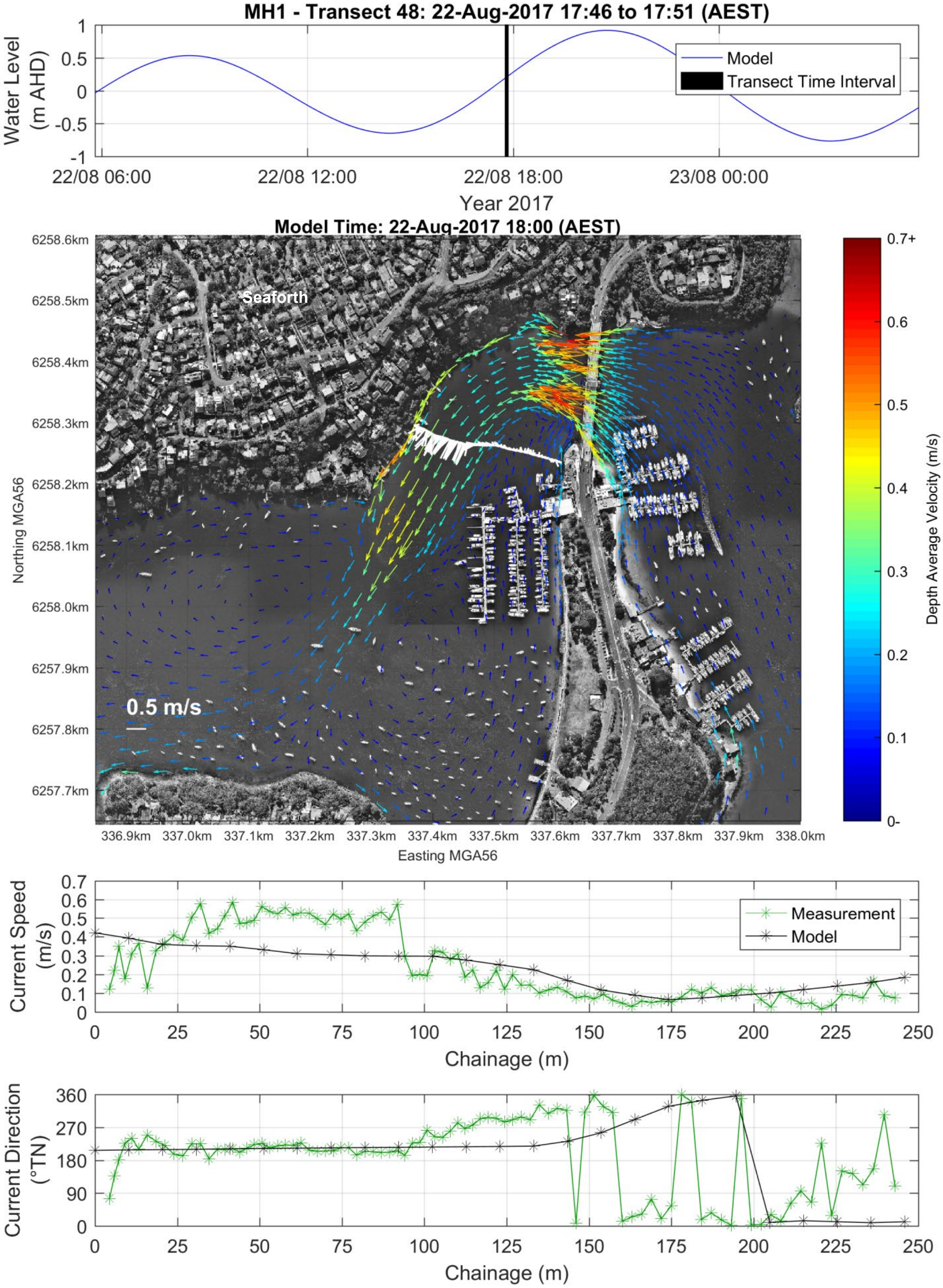

Figure 5-8: Measured and modelled current speed and direction at transect MH1 (flood)

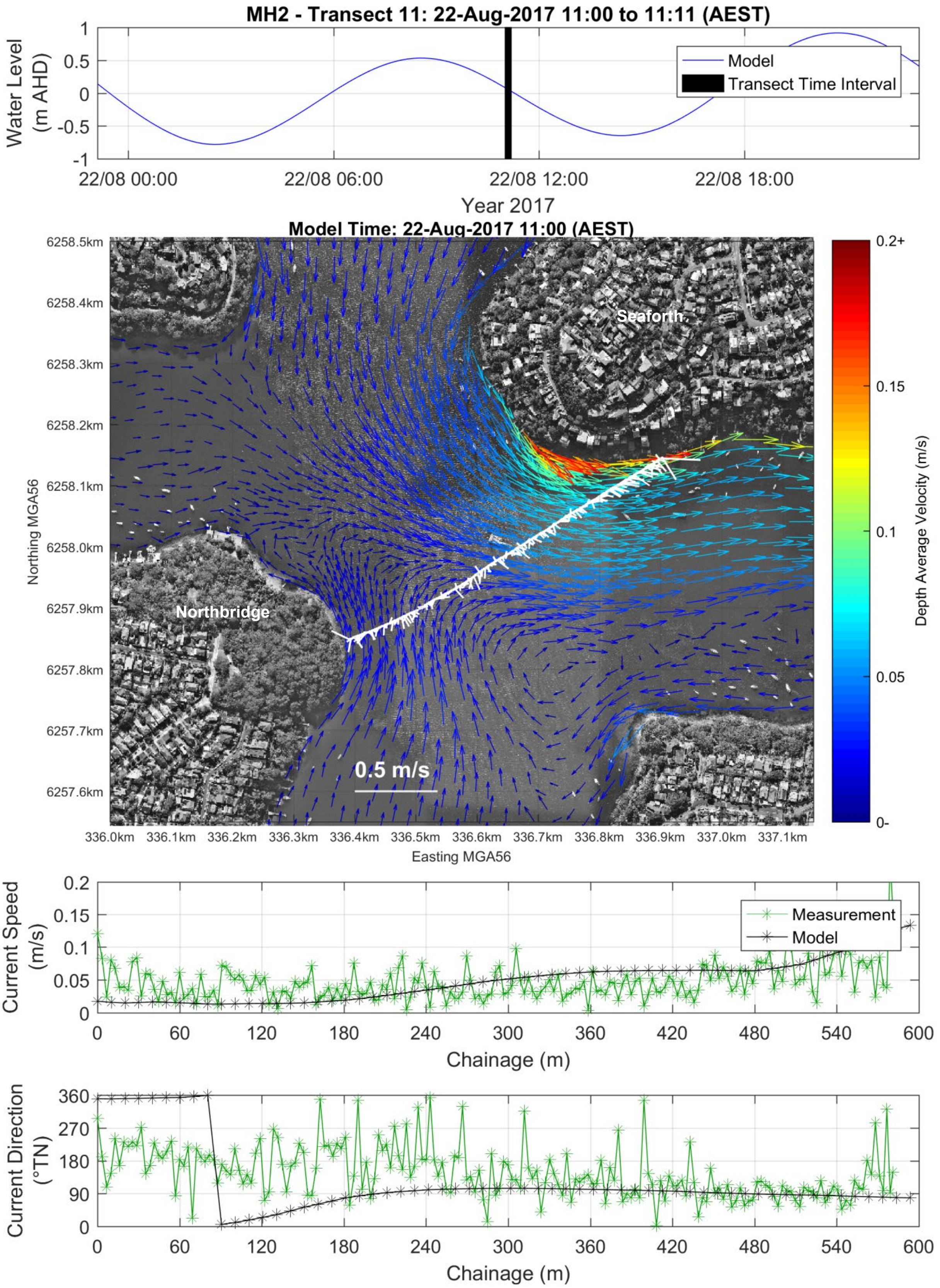

Figure 5-9: Measured and modelled current speed and direction at transect MH2 (ebb)

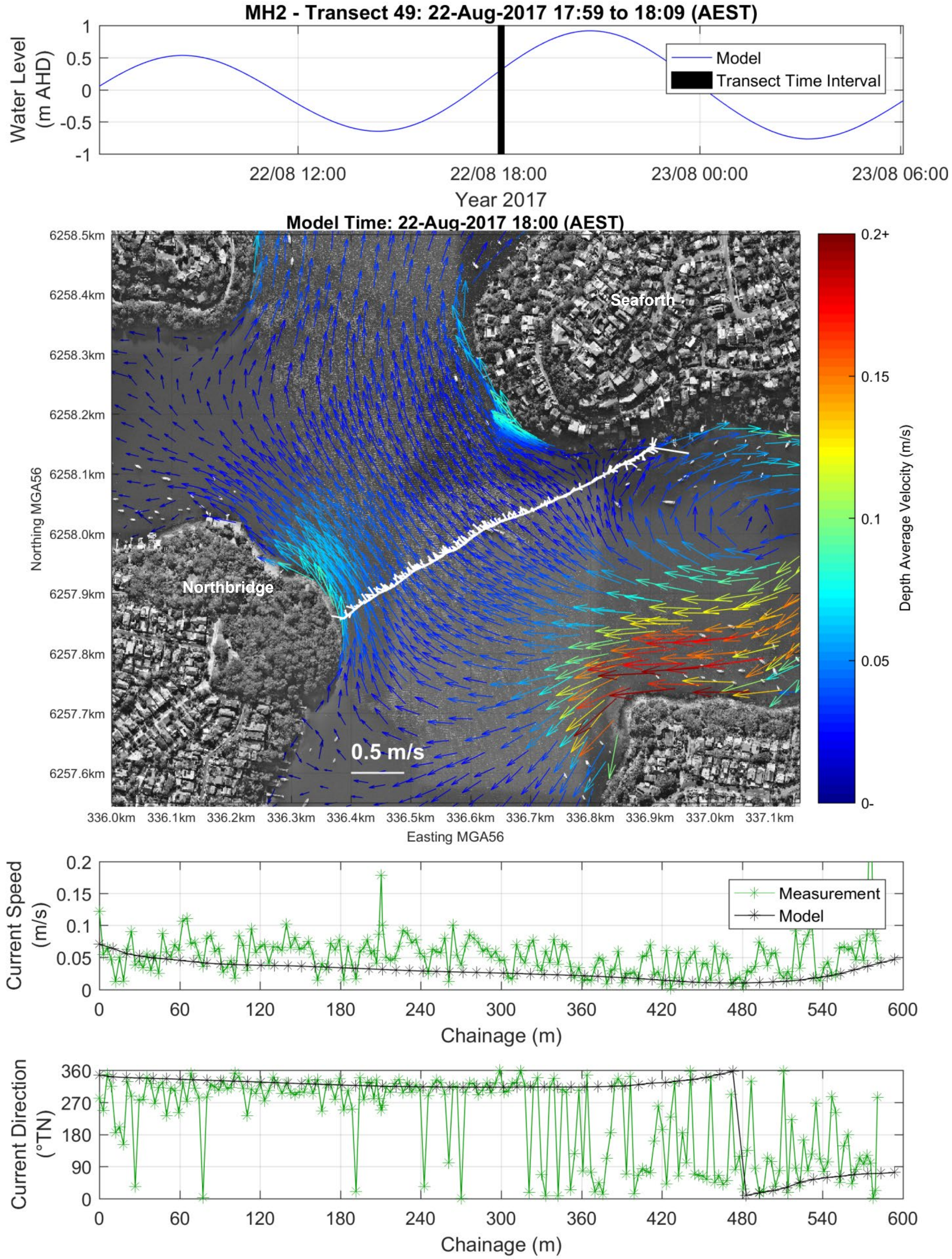

Figure 5-10: Measured and modelled current speed and direction at transect MH2 (flood)

5.3 3D calibration

The calibrated 2D model was converted into a 3D model by creating a vertical mesh comprising five sigma layers (refer to **Section 4.2.4**). The 3D model was then further calibrated against the in-situ monitoring data at MH1 and MH2. Figure 5-11 and Figure 5-12: display the measured and modelled current speed and direction at the surface, middle and bottom of the water column from 18 August 2017 to 21 September 2017.

Regarding MH1, measured current speeds were uniform across the depth profile and modelled current speeds agreed well with this. Modelled bottom layer currents were slightly underestimated which is likely attributed to height difference of the selected layers (the centre of the bottom layer of the model is closer to the actual bottom than the centre of the first ADCP measurement). Overall modelled current direction shows good agreement across the depth profile.

Regarding MH2, measured currents were not uniform along the depth profile. As mentioned in **Section 5.2**, measured currents at MH2 are likely to have been influenced by wind, which is especially apparent in the surface layer where modelled current speeds are underestimated and measured direction lack a clear (tidal) pattern. Current speed and direction for the middle and bottom layer show good agreement with measured data.

As noted above, harmonic analysis was applied to measured currents at MH1 and a low pass filter to measured currents at MH2 (all layers) in order to eliminate higher-frequency oscillations in current speed.

Following 2D and 3D model calibration the hydrodynamic model was considered appropriately calibrated and fit for application to the prediction of hydrodynamic impacts during the construction phase (see **Section 6**) and for dredge plume modelling (see **Section 7**).

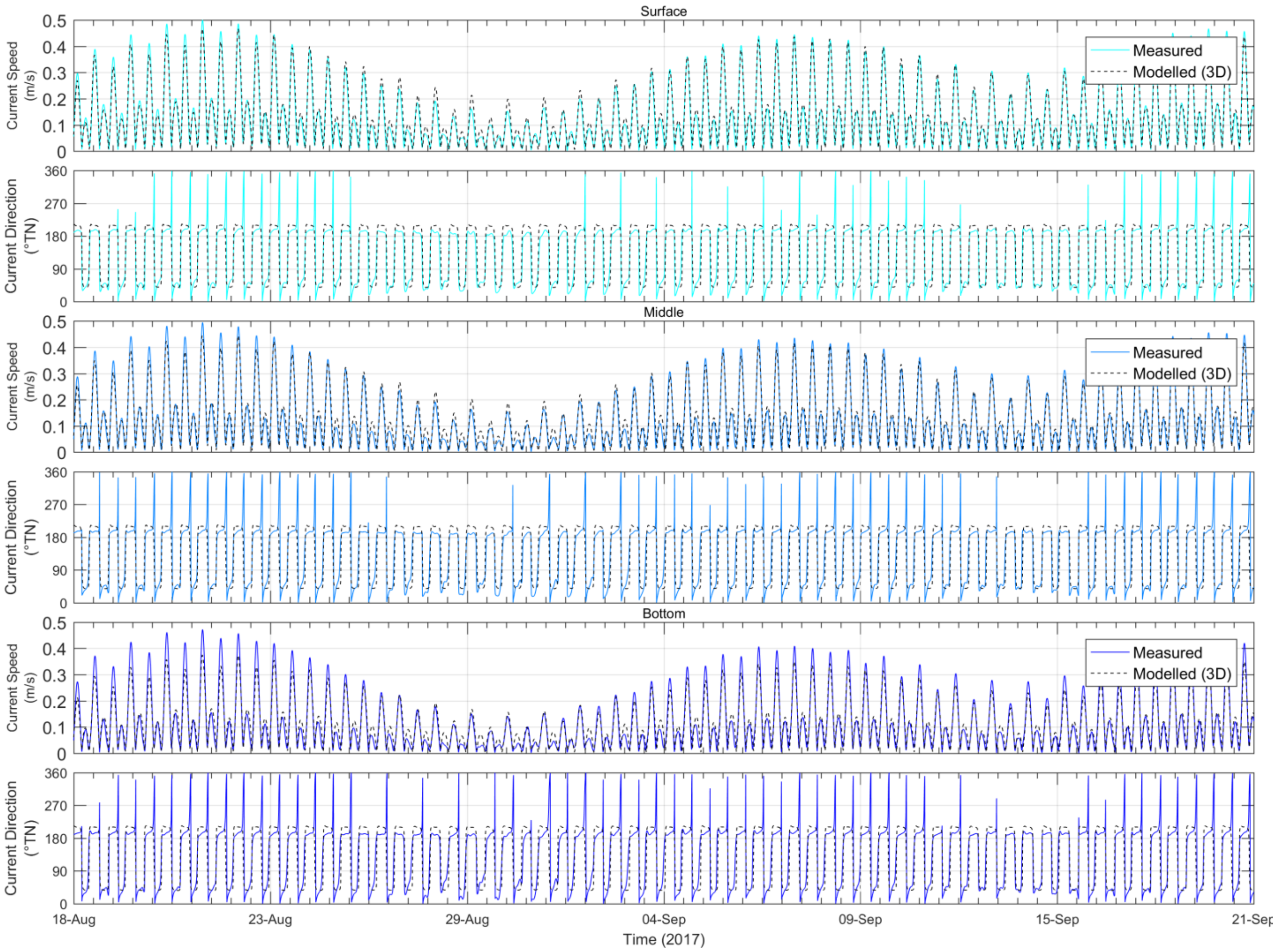

Figure 5-11: Measured and modelled current speed and direction at the surface, middle and bottom of the water column at MH1

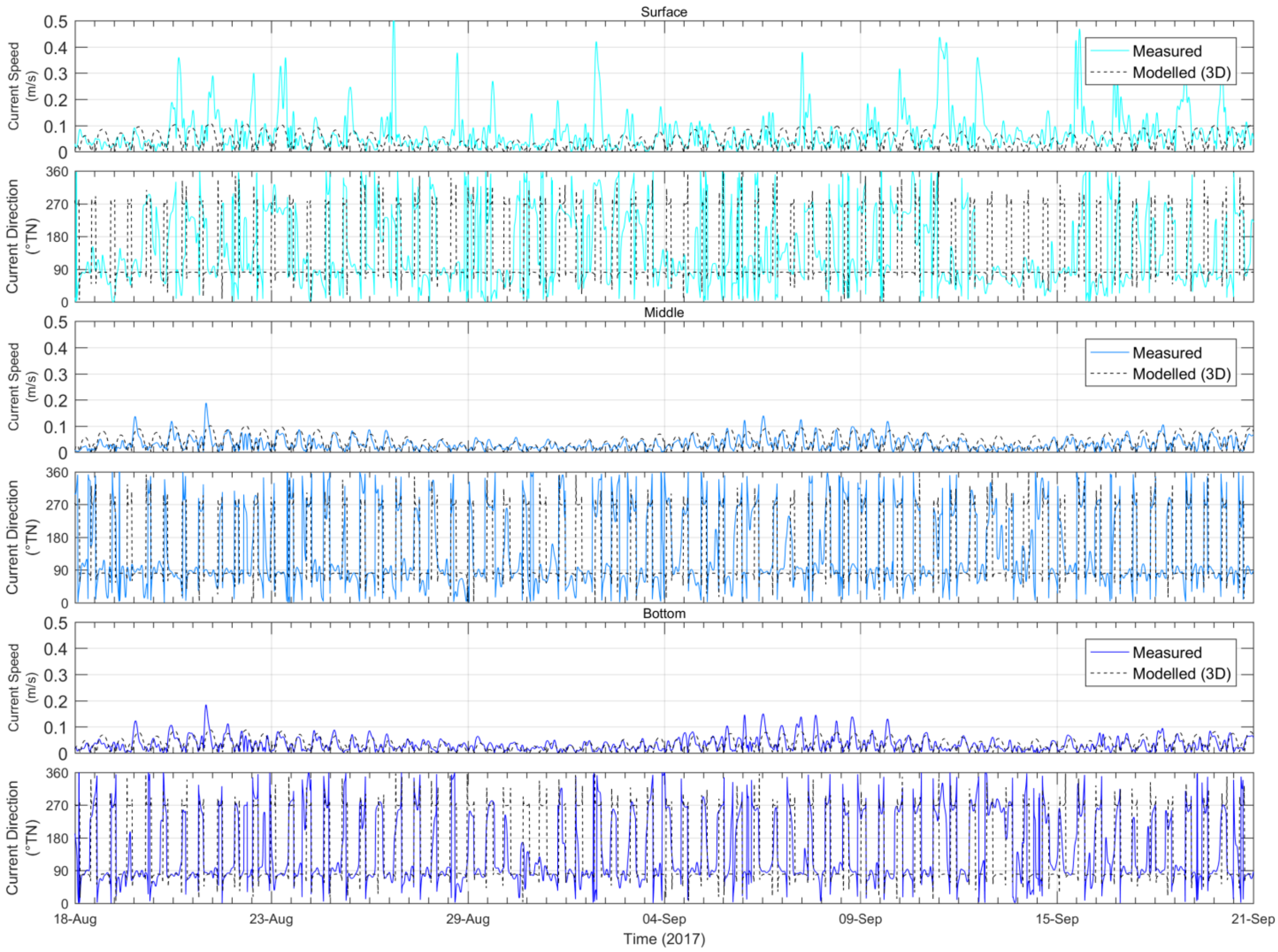

Figure 5-12: Measured and modelled current speed and direction at the surface, middle and bottom of the water column at MH2

5.4 Wind sensitivity testing

The importance of the effect of wind on current circulation within the model was assessed. Predominant wind directions (see Figure 3-5) for the area were determined during the wind analysis in **Section 3.3.3** with strong winds coming from east, west and south throughout the year.

Sensitivity testing was carried out using the calibrated model to identify the wind directions that would most affect circulation within the model and the magnitude of wind driven currents in the areas of interest. These wind only simulations were run with static wind speeds of 10 m/s from 16 cardinal directions. A wind speed of 10 m/s is about 20 knots and would be classified as a ‘fresh breeze’ in the Beaufort wind force scale. Figure 5-13 to Figure 5-16 display the modelled current speeds and vectors during four of these sensitivity tests (north, east, south and west). The results presented in these figures were used to identify which wind directions had the greatest impact on circulation.

The testing indicated that wind driven circulation was most pronounced when winds blow from the north-east to south-west. The magnitude of the currents in the area of interest can be seen in the current pattern plots in Figure 5-13 to Figure 5-16.

Based on the results of the wind circulation simulations, summer was identified as being most representative of the season that would result in the most wind driven circulation in the areas of interest. Summer also has the strongest wind speeds. From the 27 year Fort Denison dataset, a 16 week time period was selected that was representative of typical summer conditions. The period from November 2010 to February 2011 was found to have similar wind speed percentiles (see Table 5–5) and directional distribution to the average summer conditions. This representative wind time-series was applied to the model to test the sensitivity of the dredge plumes to wind. The results of the plume sensitivity to wind are presented in Section 7.4.2.

The question can arise as to whether localised intense weather (wind) impacts should be considered in the modelling. Winds associated with intense weather events, such as gales or violent storms for example, are generally short term and gusty; the more relevant winds for assessing the effects on plumes are longer prevailing steady winds (as modelled) which can generate a steady surface current.

In addition, when winds become ‘intense’ and exceed a threshold magnitude, activities such as dredging operations, barge transport of dredged material, and unloading operations, would cease due to workability, safety risk or environmental risk, in accordance with a dredge management plan which would form part of the construction environmental management plan. That is, dredging would be unlikely to be carried out during ‘intense’ wind events.

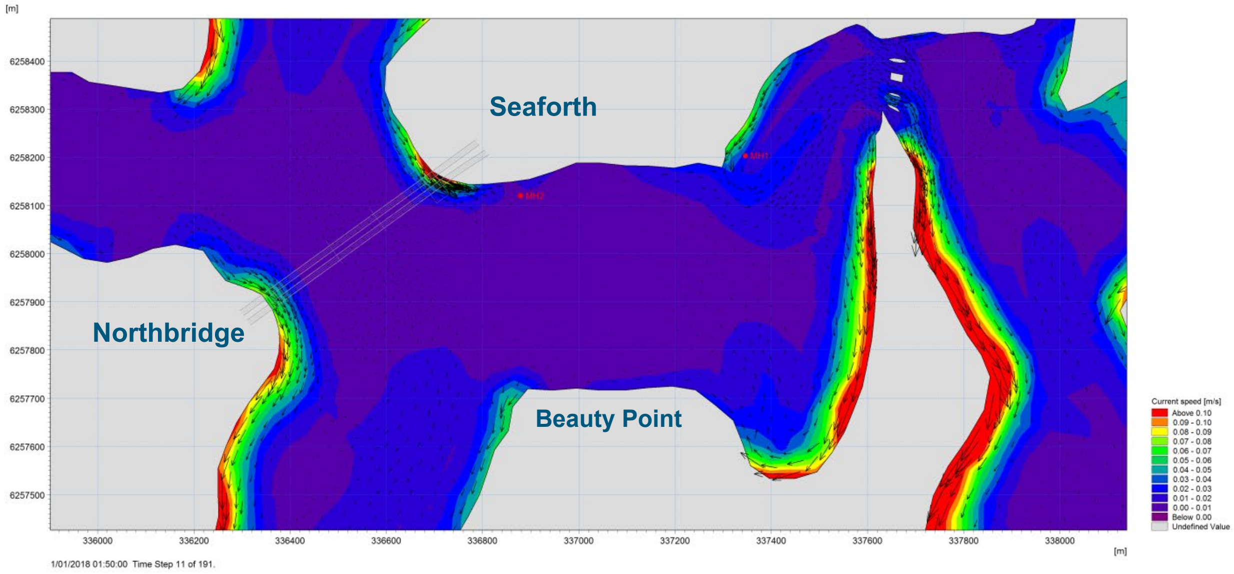

Figure 5-13: Modelled water current speed and direction during sensitivity testing with static winds from the north at 10 m/s

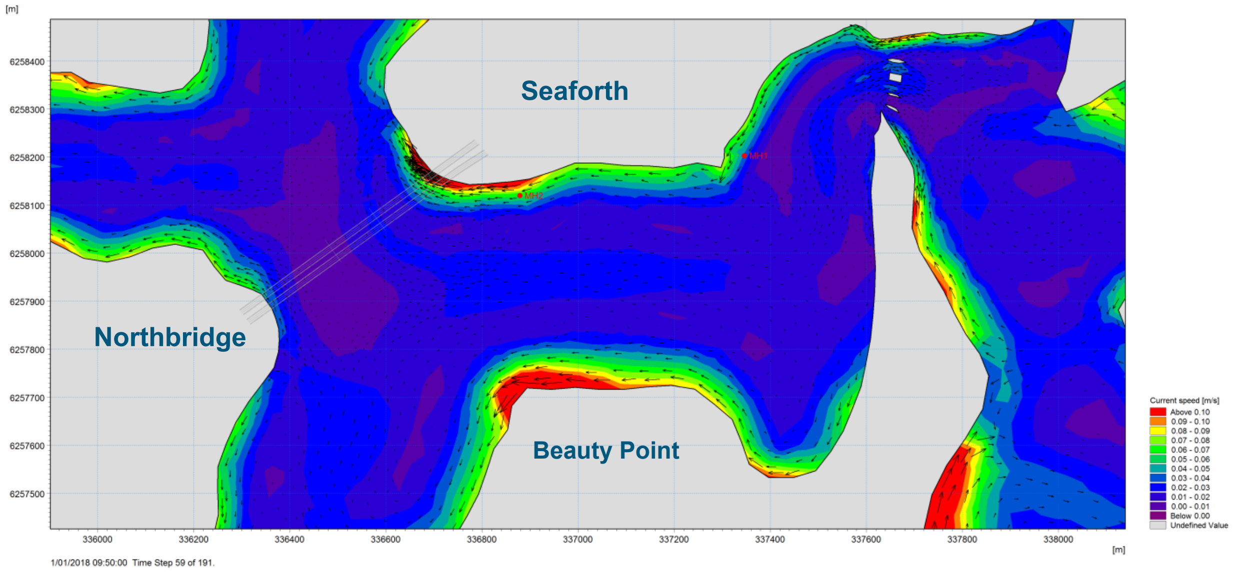

Figure 5-14: Modelled water current speed and direction during sensitivity testing with static winds from the east at 10 m/s

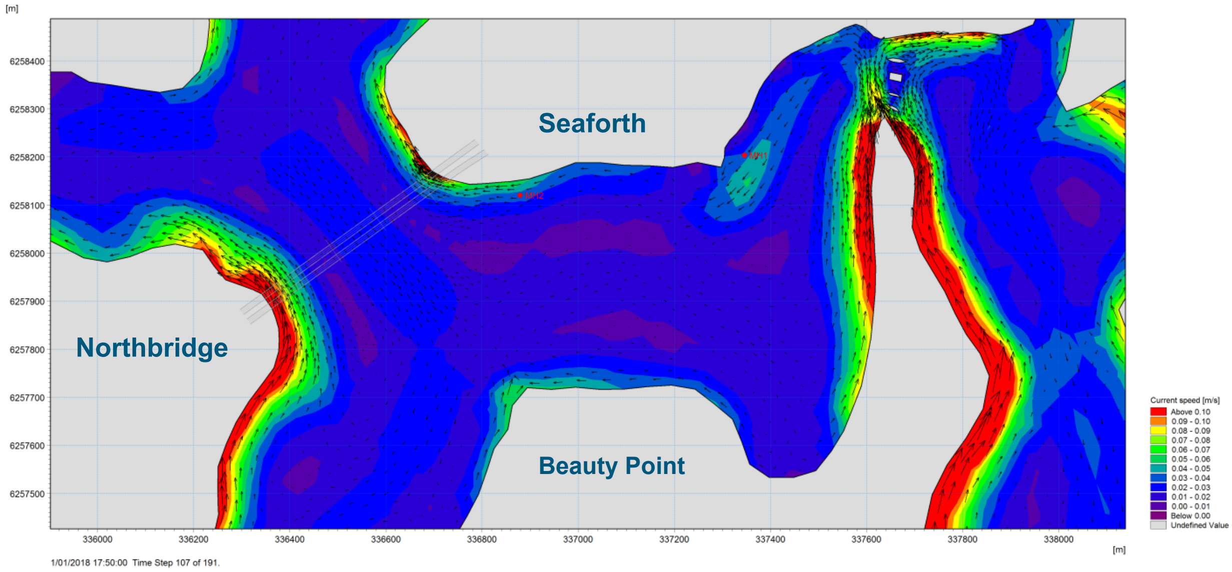

Figure 5-15: Modelled water current speed and direction during sensitivity testing with static winds from the south at 10 m/s

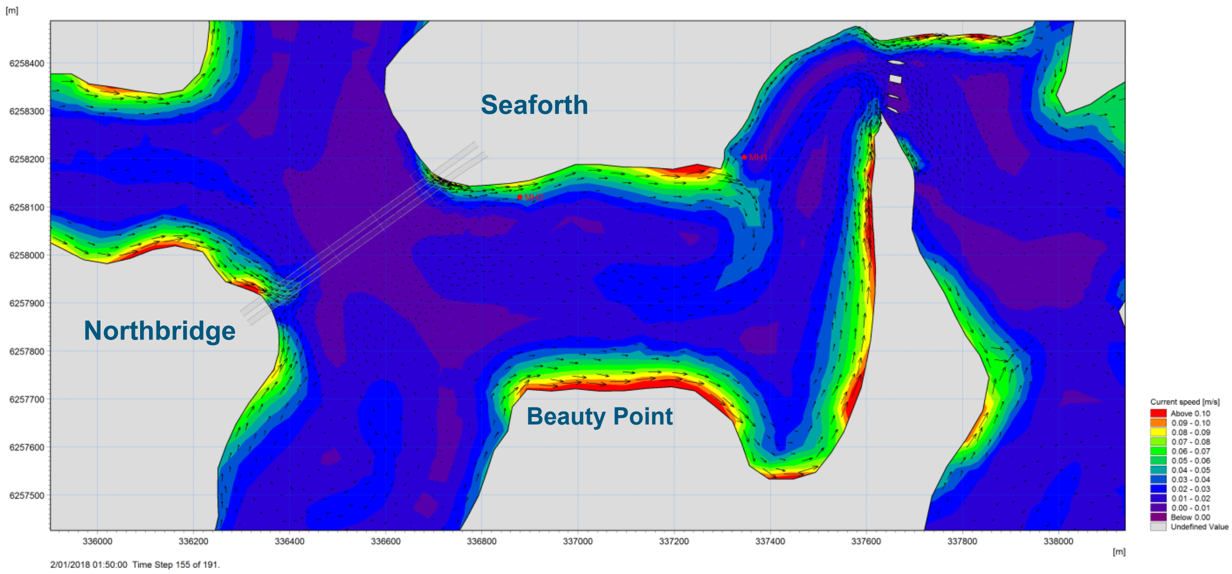

Figure 5-16: Modelled water current speed and direction during sensitivity testing with static winds from the west at 10 m/s

Table 5–5: Wind statistics for Fort Denison for 27 year data set and selected representative period

Period	Month	50th Percentile wind speed (m/s)	90th Percentile wind speed (m/s)
2010	November	4.7	7.8
2010	December	4.7	8.3
2011	January	4.7	7.8
2011	February	4.2	7.8
1990-2017	November	4.7	8.3
1990-2017	December	4.7	8.3
1990-2017	January	4.7	7.8
1990-2017	February	4.2	7.8

6 Hydrodynamics Impacts

6.1 Construction impacts

6.1.1 Overview

During the construction period two temporary cofferdams, Middle Harbour north cofferdam (BL8) and Middle Harbour south cofferdam (BL7), would be required at the connection points on either side of the crossing (see Figure 6-1). The cofferdams would be constructed through the full depth of the water. The Middle Harbour north cofferdam (BL8) would be located in water depths of about two to 13 metres and would be designed as two overlapping rectangles with a maximum length of 62 metres and maximum width of 34 metres. The Middle Harbour south cofferdam (BL7) would be located in water depths of about seven to 15 metres and would have a rectangular design with a length of 62 metres and width of 25 metres. The cofferdams would be constructed using steel tubular piles which act as a complete barrier to the flow of water. The cofferdams are expected to be in place for about 19 months. While in place, the cofferdams would influence the hydrodynamics within Middle Harbour.

Deep draft silt curtains would be placed around the cofferdams and adjacent dredging activities. Silt curtains are flexible, typically water permeable (and sometimes impermeable) barriers that act to prevent the dispersion of fine grained sediment suspended in the water column. For the project they have been designed to contain sediment suspended during cofferdam piling and dredging activities with a configuration shown in Figure 6-1. The silt curtains would be designed with a draft of 12 metres to maximise containment of fine grained sediment. The deep draft silt curtains would be in place for the duration of the cofferdam piling and adjacent dredging activities. The two deep draft silt curtains would supplement the use of shallow draft silt curtains (ie shallow draft silt curtains about two to three metres deep, sometimes referred to as a “moon pool”) that would be located around select piling and dredging plant and around ecologically sensitive areas (eg. nearby seagrass and rocky reef habitat) to provide additional protection.

The use of deep draft silt curtains at the project crossing is considered feasible because of the low current speed environment in this reach of Middle Harbour. As noted in **Section 3.3.2** the relatively deep channel in this area means tidal flows are less than 0.2 m/s. Deep draft silt curtains cannot be used in high flow environments due to difficulties restraining the curtains. Due to the depth of the deep draft silt curtains, while in place they would influence the hydrodynamics.

As outlined in **Section 4.2** the model mesh was refined around the project location to ensure that the cofferdams and deep draft silt curtains could be accurately represented in the model. The 3D hydrodynamic model, with five vertical sigma layers, was run for a period of about five weeks. The hydrodynamic modelling was then completed for the existing conditions and the design conditions:

- Base case (existing conditions scenario)
- Design scenario which included the two cofferdams in place along with the deep draft silt curtains. This is based on the project construction period scenario and incorporates the Middle Harbour north (BL8) and Middle Harbour south (BL7) cofferdams and silt curtains as per the project’s design.

To represent the cofferdams within the model, their spatial footprint was removed from the model bathymetry so that no flow was allowed to pass through. All other areas are the same as the existing conditions scenario. The silt curtains were represented within the model as being suspended within the water column and having a depth of 12 metres.

The locations and layout of the cofferdams and adjacent dredging activities were based on the project design provided by Transport for NSW, while the location and layout of the deep draft silt curtains (as shown in Figure 6-1) was based on information supplied by a dredging expert engaged by RHDHV.

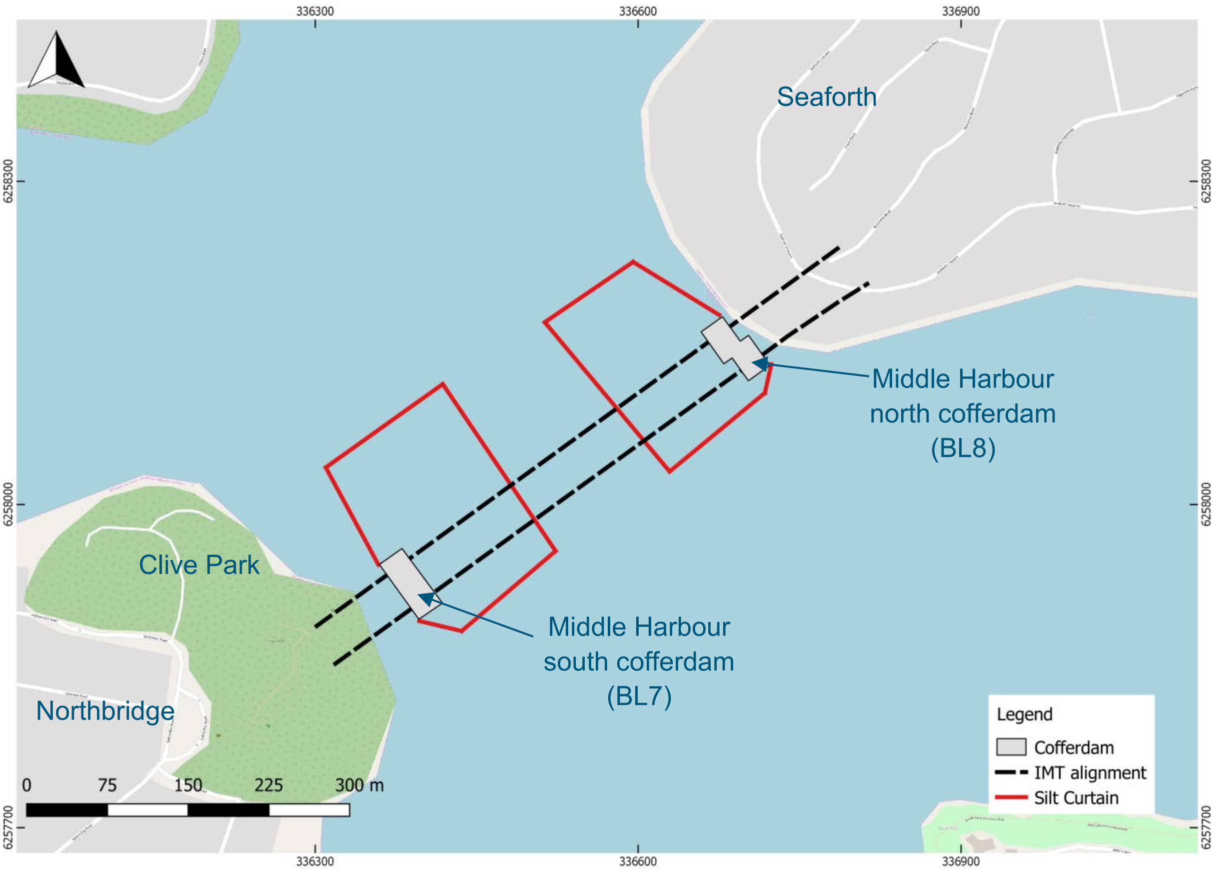

Figure 6-1: Cofferdams and deep draft silt curtains layout during construction at the project crossing

The impact of the Spit West Reserve construction support site (BL9) on hydrodynamics was also assessed. The model grid was refined around the area of proposed temporary structures at the Spit West Reserve construction support site (BL9). The 3D hydrodynamic model, with five vertical sigma layers, was run for a period of two weeks. The model was then run for two scenarios:

- Base case with refined grid around Spit West Reserve (existing conditions scenario)
- Inclusion of temporary structures (construction scenario at Spit West Reserve).

The location and layout of the marine aspects of the Spit West Reserve construction support site (BL9) was based on Figure 6-2 as well as subsequent clarifications received from Transport for NSW. With reference to Figure 6-2, the following structures were implemented into the hydrodynamic model for the construction scenario at Spit West Reserve:

1. Maximum of two immersed tube tunnel units with a draft of eight metres (draft based on maximum potential draft during concrete casting)
2. Floating access pontoons (about 18 metres by 110 metres) with a draft of two metres
3. Two short groynes that are proposed to be formed as part of the hinged access ramps.

The immersed tube tunnel units and access pontoons were represented in the model as floating structures with their respective drafts of eight metres and two metres. The areas of the groynes were removed from the model so that no flow was allowed to pass through these areas.

