Volume 2H

Appendices P to R

P: Hydrodynamics and dredge plume modelling
Q: Marine water quality
R: Flooding

Appendix

P

Appendix P

Hydrodynamics and dredge plume modelling

January 2020

Roads and Maritime Services

Western Harbour Tunnel and Warringah Freeway Upgrade

Technical working paper: Hydrodynamic and dredge plume modelling

January 2020

Prepared for

Roads and Maritime

Prepared by

Haskoning Australia PTY Ltd.

Table of Contents

Glossary

2D	Two-dimensional
3D	Three-dimensional
ADCP	Acoustic Doppler Current Profiler
AHD	Australian Height Datum
CBD	Central Business District
BHD	Back Hoe Dredge
C-M ap	Commercially available source of digitised Admiralty navigational charts
CSD	Cutter Suction Dredge
MIKE 21	Two-dimensional computer modelling software the can simulate flows, waves, sediments, ecology and water quality in rivers, lakes, estuaries, bays and open seas.
MIKE 3	Three-dimensional computer modelling software the 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
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	Western Harbour Tunnel and Warringah Freeway Upgrade
TSHD	Trailer Suction Hopper Dredge

Executive Summary

This assessment details the findings of numerical modelling to better understand the potential impact of construction activities related to the Western Harbour Tunnel and Warringah Freeway Upgrade (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) model that has been established for this project.

The 3D hydrodynamic model was used to assess potential hydrodynamic impacts and water quality impacts. The assessment of hydrodynamic impacts focuses on the construction of two temporary cofferdams, while the water quality impacts are related to the dredging required for the construction of the immersed tube tunnel.

The main outcomes of the modelling of hydrodynamic impacts during the projects construction phase are:

- During the ebb tide current speeds reduce downstream of the south-western cofferdam
- During the flood tide current speeds reduce upstream of the south-western cofferdam
- Current speeds increase along the northern bank near Greenwich Baths would occur during the flood tide only, and are more evident during spring tides
- The expected changes in tidal currents are unlikely to result in erosion of the seabed or adjacent foreshore.

The main outcomes of the modelling of dredge plume related water quality impacts during the construction phase are:

- The extent of the plume of suspended sediment caused by dredging is relatively small and is concentrated nears Balls Head Bay and Balls Head
- The extent of the potentially visible plume is very small and typically contained in the dredging footprint immediately adjacent to the north east cofferdam
- The suspended sediment released during the dredging activity is transported into Balls Head Bay and in both in an upstream and downstream direction within Sydney Harbour
- Increases in suspended sediment are expected to be low everywhere for at least 50 per cent of the dredging campaign
- The majority of the sediment deposition due to the dredging activity occurs in the dredging footprint and adjacent to the dredging footprint. Lower levels of sedimentation also occur in Balls Head Bay and within other adjacent embayments that line Sydney Harbour.

A number of environmental controls are proposed as part of dredging operations. These controls reduce or avoid the release of suspended sediments during dredging (eg appropriate dredging equipment) and to manage the suspended sediment that is released (eg floating barriers to contain suspended sediment). They reflect best environmental practice to reduce the water quality impacts of dredging. These environmental controls 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 chapter provides an overview of the Western Harbour Tunnel and Warringah Freeway Upgrade (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), a plan 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 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 Gore Hill Freeway to Balgowlah and Killarney Heights and including the surface upgrade of 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 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 harbour crossings.

1.2 The project

Roads and Maritime Services (Roads and Maritime) is seeking approval under Division 5.2, Part 5 of the *Environmental Planning and Assessment Act 1979* to construct and operate the Western Harbour Tunnel and Warringah Freeway Upgrade (the project), which would comprise two main components:

- A new crossing of Sydney Harbour involving twin tolled motorway tunnels connecting the M4-M5 Link at Rozelle and the existing Warringah Freeway at North Sydney (the Western Harbour Tunnel)
- Upgrade and integration works along the existing Warringah Freeway, including infrastructure required for connections to the Beaches Link and Gore Hill Freeway Connection project (the Warringah Freeway Upgrade).