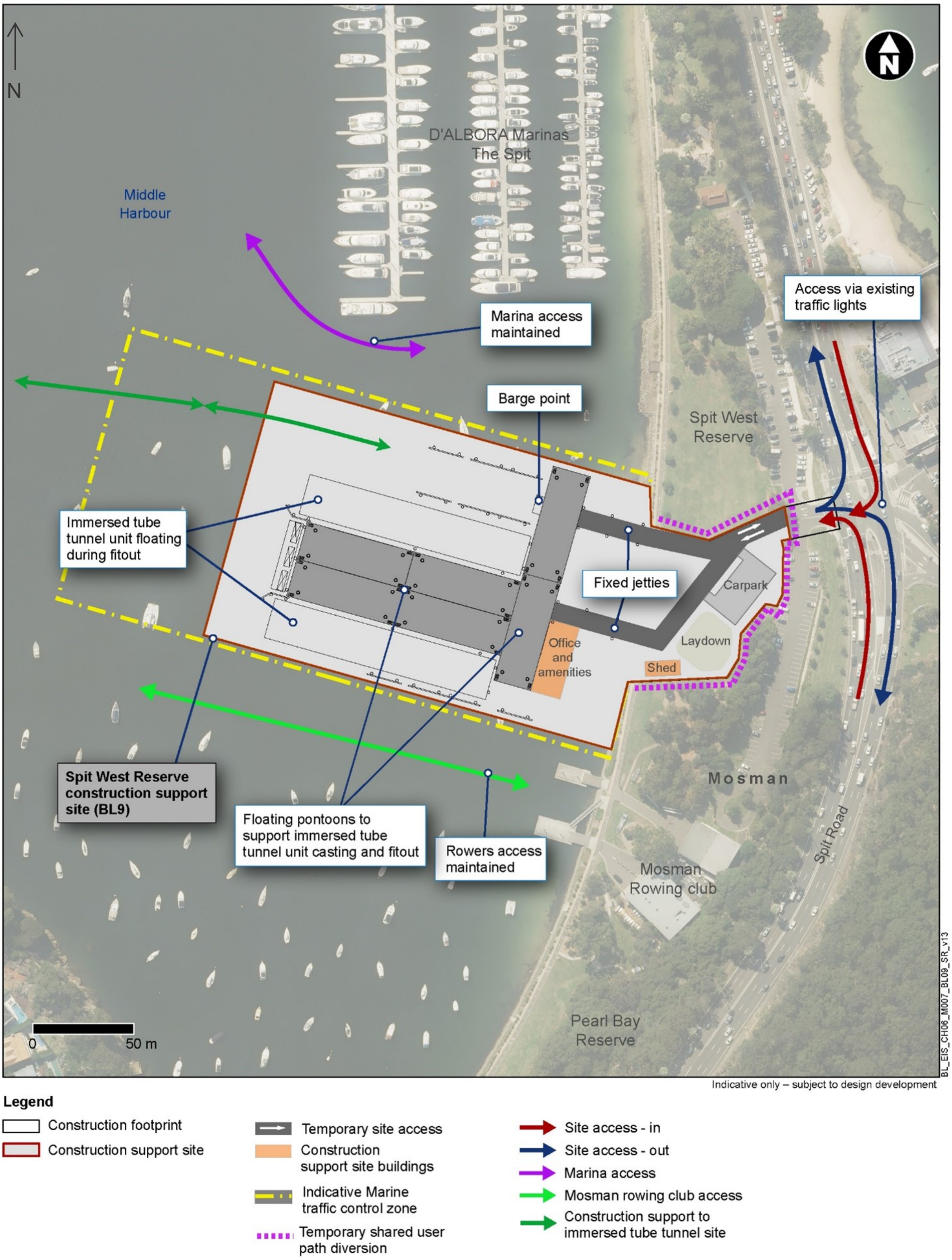

Figure 6-2: Spit West Reserve construction support site (BL9) with elements schematised into the construction scenario hydrodynamic model (source: Transport for NSW)

6.1.2 Temporary construction site at cofferdams and silt curtains

Tidal current speed and patterns at the surface and bed of the harbour during the peak ebb and flood for the existing (base case) conditions are shown in Figure 6-3 to Figure 6-8. These figures also show spatial plots of the difference in current speeds due to the silt curtains and cofferdams. In regard to these plots it is noted that:

- Current speed differences (shown as colours) compare the base case to the construction scenario. Green shows an increase in current speed due to the cofferdams while blue shows a decrease
- Current vectors shown are based on the peak speeds from the base case scenario model.

Surface currents during the ebb tide are slow with current speeds of 0.08 m/s observed in the middle of the channel between Clive Park and Seaforth Bluff. Even slower current speeds less than 0.04 m/s are observed at the proposed location of the Middle Harbour south cofferdam (BL7) while higher current speeds of 0.12 m/s to 0.20 m/s are observed at the Middle Harbour north cofferdam (BL8). During the flood current speeds in the middle of the channel are slightly lower than the ebb tide and remain low (less than 0.08 m/s) at the two cofferdam locations.

During the peak ebb tide, the Middle Harbour north cofferdam (BL8) and accompanying silt curtain reduced current speeds around Seaforth Bluff (at all depths) in a downstream direction. Current speed increased in the middle of the channel and bed of the harbour (ie beneath the silt curtain). Additionally, the Middle Harbour south cofferdam (BL7) and silt curtain cause an increase in current speeds between the temporary structures and the bank (near Clive Park) at the surface and in the middle of the water column.

During the flood tide, decreases in current speed were predicted at both the cofferdams as well as within and surrounding the silt curtains. Additionally, at the Middle Harbour north cofferdam (BL8) decreases in current speed were observed upstream of the structure along Seaforth Bluff. At the Middle Harbour south cofferdam (BL7), an increase in current speed was predicted along the bank upstream of the structure in the surface and middle layer.

During both ebb and flood tide the current reductions are more pronounced in the surface layer due to the effect of the silt curtains on the upper water column.

Comparative time series tidal current speeds for the base case and construction scenario were produced to assess the relative impact of the cofferdams and associated silt curtains. Figure 6-9 and Figure 6-10 display the time series comparisons for two locations, next to the cofferdams. The locations where results have been extracted from the model are shown in the current speed difference plots (eg Figure 6-3). The time series indicated that at the Middle Harbour north cofferdam (BL8), current speeds are reduced at the surface, middle of the water column and bed of the harbour. At the Middle Harbour south cofferdam (BL7) current speeds are reduced at the surface and middle of the water column, while at the bed of the harbour current speeds remain relatively unchanged. The plots show the relative magnitude of the current speed differences, which is higher at the surface. For example, at the north location at the bed of the harbour during the spring flood, current speeds decrease from 0.07 m/s to around 0.06 m/s, a decrease of 18 per cent.

The modelled current speeds shown in plots and described above highlight that the proposed crossing site is located in a low energy hydrodynamic environment. Therefore little to no bedload transport or resuspension of existing bed sediment is expected to naturally occur where the temporary cofferdams and silt curtains would be introduced. The geotechnical data shows that up to 30 metres of predominantly fine grained sediment is present in the centre of the channel above the bedrock (Arup WSP, 2017). From the information available it can be inferred that the centre of the channel is a depositional environment and little to no transport of bed sediment would be expected to occur in the area. Moreover, the localised increases in near bed current speeds due to the introduction of the cofferdams (ie in the gap between the silt curtains) are not expected to result in a substantial change to the sediment dynamics in this area.

While the Middle Harbour south cofferdam (BL7) is predicted to cause an increase in current speeds next to the Clive Park shoreline, particularly during the ebb tide, the increase would be confined to the surface layers and therefore would not be expected to result in erosion/accretion of the bed of the harbour in this area.

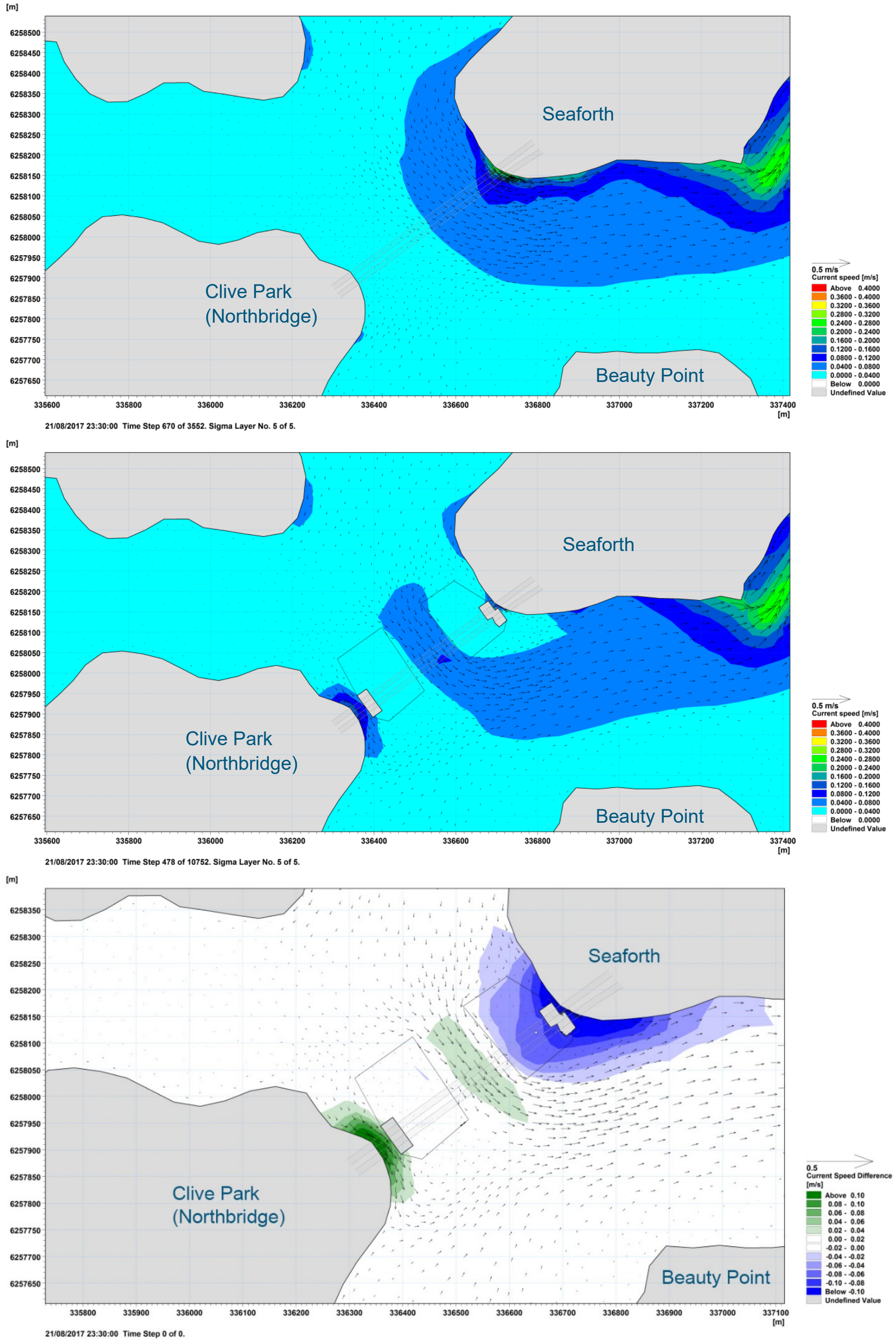

Figure 6-3: Ebb tide hydrodynamic conditions at the surface for existing scenario (top), construction scenario (middle) and current speed difference (bottom)
Note: positive change (green) indicates an increase in current speed and negative change (blue) is a decrease

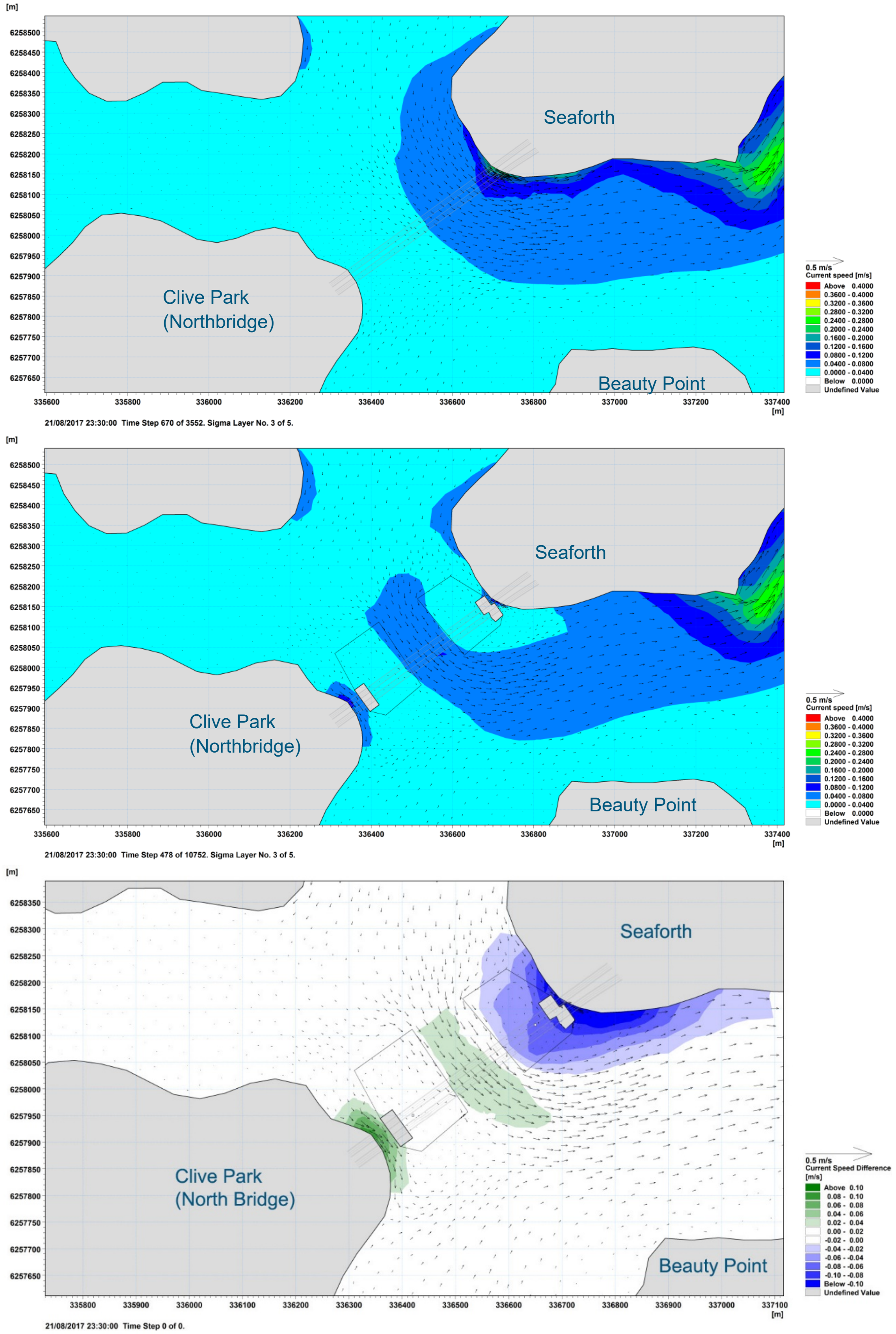

Figure 6-4: Ebb tide hydrodynamic conditions in the middle of the water column for existing scenario (top), construction scenario (middle) and current speed difference (bottom)

Note: positive change (green) indicates an increase in current speed and negative change (blue) is a decrease

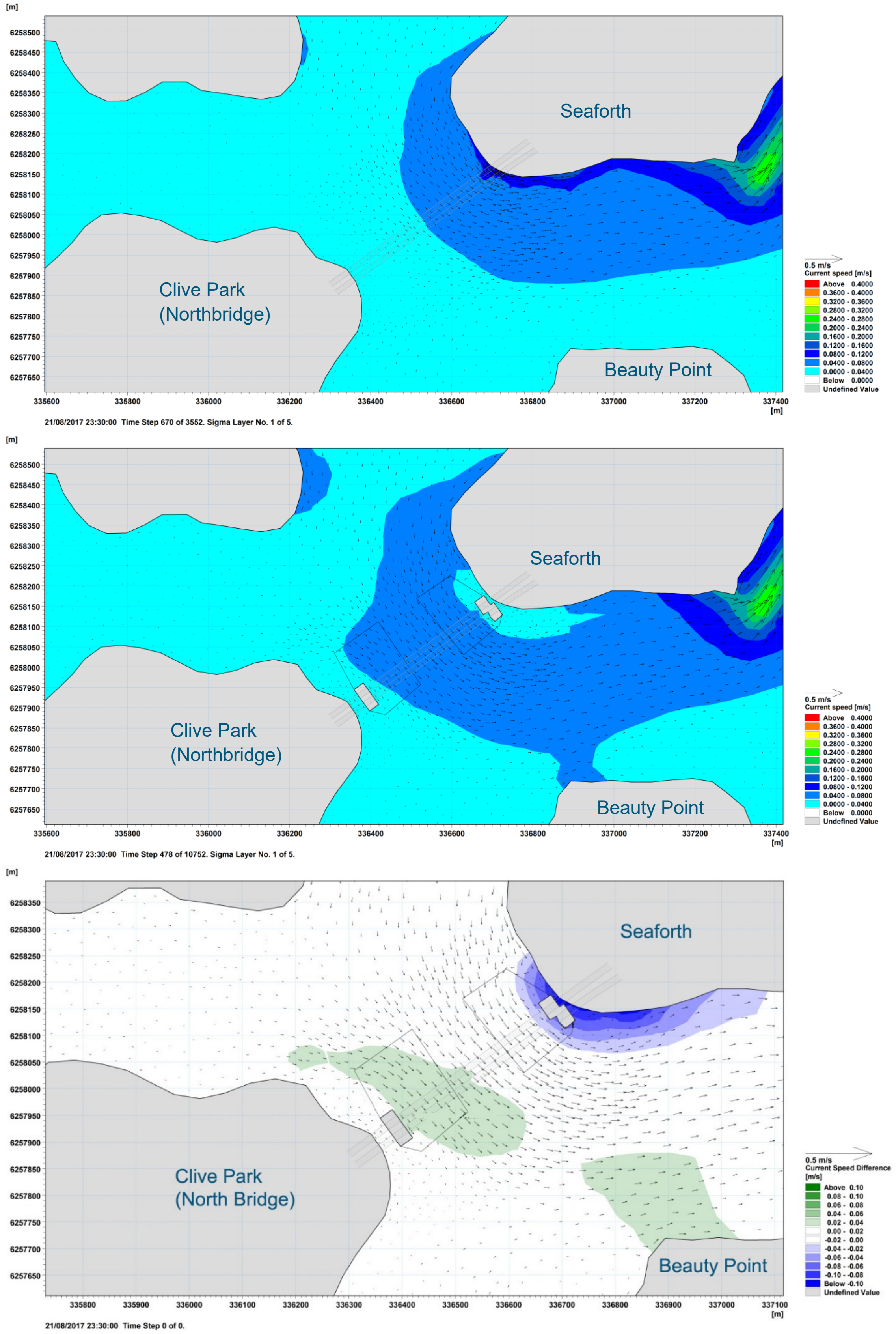

Figure 6-5: Ebb tide hydrodynamic conditions near the seabed for existing scenario (top), construction scenario (middle) and current speed difference (bottom)
Note: positive change (green) indicates an increase in current speed and negative change (blue) is a decrease

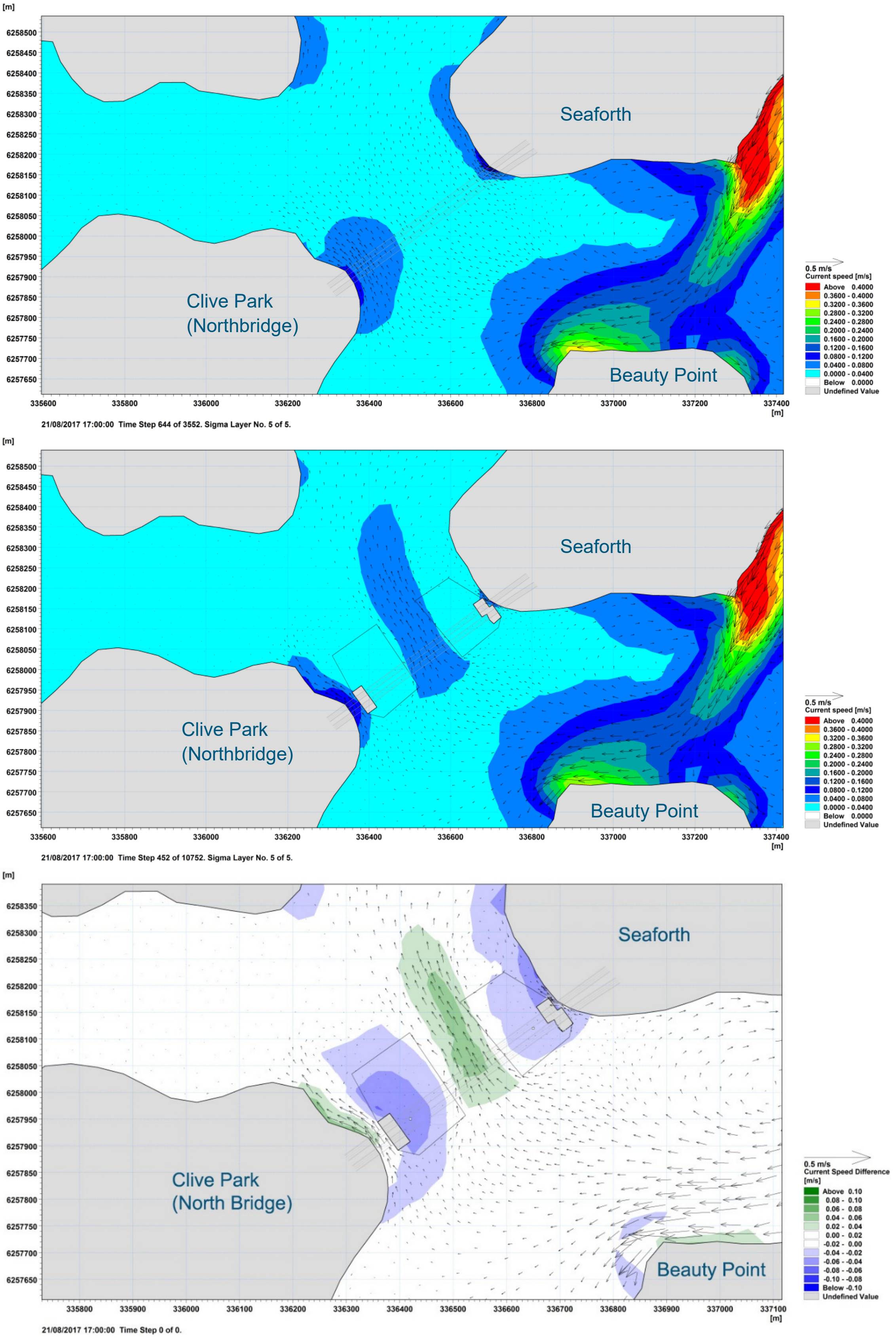

Figure 6-6: Flood tide hydrodynamic conditions at the surface for the existing scenario (top), construction scenario (middle) and difference (bottom)

Note: positive change (green) indicates an increase in current speed and negative change (blue) is a decrease

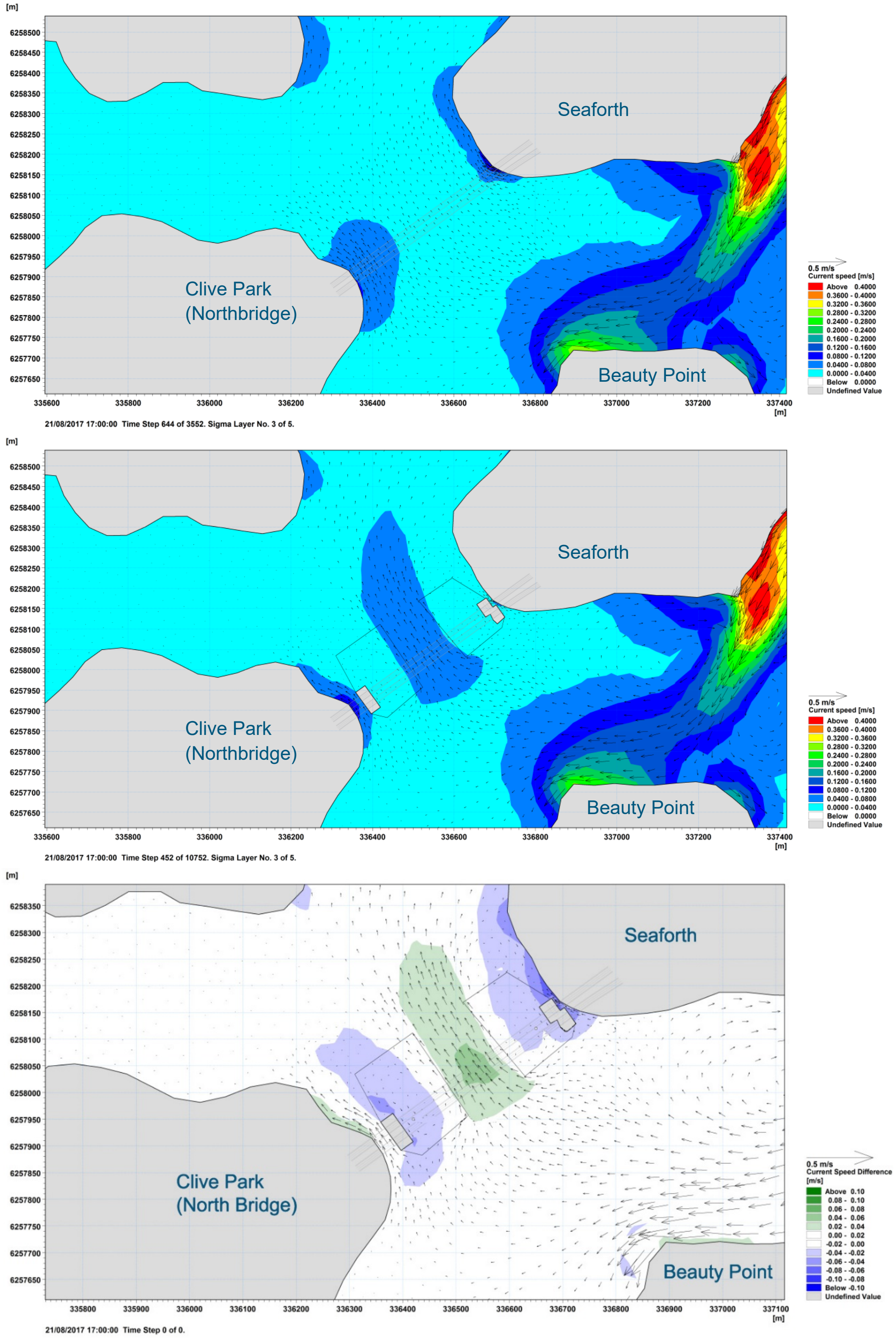

Figure 6-7: Flood tide hydrodynamic conditions in the middle of the water column for the existing scenario (top), construction scenario (middle) and difference (bottom)

Note: positive change (green) indicates an increase in current speed and negative change (blue) is a decrease

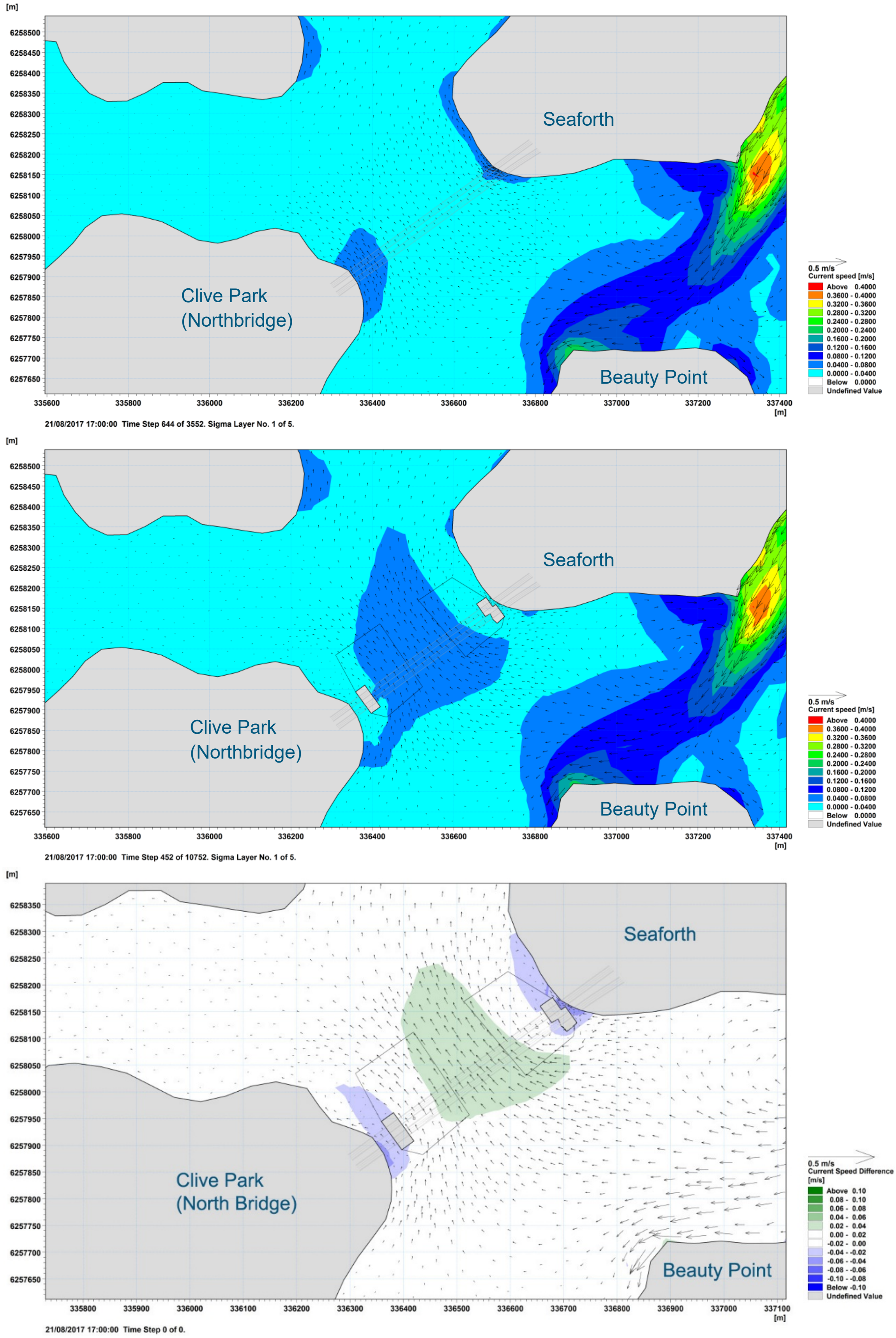

Figure 6-8: Flood tide hydrodynamic conditions near the seabed for the existing scenario (top), construction scenario (middle) and difference (bottom)

Note: positive change (green) indicates an increase in current speed and negative change (blue) is a decrease

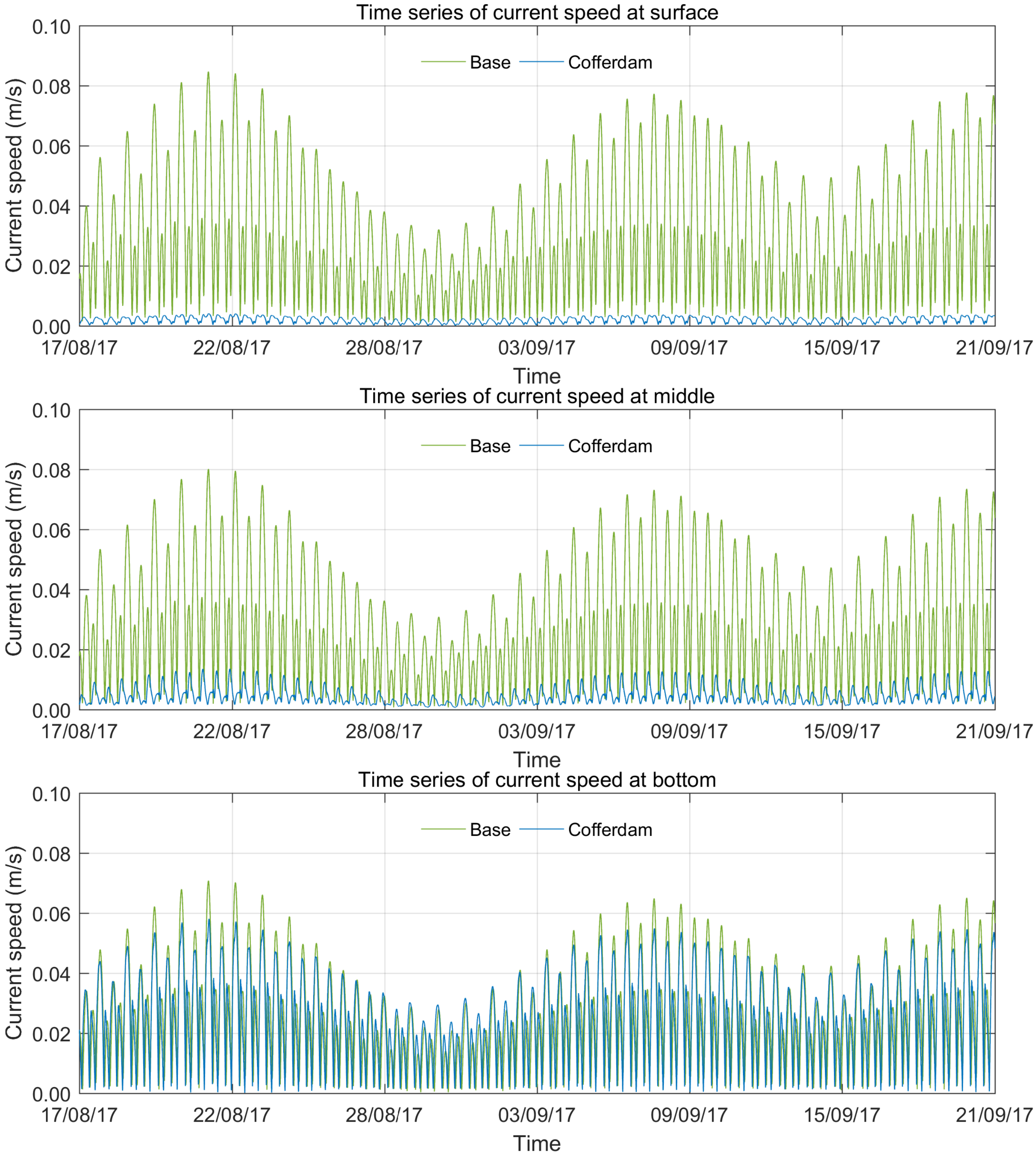

Figure 6-9: Time series plots of modelled current speed adjacent to the Middle Harbour north cofferdam (BL8)

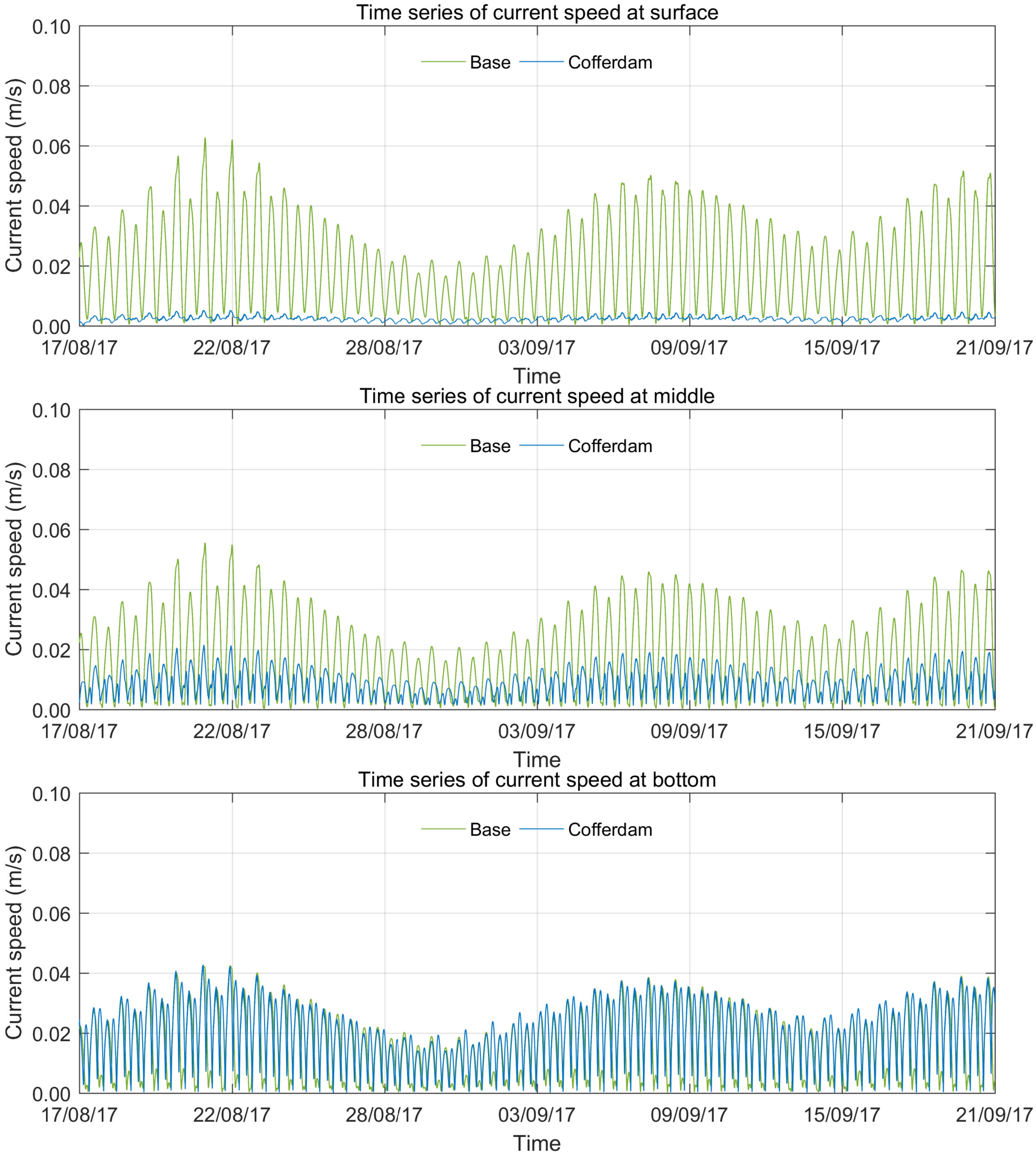

Figure 6-10: Time series plots of modelled current speed adjacent to the Middle Harbour south cofferdam (BL7)

6.1.3 Spit West Reserve construction support site (BL9)

Similar to the plots provided in **Section 6.1.2**, Figure 6-11 to Figure 6-16 provide the tidal current speed and patterns at the surface, mid-depth and bed of the harbour during the peak ebb and flood for both the existing and construction scenarios. These figures also show spatial plots of the difference in current speeds due to the Spit West Reserve construction support site (BL9).

Under the existing conditions the surface currents near the Spit West Reserve move in a northward direction during the flood tide, parallel to the shoreline at speeds which are generally less than 0.12 m/s but are as high as 0.28 m/s nearer to the shoreline. The northward flows along this shoreline area associated with a return eddy that forms in Pearl Bay during the flood tide. During the ebb tide the current speeds in this area are much lower, being less than 0.08 m/s but still slowing towards the north. During both the flood and the ebb tide, current speeds reduce slightly with depth.

Under the construction scenario with the Spit West Reserve construction support site (BL9) in place there is a general reduction in current speeds adjacent to the shoreline. This would mostly occur during the flood tide, when decreases in the current speeds of up to 0.1 m/s are predicted next to the shoreline. The reduced current speeds result from the temporary structures impeding the eddy that forms in this area. The eddy is redirected, particularly in the surface layers, towards the west and there is a small area of current speed increase to the west of the immersed tube tunnel units/casting barges.

Overall modelling of the hydrodynamic impacts at the Spit West Reserve construction support site (BL9) indicates:

- Reductions in current speed of up 0.1 m/s observed at, and to the north of, the Spit West Reserve construction support site (BL9)
- The expected reductions in current speed are greater during the flood tide than the ebb tide
- The reductions in current speed are greater at the surface than at the bed of the harbour during both the flood and ebb tide
- Due to the low current speeds observed in this area and the minor changes in current speeds expected at the bed of the harbour, no substantial change in the pattern of accretion or erosion at the bed of the harbour is expected
- No current increases were predicted along the Spit West Reserve shoreline and consequently shoreline erosion as a result of changes to tidal currents is not expected to occur.

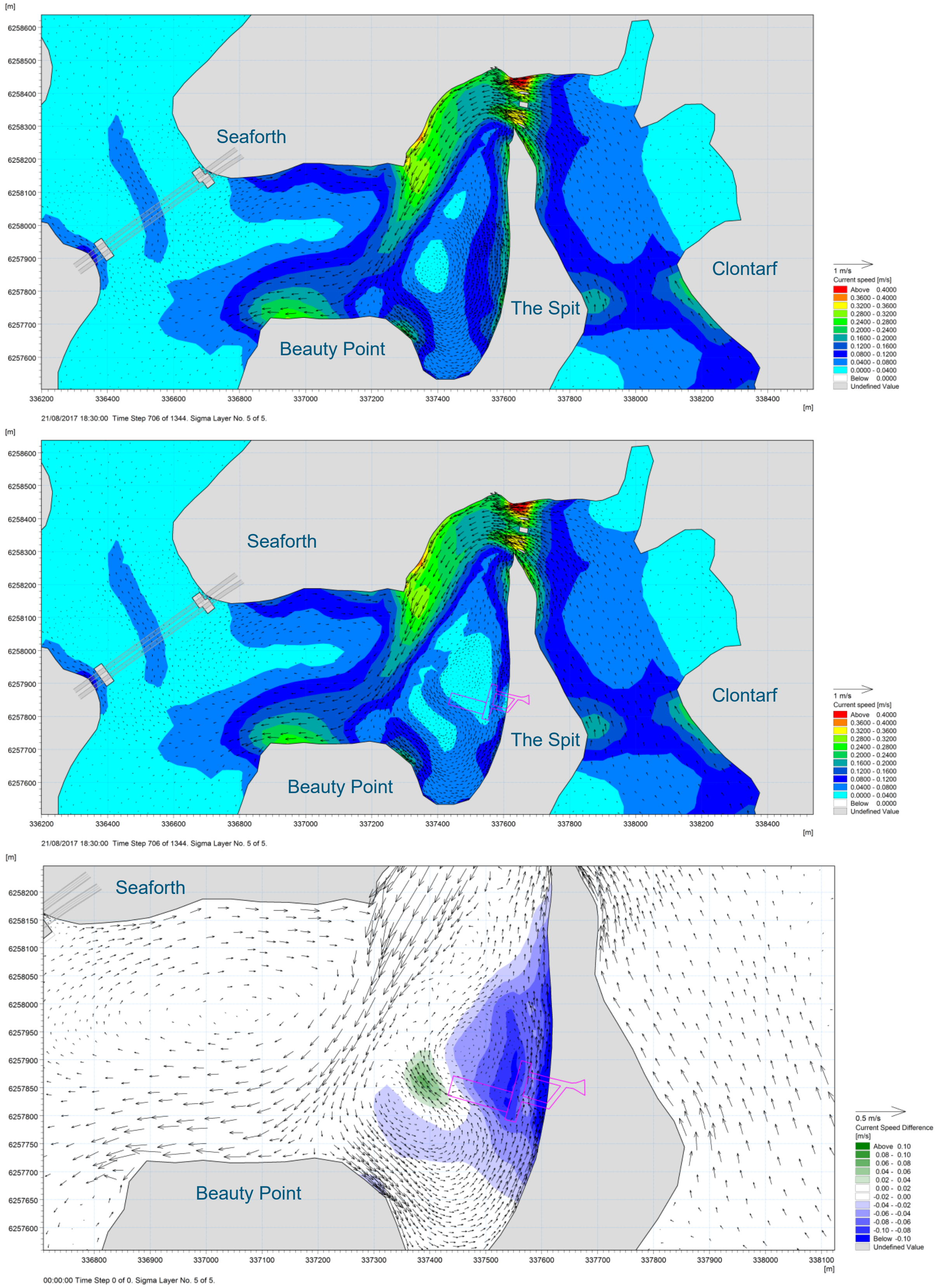

Figure 6-11: Flood tide hydrodynamic conditions in the surface layer at Spit West Reserve for existing scenario (top), construction scenario (middle) and current speed difference (bottom)

Note: positive change (green) indicates an increase in current speed and negative change (blue) is a decrease

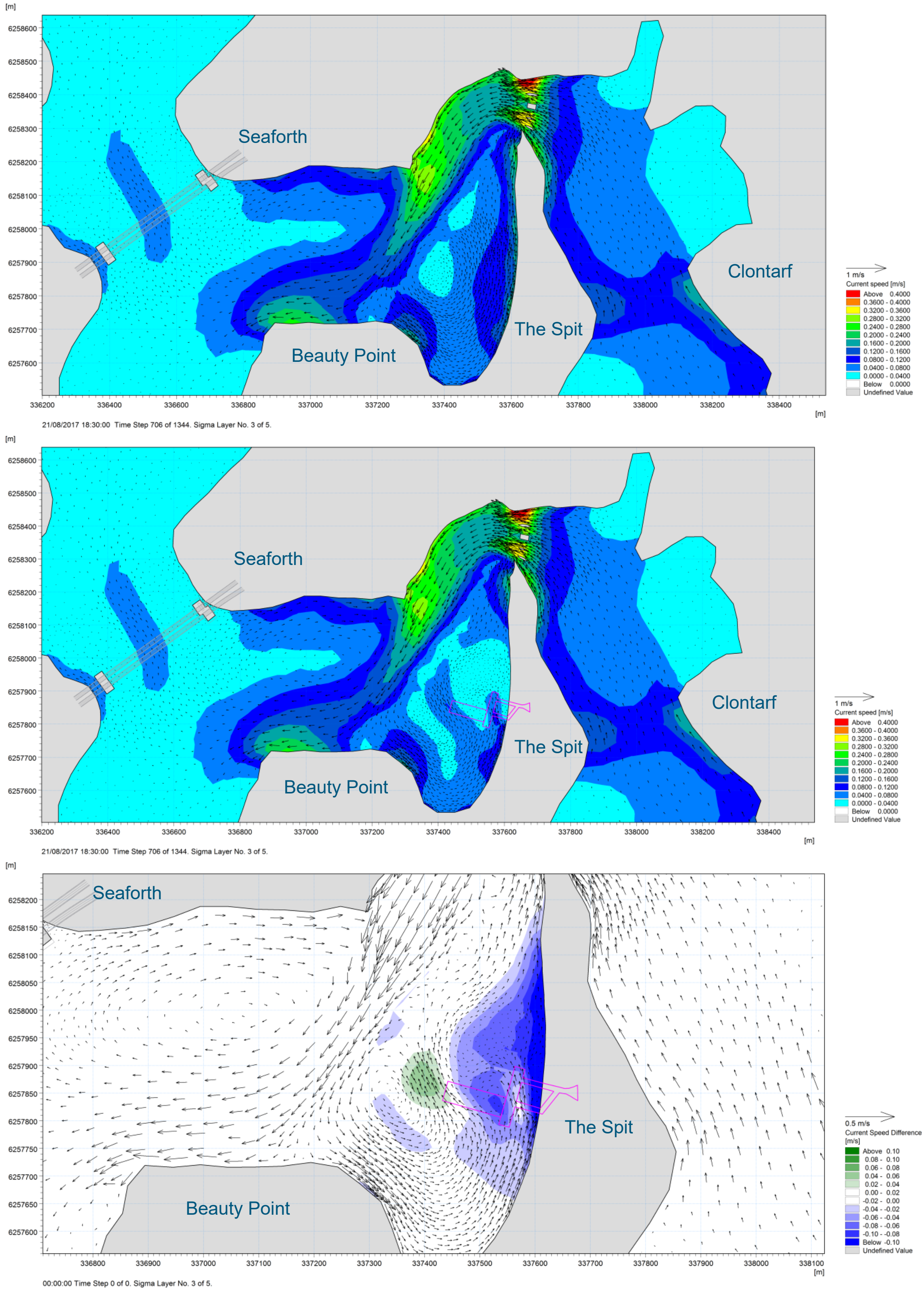

Figure 6-12: Flood tide hydrodynamic conditions in the middle of the water column at the Spit West Reserve for existing scenario (top), construction scenario (middle) and current speed difference (bottom)

Note: positive change (green) indicates an increase in current speed and negative change (blue) is a decrease

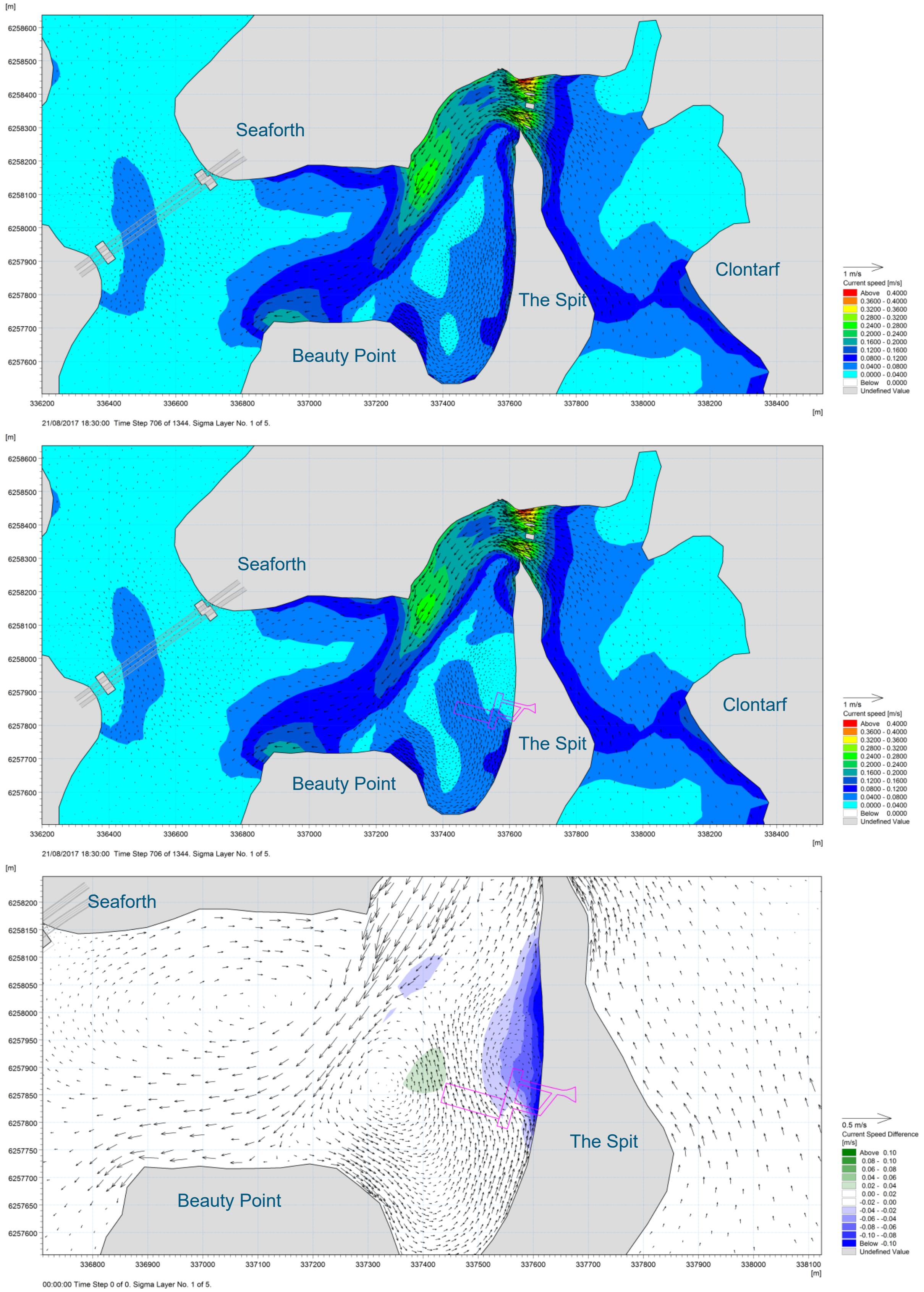

Figure 6-13: Flood tide hydrodynamic conditions near the seabed at the Spit West Reserve for existing scenario (top), construction scenario (middle) and current speed difference (bottom)

Note: positive change (green) indicates an increase in current speed and negative change (blue) is a decrease

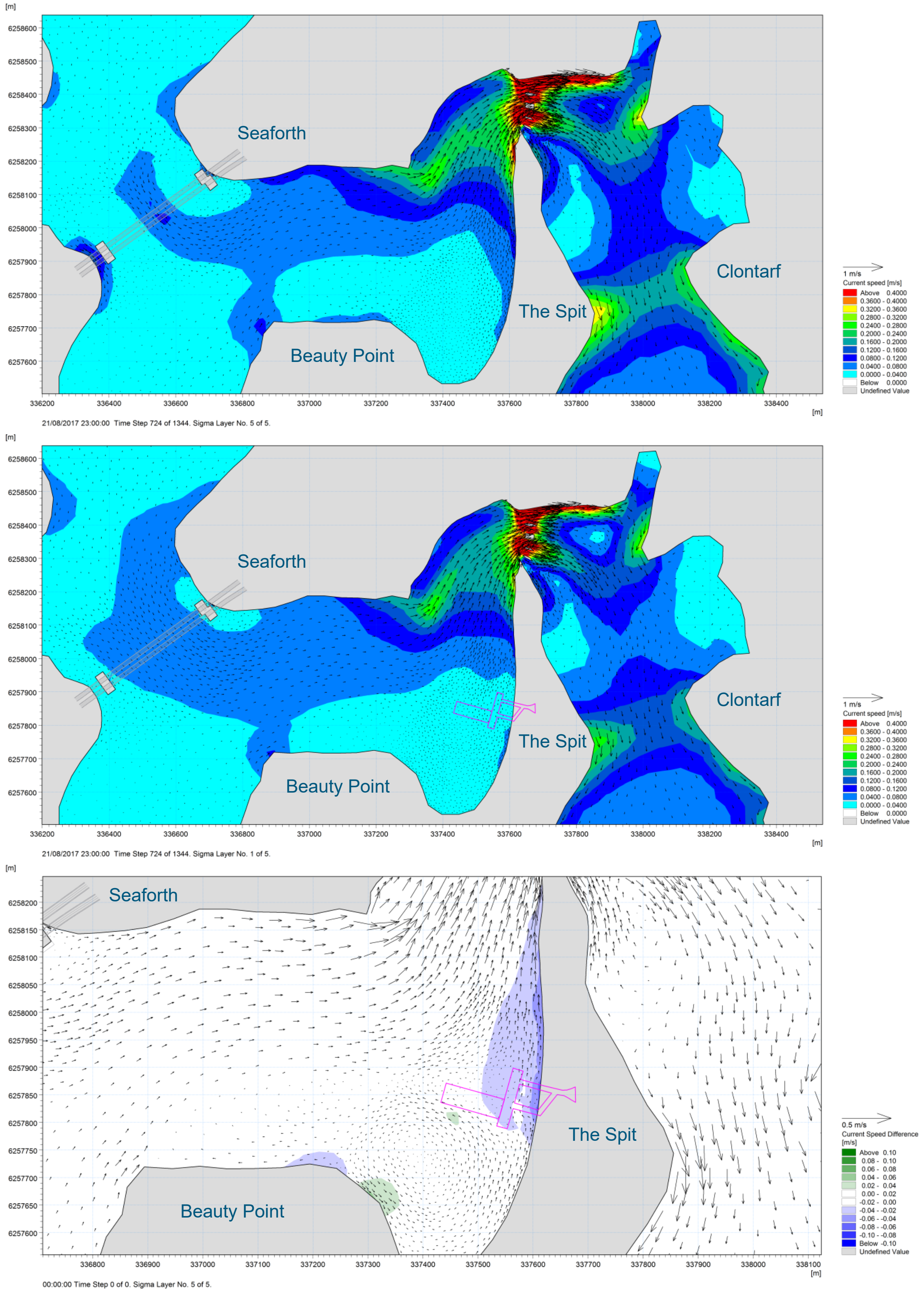

Figure 6-14: Ebb tide hydrodynamic conditions in the surface layer at the Spit West Reserve for existing scenario (top), construction scenario (middle) and current speed difference (bottom)

Note: positive change (green) indicates an increase in current speed and negative change (blue) is a decrease

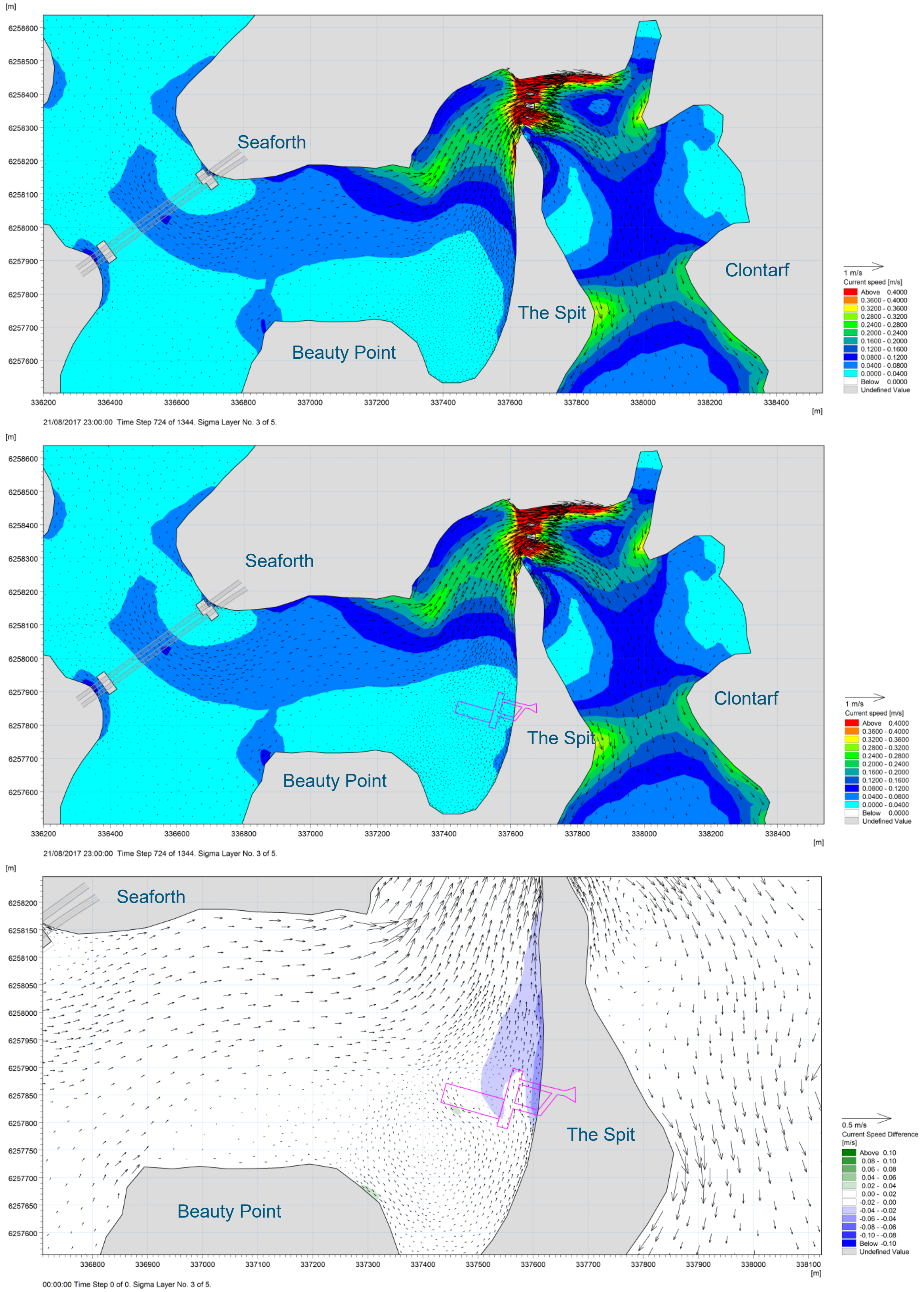

Figure 6-15: Ebb tide hydrodynamic conditions in the middle of the water column at the Spit West Reserve for existing scenario (top), construction scenario (middle) and current speed difference (bottom)

Note: positive change (green) indicates an increase in current speed and negative change (blue) is a decrease

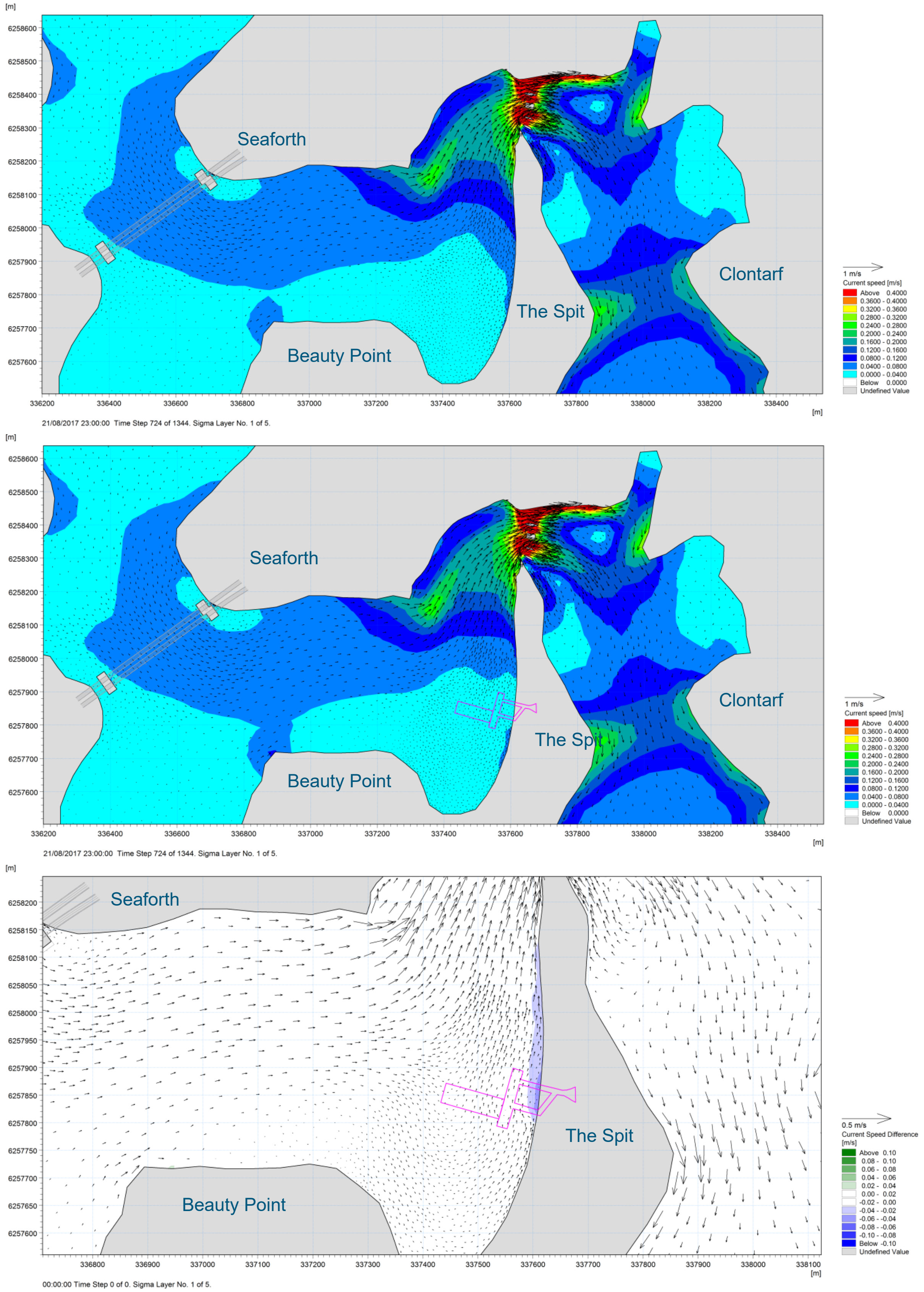

Figure 6-16: Ebb tide hydrodynamic conditions near the seabed at the Spit West Reserve for existing scenario (top), construction scenario (middle) and current speed difference (bottom)

Note: positive change (green) indicates an increase in current speed and negative change (blue) is a decrease

6.2 Operational impacts

6.2.1 Overview

The immersed tube tunnels would be situated above the bed across the deepest section of Middle Harbour, creating a sill that is about 250 metres in length, measured from bank to bank based on the intersection of the crown of the immersed tube tunnel with the existing bed of the harbour, and about 40 metres wide. The calibrated 3D hydrodynamic model was refined and used to investigate the impact the immersed tube tunnels would have on the local hydrodynamics and flushing characteristics of Middle Harbour. The 3D hydrodynamic model was run for a period of about 5.5 weeks. The model was run for two scenarios:

- Existing scenario
- Project design including the sill like feature created by the immersed tube tunnels, as shown in Figure 6-17.

The existing scenario is based on the calibrated 3D model with the bed of the harbour representative of current conditions (ie pre-tunnel construction) and based on the surveyed bathymetric data of Middle Harbour as provided by Transport for NSW.

The bathymetry in the hydrodynamic model was manually adjusted to represent the immersed tube tunnels configuration, including the gradual slope of the tunnels across Middle Harbour. At the project crossing the existing water depth is up to 32 metres; this was reduced to about 20 metres in some areas to represent the immersed tube tunnels. A grade of one per cent was used to represent the slope of the immersed tube tunnels. The level of -20 metre AHD was adopted as approximating the top (or crown) of the immersed tube tunnels in the centre of crossing.

The existing bathymetry and updated bathymetry for the project configuration are shown in Figure 6-18 and Figure 6-19, respectively.

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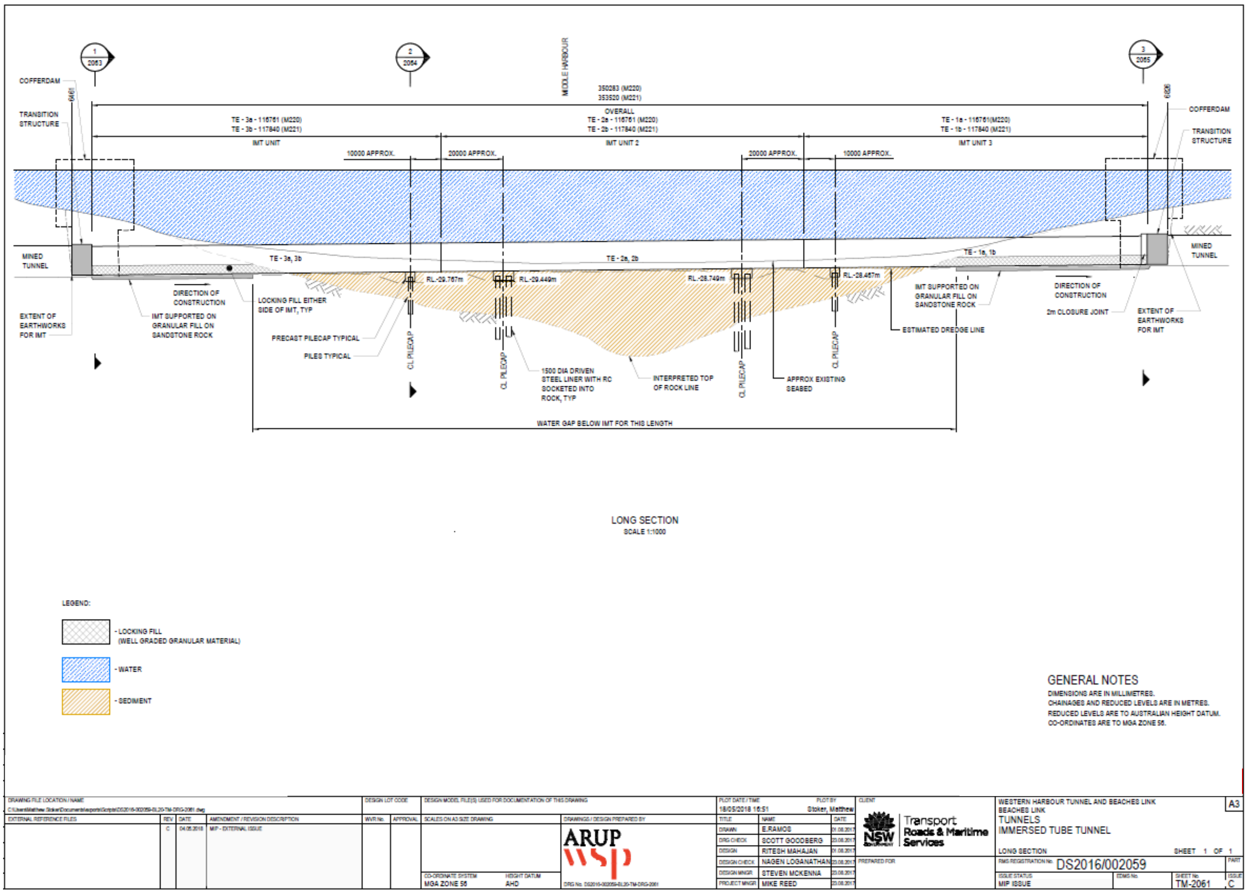

Figure 6-17: Proposed immersed tube tunnel design

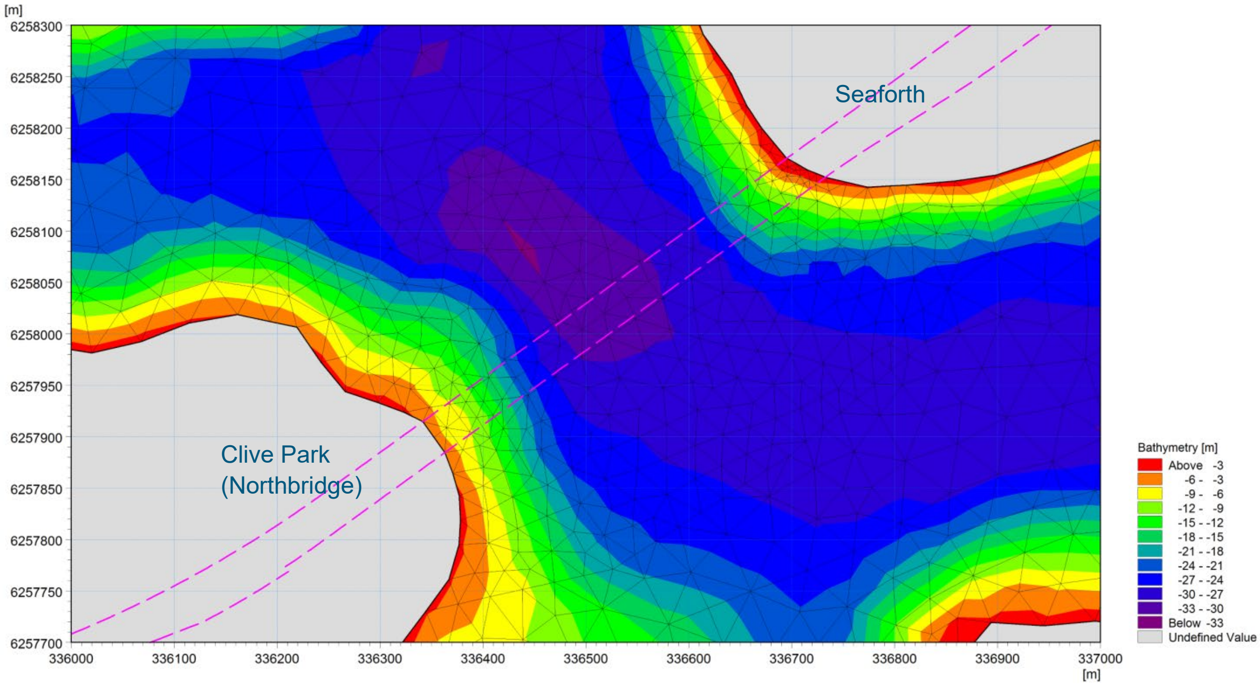

Figure 6-18: Existing conditions (proposed immersed tube tunnel alignment shown as dashed lines)

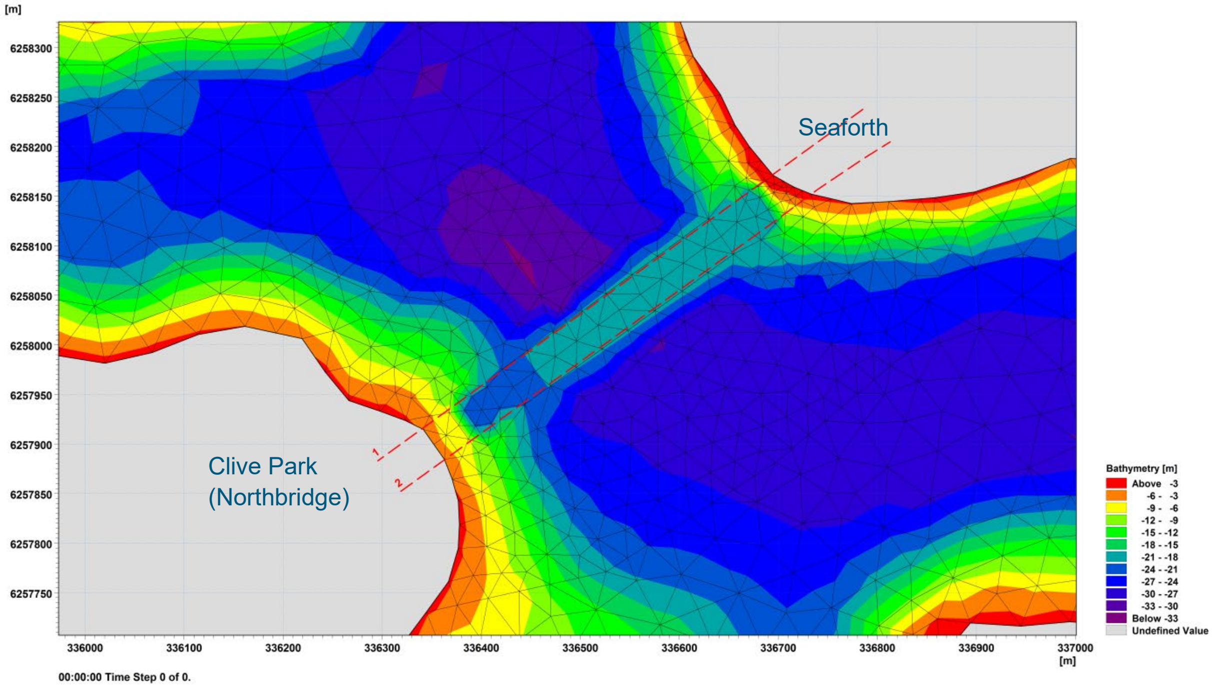

Figure 6-19: Project conditions (proposed immersed tube tunnel alignment shown as dashed lines)

6.2.2 Operational hydrodynamics impacts

Methodology

Hydrodynamic modelling with decoupled hydrodynamic output (subsequently used to assess tidal flushing) was conducted for about 5.5 weeks. The model was barotropic[2] and driven by tide only.

The vertical structure of the 3D model, from the water surface downward, comprised:

- Five sigma layers (each representing 20 per cent of the distance between the water surface and - 20 metre AHD, which is the approximate average crown level of the modelled immersed tube tunnel design in the centre of the crossing location). Each layer has a thickness of about four metres (this varies slightly with tidal levels)
- A varying number of equidistant z-layers representing the water column below -20 metre AHD (layer thickness four metres). The number of layers depends on local bed elevation.

The hydrodynamic model was simulated for the two scenarios (existing and project design) with the results used to assess the impact of the project on tidal current, water level, discharge and tidal prism.

Results – tidal currents

Spatial maps showing the current speeds and patterns for the existing scenario at peak flood and ebb tidal stages are presented in Figure 6-20 and Figure 6-21. Spatial maps showing the difference in current speeds for two selected stages of the tide (peak flood and peak ebb) are presented in Figure 6-22 to Figure 6-25. In regard to these figures it is noted:

- Current speed differences (shown as colours) compare the existing scenario to the project design scenario. Green shows an increase in current speed due to the immersed tube tunnels while blue shows a decrease
- The current vectors shown are based on the peak speeds from the existing scenario model.

Under existing conditions the modelled peak tidal current speeds (surface layer) in the area of the crossing are:

- Less than 0.05 m/s in the deeper middle section of the Middle Harbour channel for both flood and ebb tidal stages
- Up to 0.12 m/s adjacent to the northern bank (ie Seaforth side of Middle Harbour) and up to 0.10 m/s adjacent to the southern bank (ie Northbridge side of Middle Harbour) during the flood tide
- Up to 0.24 m/s adjacent to the northern bank with low (less than 0.02 m/s) current speeds adjacent to the southern bank during the ebb tide.

It is observed that there are minor overall differences in current speeds between the existing scenario and project design scenario. The differences observed between existing and project design scenario are increases in current speeds over the proposed crossing location, particularly on the northern bank (Seaforth) during both flood and ebb tides. The relative increase in current speeds adjacent to the northern bank are 33 per cent (for both flood and ebb) but the magnitude of the change is less than 0.04 m/s and 0.08 m/s, for flood and ebb, respectively. The magnitude of wind driven circulation would be expected to be greater at this location than the changes in current speeds due to the immersed tube tunnels. That is, changes due to the immersed tube tunnels would be within the range of natural variation presently observed at the site.

[2] *Fluid density is a function of pressure only*

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The modelled current speeds shown in Figure 6-20: to Figure 6-21: highlight that the project crossing is located in a low energy hydrodynamic environment. Therefore, little to no bedload transport or resuspension of existing sediment is expected to naturally occur where the immersed tube tunnels would be located above the bed of the harbour. The geotechnical data shows that up to 30 metres of predominantly fine grained sediment is present in the centre of the channel above the bedrock (refer to Figure 6-17:). From the information available it can be inferred that the centre of the channel is a depositional environment and so little to no transport of bed sediment would be expected to occur in the area. As such, the localised increase in current speed due to the project design is not expected to result in a change to the sediment dynamics in this area.