Key features of the Western Harbour Tunnel component of the project are shown in Figure 1-1 and would include:

- Twin mainline tunnels about 6.5 kilometres long and each accommodating three lanes of traffic in each direction, connecting the stub tunnels from the M4-M5 Link at Rozelle to the Warringah Freeway and to the Beaches Link mainline tunnels at Cammeray. The crossing of Sydney Harbour between Birchgrove and Waverton would involve a dual, three lane, immersed tube tunnel

- Connections to the stub tunnels at the M4-M5 Link project in Rozelle and to the mainline tunnels at Cammeray (for a future connection to the Beaches Link and Gore Hill Freeway Connection project)
- Surface connections at Rozelle, North Sydney and Cammeray, including direct connections to and from the Warringah Freeway (including integration with the Warringah Freeway Upgrade), an off ramp to Falcon Street and an on ramp from Berry Street at North Sydney
- A ventilation outlet and motorway facilities (fitout and commissioning only) at the Rozelle Interchange
- A ventilation outlet and motorway facilities at the Warringah Freeway in Cammeray
- Operational facilities including a motorway control centre at Waltham Street, within the Artarmon industrial area and tunnel support facilities at the Warringah Freeway in Cammeray
- Other operational infrastructure including groundwater and tunnel drainage management and treatment systems, signage, tolling infrastructure, fire and life safety systems, lighting, emergency evacuation and emergency smoke extraction infrastructure, CCTV and other traffic management systems.

Key features of the Warringah Freeway Upgrade component of the project are shown in Figure 1-2 and would include:

- Upgrade and reconfiguration of the Warringah Freeway from immediately north of the Sydney Harbour Bridge through to Willoughby Road at Naremburn
- Upgrades to interchanges at Falcon Street in Cammeray and High Street in North Sydney
- New and upgraded pedestrian and cyclist infrastructure
- New, modified and relocated road and shared user bridges across the Warringah Freeway
- Connection of the Warringah Freeway to the portals for the Western Harbour Tunnel mainline tunnels and the Beaches Link tunnels via on and off ramps, which would consist of a combination of trough and cut and cover structures
- Upgrades to existing roads around the Warringah Freeway to integrate the project with the surrounding road network
- Upgrades and modifications to bus infrastructure, including relocation of the existing bus layover along the Warringah Freeway
- Other operational infrastructure, including surface drainage and utility infrastructure, signage, tolling, lighting, CCTV and other traffic management systems.

A detailed description of the project is provided in Chapter 5 (Project description) and construction of the project is described in Chapter 6 (Construction work) of the environmental impact statement. The project alignment at the Rozelle Interchange shown in Figure 1-1 and Figure 1-3 reflects the arrangement presented in the environmental impact statement for the M4-M5 Link, and as amended by the proposed modifications. This project would be constructed in accordance with the finalised M4-M5 Link detailed design (refer to Section 2.1.1 of Chapter 2 (Assessment process) of the environmental impact statement for further details).

The project does not include ongoing motorway maintenance activities during operation or future use of residual land occupied or affected by project construction activities, but not required for operational infrastructure. These would be subject to separate planning and processes at the relevant times. Subject to the project obtaining planning approval, construction is anticipated to commence in 2020 and is expected to take around six years to complete.