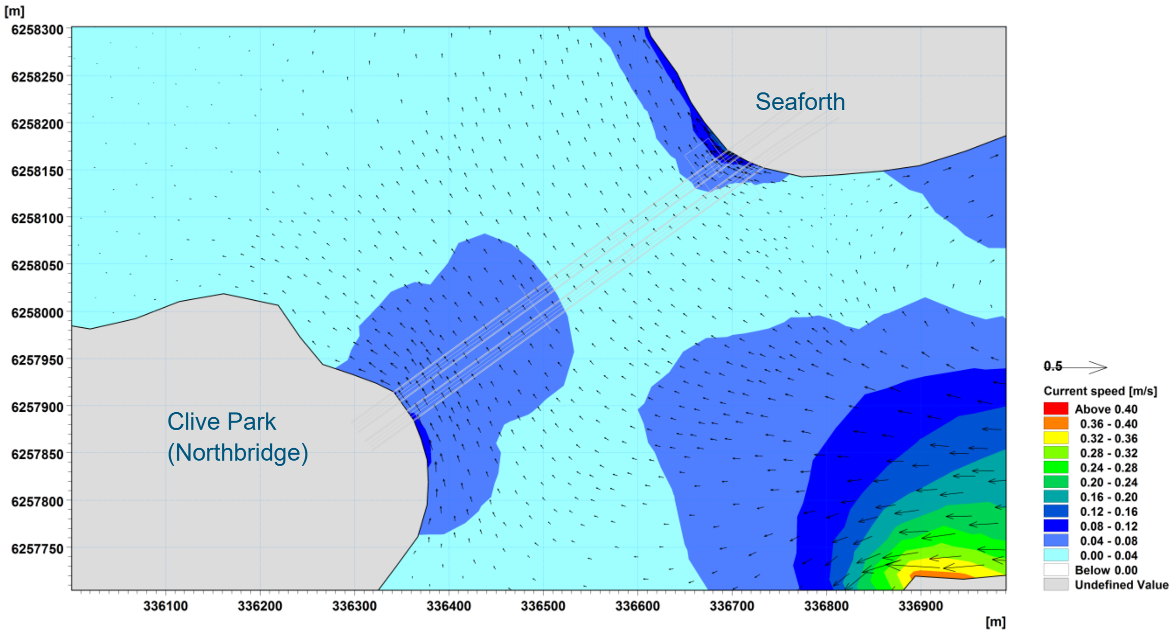


Figure 6-20: Map of current speeds and vectors at tunnel crossing location for existing conditions (Peak Flood: Surface)

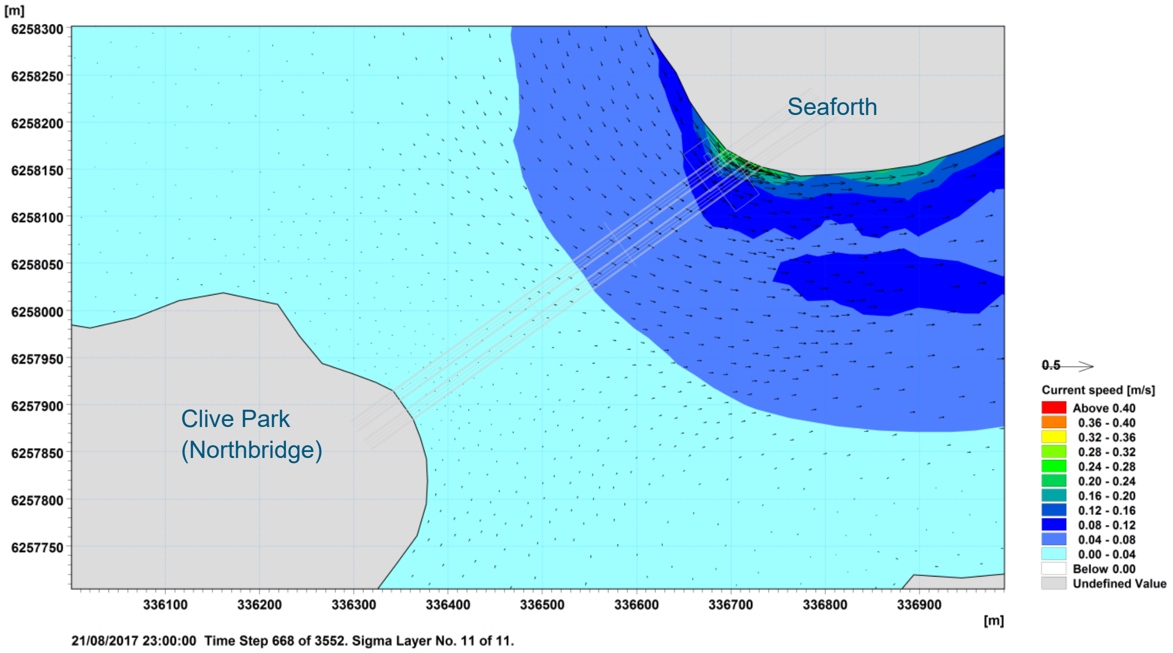


Figure 6-21: Map of current speeds and vectors at tunnel crossing location for existing conditions (Peak Ebb: Surface)

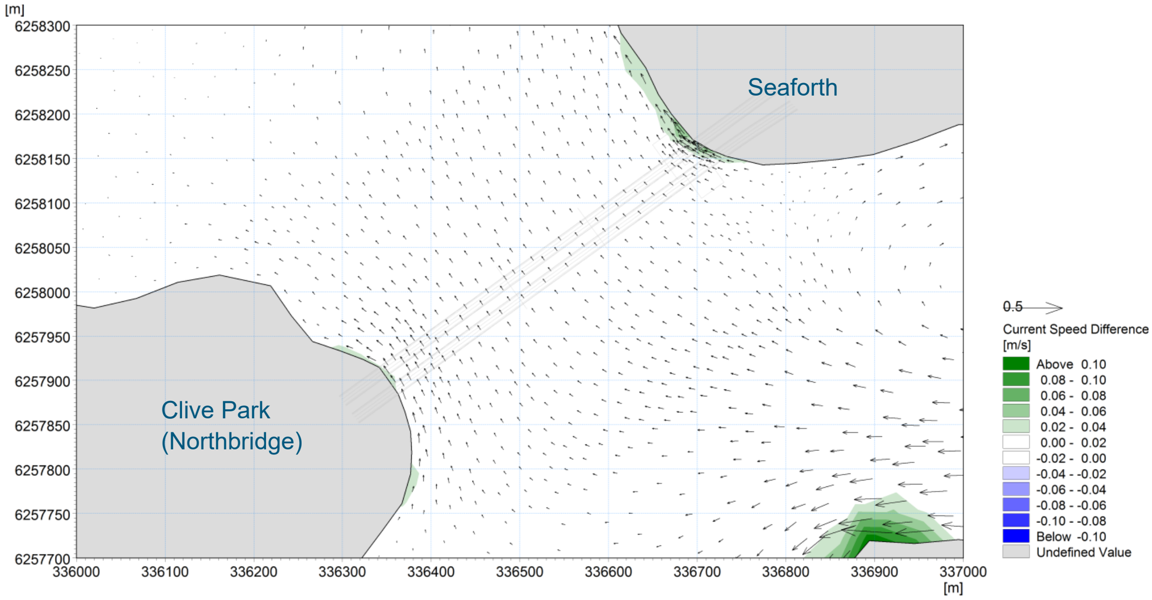


Figure 6-22: Current speed difference: project design less existing conditions (Peak Flood: Surface)

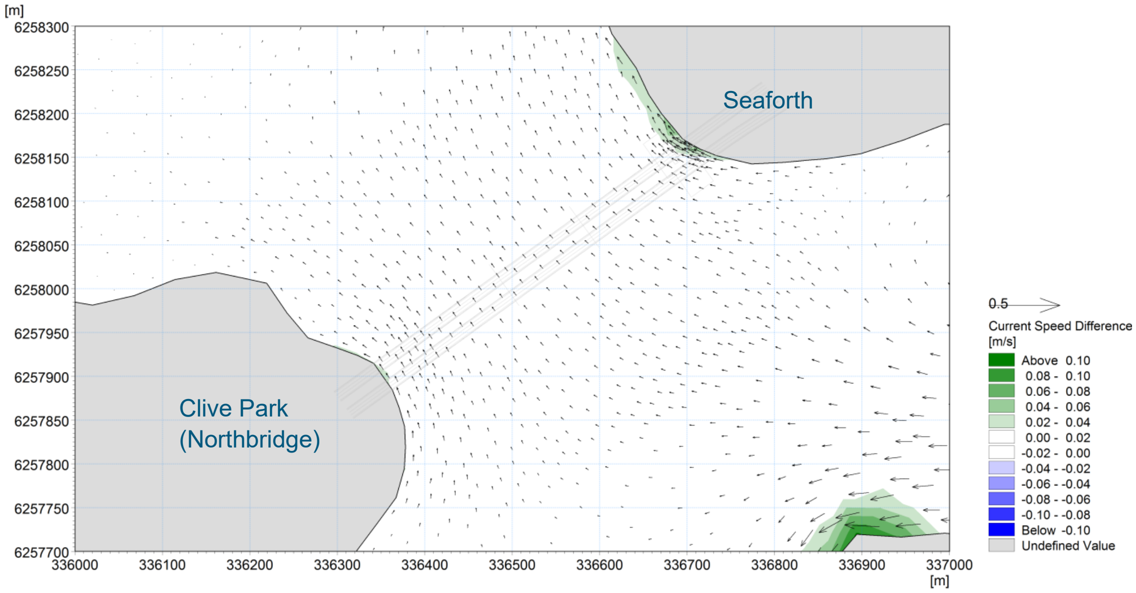


Figure 6-23: Current speed difference: project design less existing conditions (Peak Flood: in sigma layer just above the crown of the tunnel)

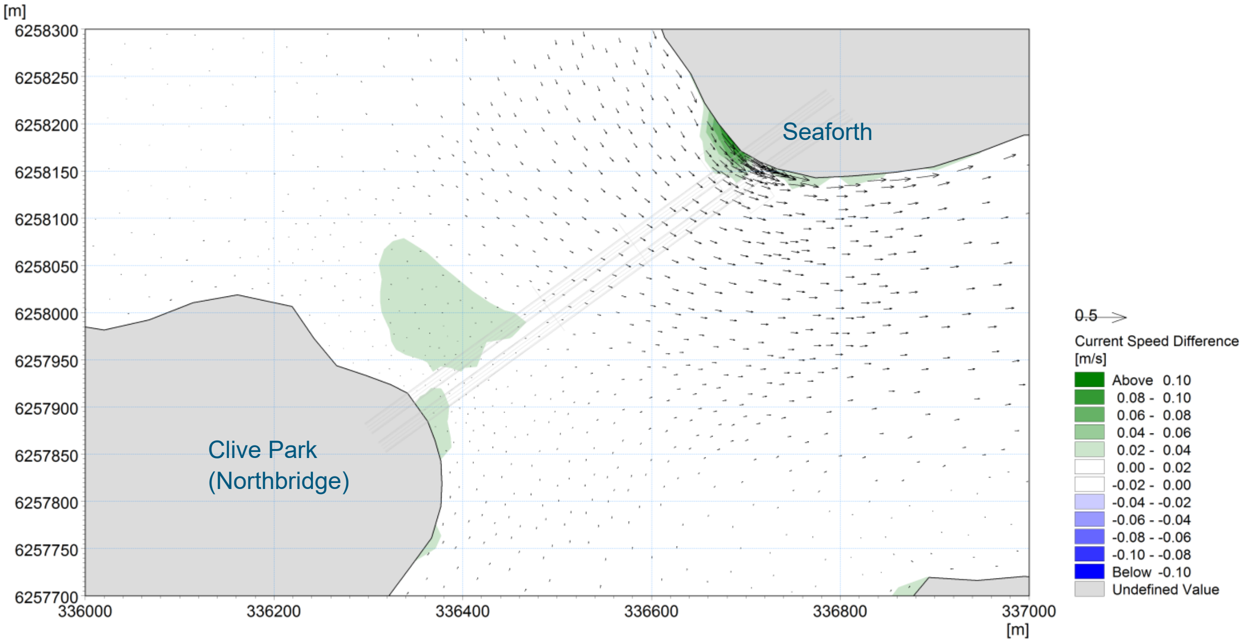


Figure 6-24: Current speed difference: project design less existing conditions (Peak Ebb: Surface)

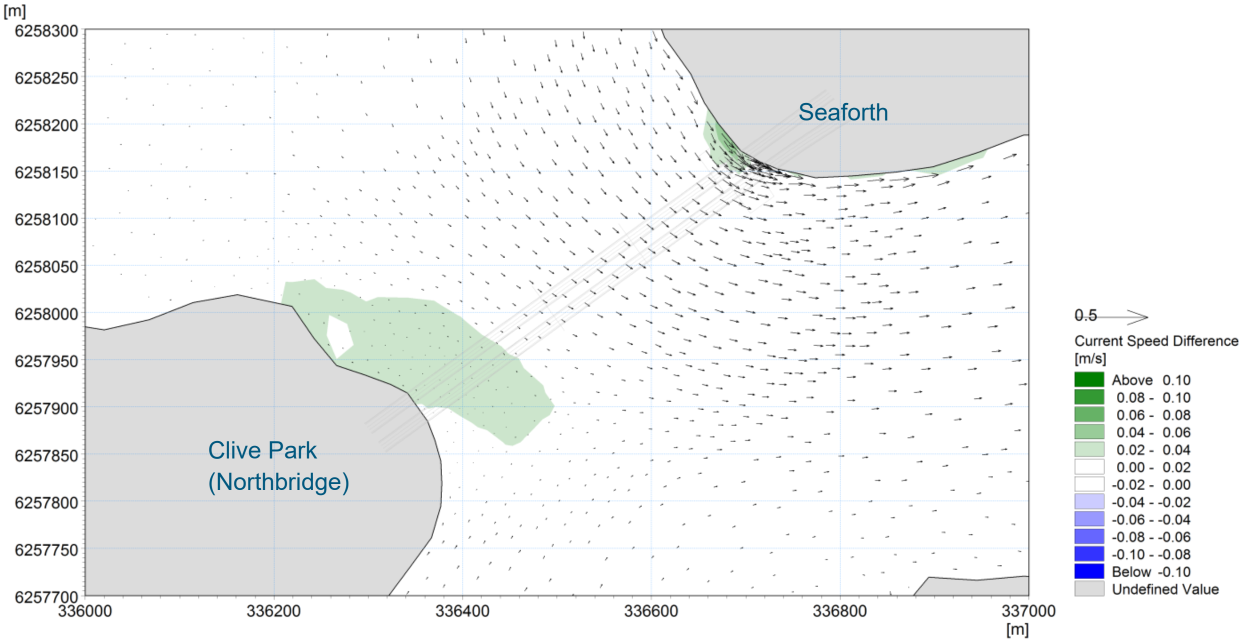


Figure 6-25: Current speed difference: project design less existing conditions (Peak Ebb: in sigma layer just above the crown of the tunnel)

Results - water level

Water levels at a site upstream of the proposed channel crossing location (site P4 – refer to Figure 6-29) were assessed and a comparison in terms of the tidal planes is presented in Table 6–1. To determine the tidal planes, harmonic analysis was completed on the modelled tidal water level variations at location P4. From the results presented in Table 6–1 it is observed that there was no difference in tidal planes between the existing scenario and the project design scenario.

Table 6–1: Tidal plane comparison (metres at Lowest Astronomical Tide)

Tidal plane	Existing scenario (metres)	Project design scenario (metres)
Mean High Water Springs (MHWS)	1.59	1.59
Mean High Water (MHW)	1.44	1.44
Mean High Water Neaps (MHWN)	1.30	1.30
Mean Sea Level (MSL)	0.94	0.94
Mean Low Water Neaps (MLWN)	0.58	0.58
Mean Low Water (MLW)	0.44	0.44
Mean Low Water Springs (MLWS)	0.30	0.30

Results – tidal discharge

The modelled tidal discharge at the tunnel crossing location (shown in Figure 6-26) is presented in Figure 6-27. This plot provides a comparison of the tidal discharge for the existing scenario and the project design scenario. It is noted that there is only a minor difference in the tidal discharge curves with and without the project.

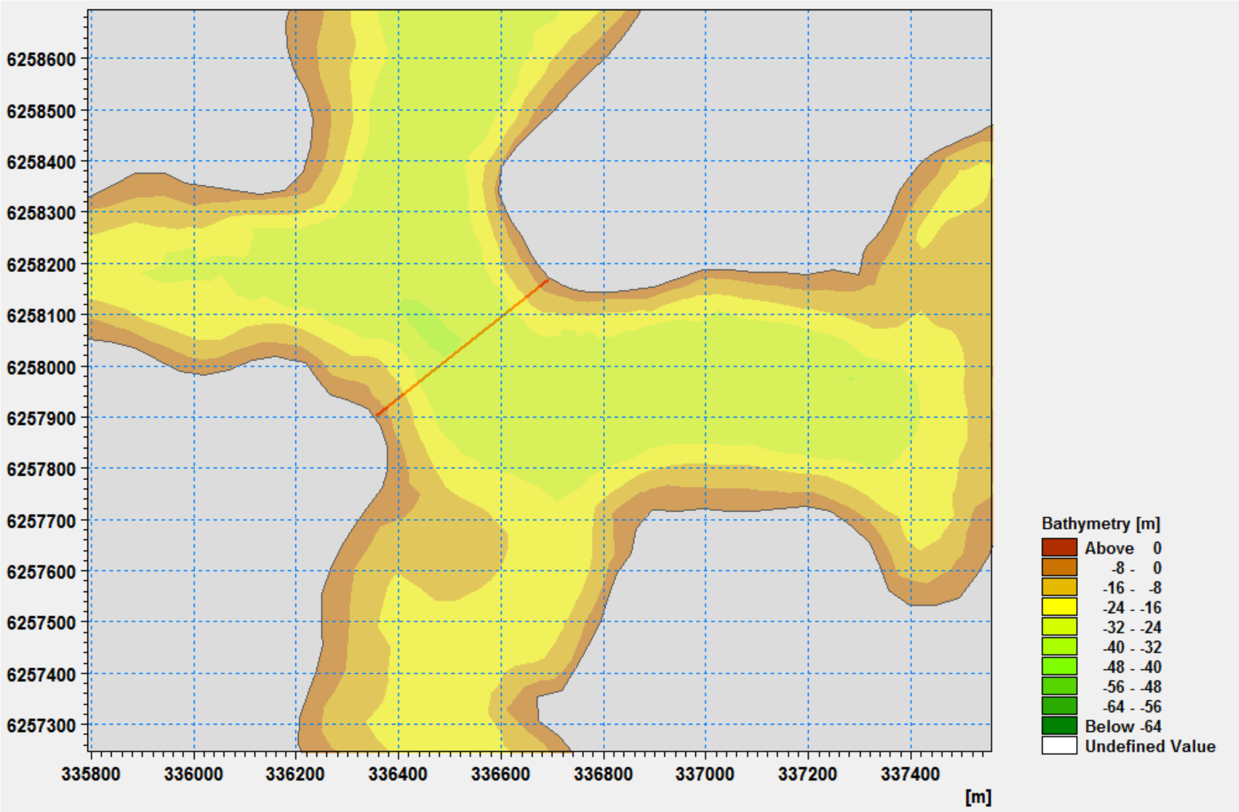


Figure 6-26: Location of cross section used to compare tidal discharges (red line located over tunnel crossing location)

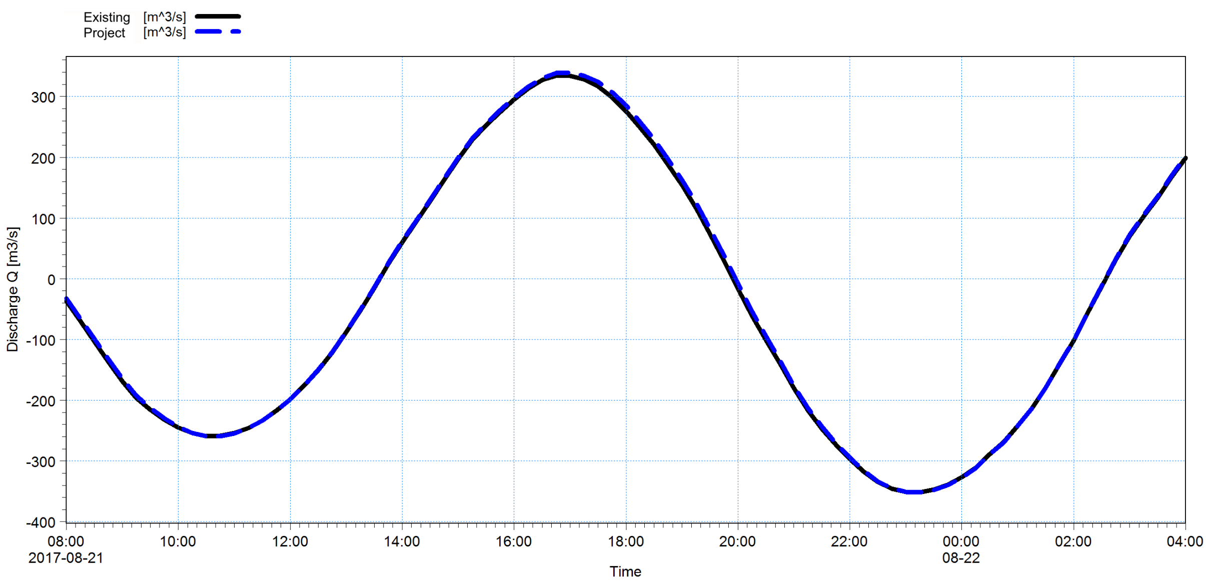


Figure 6-27: Tidal discharge curves comparing modelled scenario for the existing and project design conditions
Note: plots are almost identical and therefore largely plot over the top of each other.

Results - tidal prism

The tidal prism is the volume of water in an estuary between given high water and low water tidal planes, or, put alternatively, the volume of water that flows into and out of the estuary between these tidal planes.

To assess the potential of the project to influence the tidal prism in Middle Harbour, the tidal discharge through the channel was extracted from the model results. The tidal discharges were extracted at a cross section taken over the tunnel crossing (see Figure 6-26). The tidal prism through the cross section over a large spring tide was calculated for the existing and project conditions (Table 6–2). The results show that the immersed tube tunnels would lead to a marginal decrease in the tidal prism. The relative change in tidal prism is 0.4 per cent. This change in tidal prism would not be measurable.

Table 6–2: Modelled change in tidal prism due to the project

Modelled scenario	Tidal prism (m^3)
Existing tidal prism	5.080×10^6
Project design tidal prism	5.062×10^6

6.2.3 Tidal flushing

Methodology

Tidal flushing refers to the replacement of water in Middle Harbour upstream of the immersed tube tunnels with water from downstream of the immersed tube tunnels as the tidal fluctuations bring seawater through Port Jackson and Middle Harbour on the flood tide and carry water within Middle Harbour towards the ocean on the ebb tide. Quantitative investigations can be used to assess the impacts of the presence of the immersed tube tunnels on tidal flushing and describe the likely character of water quality responses of a water body.

Flushing simulations were carried out utilising decoupled output from the calibrated hydrodynamic model and the MIKE 3 AD (Advection/Dispersion) module. The following vertical structure was adopted for the tidal flushing simulations:

- Five equidistance sigma layers (each 20 per cent) from the surface down to -20 metres AHD
- Six z-layers from -20 metres AHD to the bed each with a layer thickness of four metres.

The hydrodynamic modelling was carried out for the barotropic case and does not include the effects of stratification on vertical mixing. The horizontal eddy viscosity was characterised using the Smagorinsky scheme with a constant value of 0.25 (Smagorinsky, 1963). The vertical eddy viscosity was also characterised using a Smagorinsky scheme and adopted a log-law formulation with minimum and maximum eddy viscosity values of 1.8×10^{-6} m^2/s and 0.4 m^2/s, respectively. Horizontal and vertical dispersion was scaled from the eddy viscosity formulation using a factor of one.

A total of four conservative tracers were used in the advection/dispersion module of the model (refer Figure 6-28) each set with an arbitrary initial concentration of 100 mg/L upstream of the proposed immersed tube tunnel crossing location in Middle Harbour. The fourth tracer T0 (not displayed) encompassed the T1, T2 and T3 areas. The water body downstream of the immersed tube tunnel crossing location has zero tracer present and as the 3D model evolves, the tracer is gradually flushed out of the Middle Harbour section upstream of the immersed tube tunnels and the concentration within that section is reduced. The area of the harbour bed, volume (relative to AHD) and the longitudinal length of each of the control volumes are provided in Table 6–3. The advection/dispersion simulations had a duration of about 16 weeks, which was completed by looping the tracers' dispersion through the results from the 5.5 week hydrodynamic simulations.

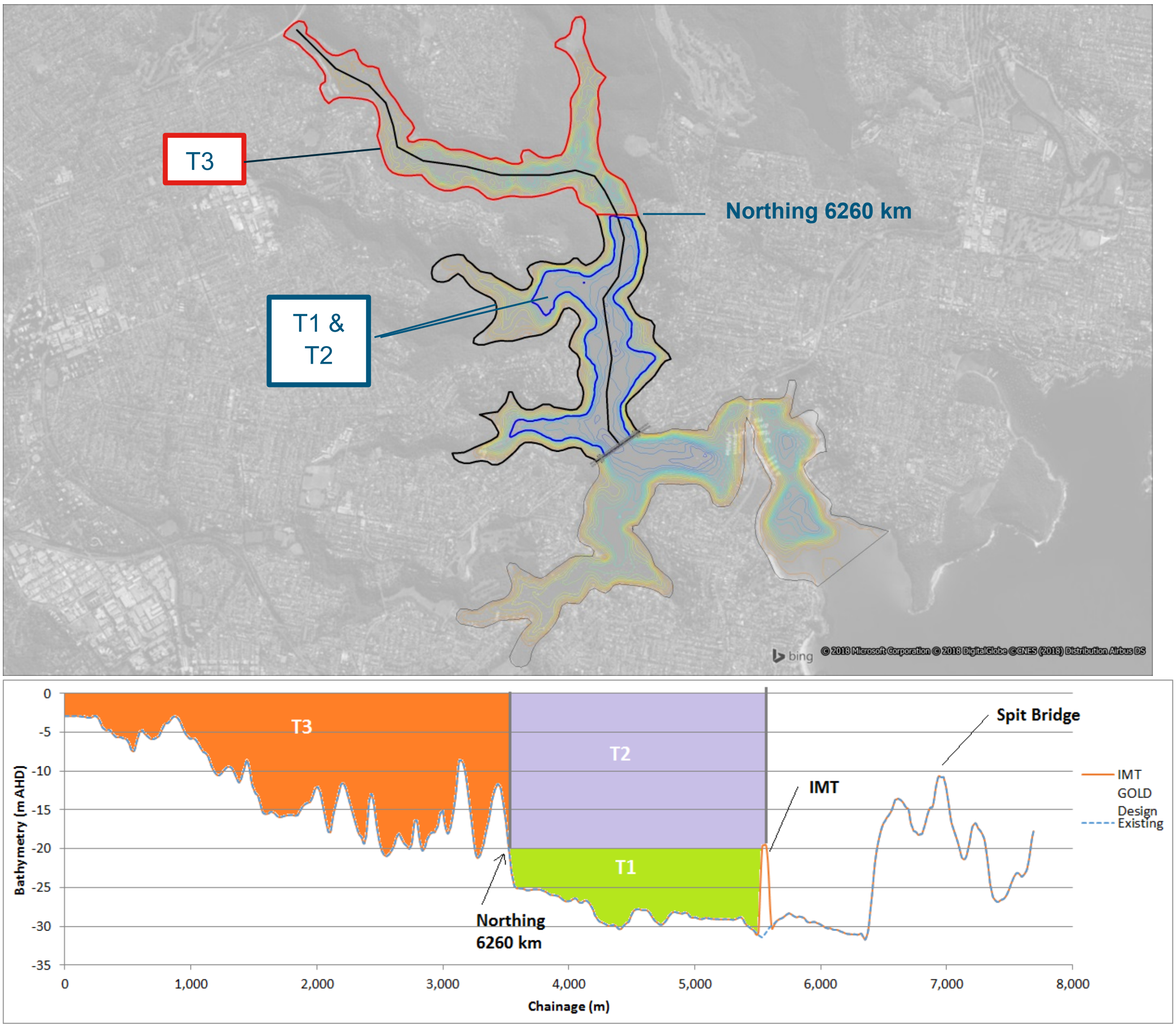


Figure 6-28: Initial tracer distribution map (top) and long section of Middle Harbour (bottom)

Table 6–3: Dimensions of control volumes used in e-folding simulations

Tracers	Control volume dimensions		
	Bed of harbour area (m^2)	Volume (m^3)	Longitudinal length (m)
T0 (overall)	3.102×10^6	4.32×10^7	5540
T1	0.806×10^6	0.56×10^7	2040
T2	1.019×10^6	2.53×10^7	2040
T3	1.277×10^6	1.23×10^7	3500

Flushing times are defined in this study in terms of e-folding times. The e-folding time refers to the time taken for a tracer to reduce to 1/e, or 0.368 of its initial concentration. At any location in a water body subject to dynamic equilibrium forces, the concentration of a particular tracer can be described by the following equation:

$$C_i = C_0 e^{-kt_i}$$

Where C_i is the concentration at time *i*, C_0 is the initial concentration, t_i is time and k is the dispersion constant. Following on from this equation, *k* can be calculated according to:

$$k = \frac{\ln\left(\frac{C_i}{C_0}\right)}{-t_i}$$

The e-folding time is then calculated as the inverse of the dispersion constant k:

e-folding = 1 / k

Results – tidal flushing

Figure 6-30 to Figure 6-33 show the time series tracer concentration for three locations (refer Figure 6-29) upstream of the proposed immersed tube tunnel crossing location under existing conditions and under the project scenario, respectively. The e-folding times (in days) were obtained by referring to the x-axis value intersecting with the line of constant concentration 1/e and extrapolation was applied where appropriate.

Table 6–4 shows the e-folding time for the Middle Harbour section upstream of the project crossing location under existing conditions as well as for the project conditions. A comparison of the model results show that the flushing times in Middle Harbour in the area upstream of the crossing are indicated to increase as a result of the immersed tube tunnels and the sill-like feature it creates on the bed of the harbour. This is particularly the case for the area immediately upstream of the immersed tube tunnels in both the surface and deeper layers of the water column.

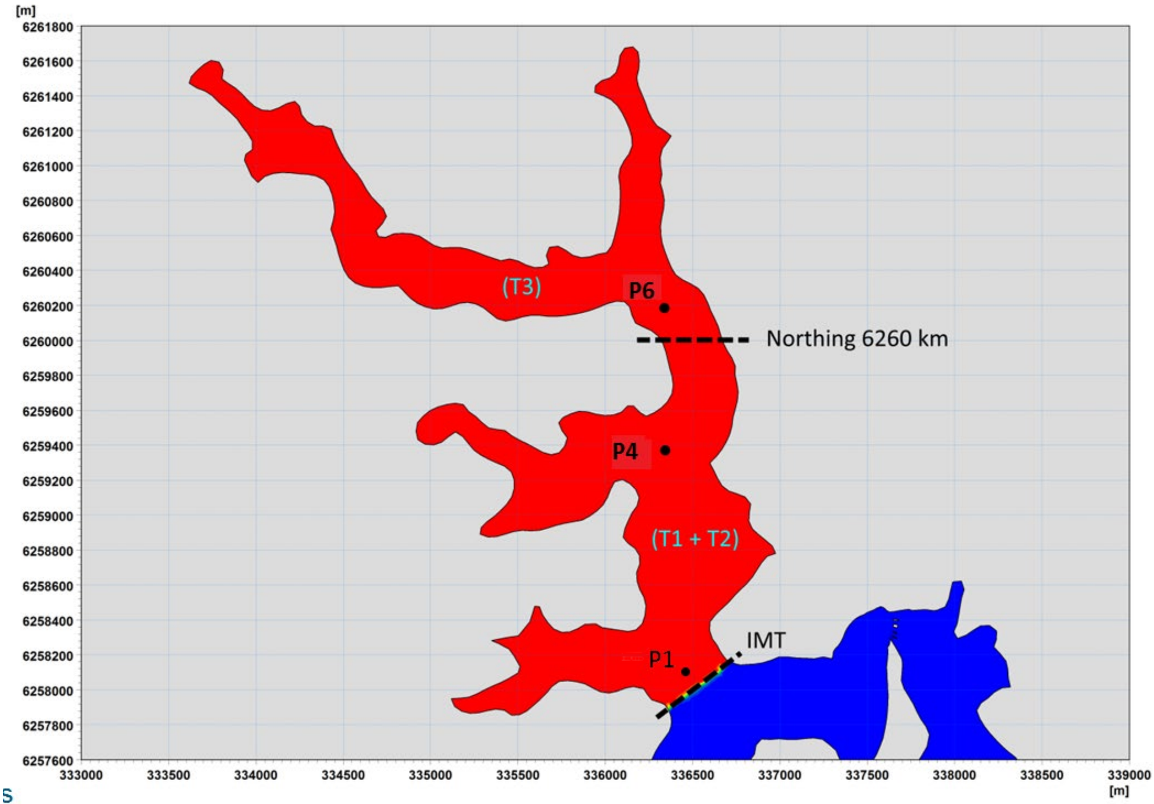


Figure 6-29: Locality plan: tidal flushing output locations

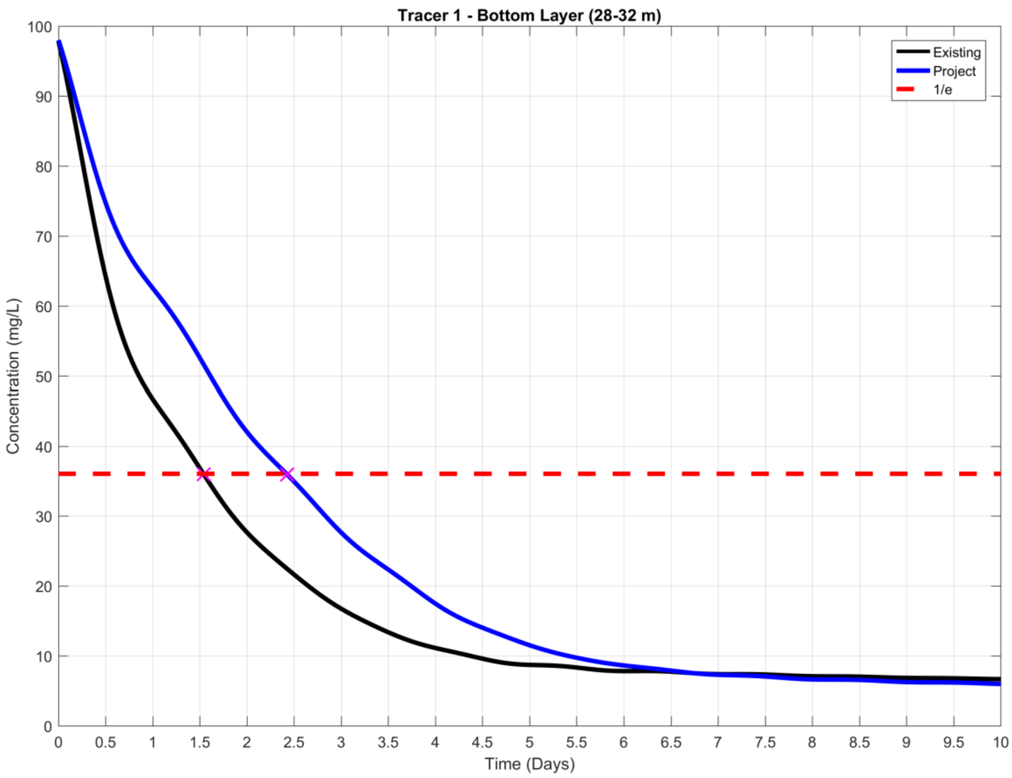


Figure 6-30: E-folding time of tracer T1 at output locations P1

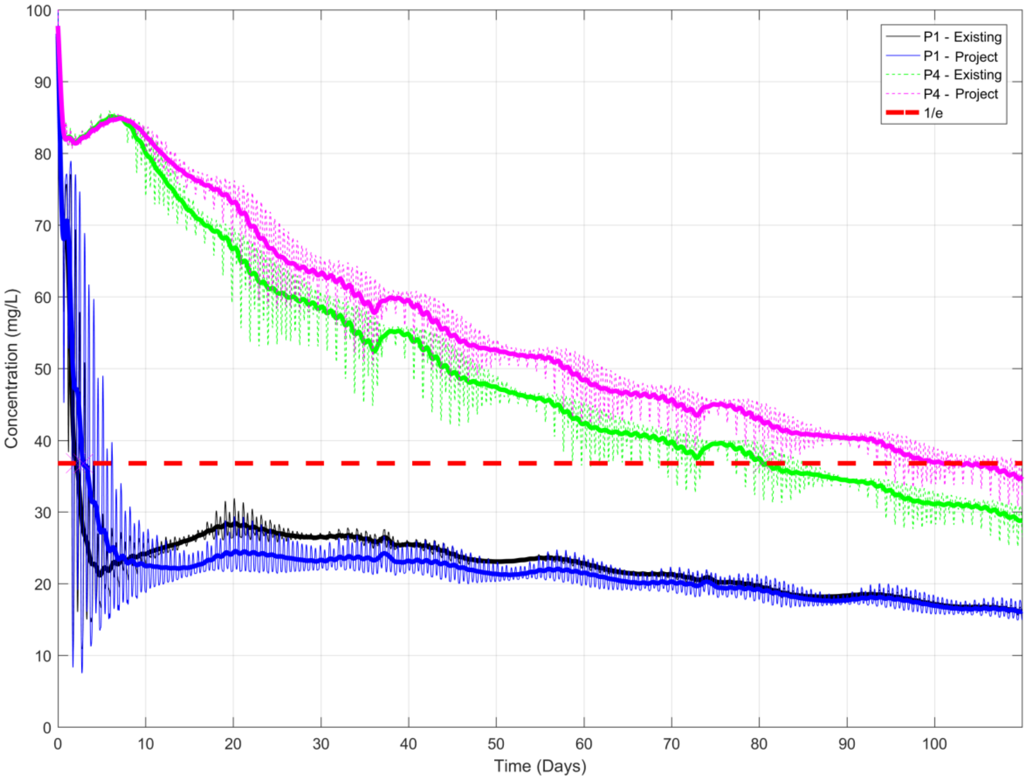


Figure 6-31: E-folding time of tracer T2 at output locations P1 and P4

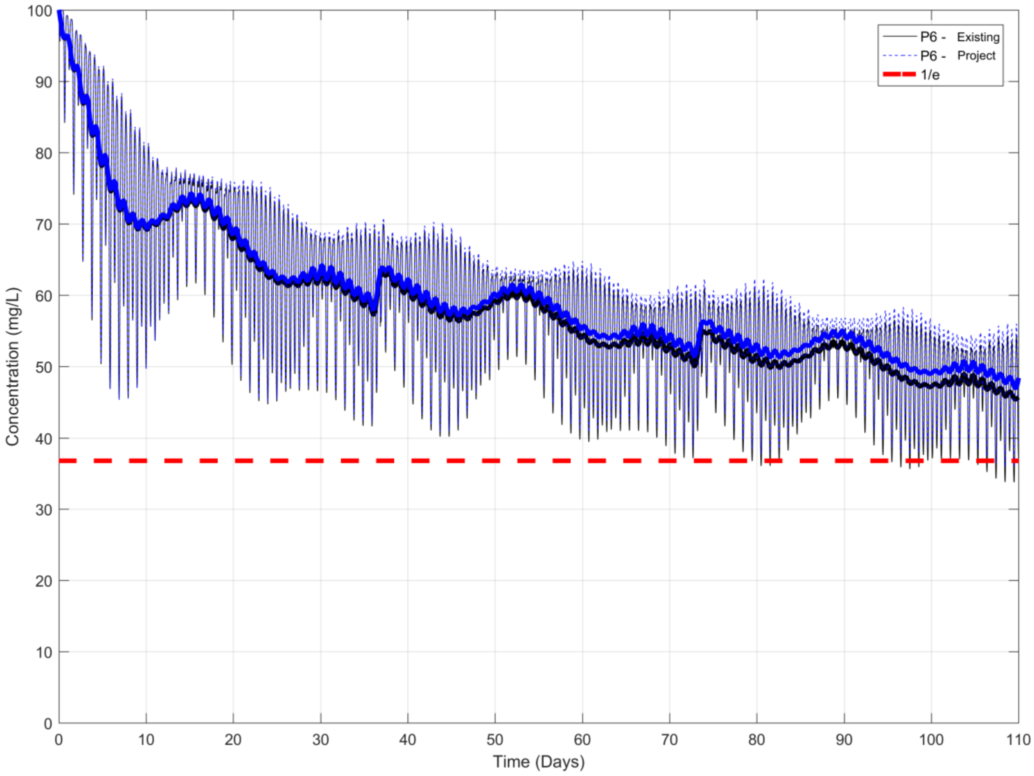


Figure 6-32: E-folding time of tracer T3 at output location P6

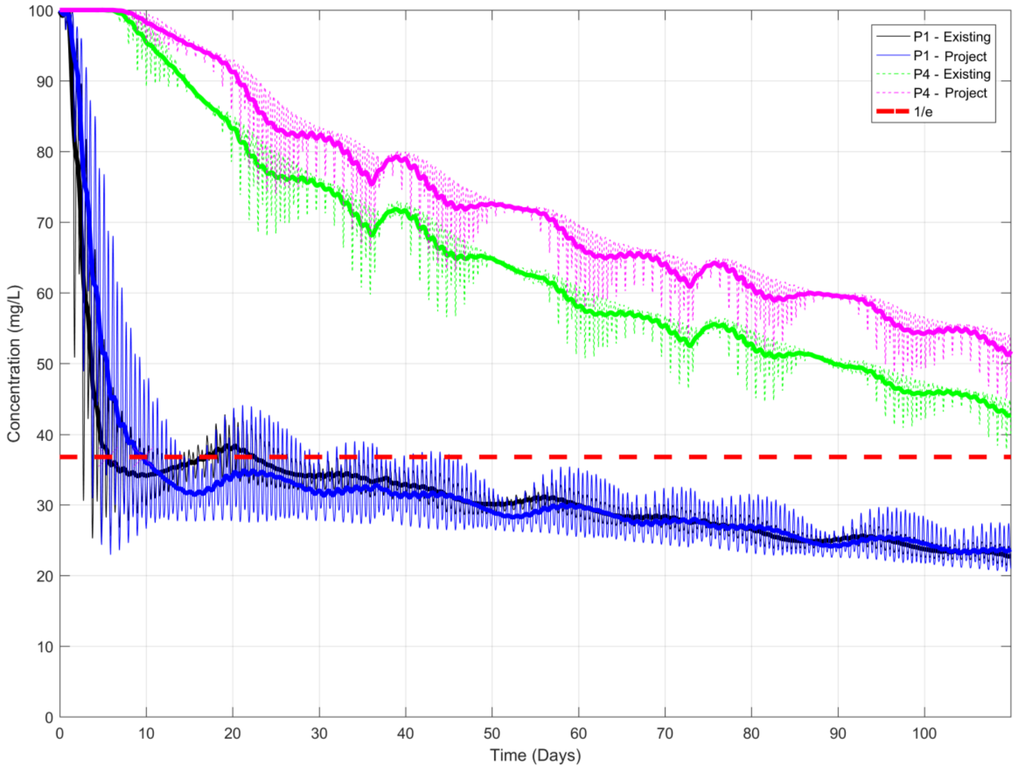


Figure 6-33: E-folding time of tracer T0 at output locations P1 and P4

Table 6–4: Comparison of flushing characteristics in terms of e-folding time (days)

Tracer and output location (depth layer)	e-folding times (days)		Percentage change in flushing time
	Existing	Project	
Tracer 1 @ P1 (Bottom layer, 28 to 32 metre water depth)	1.6	2.4	+50%
Tracer 2 @ P1 (Just above level of sill, approx. 16 to 20 metre water depth)	2.1	2.9	+38%
Tracer 2 @ P4 (Just above level of sill, approx. 16 to 20 metre water depth)	80.4	100.9	+25%
Tracer 3 @ P6 (Upper water column, approx. 8 to 12 metre water depth)	160*	163*	+2%
Tracer 0 @ P1 (Upper water column, approx. 8 to 12 metre water depth)	5.6	9.6	+71%
Tracer 0 @ P4 (Upper water column, approx. 8 to 12 metre water depth)	130*	174*	+34%

*Extrapolated

To further examine the effect of the immersed tube tunnels on the tidal mixing in the area upstream of the immersed tube tunnels sill, the P1 location was selected and further analysis of the T1, and T2 tracers completed. P1 was selected as it is 100 metres upstream of the immersed tube tunnels location and positioned over the main channel in the deep hole that is created upstream of the immersed tube tunnels (see initial T1 control volume between chainage 3500 metres and 5400 metres in Figure 6-28). The purpose of the additional analysis was to understand (i) how quickly water mixes down to the bottom in this area and then (ii) how quickly that water mixes upwards and out of the deeper layer.

Figure 6-34 to Figure 6-41 show the tracer concentration time series at the P1 location under both existing and project conditions. Using these figures various mixing and flushing time estimates were made and presented in Table 6–4. Where applicable, the time from the start of the simulation to the initial peak in concentration was calculated. The subsequent e-folding times (in days) based on the time taken for the initial peak concentration to dilute to a 1/e was then calculated. The e-folding times for the T2 tracer at layers above the immersed tube tunnels, below the immersed tube tunnels and at the bed of the harbour were beyond the exponential part of the concentration curve. For these three instances, a second parameter, the t_{40} time (or the time until of the peak concentration has decreased by 40 per cent), have also been presented. It should be noted that the surface time series have been inverted as they had logarithmic growth. The e-folding values for these plots represent the time taken for a tracer to increase to 1/e, 0.368 of the minimum concentration.