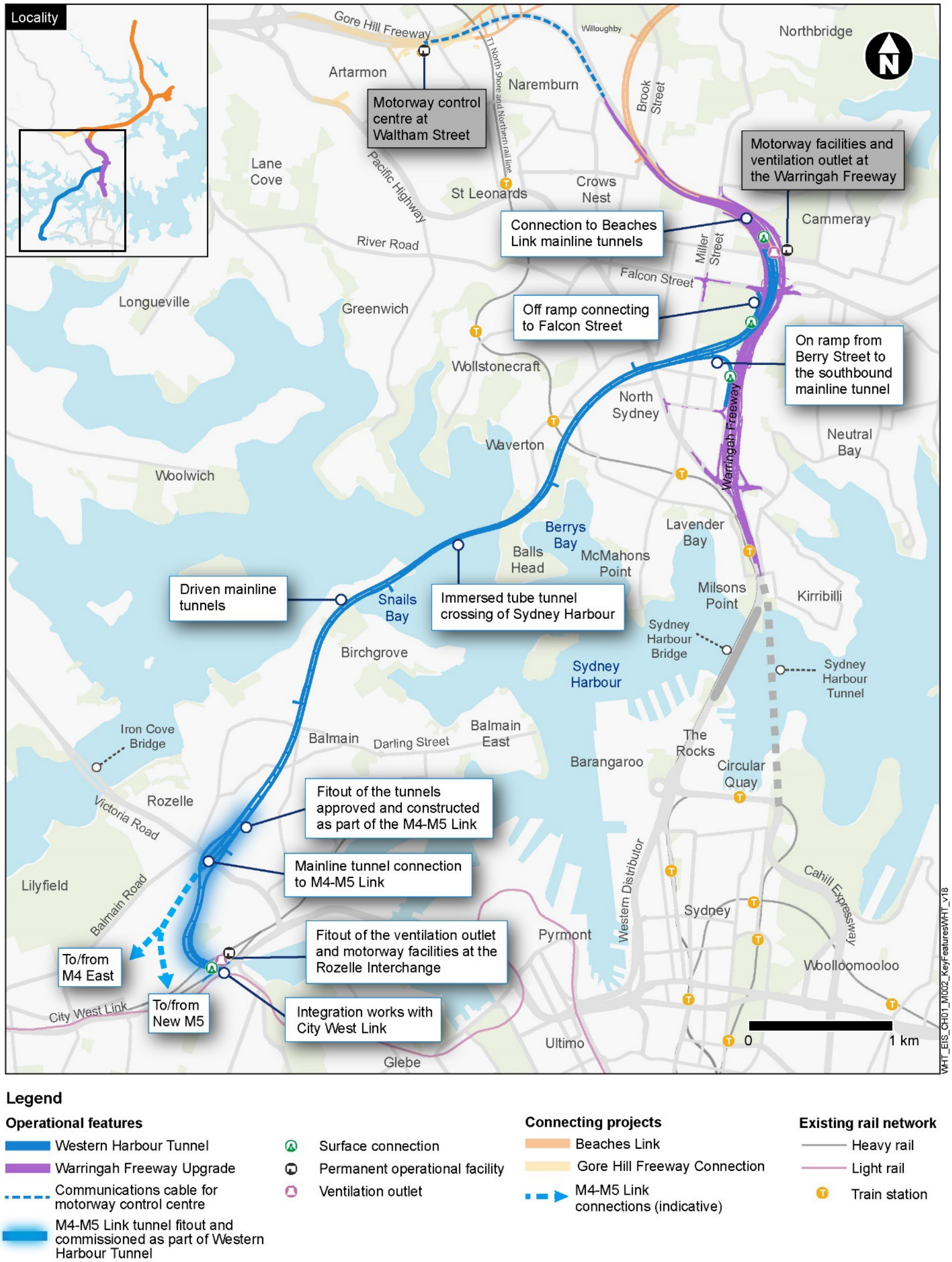


Figure 1-1: Key features of the Western Harbour Tunnel component of the project

Royal HaskoningDHV

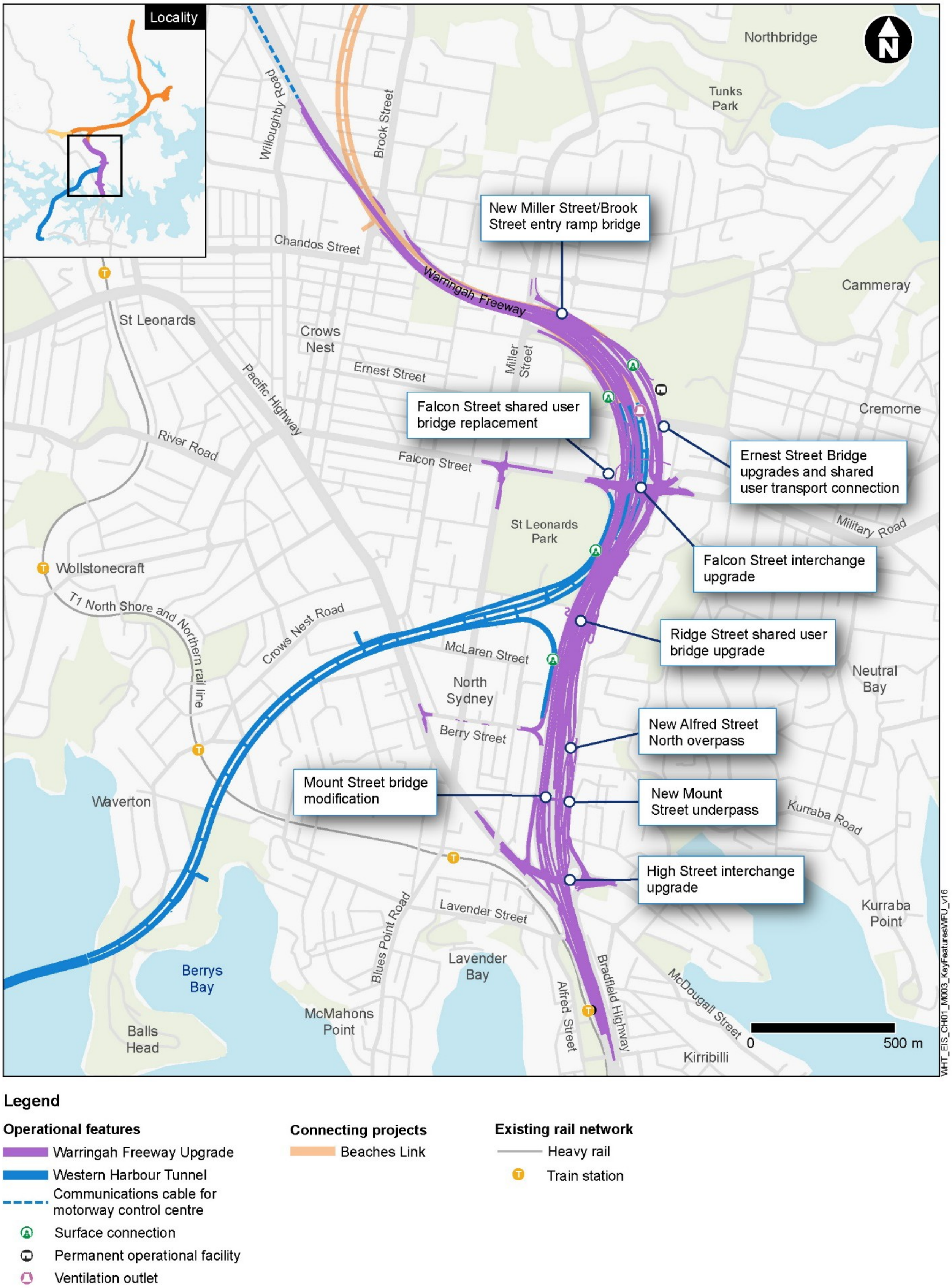


Figure 1-2: Key features of the Warringah Freeway Upgrade component of the project

1.3 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 tunnels. However, surface areas would be required to support tunnelling activities and to construct the tunnel connections, tunnel portals and operational ancillary 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) and traffic management controls, vegetation clearing, earthworks and demolition of structures, establishment of 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 within Berrys Bay and relocation of the historic vessels
- Construction of Western Harbour Tunnel, with typical activities being excavation of tunnel construction accesses, construction of driven tunnels, cut and cover and trough structures and construction of cofferdams, dredging 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 of a motorway control centre at Waltham Street in Artarmon, motorway and tunnel support facilities and ventilation outlets at the Warringah Freeway in Cammeray, construction and fitout of the project operational facilities that form part of the M4-M5 Link Rozelle East Motorway Operations Complex, a wastewater treatment plant at Rozelle and the installation of motorway tolling infrastructure