The T1 tracer results indicate that upstream of the immersed tube tunnels, water initially within the T1 control volume (in the deeper layers upstream of the sill) mixed upwards rapidly (ie over the course of three to four days). For example:

- At the layer above the immersed tube tunnels, concentrations start almost at zero and peak 0.6 days later for both cases. For the existing conditions flushing occurs slightly faster with an e- folding time of 1.4 days as opposed to 2.3 days for the project design scenario
- The tracer at the bed of the harbour is mixed upward through the water column, with e-folding times at the bed of 1.5 days and 2.4 days for the existing and project design scenario, respectively.

The T2 tracer results indicate that upstream of the immersed tube tunnels, water initially within the T2 control volume (upper layers upstream of the sill) mixed downward relatively rapidly (ie concentration at the bed of the harbour peaks after two days). For the water that has mixed down into the deeper layers, the results indicate that the initial reduction in concentration occur over a further 3.5 to 4.1 days, for the existing and project design scenarios at the bed of the harbour for t_{40}.

Overall, the results of the tidal flushing analysis indicate that the volume of water upstream and below the level of the sill would take longer to flush as a result of the project. However, the flushing times are still expected to be relatively rapid.

Table 6–5: Comparison of flushing characteristics in terms of e-folding time and t_{40} time (where applicable)

Tracer	Layer	Conditions	Start time (days)	e-folding (days)	t_{40} (days)
Tracer 1 (T1)	Surface* (See Figure 6-34)	Existing	2.0	1.1	
		Project	2.1	3.7	
	Above immersed tube tunnels (See Figure 6-35)	Existing	0.6	1.4	
		Project	0.6	2.3	
	Below immersed tube tunnels (See Figure 6-36)	Existing	1.0	1.7	
		Project	1.1	2.6	
	Seabed (See Figure 6-37)	Existing	0	1.5	
		Project	0	2.4	
Tracer 2 (T2)	Surface* (See Figure 6-38)	Existing	2.2	2.0	
		Project	3.3	n/a	
	Above immersed tube tunnels (See Figure 6-39)	Existing	1.1	79.0	1.1
		Project	1.1	63.7	2.3
	Below immersed tube tunnels (See Figure 6-40)	Existing	0.8	67.3	2.7
		Project	2.0	80.3	4.0
	Seabed (See Figure 6-41)	Existing	2.1	74.4	3.5
		Project	2.3	81.8	4.1

*inverted as there was a minima and then logarithmic growth

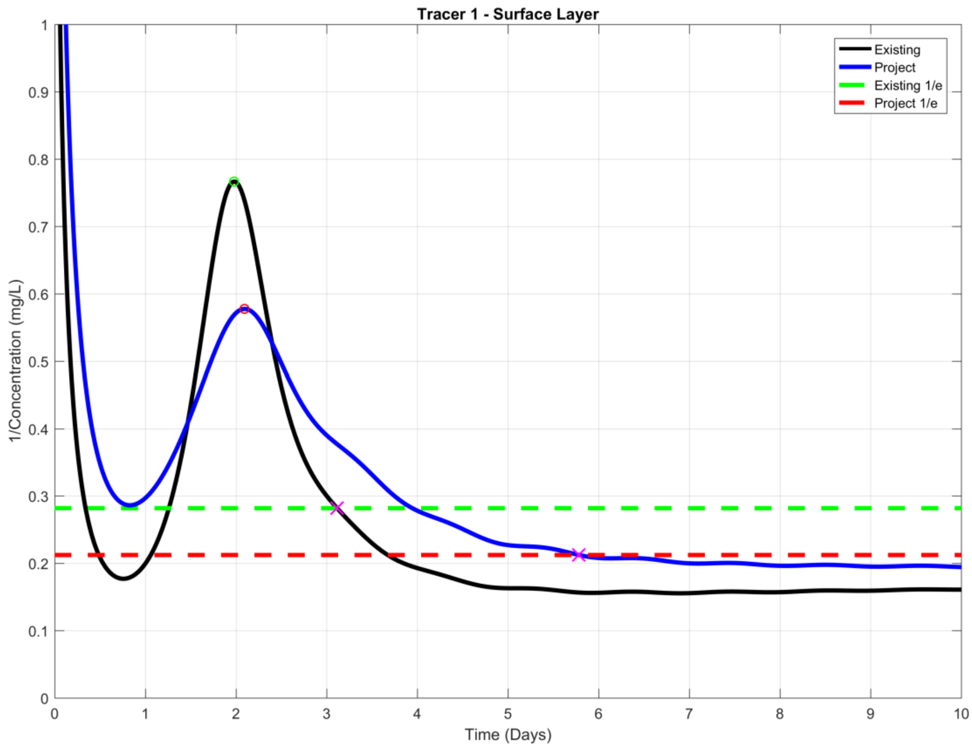


Figure 6-34: E-folding time of tracer T1, 100 metres upstream of the immersed tube tunnels at the sigma layer at the surface
Note: Concentration has been inverted (ie y-axis shows 1/concentration)

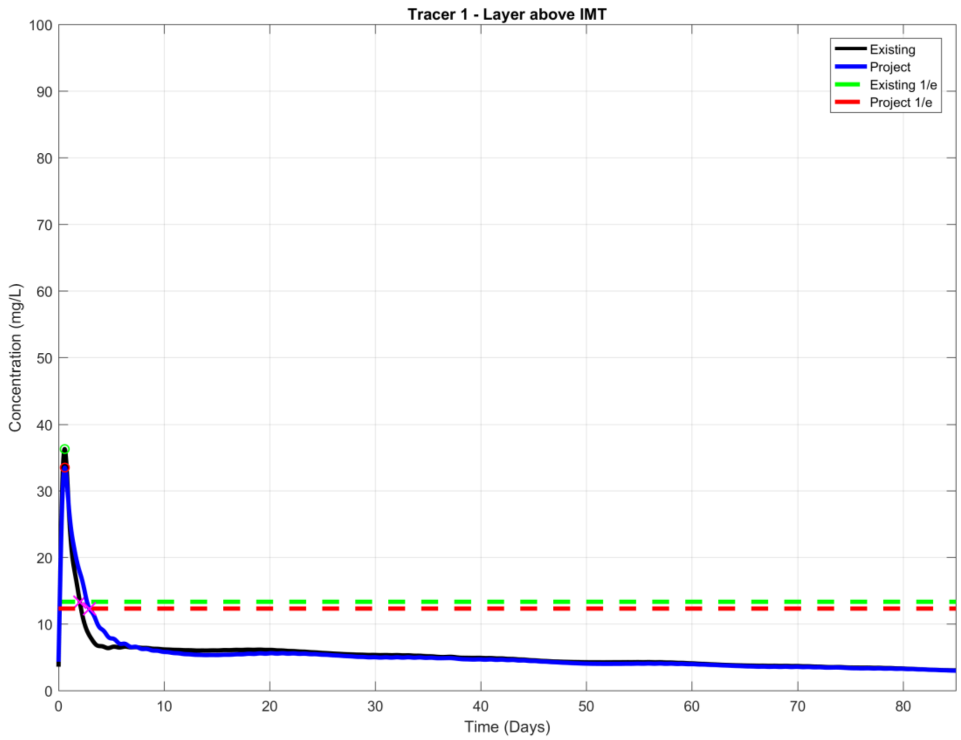


Figure 6-35: E-folding time of tracer T1, 100 metres upstream of the immersed tube tunnels at the sigma layer just above the immersed tube tunnels sill
Note: On these and similar figures below, the 'o' markers on the time series indicate the peak concentration used to determine the constant concentration line. The 'x' makers indicate the e-folding time

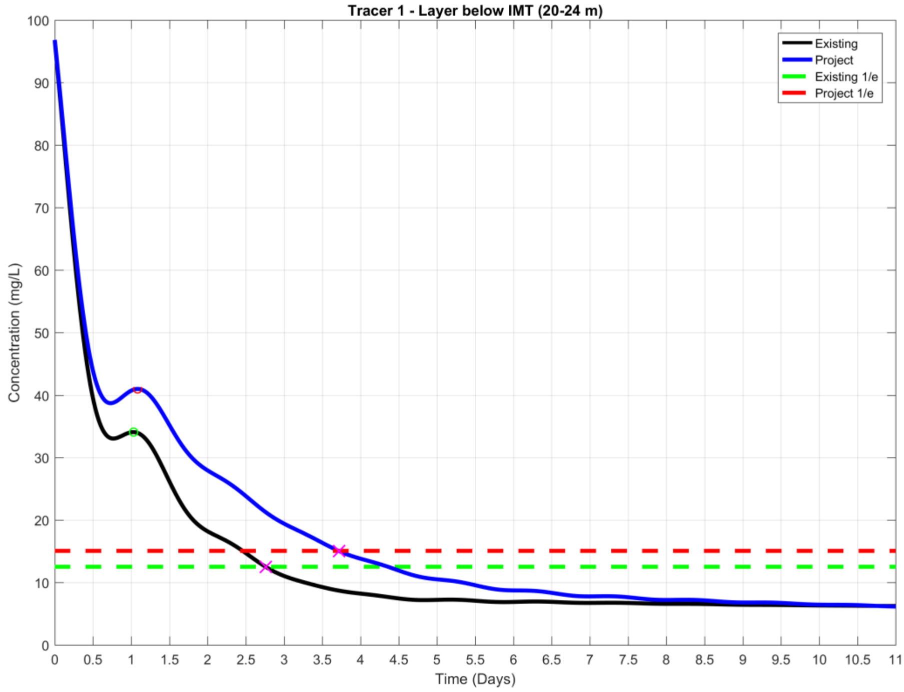


Figure 6-36: E-folding time of tracer T1, 100 metres upstream of the immersed tube tunnels at the z-layer just below the immersed tube tunnels sill (20 to 24 metres)

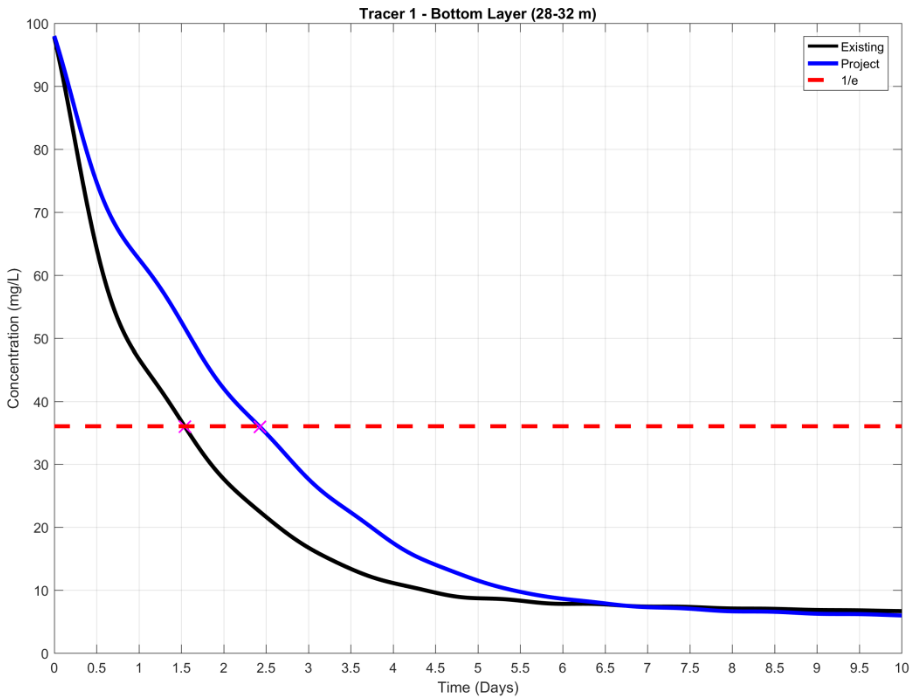


Figure 6-37: E-folding time of tracer T1, 100 metres upstream of the immersed tube tunnels at the z-layer at the bed of the harbour (28 to 32 metres)

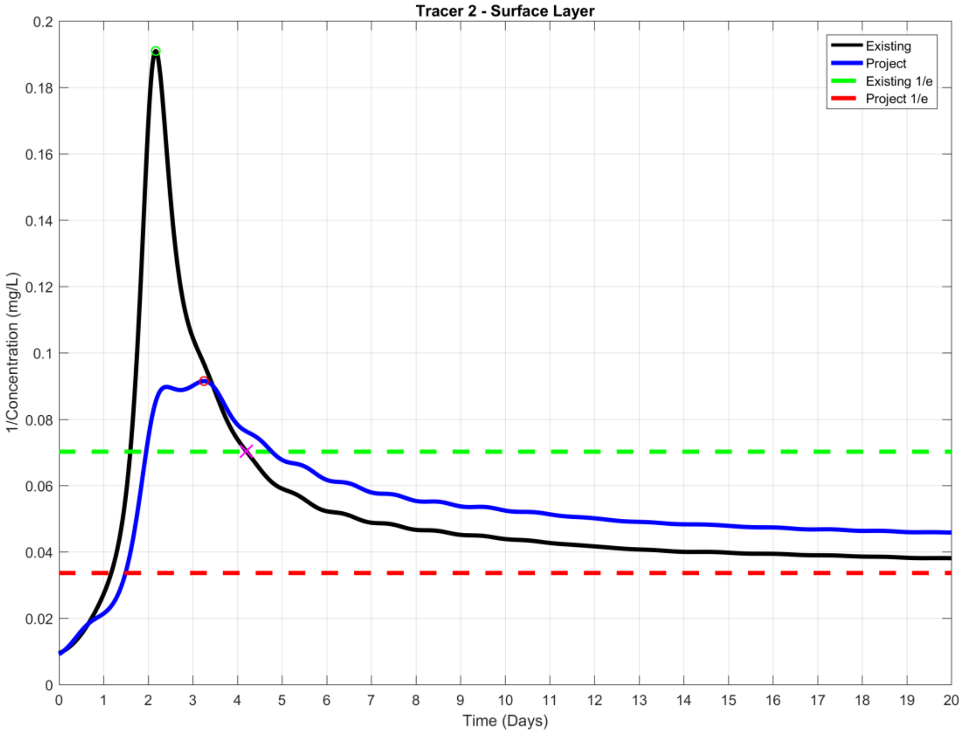


Figure 6-38: E-folding time of tracer T2, 100 metres upstream of the immersed tube tunnels at the sigma layer at the surface

Note: Concentration has been inverted (ie y-axis shows 1/concentration)

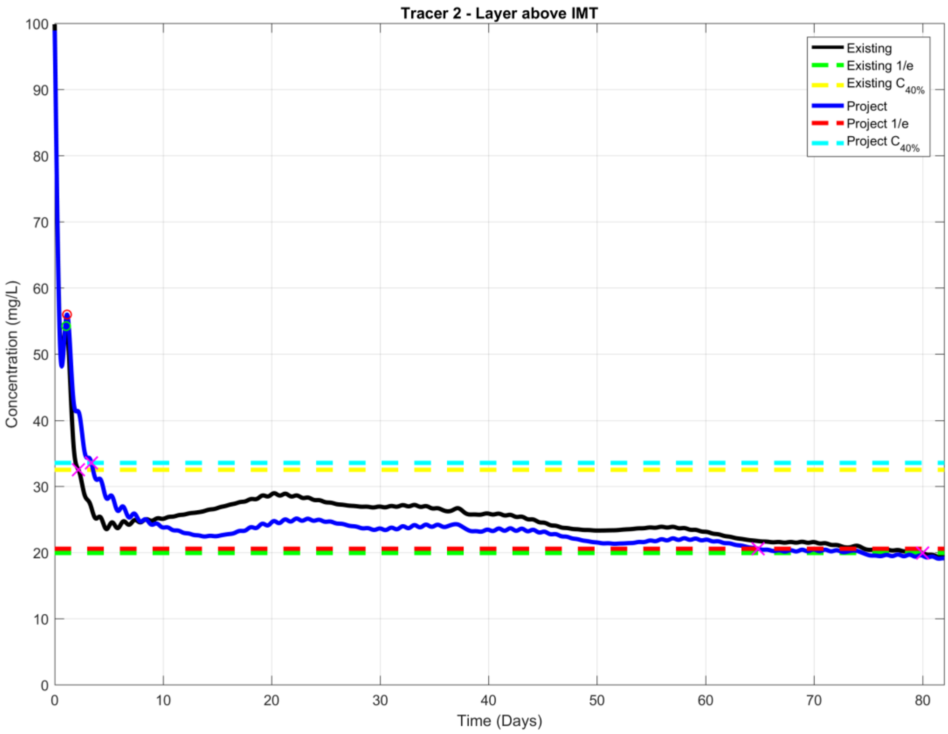


Figure 6-39: E-folding and t40 time of tracer T2, 100 metres upstream of the immersed tube tunnels at the sigma layer above the sill

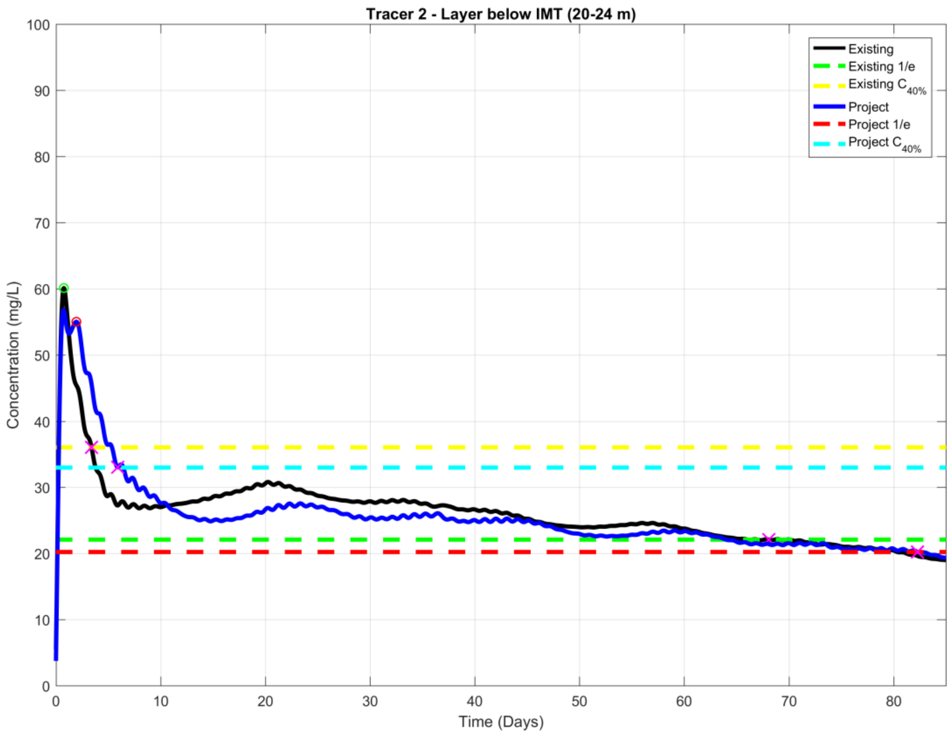


Figure 6-40: E-folding and t40 time of tracer T2, 100 metres upstream of the immersed tube tunnels at the z-layer below the sill (20 to 24 metres)

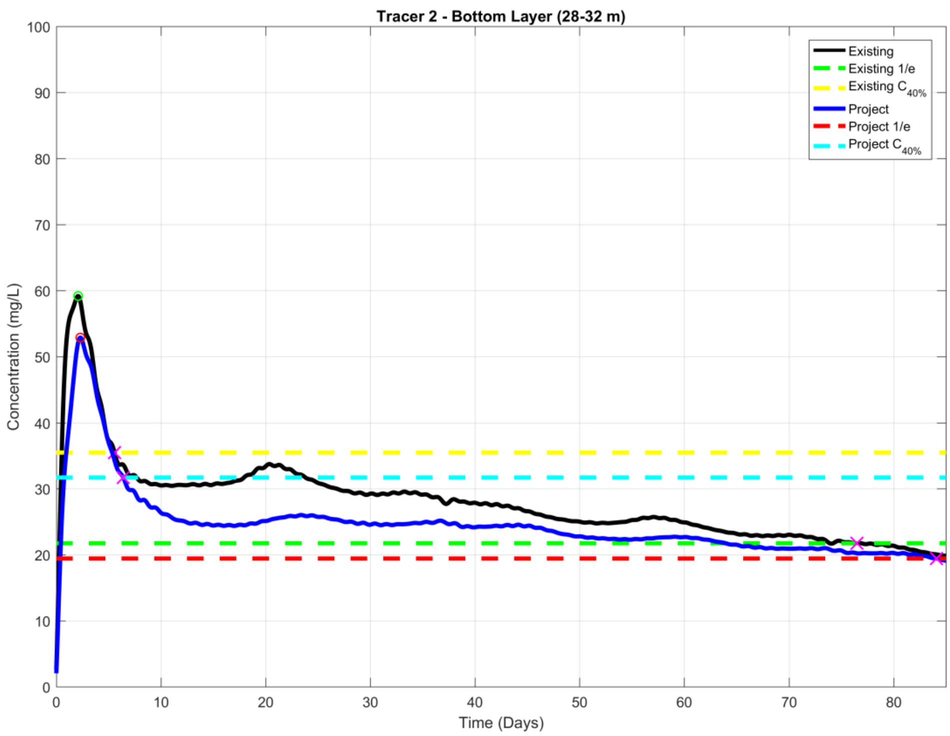


Figure 6-41: E-folding and t40 time of tracer T2, 100 metres upstream of the immersed tube tunnels at the z-layer at the bed of the harbour (28 to 32 metres)

7 Dredging plume modelling

7.1 Overview

Construction of the immersed tube tunnels would require dredging activities to be carried out. To assess potential impacts resulting from these dredging activities, numerical modelling was carried out to represent the potential movement of sediments released into the water column through dredging. The modelling simulated the dispersal of suspended sediments by ambient currents as well as the subsequent deposition of sediment suspended by dredging. The modelling simulated sediment suspended by dredging (ie dredge SSC), and therefore shows the potential impact dredging has on water quality. Ambient SSC and sedimentation were not simulated. The disposal of the dredged material was not a relevant consideration in the dredge plume modelling. Dredged material would be disposed of either offshore at an approved designated offshore disposal site or on land. For further details refer to Chapter 2 (Assessment process) of the environmental impact statement.

To represent the dredging activities in the numerical model it is necessary to define the quantity, characteristics, location, duration and frequency of the material released. RHDHV, in consultation with an appointed dredging expert, developed a dredging strategy, enabling a realistic representation of the actual dredging works. The amount of material that would become suspended in the water column during the dredging activities is referred to as the source term.

The modelled source terms are dependent on a number of parameters which relate to several aspects and processes, including the fines content of the material to be dredged, the breakup of the dredge material under mechanical action and hydraulic transport. Other factors, including dredger efficiency, production rates and cycle times also feed into the magnitude of the source term.

The following subsections summarise the interpretation of available geotechnical information, estimated dredge material quantities, dredging methodology and properties for the purpose of representing the relevant source terms in the numerical model.

7.1.1 Material to be dredged (properties and quantities)

The quantities and characteristics of the material to be dredged influences both the quantity of material released and the make-up of fine sediment released into the water column. A summary of the estimated dredging volumes required for the construction of the project is provided in Table 7–1. The volumes and sediment properties were based on information obtained from detailed geotechnical investigations as provided by Transport for NSW.

Geotechnical investigations typically identified areas of silty sands and silty clays overlying rock (sandstone). Areas of silty sand were largely identified at the banks of the proposed crossing location, while the surface sediments in the centre of the crossing were principally silty clays. Estimates of the various materials and quantities which would be dredged for the project are summarised in Table 7–1.

Table 7–1: Estimated dredge material quantities and properties

	Dredge material	In-situ volume (m^3)	Dry density (kg/m^3)	In-situ fines (%)*
Sediment unsuitable for offshore disposal***	Silty clay	8000	750	83
	Silty sand	4000	1280	20
Sediment and rock suitable for offshore disposal	Silty clay	6000	750	83
	Silty sand	35,000	1280	20
	Soft (weathered) rock (Dredged with backhoe dredge)	1500	2300	3
	Hard (less weathered) rock (Crushing)	73,500	2300	14
	Rehandled rock	73,500**	1600	14

* Coarse silts and finer (less than 63 microns).

** This volume reflects the solid in-situ volume rather than the bulked volume following crushing by the drum cutter. The bulked volume would be more (around 106,000 m^3)

*** Preliminary testing has indicated that upper layer materials may not be suitable for offshore disposal due to existence of contaminants. Further sampling and testing would be carried out to confirm the preliminary findings and actual volumes. Should the further sampling and testing confirm the preliminary findings the material would be removed, treated and disposed at a suitably licensed waste disposal facility on land.

7.2 Dredge description

RHDHV engaged a dredging expert to assist in developing a realistic dredging strategy that would likely be employed to deliver the required dredging works. A detailed strategy was developed from the perspective of a dredging contractor whereby the most efficient means of dredging, with regard to environmental constraints, would be achieved. The methodology was informed by the proposed design, available geotechnical information and availability of suitable plant.

The dredging plant and equipment that is proposed to be mobilised includes:

- Very large backhoe dredge (BHD) with closed bucket and standard (open) bucket attachments (note that the type of closed bucket may constitute an environmental bucket or clamshell, which is a particular form of closed bucket that closes along a horizontal plane)
- Very large BHD fitted with drum cutters or pre-cutting blade
- Sweep bar unit and airlift
- Non-propelled (dumb) barges
- Self-propelled split hopper barges.

Photos of the typical plant are provided below.

The proposed dredging method also included a number of environmental management measures to ensure best practice from an environmental perspective would be achieved. These measures include but are not limited to:

- The installation of silt curtains around some dredging plant
- The installation of silt curtains along the adjacent Clive Park and Seaforth Bluff shorelines providing added protection to nearby ecologically sensitive areas (eg. seagrass and rocky reef habitat)
- Ensuring no overflow from the receiving hopper barges
- The use of specialist dredge equipment (ie closed environmental clamshell – refer below)

- Incorporation of accurate positioning systems, such as Real Time Kinematic, on the BHD which provides the operator with real time positioning of the dredging bucket relative to the bed to minimise disturbance during lowering of the bucket and allow precision dredging
- Implementation of a detailed water quality monitoring program
- A full time supervision and inspection regime
- Implementation of a dredge management plan, which would form part of the construction environmental management plan.

The environmental clamshell has been specifically designed for dredging of sediments containing elevated contaminants and provides three significant advantages compared to conventional open buckets:

- Minimisation of suspended sediments
- Minimisation of spill
- Precision (accurate dredging).

Accurate dredging is achieved by real time monitoring and control systems and the fact that the environmental clamshell closes horizontally to provide a level cut as opposed to a conventional semi-circular or arched cut. In this way, relatively thin layers of sediment can be removed in a controlled manner.

The dredge methodology proposed for the dredging of the Middle Harbour crossing is summarised below:

1. All material unsuitable for offshore disposal would be dredged by a very large BHD working in conjunction with hopper barges (refer Figure 7-1). The BHD would be fitted with a closed environmental clamshell and would load dredged material directly into hopper barges positioned immediately adjacent to the dredge (with no overflow). The material would be transported to a load out facility on shore (location to be determined) for treatment and land disposal. No barge overflows or losses from barges would be allowed. While this leads to lower production rates, the loss of sediment into the water column would be greatly reduced.
2. Following validation that all material unsuitable for offshore disposal has been removed, dredging of underlying material that is considered to be suitable for offshore disposal would be completed by a very large BHD. The BHD would remove the remaining non-cohesive materials comprising soft clay/silt and silty sand, and would also remove cohesive materials including hard clays and soft rock (one to five MPa strength). The BHD would be fitted with a closed environmental clamshell (for soft clay/silt, silty sand) and a standard open bucket (for hard clays and soft rock) (refer Figure 7-1). Material would be loaded into a fleet of split hopper barges (with no overflow) having a suitable capacity and number to match the BHD production rate. The hopper barges would be used to transport the dredged material overwater to the designated offshore disposal site. The hopper barges would be self-propelled (refer Figure 7-3).
3. Hard rock (greater than five MPa strength) could be pre-cut with a large steel blade attached to the BHD before being crushed with a very large BHD fitted with a set of drum cutters (refer Figure 7-4) on its dipper arm (instead of a bucket). The drum cutter would be lowered by the BHD to crush the rock into small fragments. The crushed rock would be left within the tunnel trench for subsequent removal by a very large BHD fitted with a standard open bucket. The BHD would need to work in close coordination with the drum cutter BHD to remove crushed rock before the drum cutter BHD returns to perform another cut. The BHD with standard bucket would load the crushed rock into self-propelled split hopper barges (with no overflow) for transport and disposal of the material at the designated offshore disposal site.

Figure 7-1: Example backhoe dredge loading hopper barge with closed 'bucket' (environmental clamshell) attachment

Figure 7-2: Example of a very large backhoe dredge operating with standard open bucket attachment

Figure 7-3: Example of self-propelled split hopper barge

Figure 7-4: Example of drum cutter attachment on an excavator stick

7.2.1 Dredge schedule

A schedule has been prepared for completion of dredging. This proposed schedule was used to determine the duration and frequency of material being released into the model and is summarised below. For the purpose of developing the dredge plume models, it was assumed that dredging would occur Monday to

Friday between the hours of 7am and 6pm. It is noted that dredging may potentially occur outside these hours, provided it is consistent with the project planning approval and environmental protection licences.

Stage 1 – Sediment and soft (weathered) rock removal

- Dredging of material with the very large BHD would occur five days per week (Monday to Friday) for a total of 11 hours a day during daylight hours (7am to 6pm)
- Actual dredging hours for sediments and soft rock is estimated to be 7.5 hours a day, which allows for an estimated average time of start-up activities and downtime of 3.5 hours a day
- The very large BHD would stay positioned at the one location until all sediment is removed to the target dredge level, after which the BHD would move incrementally across the crossing (assuming a south to north direction).

Stage 2 – Hard rock removal

- A BHD fitted with a drum cutter would be used to crush regions of hard rock within the dredge footprint. The drum cutter BHD would operate five days per week (Monday to Friday) for a total of 11 hours per day during daylight hours. Actual dredging hours utilising the drum cutter BHD are estimated to average 6.5 hours a day, which allows for an estimated average time for start-up activities and downtime of 4.5 hours a day
- Intermittently throughout the crushing process, the very large BHD with standard open bucket would handle crushed rock through placement into hopper barges for offshore disposal.

The rate at which the material is dredged is dependent on the properties of the material being dredged and the type of equipment used. A detailed description of the assumed working hours and production rates for each plant utilised has been developed. Based on these assumptions and estimates, the overall program for capital dredging extends over a period of 37 weeks, excluding mobilisation and site establishment.

It is noted that the particular daily dredging hours referred to above, as opposed to 24/7 dredging which is otherwise common on larger dredging projects, is an environmental benefit in that it reduces the mass rate of sediment released to the water column which potentially gives rise to elevated turbidity.

The downtime allowance above covers a range of matters including moving position, adverse weather conditions such as strong winds, maintenance of equipment, and breakdowns.

7.2.2 Material release to the water column by dredging

As previously discussed, the dredging methodology involves no overflow of material from barges as part of the loading process. Furthermore, it is intended that the disposal of all dredged material is either to be onshore or be placed offshore, at a designated site outside of Port Jackson. Therefore, the only potential source for material to be suspended into the water column would be through the direct dredging/loading process (ie at the drum cutter face or BHD bucket).

The rate of material suspended into the water column from dredging activity is dependent on both the method of dredging and the properties of the materials being dredged. Table 7–2 presents the assumed losses of fines during each of the proposed dredging activities. The percentages provided relate to the rate of dredge material lost and suspended in the water column as a proportion of the total production rate.

The source terms applied for the various dredging activities and sediment types were derived based on a review of geotechnical information and information from other dredging projects.

Table 7–2: Assumed losses of fines from dredging activities

Method of dredging	Material dredged	Material losses (per cent)
BHD - Environmental Clamshell[1]	Silty clay	1.5
BHD - Environmental Clamshell[1]	Silty sand	1.5
BHD - Closed Bucket	Silty clay	1.8
BHD - Closed Bucket	Silty sand	1.5
BHD - Closed Bucket	Sandstone (soft, weathered rock)	2.0
Drum Cutter (crushing only)	Sandstone (less weathered or hard rock)	3.5
BHD (rehandle crushed rock)	Sandstone (less weathered or hard rock)	1.5

[1] Used for the dredging of sediments unsuitable for offshore disposal.

Source terms, given in Table 7–2 as a percentage of the total quantity to be dredged, have been converted into a rate of fine sediment released (in kilograms per second (kg/s)) into the water column for application in the model. The conversion was informed by:

- Measured dry density data from the geotechnical information provided by Transport for NSW as well as information from previous RHDHV projects
- Particle size distribution information from the geotechnical site investigations (which have been carried out using a freshwater hydrometer) and surface sediment samples collected on behalf of Transport for NSW and analysed using laser analysis methods as well as information reported from other nearby projects (eg Sydney Metro City & Southwest).

Further analysis would be carried out to confirm the properties of the material to be dredged, in particular for the case of the particle size distribution for the softer sediments.

The depth layer at which material is released into the model is dependent on the type of dredge plant being used. Suspended sediment is released only into the bottom water layer for dredging activities involving the drum cutter. For the BHD, sediment release is modelled with a uniform release throughout the water column. These approaches are considered to be physically realistic release distributions through the water column.

It is intended that two deep draft silt curtains (ie deep draft silt curtains 12 metres deep) would be used, one at each side of the crossing (ie adjacent to the cofferdams) in the area where most dredging would be required. The use of deep draft silt curtains is permissible due to low flows in the area. The deep draft silt curtains would limit the surface sediment plume when the BHD bucket is lifted through the water column and above the water surface. The deep draft silt curtains were replicated in the model through the inclusion of a physical structure which limits the advection of sediment in the top 12 metres of the water column. In addition to the deep silt curtains, shallow draft silt curtains (ie shallow draft silt curtains about two to three metres deep, sometimes referred to as a 'moon pool') would also be attached to the BHD itself and the dredge bucket would enter and leave the water column within the surface silt curtains.

There would be brief periods at the start of the dredging activities when the BHD operates outside the deep silt curtains. In these instances it is intended that the shallow silt curtains would be used around the dredge. Inclusion of shallow draft silt curtains was assumed when deriving the source terms resulting from the dredging (as opposed to the shallow draft silt curtains being included as structures in the hydrodynamic model).

As noted previously, shallow draft silt curtains would also be installed adjacent to nearby ecologically sensitive areas (eg. seagrass and rocky reef habitat) to provide additional protection.

7.3 Dredge plume modelling

The dredge plume results have been generated using RHDHV's calibrated MIKE 3 Flexible Mesh (FM) 3D hydrodynamic model of Port Jackson for the project crossing described in **Section 4** and **Section 5**. For the purpose of dredge plume modelling:

- Tidal hydrodynamics were simulated for a 16-week period
- The model's bathymetry was based on survey provided by Transport for NSW
- Five sigma layers were used
- Two deep draft silt curtains (12 metres deep) and two temporary cofferdams (Middle Harbour south cofferdam (BL7) and Middle Harbour north cofferdam (BL8)) were included.

The MIKE 3 FM Mud Transport (MT) module was used to model the dispersion of fine sediment released into the water column during dredging. The model applies a moving source term to represent how the dredger moves to dredge different areas.

Only fine sediments (less than 63 microns) were included in the model, which was setup to include four sediment fractions: clay (less than two microns), fine silt (two to 6.3 microns), medium silt (6.3 to 20 microns) and coarse silt (20 to 63 microns). The sediment fractions used in the model were based on size classes detailed in *ISO 14688-1:2002 Geotechnical investigation and testing — Identification and classification of soil — Part 1: Identification and description*. The percentage distribution of the four fractions was based on the particle size distribution described above.

The spatial distribution and the quantities of different materials to be dredged were derived from the geotechnical site investigations. Based on the sequencing of dredging activities (both dredge plant and sediment/rock types) and the spatial distribution, a number of time series with moving source terms were derived and applied to the dredge plume model.

The time series of source terms as a rate of release of fine sediment (in kg/s) is provided in Figure 7-5.

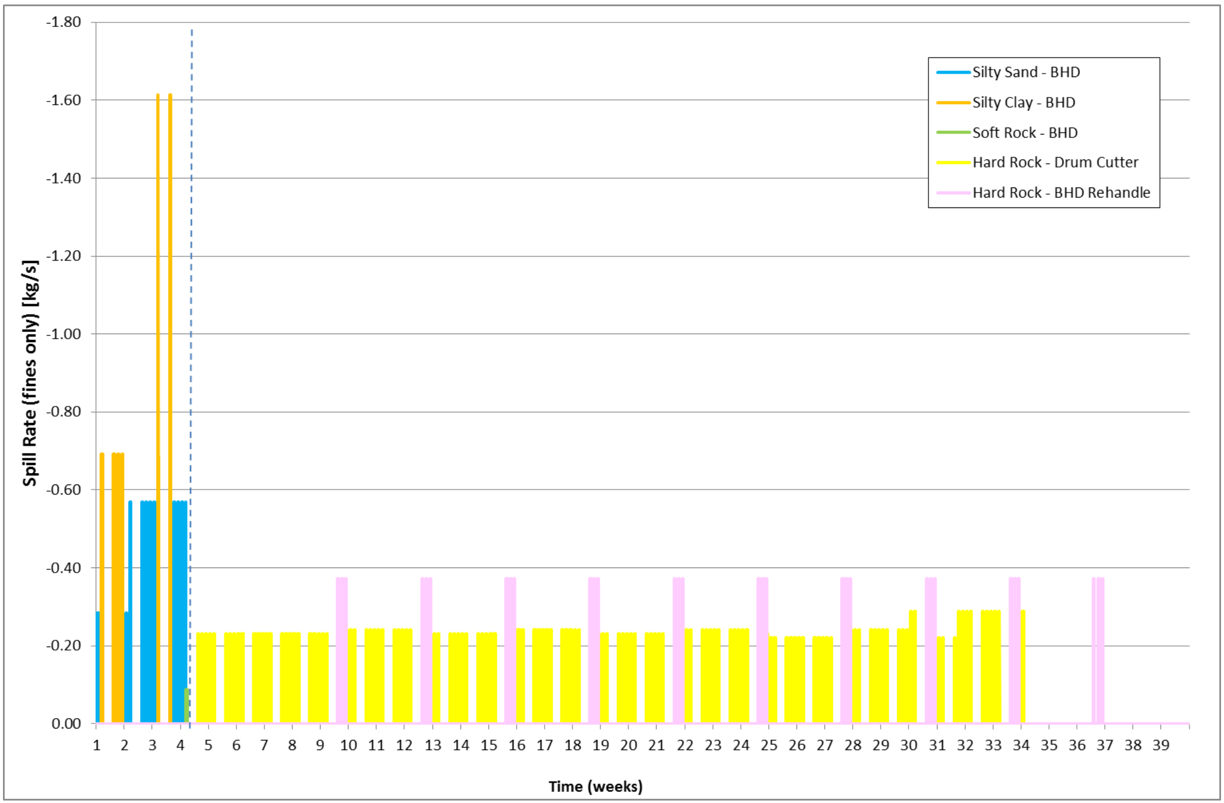


Figure 7-5: Summary of source terms for fine sediment released by dredging at the crossing at Middle Harbour

The entire dredging program was simulated by looping each of the source term time series through the 16 weeks of tidal hydrodynamics until the entire dredge duration was modelled for the crossing.

The following assumptions were made as part of this assessment:

- The dredge plume model does not include any background ambient suspended sediment concentration (SSC)
- Suspended sediment is released into the bottom water layer for the drum cutter for all soil types. The BHD releases sediment uniformly throughout the water column
- No overflow of any barges used with the BHD occurs
- The model includes the deposition and subsequent resuspension of sediment released from the dredging activity. Any sediment deposited on the bed has been assumed to be readily available for resuspension, with a critical erosion threshold of 0.1 newtons per square metre (N/m^2) applied
- The plume resulting from the immersed tube tunnel bedding and backfilling activities would be small compared to the dredging activity and these were not included in the modelling
- Any clay particles released by the dredging activity would be subject to flocculation. The amount of flocculation was assumed to be low, only allowing relatively small flocs to form. The settling velocity of the clay sized particles in the model was been calculated assuming the size of flocs which would be expected to form with a SSC of 10 mg/L.

7.4 Water quality modelling results

7.4.1 Suspended sediment

The SSC due to dredging (ie dredge plume) model results were processed to calculate the spatial percentile exceedance maps over the duration of the dredging. The percentile plots do not show an actual dredge plume at any point in time, they are duration-based plots which show statistical summaries of the dredge plume over the selected dredge period. The percentile plots show the value for which SSC throughout the dredging duration is ‘less than’, a given percentage of the time. For example:

- The 90th percentile shows the value that is predicted to be exceeded for 10 per cent of the time, or 16.8 hours in a week
- The 95th percentile shows the value that is predicted to be exceeded for five per cent of the time, or 8.4 hours in a week.

The percentile plots were processed for specific periods that related back to the varying dredging activities (refer to **Section 7.2.1**). These periods are indicated on Figure 7-5 and listed as:

- The entire 37 weeks of the dredging program
- Weeks one to four of dredging when the BHD is working in soft sediments with high source terms
- Weeks five to 37 when the BHD and drum cutter work to remove rock on either side of the crossing with moderate source terms.

Annexure B provides a complete set of percentile SSC plots. This includes percentile plots of the modelled SSC due to dredging for the 90th and 95th percentiles for three vertical layers. Example plots showing the 95th percentile over the entire dredging duration are shown in Figure 7-6. The colour scale applied to the plots has been designed to show when the plume is likely to start to become clearly visible; based on previous experience this was estimated to be at 20 mg/L (assuming a background SSC in order of five mg/L) when the colour scale changes from blue to green. However it should be noted that the SSC at which a dredge plume can be considered visible varies depending on the material being dredged, ambient water colour and ambient atmospheric conditions. It is also subjective as it depends on the observer.

In addition to the percentile plots, time series plots of the SSC due to dredging are shown for the dredge duration. The time series plots, for the surface layer, are shown for Clive Park, MH1, MH2 and Clontarf (Figure 7-7) model extraction locations. These locations are also shown on the dredge plume SSC plots (see Figure 7-6). They have been selected as they represent sites that have been or are being monitored as part of the project, they are near sites which could be impacted by the dredging, or are locations directly adjacent to the dredging (ie Clive Park). Table 7–3 presents summary statistics of the modelled SSC for the four selected locations. The surface layer is presented as this is most relevant to visible plumes and for comparison to ambient monitoring data which is typically collected at the surface.

Table 7–3: Summary of suspended sediment concentration statistics in the surface layer at four model extraction locations

Location	Percentile SSC (mg/L)				
	20th percentile	50th percentile	90th percentile	95th percentile	99th percentile
Clontarf	0.02	0.34	0.64	0.78	1.17
MH1	0.03	0.54	1.22	1.63	2.94
MH2	0.03	0.56	1.16	1.45	2.50
Clive Park	0.02	0.33	0.94	1.16	1.71

The percentile plots of the SSC caused by dredging show:

- When analysed over the entire dredge duration and viewed at the 95th percentile level (ie less than five per cent of the time), the spatial extent of the dredge plume (SSC greater than two mg/L) would be relatively small. There are two main factors that influence this result:
  - The deep draft silt curtains would effectively capture sediment in the surface layer and to a lesser degree in the middle layer and near the harbour bed layers. The surface plume would be essentially contained within the deep draft silt curtains
  - Current speeds are generally low in the area with the tidal stream flowing along Seaforth Bluff during the ebb and flood tide. Sediment in the lower layers would be transported within this tidal stream adjacent to the Seaforth shoreline.
- The suspended sediment released during the dredging activity would be transported both in an upstream and downstream direction, with a downstream dominance, particularly along the Seaforth shoreline. This is a result of the tidal currents that are predominately aligned with the main longitudinal axis of the estuary
- The dredge plume extents would be greater in the near bed layer (layer one) than at the surface (layer five) (see Figure 7-6).

Time series plots of the dredging SSC provides an understanding of the influence of the SSC from dredging on the natural environment. The results show:

- At Clive Park-Northbridge, a site immediately outside the south west (Northbridge) silt curtain, elevated SSC would be observed at the surface. Values of up to 3.5 mg/L are predicted for brief periods during the dredging campaign. For 99 per cent of the dredge time, SSC in the surface layer are predicted to be less than 1.7 mg/L
- For MH1, which is located further downstream from the dredging footprint, the 95th percentile SSC in the surface layer is predicted to be 2.9 mg/L. Peak SCC values of up to 5.4 mg/L are predicted
- At MH2, which is closer to the dredge footprint than MH1, peak values of up to eight mg/L in the surface layer occurred during the initial four weeks of dredging when the BHD is removing surface sediments. For 95 per cent of the dredge time SSC would be less than 2.5 mg/L
- At Clontarf, SSC values would be consistently low (less than 2.1 mg/L) through the dredge campaign. For 99 per cent of the time, dredge SSC in the surface layer are predicted to remain below 1.2 mg/L.

The SSC modelling results consistently show that SSC would be generally low (less than five mg/L) for areas outside of the silt curtains with higher concentrations predicted in the bottom layer. The waters adjacent to Seaforth Bluff would most likely experience moderate increases in SSC, particularly in the downstream direction.

The additional silt curtains along the two shorelines were not modelled and would be assumed to give added protection to nearby ecologically sensitive areas (eg. seagrass and rocky reef habitat).

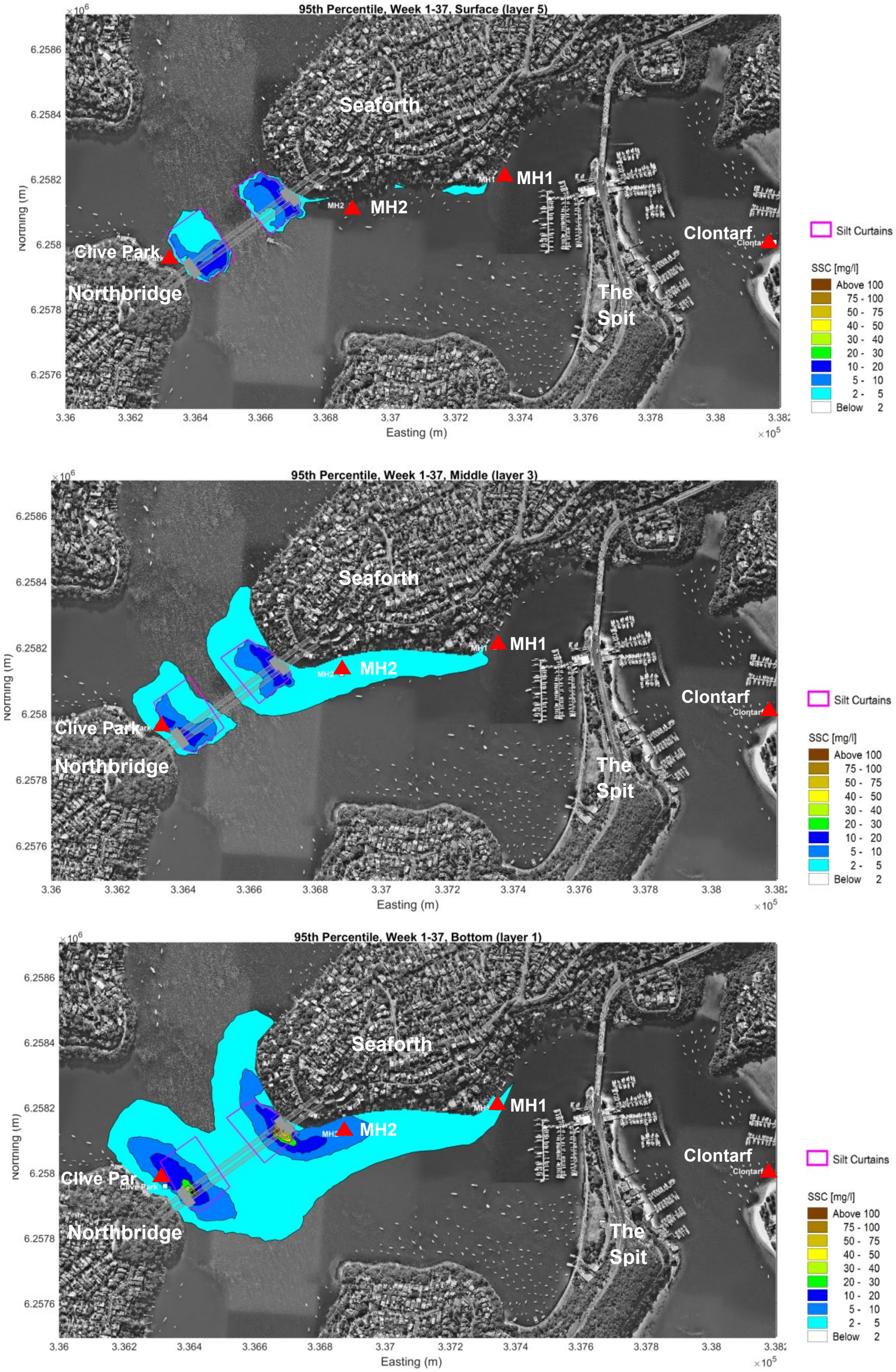


Figure 7-6: 95th percentile, for surface (top), mid-water column (middle) and near the bed of the harbour (bottom) for the entire dredging period (weeks 1 to 37)

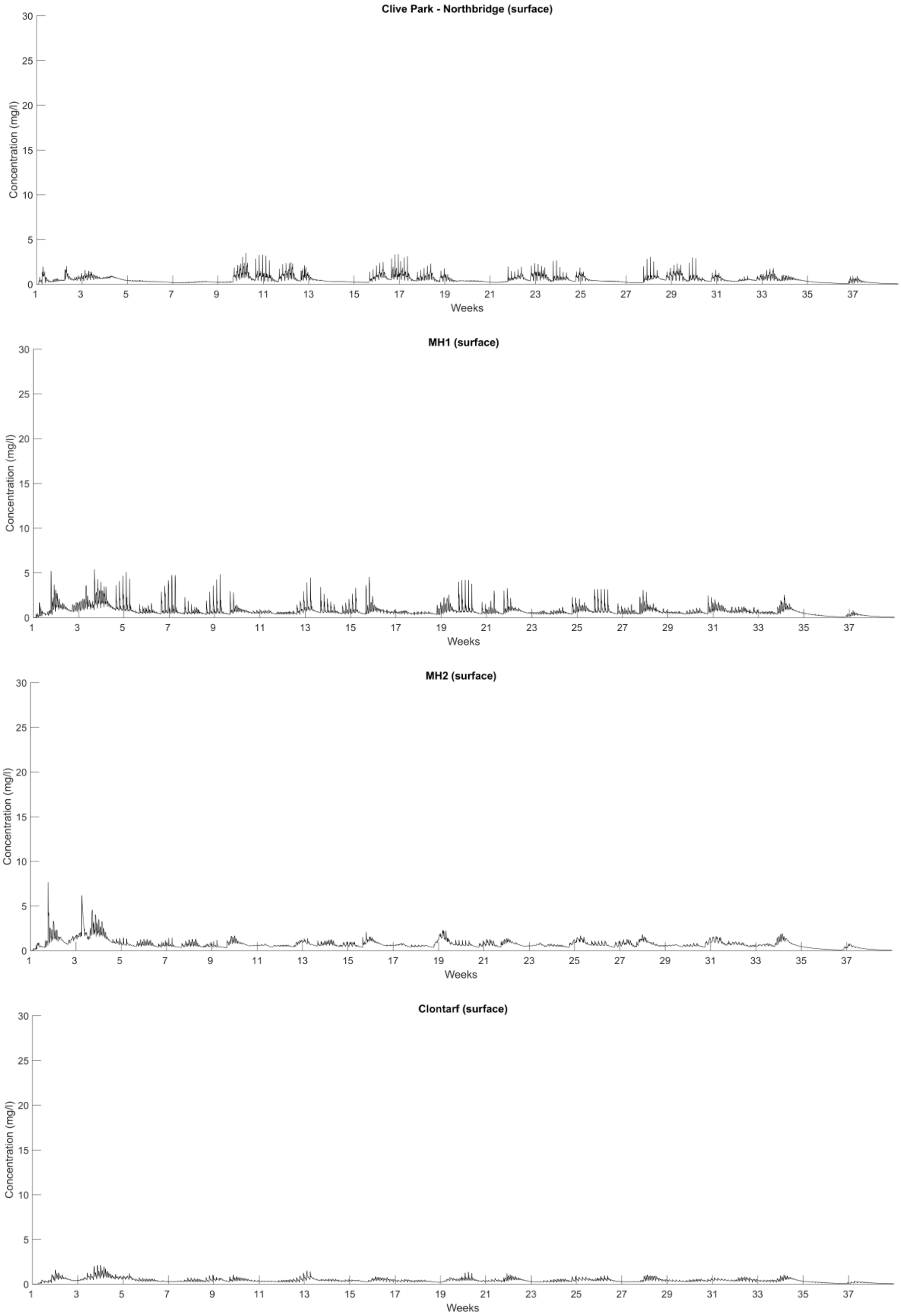


Figure 7-7: Time series of dredge suspended sediment concentrations at four sites across the Middle Harbour region (surface layer)

7.4.2 Sensitivity of dredge plumes to winds

The influence of the dredge SSC to wind driven circulation was tested. A six week hydrodynamic simulation was completed which included wind forcing as well as tidal hydrodynamic (as described in **Section 7.3**). The applied wind forcing was based on measured wind at Fort Denison and using the representative typical summer conditions as determined in **Section 5.4**. The hydrodynamic simulation with both tide and wind forcing (tide + wind) was then used to simulate the dredge plume SSC for the first six weeks of the dredging activity.

Table 7–4 presents a summary of statistics at MH1 and MH2 locations for the tide only and tide + wind hydrodynamic simulations. Time series results of SSC at the MH1 and MH2 locations are presented in Figure 7-8. The MH1 and MH2 locations were selected for this comparison as one site (MH1) is located in the main tidal channel, where strong tidal flows are expected to limit the sensitivity of dredge SSC to wind. MH2 is located in an area where tidal velocities are low and the sensitivity of dredge SSC to wind would be expected to be more significant. Results are provided for the surface layer, where the effect of wind would be expected to be greatest.

The results show only minor sensitivity to wind at MH1, with the SSC percentiles being lower for the tide + wind case than the tide only case. The plot also indicates that dredge SSC for the tide + wind results are lower for the majority of the simulation, only being higher than the dredge SSC for a few short bursts over the six week simulation. The results at the MH2 location show a higher sensitivity to wind with the tide + wind SSC percentiles being lower than the tide only SSC percentiles at the 20th and 50th level but higher for the upper percentiles. Referring to the time series, the most noticeable differences occur at about week five, when the BHD is working on the Seaforth side of the channel. Overall the results indicate that sensitivity to wind is limited to brief periods in the main deep channels of Middle Harbour where tidal currents are weak.

Table 7–4: Summary of suspended sediment concentration statistics in the surface layer for the four model extraction locations over the six week simulation

Percentile	MH1		MH2	
	Tide only SSC (mg/L)	Tide + wind SSC (mg/L)	Tide only SSC (mg/L)	Tide + wind SSC (mg/L)
20th percentile	0.55	0.42	0.57	0.49
50th percentile	0.95	0.71	0.95	0.93
90th percentile	2.14	1.71	2.31	2.49
95th percentile	2.83	2.20	3.02	3.13
99th percentile	4.07	3.37	4.43	5.20

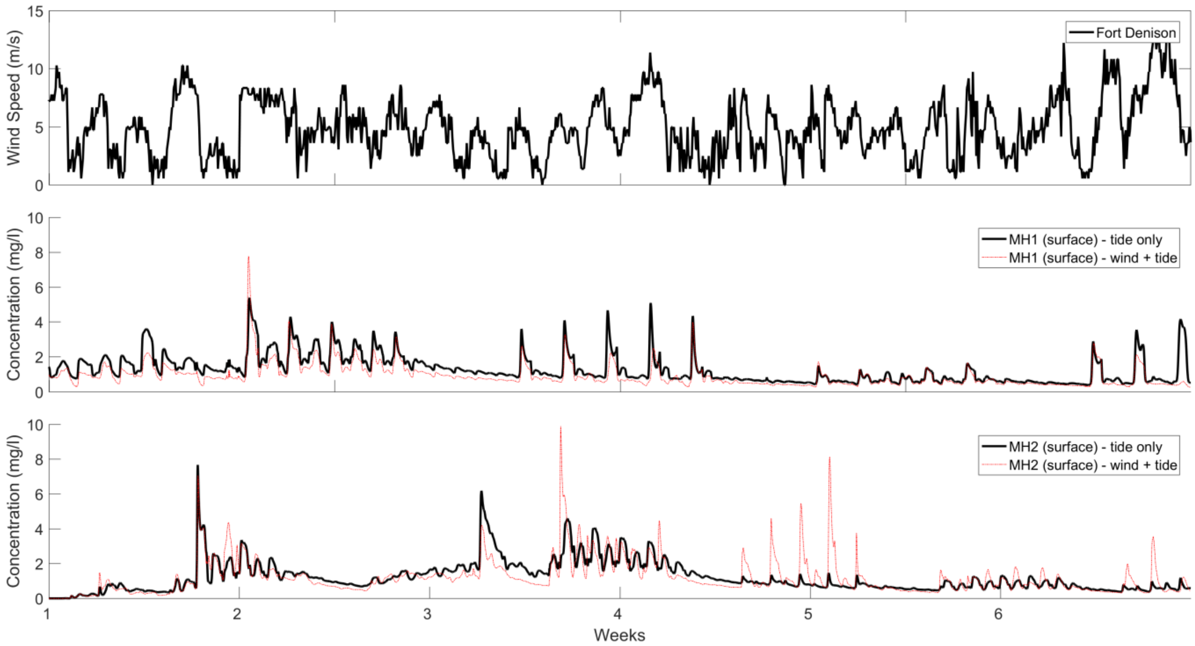


Figure 7-8: Time series of dredge suspended sediment concentration comparing the tide only with tide + wind dredge plume simulations at MH1 and MH2

7.4.3 Sediment deposition

Sediments suspended by dredging and dispersed by ambient currents deposit back on the bed of the harbour in suitable environments. A map of deposition on the bed of the harbour two weeks after the cessation of the dredging campaign is presented in Figure 7-9. This plot clearly shows the spatial distribution and magnitude of the deposition which is predicted to occur due to dredging. The two week period after the dredging has finished allows time for any sediment that is in suspension at the end of dredging to settle to the bed of the harbour.

A deposition map for sediment dredged within the first 1.2 weeks is presented in Figure 7-11. Sediment dredged over this period represents material that is not suitable for offshore disposal. The deposition map shows that this material would be contained within the silt curtains and limited to a thickness of less than five millimetres. No deposition of material unsuitable for offshore disposal would be expected to occur outside the immediate project crossing location.

Time series plots of deposition at the two model extraction locations, Clive Park-Northbridge and MH2, are provided in Figure 7-12 and Figure 7-13. These plots present the cumulative deposition as well as the rate of sedimentation. The average daily rate of sedimentation and maximum daily sedimentation rates were calculated over the 37 week dredge duration.

The deposition results show:

- The majority of the deposition due to the dredging activity occurs in the dredging footprint and adjacent to the dredging footprint due to the low current speeds throughout the area. High deposition would be concentrated in front of the two cofferdams where most of the rock dredging and rehandling occurs
- Low levels of sedimentation (one to five millimetres) would predominantly be located downstream of the dredge footprint due to stronger currents during the ebb tide, with some deposition

occurring beyond the Spit Bridge near Clontarf. Low level sedimentation would occur upstream, up to 600 metres from the crossing

- The pattern of sedimentation reflects the lower tidal current speeds in the deep main channel of Middle Harbour and the plot of bed shear stress (see Figure 7-10) confirms this area is a depositional environment (lower bed shear stress shown in white areas). Deposition is not predicted to accumulate in the area around the Spit Bridge where tidal currents and bed shear stresses are high
- Deposition rates at the Clive Park-Northbridge site would remain low throughout the 37 week dredge period, despite its location close to the dredge footprint. The site is positioned just outside the silt curtain and highlights the effectiveness of the silt curtains.

Note that shoreline shallow silt curtains are proposed but were not modelled and would give added protection to nearby ecologically sensitive areas (eg. seagrass and rocky reef habitat).

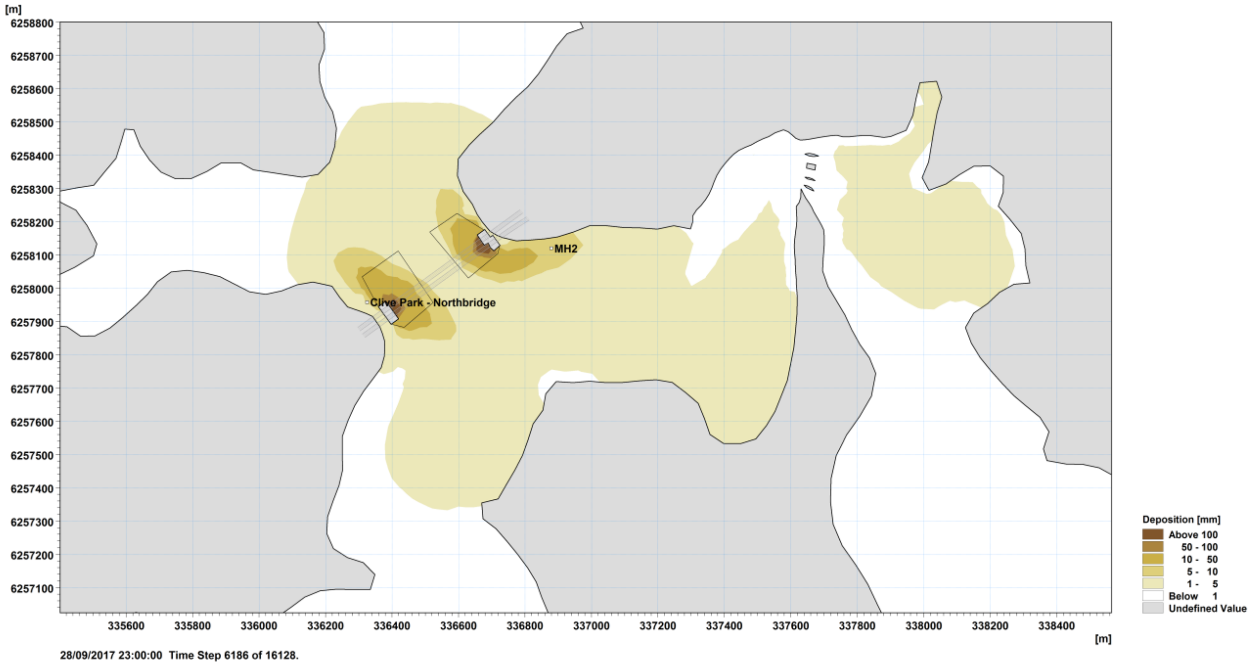


Figure 7-9: Deposition (millimetres) two weeks after the cessation of dredging

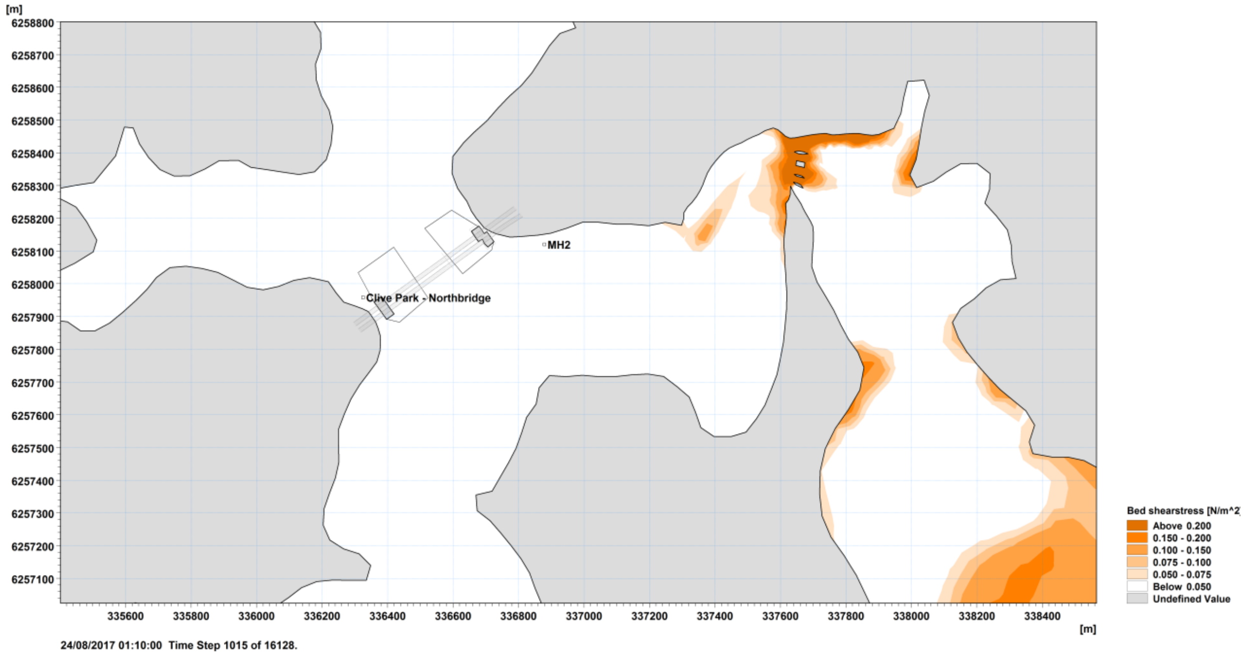


Figure 7-10: Bed shear stress during peak spring tides (ebb tidal stage)

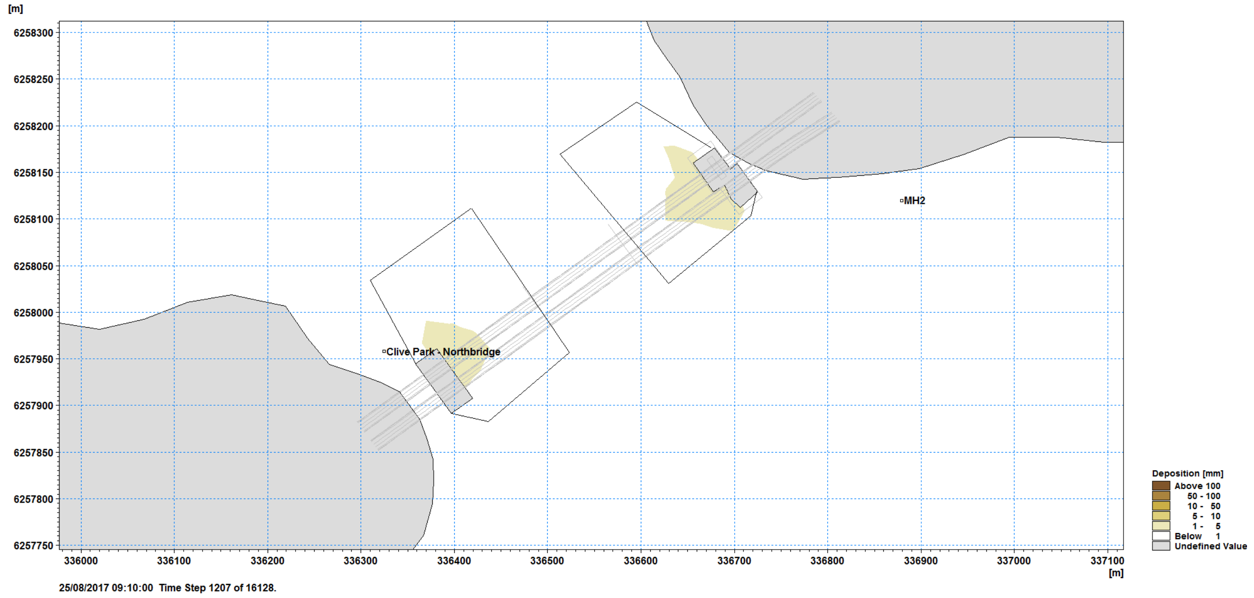


Figure 7-11: Deposition (millimetres) of material which is not suitable for offshore disposal

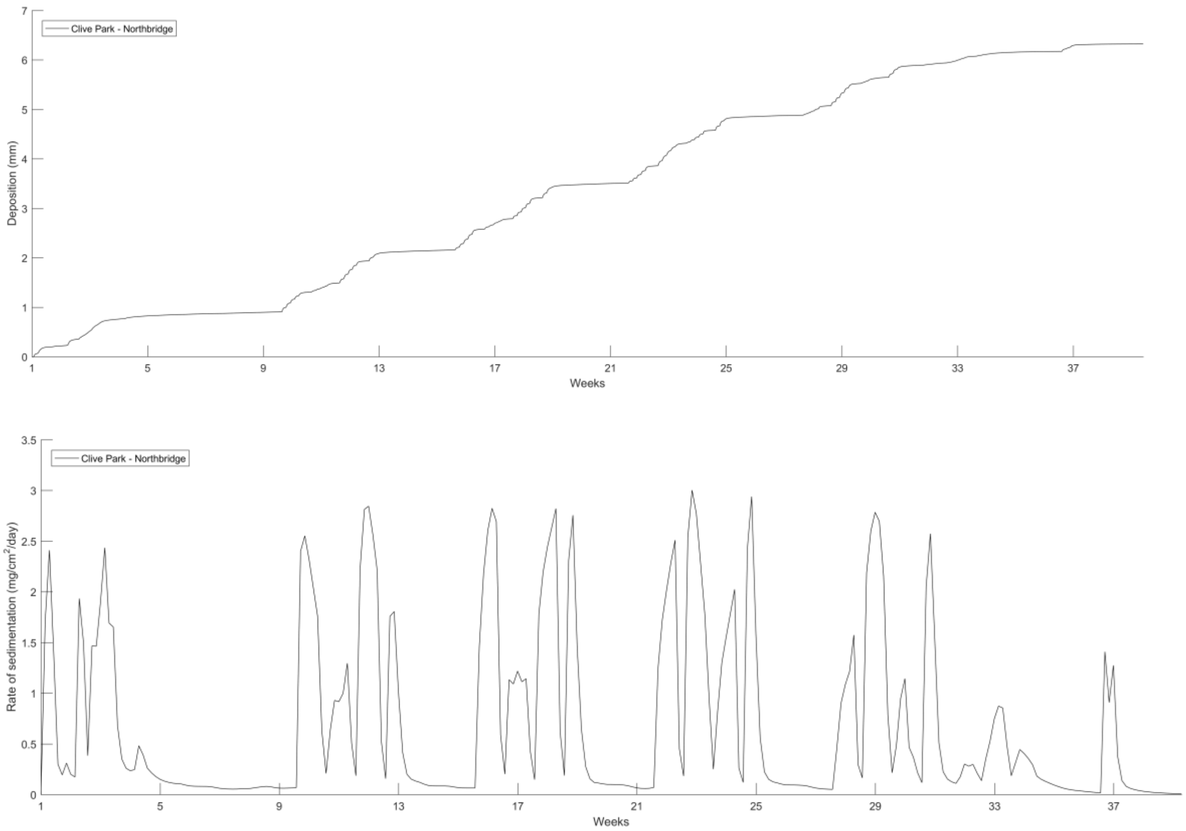


Figure 7-12: Time series of accumulative deposition (top) and rates of sedimentation (bottom) caused by dredging at the Clive Park-Northbridge model extraction location

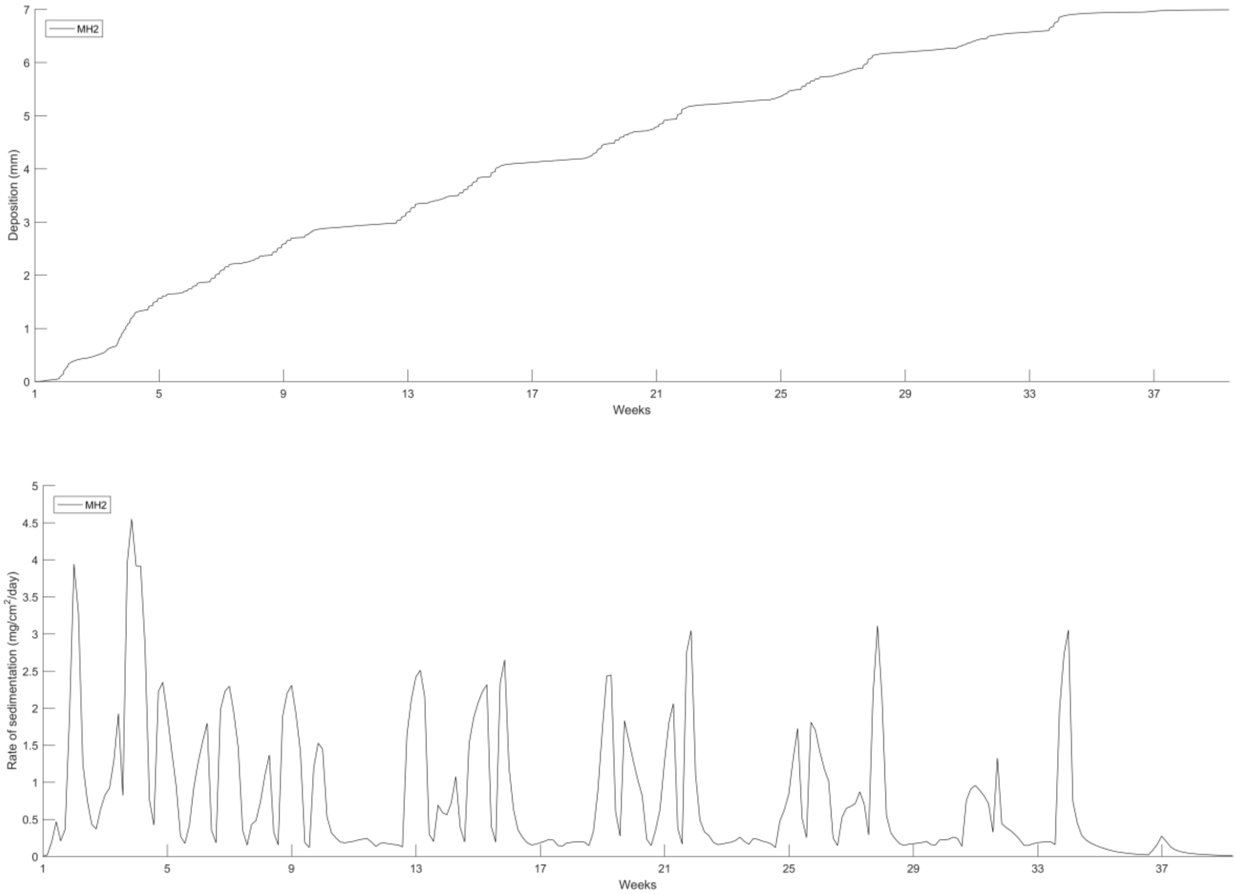


Figure 7-13: Time series of accumulative deposition (top) and rates of sedimentation (bottom) caused by dredging at MH2 model extraction location

8 Summary and discussion

8.1 Summary

This assessment has detailed the findings of numerical modelling to better understand the potential impact that construction activities and operation of the project may have on the hydrodynamic and water quality of the marine environment.

To inform the assessment, available historical data was reviewed and additional project specific data was collected. The project specific hydrodynamic data was then used to calibrate a 3D model that was established for the project. The model was successfully calibrated to tidal water levels, current (both in-situ and spatial) and discharge. Following calibration the model was considered fit for purpose and applied to inform the impact assessment.

8.1.1 Hydrodynamic impacts

The assessment of hydrodynamic impacts during the construction period included consideration of two temporary cofferdams (Middle Harbour south cofferdam (BL7) and Middle Harbour north cofferdam (BL8)), adjacent dredging activities and associated deep silt curtains that are proposed at the crossing location. It also considered the Spit West Reserve construction support site (BL9). The assessment of hydrodynamic impacts during the operational period focused on the effect of the permanent sill created by the immersed tube tunnels being situated above the bed of the harbour.

The modelling of hydrodynamic impacts during the construction phase has shown the following:]

- During the peak ebb tide, the Middle Harbour north cofferdam (BL8) and accompanying deep draft silt curtain would reduce current speeds around Seaforth Bluff (at all depths) in a downstream direction. Current speeds would increase in the middle of the channel and at the bed of the harbour (ie beneath the silt curtain). Additionally, the Middle Harbour south cofferdam (BL7) and associated deep silt curtain would cause an increase in current speeds between the temporary structures and the bank (near Clive Park) at the surface and in the middle of the water column
- During the flood tide, decreases in current speed were predicted at the Middle Harbour north cofferdam (BL8) and Middle Harbour south cofferdam (BL7) as well as within and surrounding the silt curtains. Additionally, at the Middle Harbour north cofferdam (BL8), decreases in current speed were predicted to occur upstream of the structure along Seaforth Bluff. At the Middle Harbour south cofferdam (BL7), an increase in current speed was predicted along the bank upstream of the structure in the surface and middle layer
- Due to the pre-existing low energy hydrodynamic environment, the changes in current speeds observed during ebb and flood tides are not expected to have a substantial impact on the surrounding environment
- During both ebb and flood tide, the current speed reductions would be more pronounced in the surface layer due to the effect of the silt curtains on the upper water column. As these changes are more pronounced in the surface layer, it is not expected that any major erosion or accretion of the bed of the harbour would occur in this area
- The temporary and localised increases in current speeds near the bed of the harbour due to construction of the project crossing (ie in the gap between the silt curtains) are not expected to result in a substantial change to the sediment dynamics in this area.

The modelling of hydrodynamic impacts during the construction phase at Spit West Reserve construction support site (BL9) has shown the following:

- Under the existing conditions the surface currents in the vicinity of the Spit West Reserve construction support site (BL9) move in a northward direction during the flood tide. The northward flows along this shoreline are associated with a return eddy that forms in Pearl Bay during the flood tide. During the ebb tide the current speeds in this area are much lower, being less than 0.08 m/s, but still flowing towards the north. During both the flood and the ebb, current speeds reduce slightly with depth
- With the Spit West Reserve construction support site (BL9) in place there is expected to be a general reduction in current speeds adjacent to the shoreline. This would mostly occur during the flood tide, when decreases in current speeds of up to 0.1 m/s are predicted adjacent to the shoreline. The reduced current speeds result from the temporary structures impeding the eddy that forms in this area. The eddy is redirected, particularly in the surface layers, towards the west and there is a small area of current speed increase to the west of the immersed tube tunnel units/casting facility
- Due to the low current speeds observed in this area and the minor changes in current speeds expected at the bed of the harbour, no substantial change in the pattern of accretion or erosion at the bed is expected
- No current speed increases were predicted along the Spit West Reserve shoreline and consequently shoreline erosion as a result of changes to tidal currents is not expected to occur.

The immersed tube tunnels would be situated above the bed of the harbour across the deepest section of Middle Harbour, creating a sill. In regard to operational hydrodynamic impacts due to the project:

- It is observed that there would be minor overall differences in current speeds between the existing scenario and the project design scenario. The differences observed between the existing and project design scenarios are increases in current speeds over the proposed project crossing location, mostly noted on the northern bank (Seaforth) during both flood and ebb tides. The relative increase in current speeds adjacent to the northern bank are 33 per cent (for both flood and ebb) but the magnitude of the change is less than 0.04 m/s and 0.08 m/s, for flood and ebb, respectively. The magnitude of wind driven circulation would be expected to be greater at this location than the changes in current speeds due to the project
- Similarly, the results indicate only minor differences between existing and project design scenarios, for:
  - Tidal water levels and tidal planes
  - Tidal discharge at the tunnel crossing location
  - Tidal prism (marginal decrease of 0.4 per cent).
- Tidal flushing analysis indicates that the volume of water upstream and below the level of the sill would take longer to flush as a result of the project. However, the flushing times are still expected to be relatively rapid. The water quality/ecological impacts of the increased flushing time are addressed in Appendix Q (Technical working paper: Marine water quality) of the environmental impact assessment.

8.1.2 Water quality impacts

The assessment of water quality impacts focuses on the increase in SSC due to the proposed dredging activity required to construct the immersed tube tunnel at the proposed crossing.

The modelling of dredge plume related water quality impacts during the construction phase has shown the following:

- When analysed over the entire dredge duration of 37 weeks and viewed at the 95th percentile level (ie less than five per cent of the time), the spatial extent of the dredge plume (SSC greater than two mg/L) would be relatively small. There are two main factors that influence this result:
    - The deep draft silt curtains effectively capture sediment in the surface layer and to a lesser degree in the middle layer and in the near harbour bed layer. The surface plume is essentially contained within these silt curtains
    - Current speeds are generally low in the area with the tidal stream flowing along Seaforth Bluff during the ebb and flood tide. Sediment in the lower layers is transported within this tidal stream adjacent to the Seaforth shoreline.
- The suspended sediment released during the dredging activity would be transported in both an upstream and downstream direction, with a downstream dominance, particularly along the Seaforth shoreline. This is a result of the tidal currents that are predominately aligned with the main longitudinal axis of the estuary
- The dredge plume extents would be greater in the layer near the bed of the harbour than at the surface
- At Clive Park, Northbridge, located immediately outside the silt curtain on the southern side of the proposed crossing location, elevated SSC is predicted at the surface. Values of up to 3.5 mg/L are predicted for brief periods during dredging activities. For 99 per cent of the dredge time, SSC in the surface layer are predicted to be less than 1.7 mg/L
- For MH1, which is located further downstream from the dredging footprint, the 95th percentile SSC in the surface layer is predicted to be 2.9 mg/L. Peak SCC values of up to 5.4 mg/L are predicted. At MH2, which is closer to the dredge footprint than MH1, peak values up to eight mg/L in the surface layer are predicted to occur during the initial four weeks of dredging when the BHD is removing surface sediments. For 95 per cent of the dredge time, SSC is predicted to be less than 2.5 mg/L. At Clontarf, SSC values were predicted to be consistently low (less than 2.1 mg/L) throughout the duration of dredging. For 99 per cent of the time, dredge SSC in the surface layer is predicted to remain below 1.2 mg/L
- The SSC modelling results consistently show that SSC is generally low (less than five mg/L) for areas outside of the silt curtains with higher concentrations predicted in the bottom layer. The waters adjacent to Seaforth Bluff would be most likely to see moderate increases in SSC, particularly in the downstream direction
- The majority of the deposition due to the dredging activity would occur in the dredging footprint and adjacent to the dredging footprint due to the low current speeds throughout the area. High deposition would be concentrated in front of the two cofferdams (Middle Harbour north cofferdam (BL8) and Middle Harbour south cofferdam (BL7)) where the majority of rock dredging and rehandling would occur
- Although there is limited existing data for turbidity during high rainfall events in Middle Harbour, the expected construction source plumes are likely less of an impact than turbidity that occurs due

to high rainfall events. One such high rainfall event occurred in early February 2020 (refer Figure 3-8 and Figure 3-9).

8.2 Discussion

A number of environmental management measures are proposed to be implemented as part of dredging operations. These management measures aim to manage the generation of suspended sediments by appropriate selection of dredging techniques and containment of turbidity with physical barriers. These reflect best environmental practice to reduce the water quality impacts of dredging. The hydrodynamic and dredge plume modelling carried out has incorporated these management measures. Proposed environmental management measures for mitigation of potential turbidity impacts are summarised below.

In regard to the dredging by the very large BHD:

- Actual dredging production hours would be about 7.5 hours per day, five days per week
- An environmental and/or closed bucket should be used for all suitable material (ie all other than rock material). This attachment is specifically designed to minimise the volume of water entrained into the in-situ sediment mass and reduce the generation of turbidity from the release of suspended sediments whilst the bucket is raised up through the water column and swung over to a receiving hopper barge. The environmental clamshell bucket should not have teeth, should be fitted with a venting system that allows water and air to pass through the bucket during its descent, and completely enclose the dredged sediment. The clamshell mechanism would be operated by hydraulic means. An open bucket may be required for dredging of rock, either directly dredging weak rock or rehandling of hard rock crushed by the drum cutter, as damage to closed buckets can be frequent and extensive in this material type
- No overflow has been assumed when loading barges. This reduces losses to the water column
- BHD dredging operations should be completed within a floating silt curtain enclosure (or 'moon pool') that is secured to the dredge barge. This should comprise a fixed or floating boom upon which a shallow draft silt curtain (ie shallow draft silt curtain about two to three metres deep) is attached to provide a controlled area for the dredge operator to work within
- Additional containment of suspended sediments should be provided by the installation of deep draft silt curtains (ie deep draft silt curtains about 12 metres deep) situated at either side of the proposed project crossing location where the majority of the dredging would occur. The deep draft silt curtains are considered feasible at the project crossing location due to the low flow environment (ie current speeds of less than 0.2 m/s).

In regard to the dredging by the drum cutters:

- Actual dredging production hours would be 6.5 hours per day, five days per week
- Crushing of rock by drum cutters could be assisted by pre-cutting with a rock saw on a BHD
- Crushing of rock should be completed by drum cutters attached to the stick of a BHD to minimise dispersal of rock fines into the water column that may be caused by hydraulic dredging techniques
- The drum cutters would leave cut material on the bed of the harbour for direct rehandling by BHD. This means that the BHD would need to remove the material left by drum cutters before the next layer can be dredged. This is considered an environmentally appropriate approach, as it requires that only one dredger (either BHD doing rehandling or BHD fitted with drum cutter) would be

working at any one time. This would eliminate a second source point of release of sediments into the water column.

- Deep draft silt curtains should be used at either side of the crossing where rock dredging occurs
- Hopper barge overflows should not be used during dredging, transit or disposal operations.

The above aspects of the dredging methodology would result in an overall reduction in the extent and intensity of the dredge plumes, which is reflected in the modelling results presented in this report.

Additional environmental management measures proposed for mitigation of potential hydrodynamic and water quality impacts associated with dredging for the project could include:

- Dredging equipment should be fitted with accurate positioning systems to enable precision dredging
- Implementation of a water quality monitoring program
- Full time supervision and an inspection regime for dredging activities should be conducted
- Preparation and implementation of a dredge management plan as part of the construction environmental management plan for the project
- Implementation of floating silt curtains for protection of ecologically sensitive areas (eg seagrass and rocky reef habitat along the Clive Park foreshore) adjacent to the dredging footprint.

9 References

Arup WSP Memo (2017) *Provision on PSD, Dry Density and CPT data to RMS for plume generation modelling.* WHTBL-PW00-GT-MEM-0065[A] dated 21 December 2017.

BoM (Bureau of Meteorology) (2015), Annual Climate Statement 2015

BoM (Bureau of Meteorology) (2017), Annual Climate Statement, 2017

Cardno 2017 *Preliminary Risk Screening of Potential Marine Ecology Impacts from Dredging and Ongoing Operational of the Western Harbour Tunnel (WHT) and Beaches Link (BL) Projects*. Letter to Jacobs dated 14 July 2017.

Cardno 2020 *Beaches Link and Gore Hill Freeway Connection Technical working paper: Marine water quality*, October 2020.

Danish Hydraulic Institute (DHI), 2017 *MIKE 3 Flow Model: Hydrodynamic Module User Guide*; DHI Water and Environment: Hørsholm, Denmark, 2017.

Erftermeijer, P.L.A., Riegl, B., Hoeksema, B.W. and Todd, P.A., 2012. *Environmental impacts of dredging and other sediment disturbances on corals: A review.* Marine Pollution Bulletin 64, 1737-1765.

Hatje, V., Birch, G.F., Hill, D.M., (2001). *Spatial and temporal variability of particulate trace metals in Port Jackson estuary, Australia. Estuarine*, Coastal and Shelf Science 53, 63–77.

GBRMPA, 2010. *Water quality guidelines for the Great Barrier Reef Marine Park.* Revised Edition 2010. Great Barrier Reef Marine Park Authority, Townsville.

J. J. Williams and L. S. Esteves (2017) *"Guidance on Setup, Calibration, and Validation of Hydrodynamic, Wave, and Sediment Models for Shelf Seas and Estuaries"* Advances in Civil Engineering, Volume 2017, Article ID 5251902

Manly Hydraulics Laboratory (2016), *NSW Ocean and River Entrance Tidal Levels Annual Summary*, ISSN 2205-555X.

RHDHV (2017a), *WHTBL Feasibility and Business Case - Technical and Environmental Advisory Services*. Prepared for WSP PB/Arup JV, 31 October 2017.

RHDHV (2017b), *WHTBL Feasibility and Business Case - Hydrodynamic Data Collection Report*. Prepared for WSP PB/Arup JV, 30 November 2017.

Roads and Maritime Services (2017) *IMT GOLD Design Package – Drawings and Autocad Files*, *RMS registration number DS2016/002059,* dated 20/11/2017 and received via email (22/12/2017)

Robinson, C., Suggett, D., Ralph, P. J., and Doblin, M. A. (2014). *Performance of fast repetition rate fluorometry based estimates of primary productivity in coastal waters*. Journal of Marine Systems 139, 299–310. doi:10.1016/J.JMARSYS.2014.07.016

Teledyne RDI (2013), *ADCP Sentinel V Operation Manual*. P/N 95D-6002-00.

Smagorinsky, J. (1963): *General Circulation Experiments with the Primitive Equations, 1, The Basic Experiment*, Mon. Weather Rev., Vol. 91, pp. 90-164.

Annexure A – RHDHV hydrodynamic data collection summary

Scope of works

The scope of the hydrodynamic and water quality monitoring is:

- Two in-situ monitoring sites are located at the project crossing. The in-situ measurements have been used to capture temporal variability in hydrodynamic and water quality conditions due to tidal and non-tidal influences. Each site provides continuous measurements of water level, current velocity and acoustic backscatter using an ADCP (Acoustic Doppler Current Profiler) type instrument. Additionally one of the in-situ monitoring sites measures water quality parameters (primarily turbidity). The water quality data collected as part of RHDHV's monitoring was carried out to inform an understanding of the concurrent turbidity at the in-situ monitoring locations. This summary covers a period of about 11 weeks
- Vessel mounted ADCP transects were carried out at the project crossing location during spring tidal conditions to determine spatial variability in currents and discharge throughout a tidal cycle
- Surface sediment samples on the bed of the harbour were collected at the project crossing location and analysed for particle size distribution.

Deployment summary

Table 9–1 presents a summary of the fieldwork days completed as part of the monitoring campaign. Figure 9-1 shows the monitoring locations at the project crossing. The monitoring period reported herein was between 17 August 2017 and 1 November 2017, totalling a monitoring period of 76 days. Servicing of the instruments took place midterm on 21 September 2017. RHDHV further serviced and redeployed all monitoring sites on 1 November 2017.

Details of the co-ordinates, instruments and measured parameters of all in-situ monitoring sites are provided in Table 9–2 and Table 9–3 for the first and second deployment period respectively. Figure 9-2 presents images from the fieldwork during the reporting period.

Table 9–1: Activities completed during field operations

Fieldwork dates	Activities completed	Vessel / marine services supplier	RHDHV field personnel
17 August 2017	Deployment of MH1 and MH2	Polaris Marine	H. Loehr
22 August 2017	ADCP transects/ sediment sampling	Geochemical Assessments	H. Loehr
21 September 2017	Servicing and redeployment of MH1 and MH2	Polaris Marine	H. Loehr
1 November 2017	Servicing and redeployment of MH1 and MH2	Polaris Marine	H. Loehr

Note: ADCP are used for measuring currents. Refer below for further details.

Table 9–2: Co-ordinates of in-situ monitoring locations during the first deployment period

Site name	Co-ordinates (WGS84)		Instrument	Parameters
	Marker buoy	Instrument		
MH1	-33.80378, 151.24318	-33.80406, 151.24316 (*note instrument moved 30m to south-west on 26 Aug 2017)	AWAC/ Hydrolab	Currents, water level, temperature, water quality
MH2	-33.80453, 151.23776	-33.80453, 151.23776	Aquadopp Z-Cell	Currents, temperature

Table 9–3: Co-ordinates of in-situ monitoring locations during the second deployment period

Site name	Co-ordinates (WGS84)		Instrument	Parameters
	Marker buoy	Instrument		
MH1	-33.80373, 151.24314	-33.80384, 151.24306	AWAC/ Hydrolab	Currents, water level, temperature, water quality
MH2	-33.80453, 151.23776	-33.80453, 151.23776	Aquadopp Z-Cell	Currents, temperature

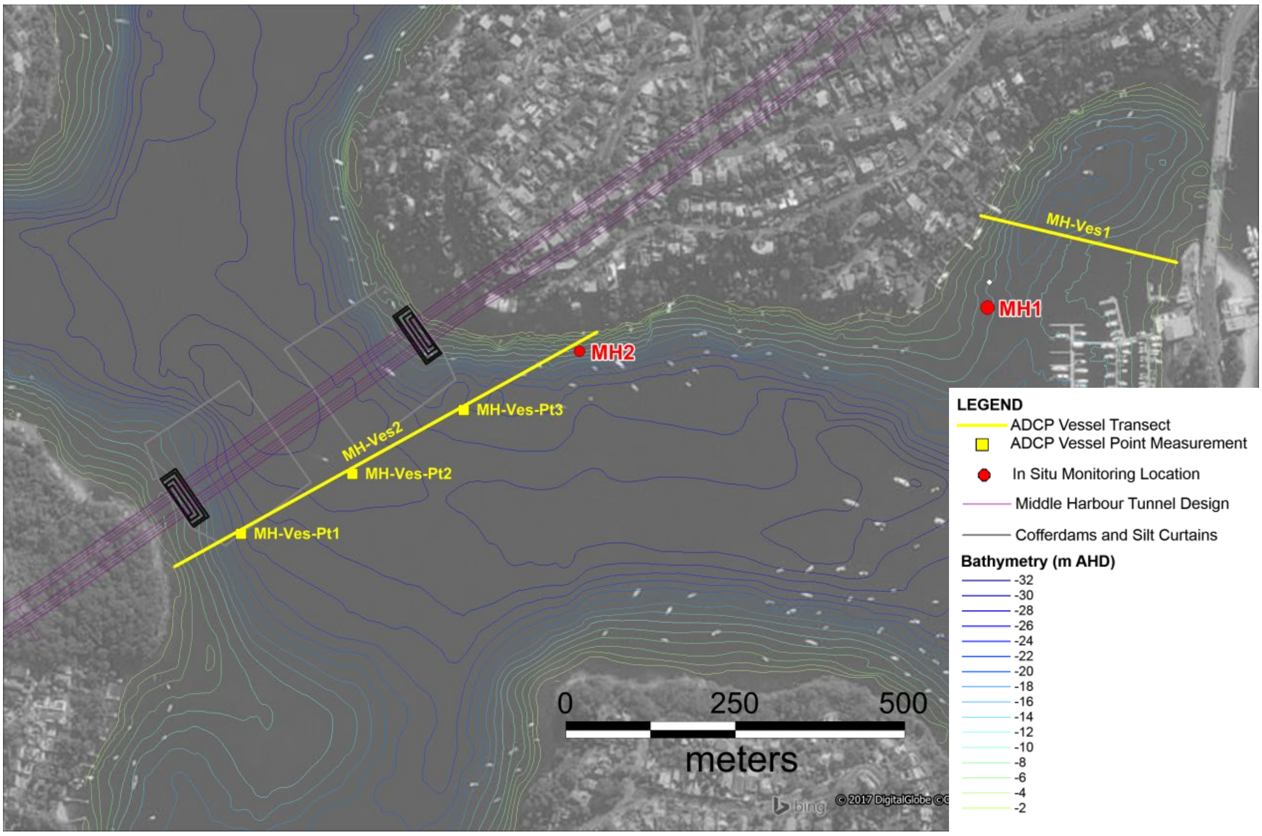


Figure 9-1: Map showing the hydrodynamic and water quality monitoring locations at the project crossing location

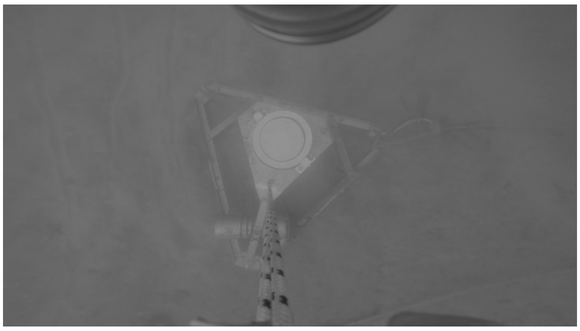
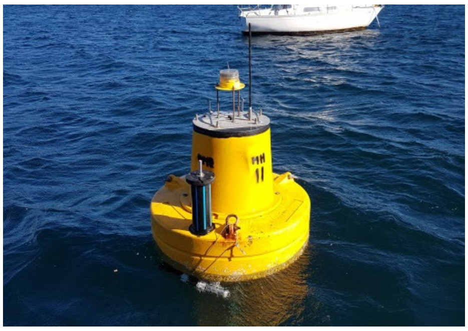
 Underwater photograph of bottom mounted ADCP at MH1	 Aquadopp Z-Cell mounted underneath buoy at MH2
 Bottom mounted ADCP during deployment at MH1	 Vessel mounted ADCP transect setup

Figure 9-2: Fieldwork photos during the instrument deployments and vessel transects

In-situ monitoring sites

ADCP instrument setup

Teledyne RDI Sentinel V20 (V50) ADCP, Nortek AWAC and *Aquadopp Z-Cell* instruments were used in this study. With the exception of the TRDI V50, these instruments have an operational frequency of 1000 kHz. The TRDI V50 has an operational frequency of 500 kHz. The instruments were configured to measure currents, water depth variation and water temperature. ADCPs measure the flow velocity of water by transmitting short sound pulses and measuring the Doppler shift of the reflected signal. The acoustic signal is reflected by 'scatters' (small particles) assumed to be passively flowing in suspension. Current velocities are measured across the instruments acoustic range in vertical bins (ie the ADCP measured the velocity profile with depth).

The ADCP also has sensors to measure relative pressure and water temperature, as well as pitch, roll and instrument heading. A technical specification for the TRDI ADCP instruments is provided in Teledyne RDI (2013). A technical specification for the Nortek AWAC and Aquadopp instruments is provided in Nortek AS

(2013). Details of the instrument setups and mounting configuration adopted for this study are provided in Table 9–4.

Table 9–4: Instrument specifications for the deployed ADCP type instruments

Parameter description	MH1	MH2
Operation frequency	500kHz/ 1000 kHz	1000 kHz
Mounting	Upward looking from bed of the harbour	Downward looking from buoy
Approx. depth (relative to AHD)	13, 11 and 11 metres	11 metres
Vertical resolution (Bin Size)	1 metre	1 metre
Blanking distance	0.4 metres	0.4 metres
Current measurement interval	10 minutes	10 minutes
Current averaging interval	120 seconds	120 seconds

Turbidity sonde setup

Turbidity measurements were made using Hydrolab DS5 multi-parameter sondes. The sondes support self-cleaning turbidity, pH and temperature sensors in one compact instrument package. The sonde was attached to the mooring approximately two metres above the bed of the harbour. All parameters were measured at 30 minute intervals. Details of the instrument setup adopted for this study are provided in Table 9–5.

Table 9–5: Instrument specifications for the deployed turbidity sonde

Hydrolab DS5
Sensor about 2 metres above the bed of the harbour
30 minutes

ADCP vessel transects

A vessel mounted ADCP was used to map current velocities along predefined transect lines at Middle Harbour on 22 August 2017. The two predefined transects (see Figure 9-1) were repeated as frequently as possible over an eight to 11 hour period during the spring tide in order to produce tidal discharge curves.

Table 9–6: ADCP vessel transects completed on 22 August 2017

Transect ID	Length (metres)
MH_Ves1	245
MH_Ves2	590

At Middle Harbour, deep draft silt curtains are proposed during construction. This type of silt curtain requires a low current environment. Low currents speeds were expected along transect MH_Ves2 and given the requirements of the deep silt curtains, particular emphasis was placed on recording accurate current speeds. To ensure that any vessel-motion induced errors could be reduced additional two-minute stationary current speed measurements were carried out at three points along the MH_Ves2 transect, which is representative of the Middle Harbour crossing. These vessel-based point measurements (ie MH_Ves2_Pt1, MH_Ves2_Pt2 and MH_Ves2_Pt3) can be seen in Figure 9-1. As with the predefined transect lines, the ADCP vessel-based 'point' measurements were also repeated over the tidal cycle as frequently as practically possible to accurately capture the full range of velocities over the tidal cycle.

Water quality profile

A single vertical water quality profile was carried out opportunistically on 21 September 2017 using the Hydrolab WQ sonde. This was taken in a location of relatively deep water (about 30 metres) at the location of MH_Ves2_Pt2 near the middle of the Middle Harbour crossing (see Figure 9-1). The parameters measured included conductivity, temperature, pH and turbidity.

Sediment sampling

A map showing the sediment sampling locations is provided in Figure 9-3. A total of 10 sediment samples were collected from the surface of the harbour bed using a grab sampler. A detailed grain size analysis was carried out for selected samples by Geochemical Assessments.

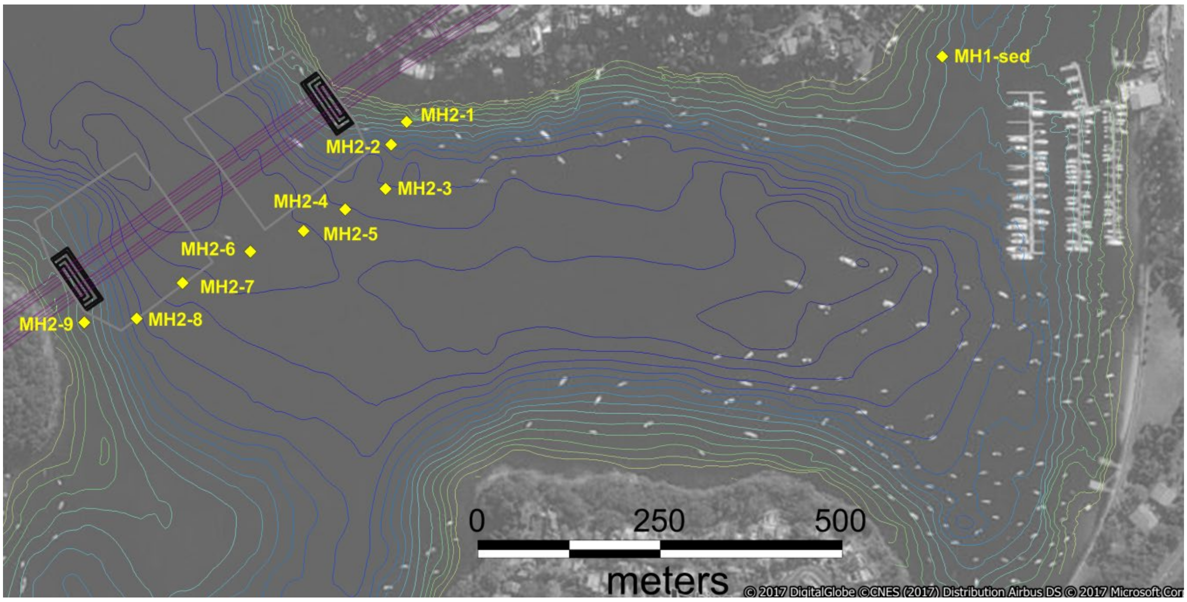


Figure 9-3: Location of sediment samples

Quality assurance and data processing

All data have been quality controlled and processed to remove all erroneous data points using RHDHV in-house tools (see **Figure 9-4**). The total raw data capture for the RHDHV field deployments was 92 per cent while the average data return of good data was 86 per cent (Table 9–7). During the second deployment, the turbidity sensor at MH1 did not record valid data, however, valid pH and temperature data were recorded by the sonde.

The vessel-based ADCP transect data was processed using TRDI's processing software WinRiver II.

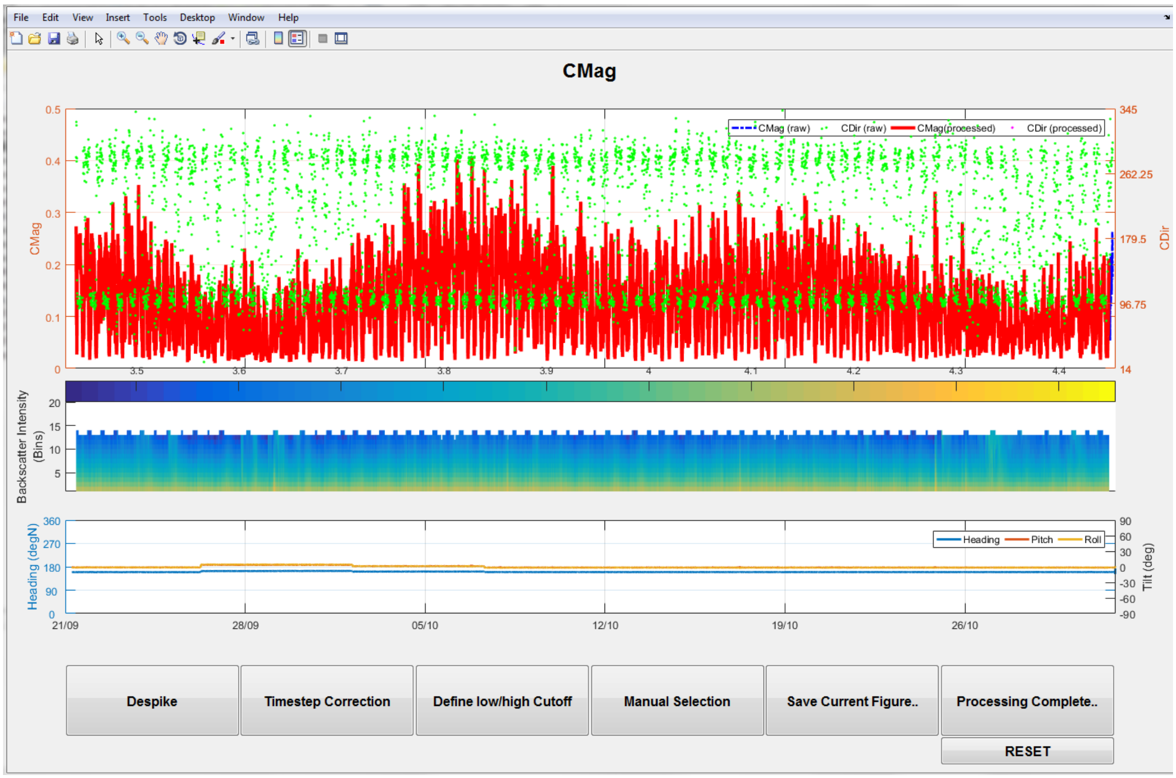


Figure 9-4: RHDHV in-situ data QA/QC processing tool used to remove erroneous field data

Table 9–7: Overview of data return for RHDHV in-situ deployments at both crossing sites

Site	Parameter	Monitoring duration (days)	Raw data capture (%)	QA data return (%)
MH1	Currents	76	100.0	100.0
	Turbidity	76	71.1	34.7
MH2	Currents	76	100.0	100.0
	Turbidity	-	-	-
Overall data return		Total:	90.3	78.2

Results

In-situ measurements

Time series plots

Time series plots of water level, current speed and current direction for the in-situ sites at MH1 and MH2, can be seen in Figure 9-5 and Figure 9-6. Concurrent wind data from BoM (Sydney Observatory Hill site) is also presented in these plots.

A time series of measured turbidity at Middle Harbour is also available in Figure 9-7.

Current roses and scatter plots

Rose plots for depth averaged current speeds and directions at the four sites are provided in Figure 9-8. The joint-occurrence of currents speeds and directions are demonstrated in Figure 9-9.

Depth profile

An analysis of the current profile throughout the water column at the in-situ monitoring sites has been carried out. It was found that uniform flow throughout the water column occurs during peak flood and ebb however flow separation occasionally occurs during periods of high wind speeds.

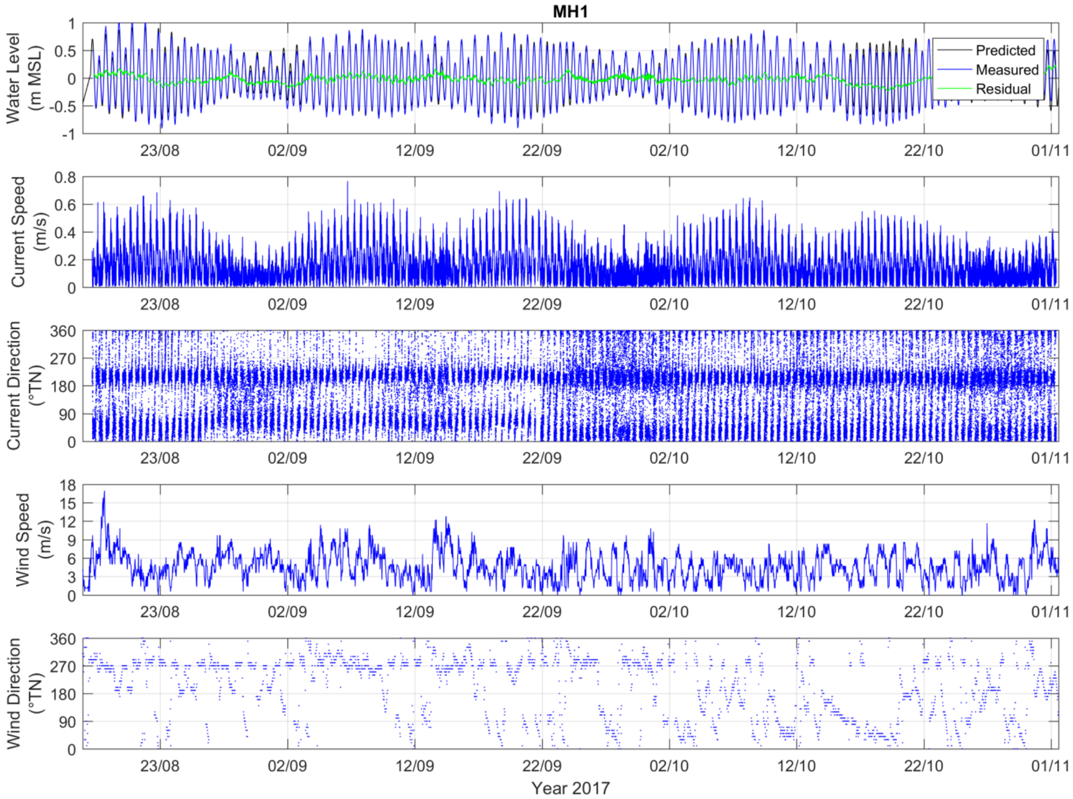


Figure 9-5: Time series of measured metocean data at site MH1 complemented with measured wind data from BoM

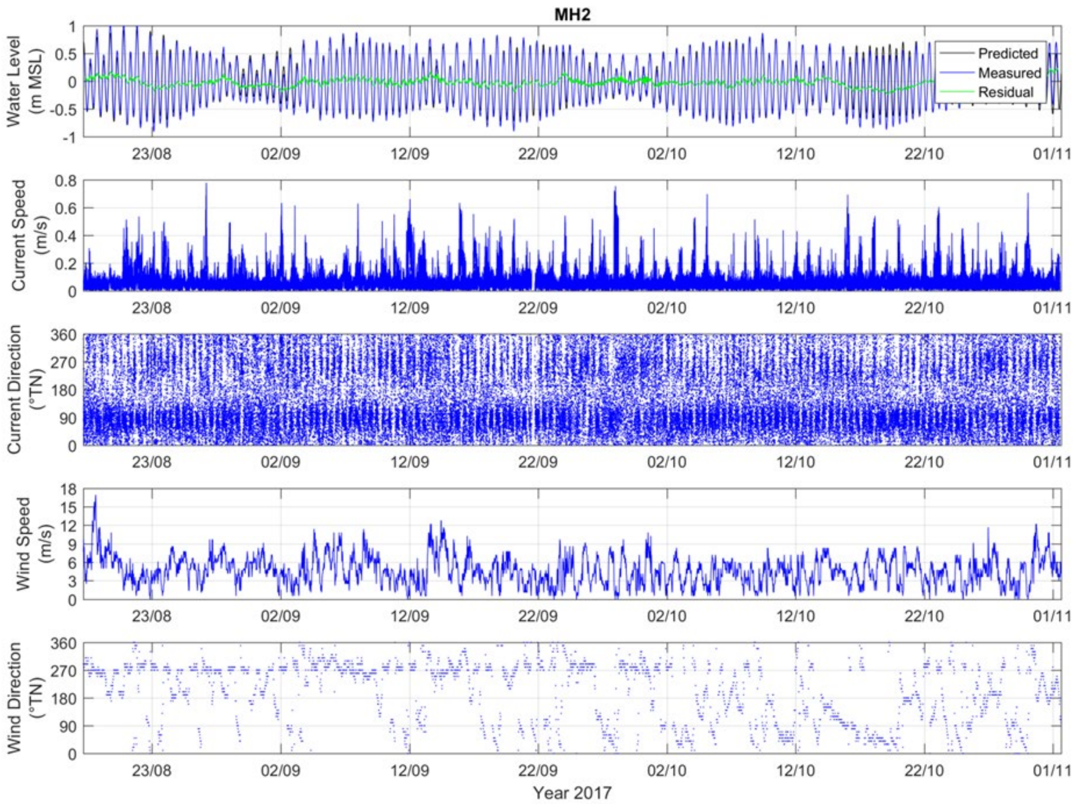


Figure 9-6: Time series of measured metocean data at site MH2 complemented with measured wind data from BoM (MH2 is a buoy mounted instrument so does not record water level. The water level data in this plot comes from MH1 site and is included for reference only)

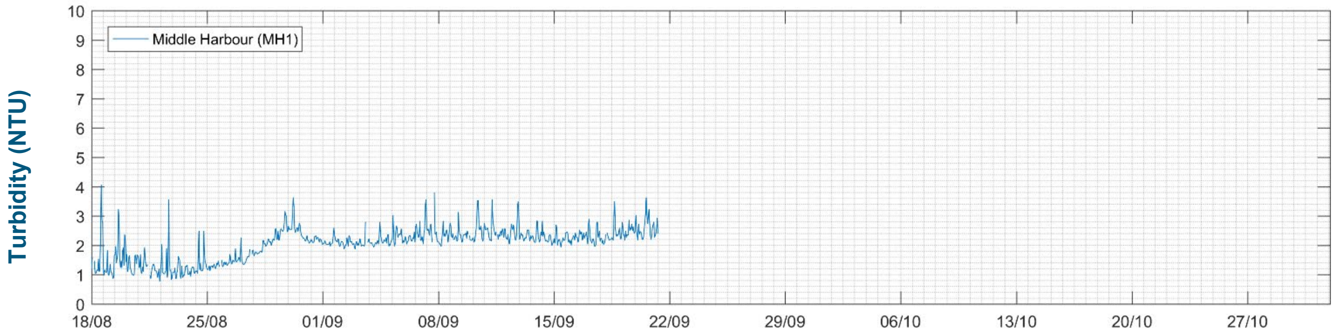


Figure 9-7: Time-series of measured turbidity at Middle Harbour

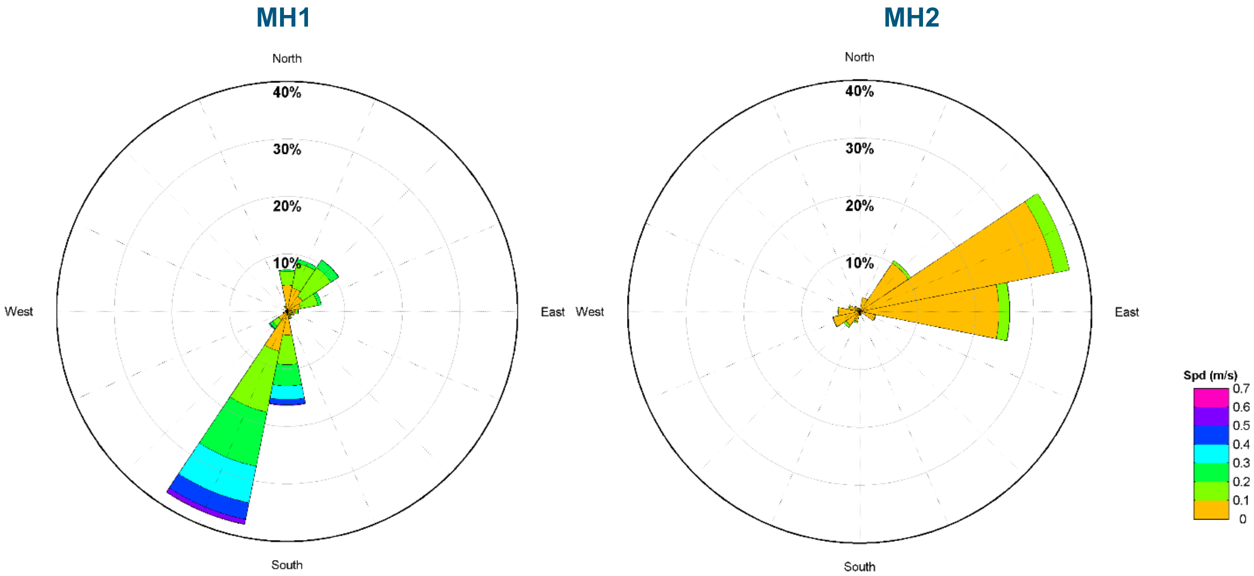


Figure 9-8: Rose plots showing current speed and direction during the deployment period at the two in-situ sites

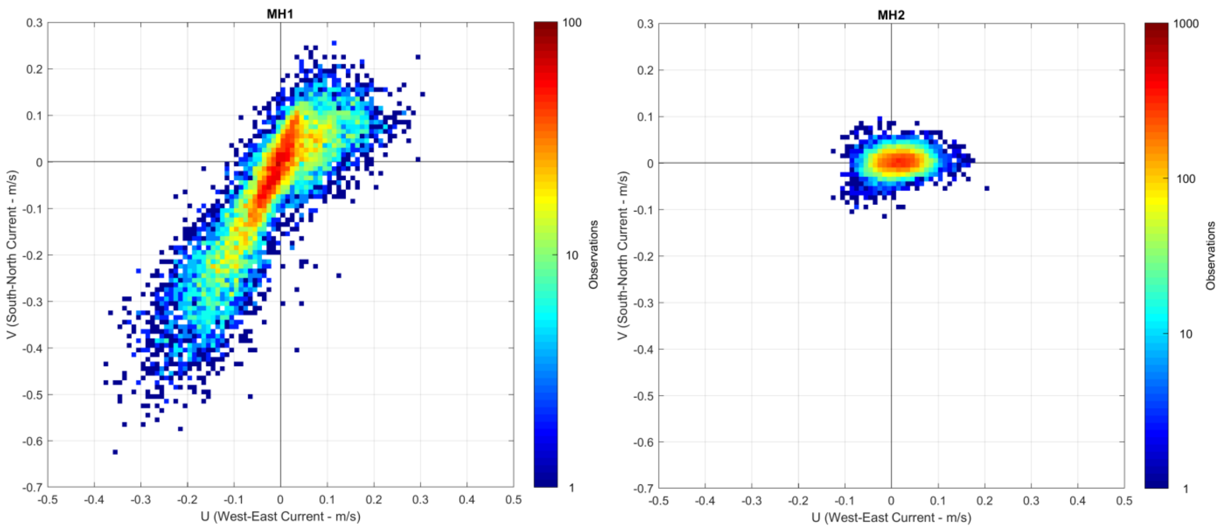


Figure 9-9: Current speed vs current direction at MH1 and MH2.The colour code demonstrates the number of observations.

Descriptive statistics

A summary of the descriptive statistics taken from the in-situ current velocity data is provided in Table 9–8. Tidal planes were estimated from the measured water level data by carrying out harmonic analysis and are presented in Table 9–9.

Table 9–8: Summary of current speed statistics at the two sites for the deployment period

Parameter	Statistic	MH1	MH2
Flood current speed (m/s)	Maximum	0.72	0.15
	95th percentile	0.42	0.07
	Mean	0.17	0.03
Ebb current speed (m/s)	Maximum	0.37	0.21
	95th percentile	0.21	0.08
	Mean	0.09	0.04

Table 9–9: Approximate tidal planes derived from harmonic analyses of the three month water level data at the two sites and literature tidal planes at Fort Denison[3]

Tidal plane	Fort Denison water level (m AHD)*	MH1 water level (m MSL)	MH2 water level (m MSL)
Mean High Water Springs (MHWS)	0.69	0.66	0.66
Mean High Water (MHW)	0.56	0.52	0.52
Mean High Water Neaps (MHWN)	0.44	0.37	0.37
Mean Sea Level (MSL)	0.01	0.00	0.00
Mean Low Water Neaps (MLWN)	-0.39	-0.37	-0.37
Mean Low Water (MLW)	-0.51**	-0.51	-0.51
Mean Low Water Springs (MLWS)	-0.64	-0.66	-0.66

*Taken from Estuarine Planning Levels Study - Foreshore Region of Leichhardt Local Government Area (Cardno, 2010)
** Interpolated

[3] *Note: no datum has been defined for the water levels measured as part of this study and values are presented in meters to approximate mean water level.*

A summary of the statistical distribution of turbidity (NTU) measured at MH1 is provided in Table 9–10. A time series plot of turbidity at MH1 is presented in Figure 9-7.

Table 9–10: Summary of measured turbidity statistics at MH1

Statistic	NTU
Minimum	0.8
Maximum	4.1
Mean	2.1
Median	2.2
Standard deviation	0.6
5th percentile	1.1
99th percentile	3.5
Data length (days)	68

ADCP vessel transects

Current velocities

The ADCP vessel transect results are presented in Figure 9-11 to Figure 9-14. The figures show the predicted tidal water level, a map with depth averaged velocity vectors across the transect and current speed with depth. Transects included represent the peak measured flood and ebb flows at transects MH_Ves1 and MH_Ves2.

Figure 9-10 presents the 'point' current speeds measured along the Middle Harbour transect (MH_Ves2). The current speeds in this plot are depth averaged at each of the vessel-based 'point' measurement sites along the MH_Ves2 transect. The values in the current speed curve align with the data presented in the ADCP vessel velocity transects. Measured depth averaged current velocities did not exceed 0.15 m/s during the spring tide on 22 August 2017 at any of the vessel-based 'point' measurement sites along the MH_Ves2 transect. Higher current speeds were observed closer to the northern bank (ie on the Seaforth side of Middle Harbour).

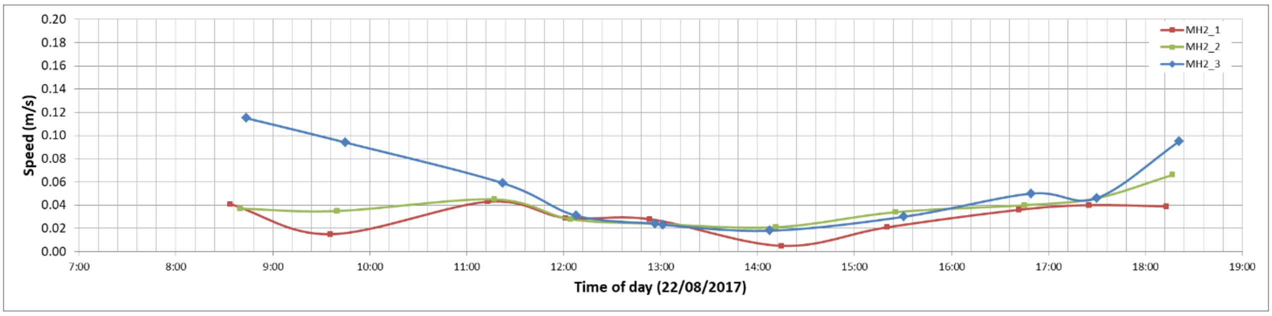


Figure 9-10: Measured current speed at the transect MH_Ves2 on 22 August 2017

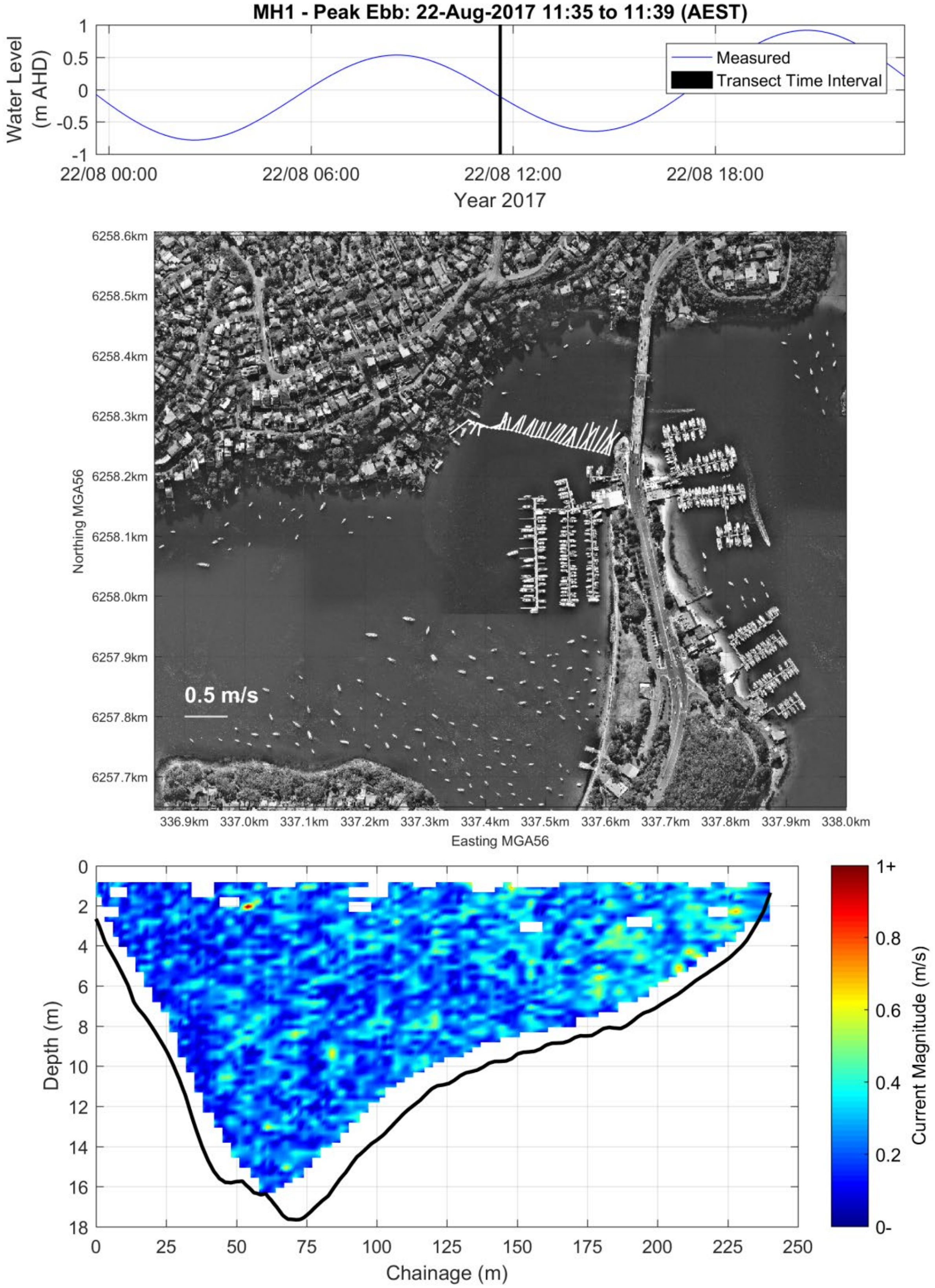


Figure 9-11: Measured spatial currents along MH-Ves1 transect during peak ebb conditions on 22 August 2017

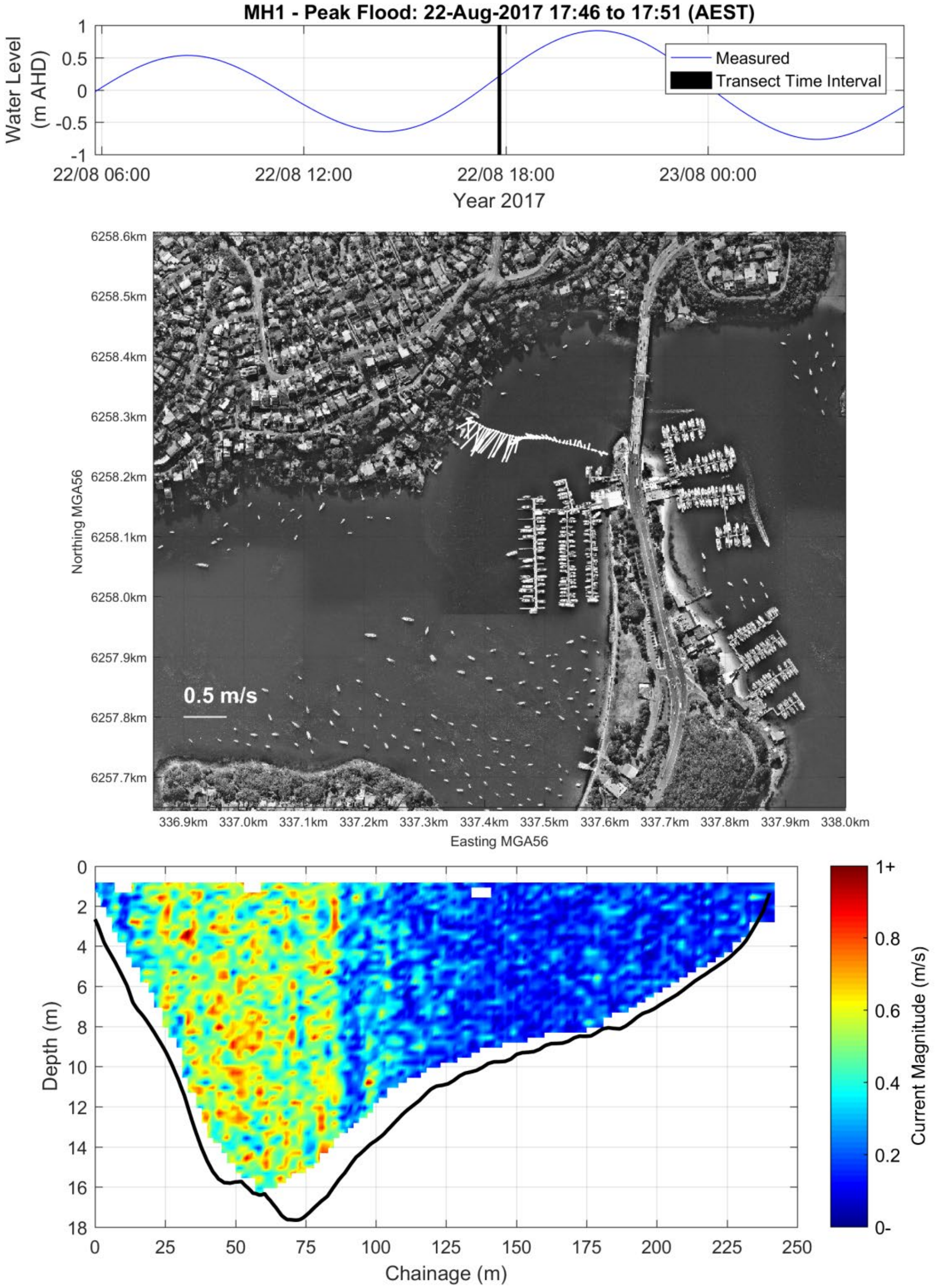


Figure 9-12: Measured spatial currents along MH-Ves1 transect during peak flood conditions on 22 August 2017

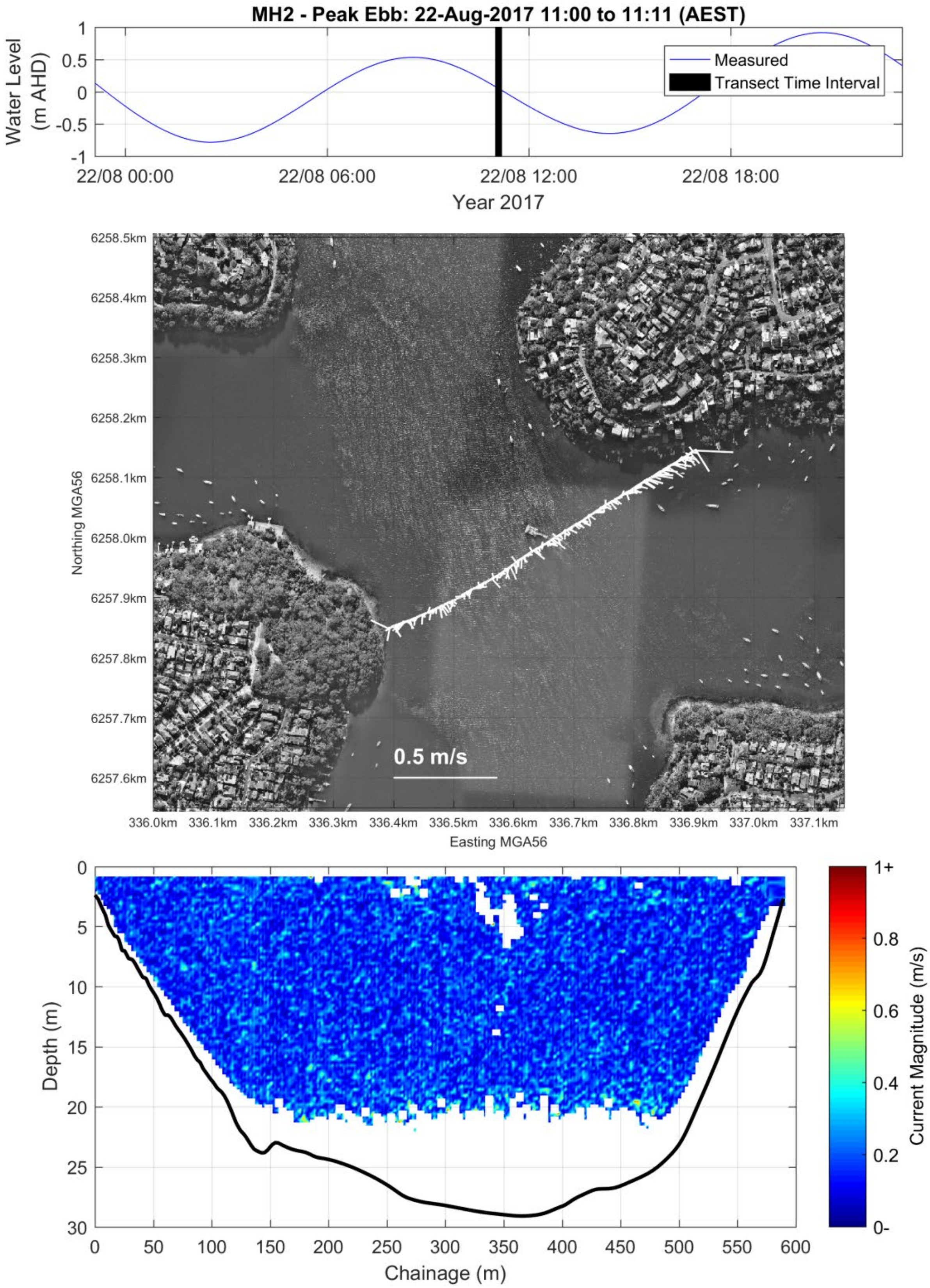


Figure 9-13: Measured spatial currents along MH-Ves2 transect during peak ebb conditions on 22 August 2017

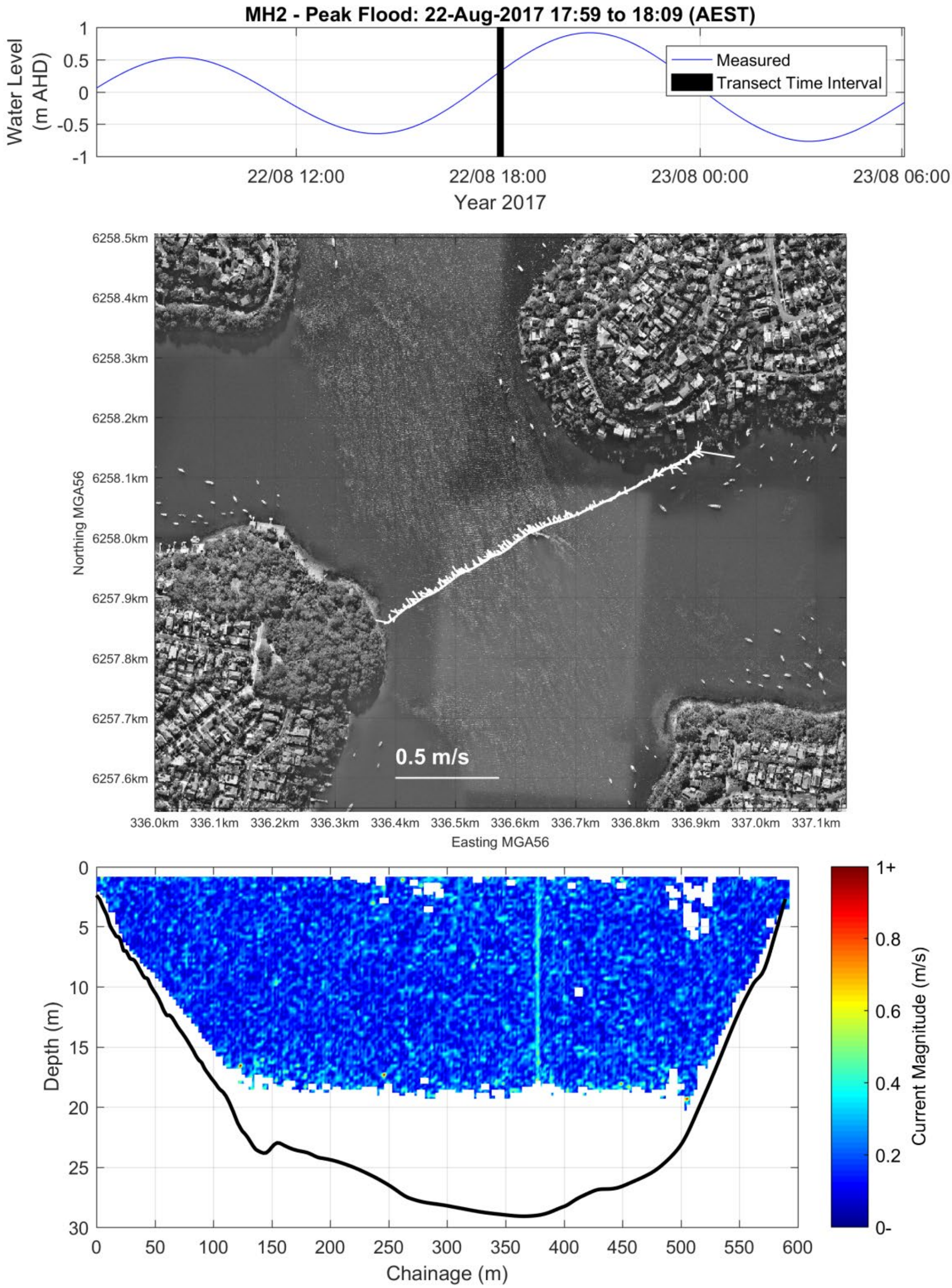


Figure 9-14: Measured spatial currents along MH-Ves2 transect during peak flood conditions on 22 August 2017

Discharge

The measured discharge at the predefined transect locations (shown in Figure 9-1) is presented in Figure 9-15. In this figure ebb flow is shown as positive and flood flow as negative. A summary of the maximum flood and ebb discharges are provided in Table 9–11.

Table 9–11: Maximum ebb and flood discharges from the ADCP vessel transects completed on 22 August 2017

Transect ID	Maximum measured discharge (m^3/s)	
	Ebb (positive)	Flood (negative)
MH_Ves1	422	523
MH_Ves2	294	395

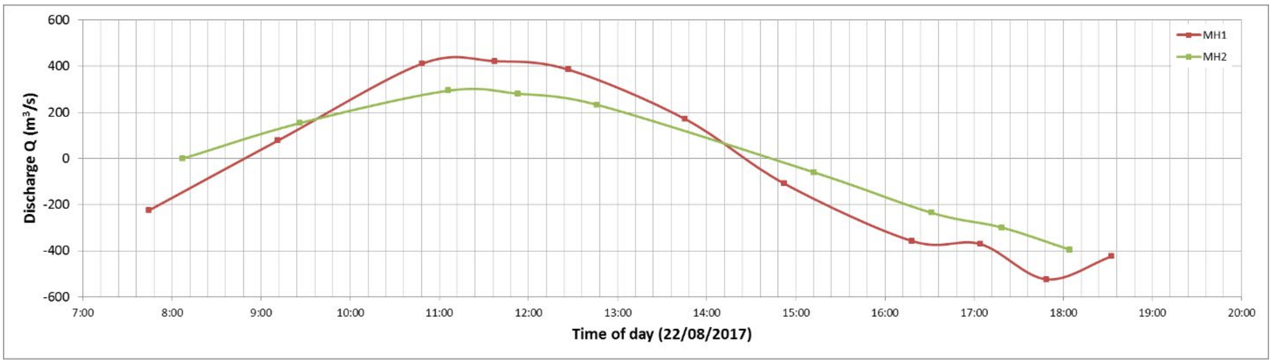


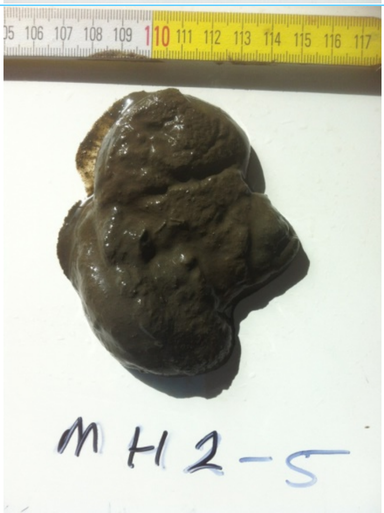
Figure 9-15: Measured discharge at transect MH1 and MH2 on 22 August 2017

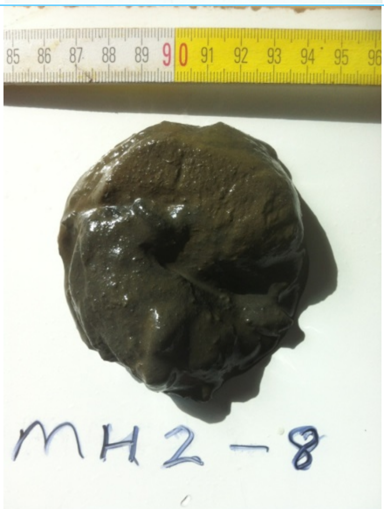
Sediment samples

Geochemical Assessments photographed and characterised each sediment sample collected during the sampling program (see Table 9–12). Based on this characterisation, a sub-set of four representative samples were subject to wet and dry sieving to separate the coarser fraction and provide a particle size distribution of the coarse material. A laser particle analysis was carried out for the silt and clay fractions (see Table 9–13).

Table 9–12: Sediment samples and description.

Sample ID	Photo	Description
MH-1		Muddy sand Shells to 2.5 cm
MH2-1		Gravelly, muddy sand (about 70% s, 15% g and 15% m). Shells to 2.5 cm. Grey – brown.
MH2-2		Muddy sand, trace gravel (>70% sand). Grey – green - brown. Medium-grained, quartzose sand.

Sample ID	Photo	Description
MH2-3		Sandy mud (>90% mud). Grey – green, very soft.
MH2-4		Mud, trace fine sand, nil gravel. Grey –brown-green. Soft. Worm tubes and faecal pellets.
MH2-5		Mud, trace fine sand, nil gravel. Grey –brown-green. Soft. Faecal pellets.

Sample ID	Photo	Description
MH2-6		Mud, trace fine sand, nil gravel. Grey –brown-green. Soft.
MH2-7		Mud, trace fine sand, nil gravel. Grey –brown-green. Very soft. Abundant faecal pellets.
MH2-8		Sandy mud. (<10% s). Brown – grey. Soft. Some faecal pellets.

Sample ID	Photo	Description
MH2-9		(Muddy) sand (<5% m), trace gravel. Brown. Medium to coarse, quartzose sand.

Table 9–13: Detailed grain size analysis for the four representative samples identified by Geochemical Assessments

GEOCHEMICAL ASSESSMENTS

Unit 22, 28 Barcoo St
Roseville, NSW 2069
Phone: 0403977209

LABORATORY TEST RESULTS

Client:	RHDHV	Client Job No.:	
Project:		Date Received: 22 August 2017	
Batch No.:	1		
Type:	Sediment		

PARTICLE SIZE SUMMARY

Sample	<63 micron	63 to 125 micron (%)	125 to 250 micron (%)	250 to 500 micron (%)	500 to 1000 micron (%)	1000 to 2000 micron (%)	>2000 micron (%)	>4000 micron (%)
MH 2-3	86.3	7.9	5.0	0.5	0.3	<0.1	0.0	0.0
MH 2-5	88.6	7.5	2.6	0.6	0.5	0.1	0.0	0.0
MH 2-7	87.2	8.2	3.4	0.7	0.4	<0.1	0.0	0.0
MH 2-8	79.2	9.2	7.7	3.0	0.8	0.1	0.0	0.0
MH 2-8 Dup	76.5	10.4	8.0	3.8	1.1	0.2	0.0	0.0

Test Procedure : AS 1289 3.6.1 wet sieving
Prepared by: SET Checked by: HDT

References

Cardno, (2010). Estuarine Planning Levels Study – Foreshore Region of Leichhardt Local Government Area. Prepared for Leichardt Municipal Council.

Nortek AS, (2013). Principles of Operations – Measuring Currents and Waves.

Teledyne RDI (2013). ADCP Sentinel V Operation Manual. P/N 95D-6002-00.

Annexure B – Dredge plume results

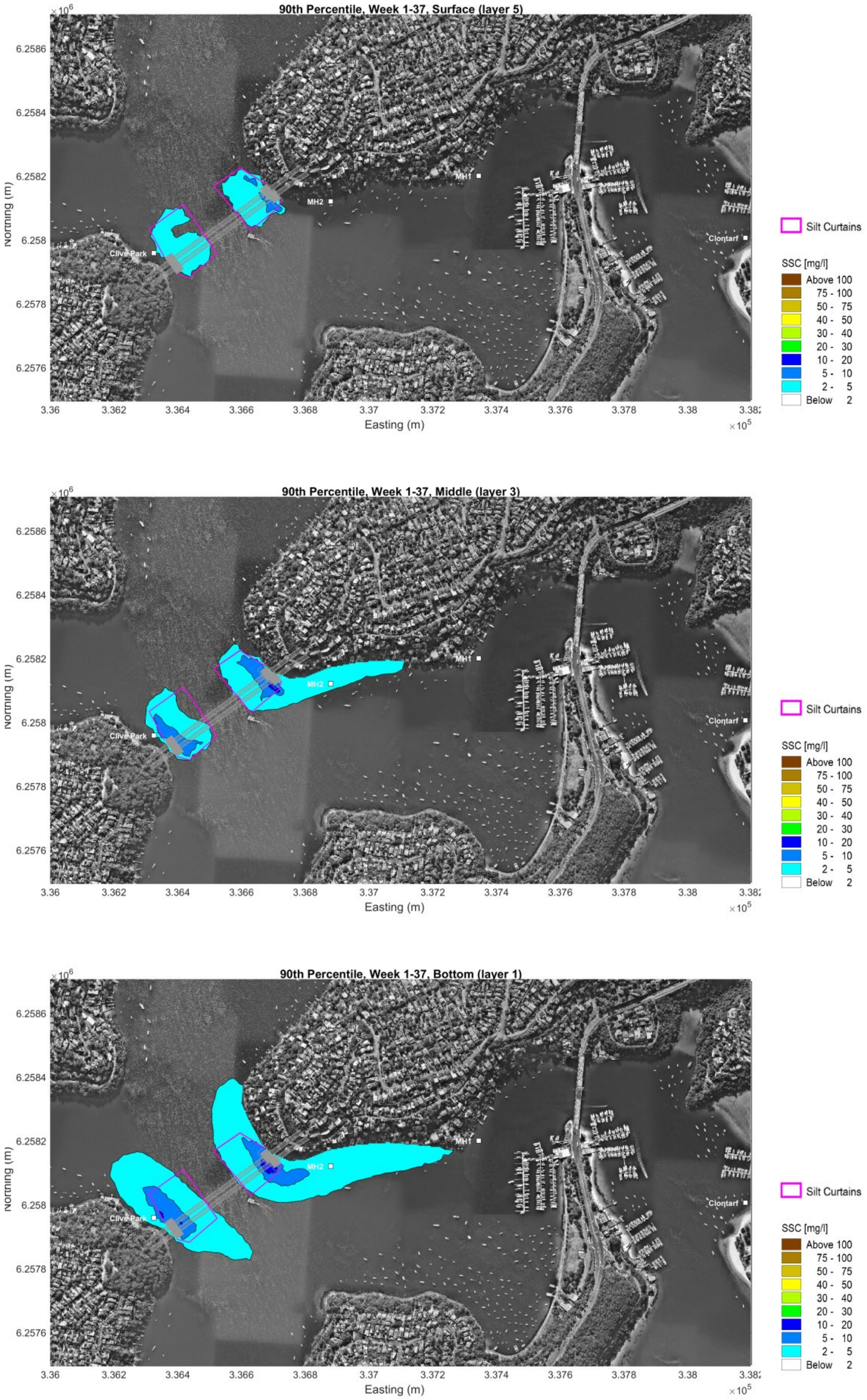


Figure 9-16: 90th percentile, for surface (top), mid-water column (middle) and near the seabed (bottom) for the entire dredging period (weeks 1 to 37)

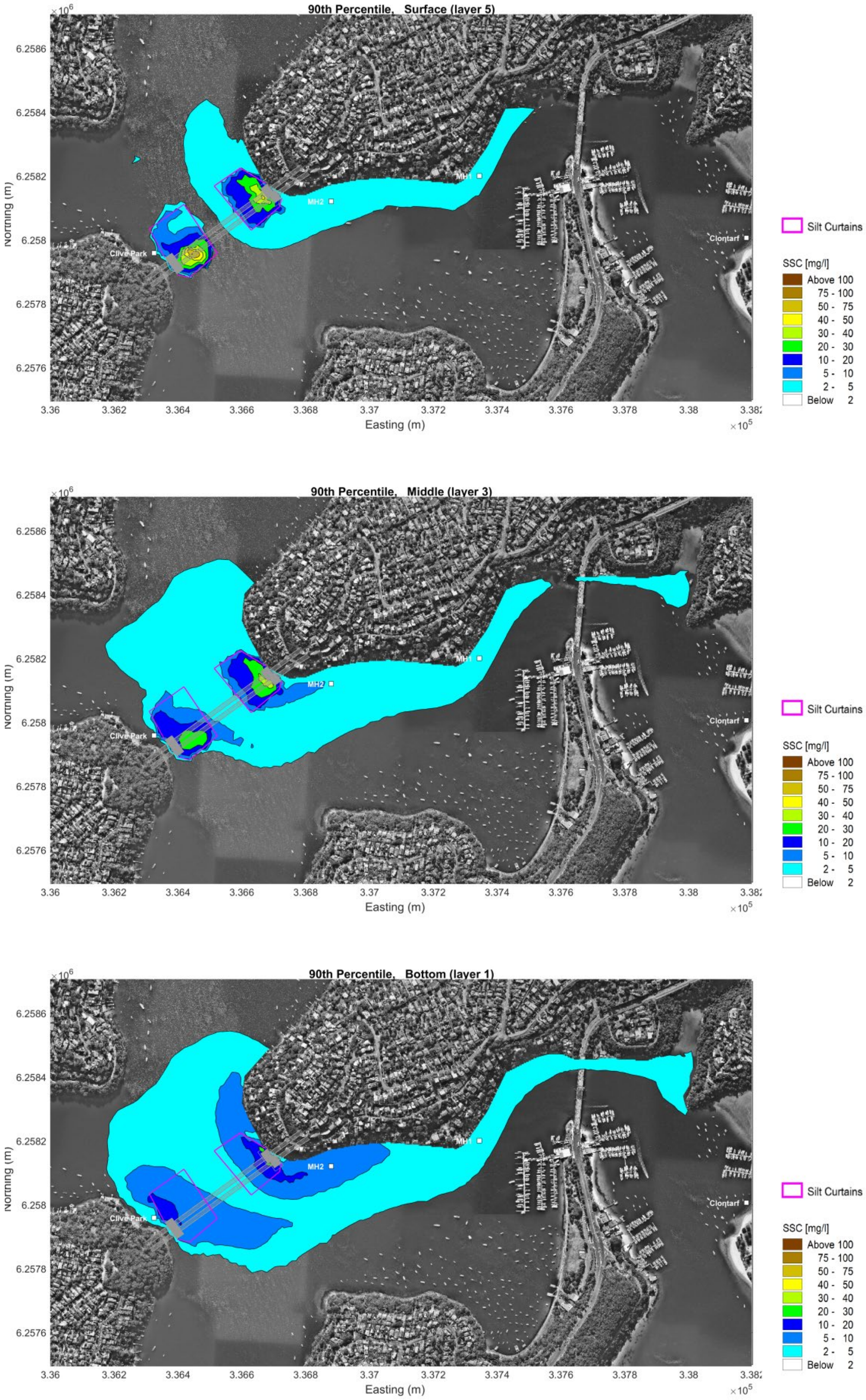


Figure 9-17: 90th percentile, for surface (top), mid-water column (middle) and near the seabed (bottom) for the BHD only dredging period (weeks 1 to 4)

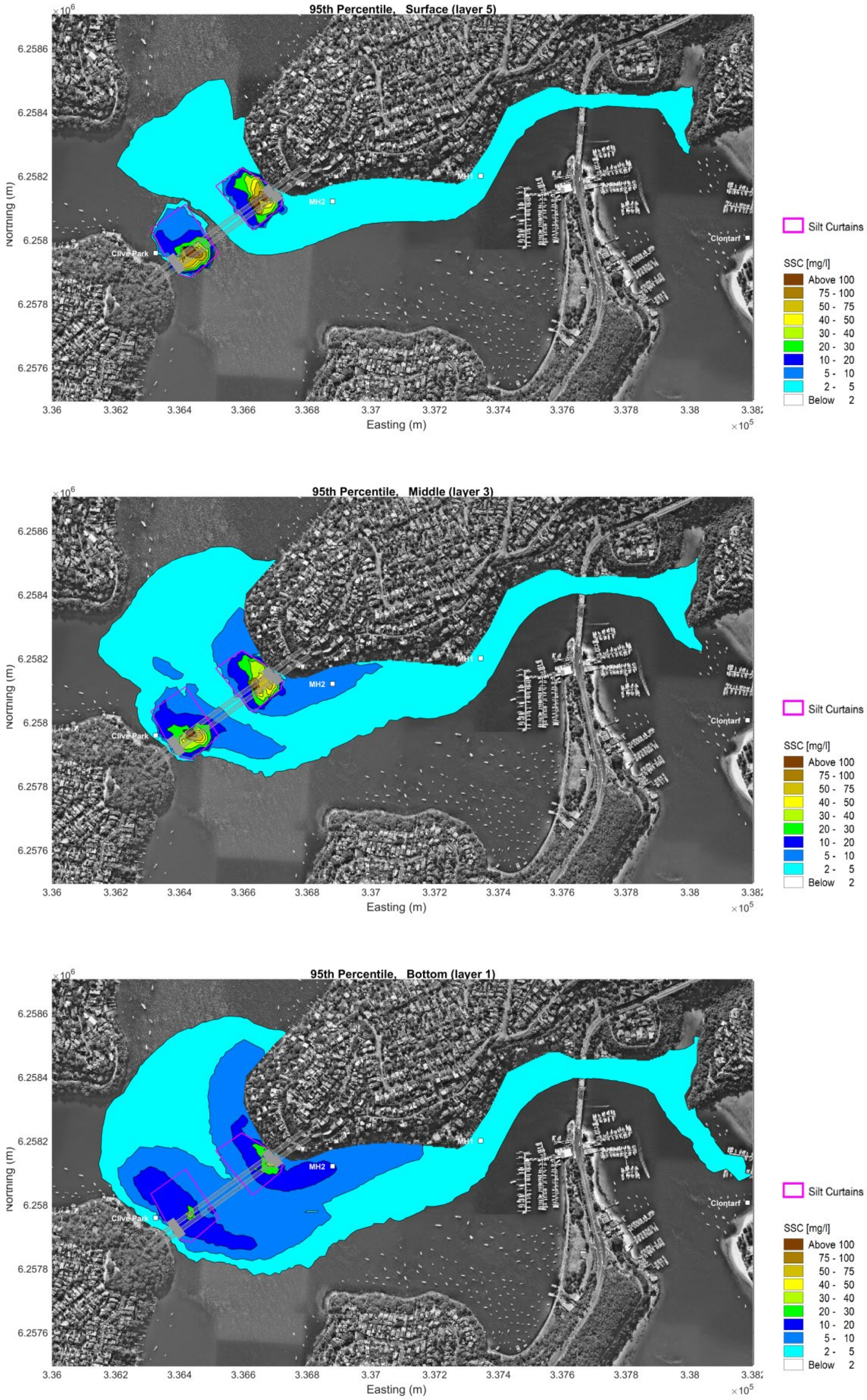


Figure 9-18: 95th percentile, for surface (top), mid-water column (middle) and near the seabed (bottom) for the BHD only dredging period (weeks 1 to 4)

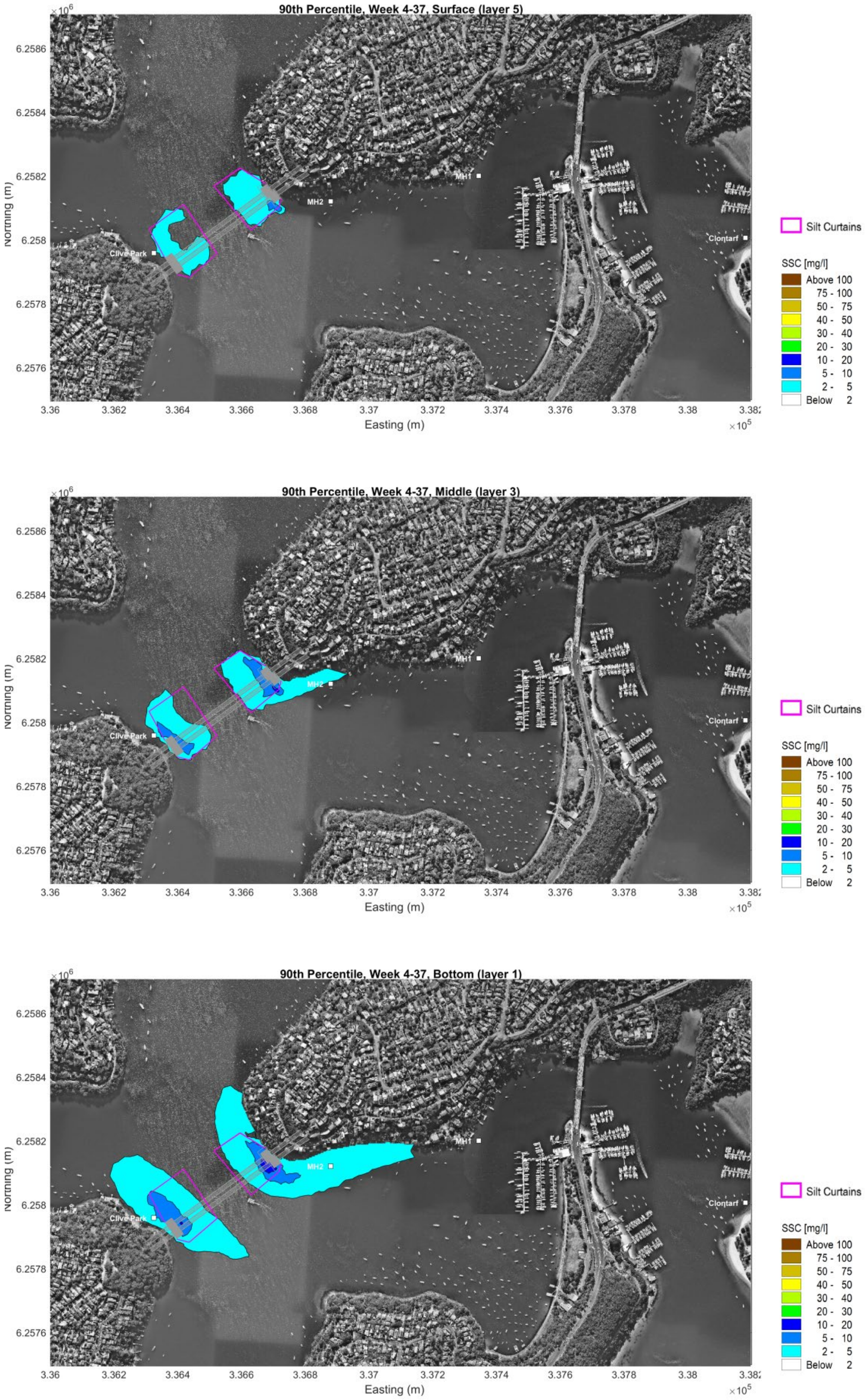


Figure 9-19: 90th percentile, for surface (top), mid-water column (middle) and near the seabed (bottom) for the drum cutter and BHD rehandle portion of the dredging period (weeks 4 to 37)

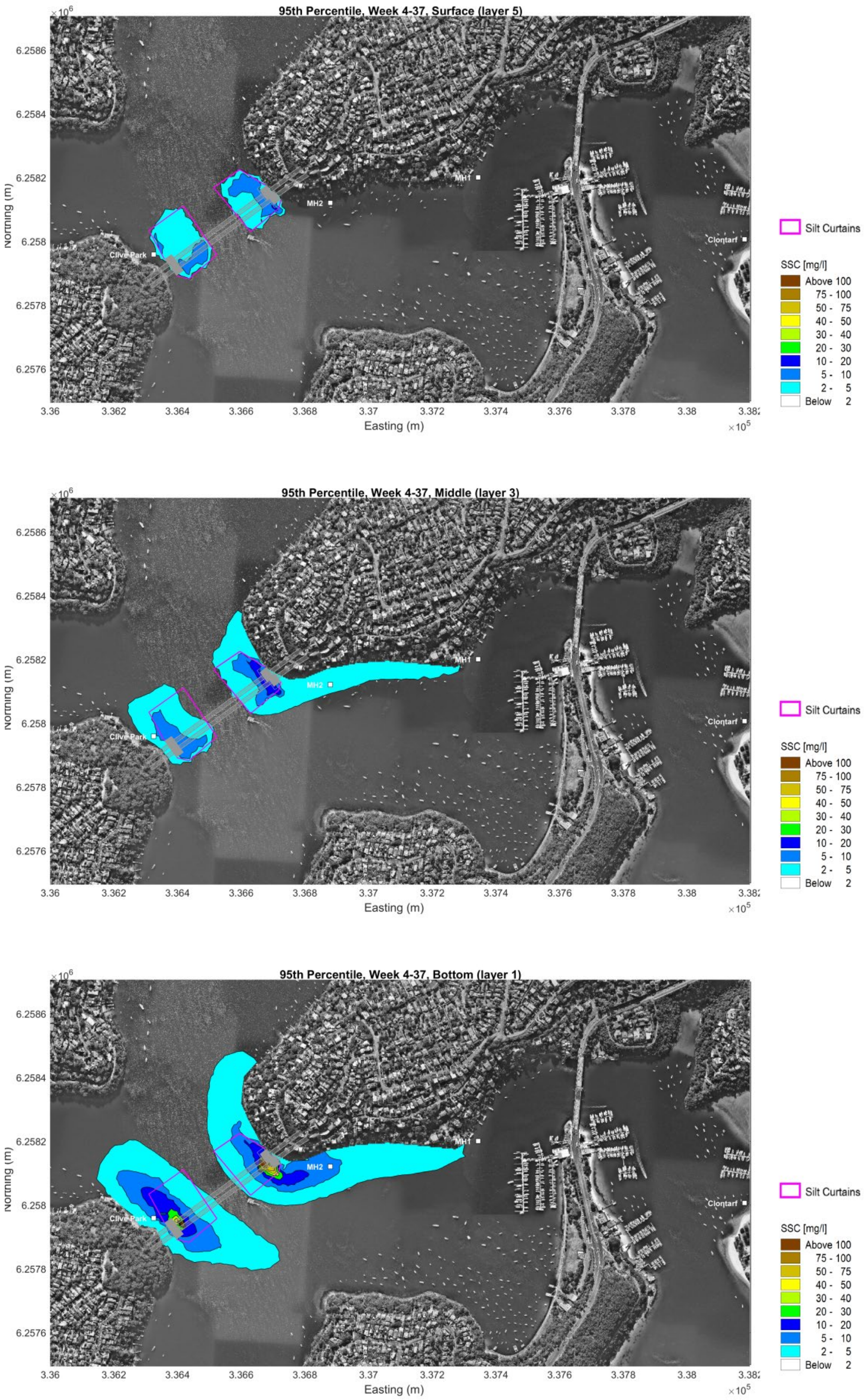


Figure 9-20: 95th percentile, for surface (top), mid-water column (middle) and near the seabed (bottom) for the drum cutter and BHD rehandling portion of the dredging period (weeks 4 to 37)