- Construction of the Warringah Freeway Upgrade, with typical activities being earthworks, bridgeworks, construction of retaining walls, stormwater drainage, pavement works and linemarking and the installation of road 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 Figure 1-3), and would include tunnelling and tunnel support sites, civil surface sites, cofferdams, mooring sites, wharf and berthing facilities, laydown areas, parking and workforce amenities. Construction support sites for Western Harbour Tunnel would include:

- Rozelle Rail Yards (WHT1)
- Victoria Road (WHT2)
- White Bay (WHT3)
- Yurulbin Point (WHT4)
- Sydney Harbour south cofferdam (WHT5)
- Sydney Harbour north cofferdam (WHT6)
- Berrys Bay (WHT7)
- Berry Street north (WHT8)
- Ridge Street north (WHT9)
- Cammeray Golf Course (WHT10)

- Waltham Street (WHT11).

During the construction of the Warringah Freeway Upgrade, smaller construction support sites would be required to support the construction works (as shown on Figure 1-3). These include:

- Blue Street (WFU1)
- High Street south (WFU2)
- High Street north (WFU3)
- Arthur Street east (WFU4)
- Berry Street east (WFU5)
- Ridge Street east (WFU6)
- Merlin Street (WFU7)
- Cammeray Golf Course (WFU8)
- Rosalind Street east (WFU9).

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

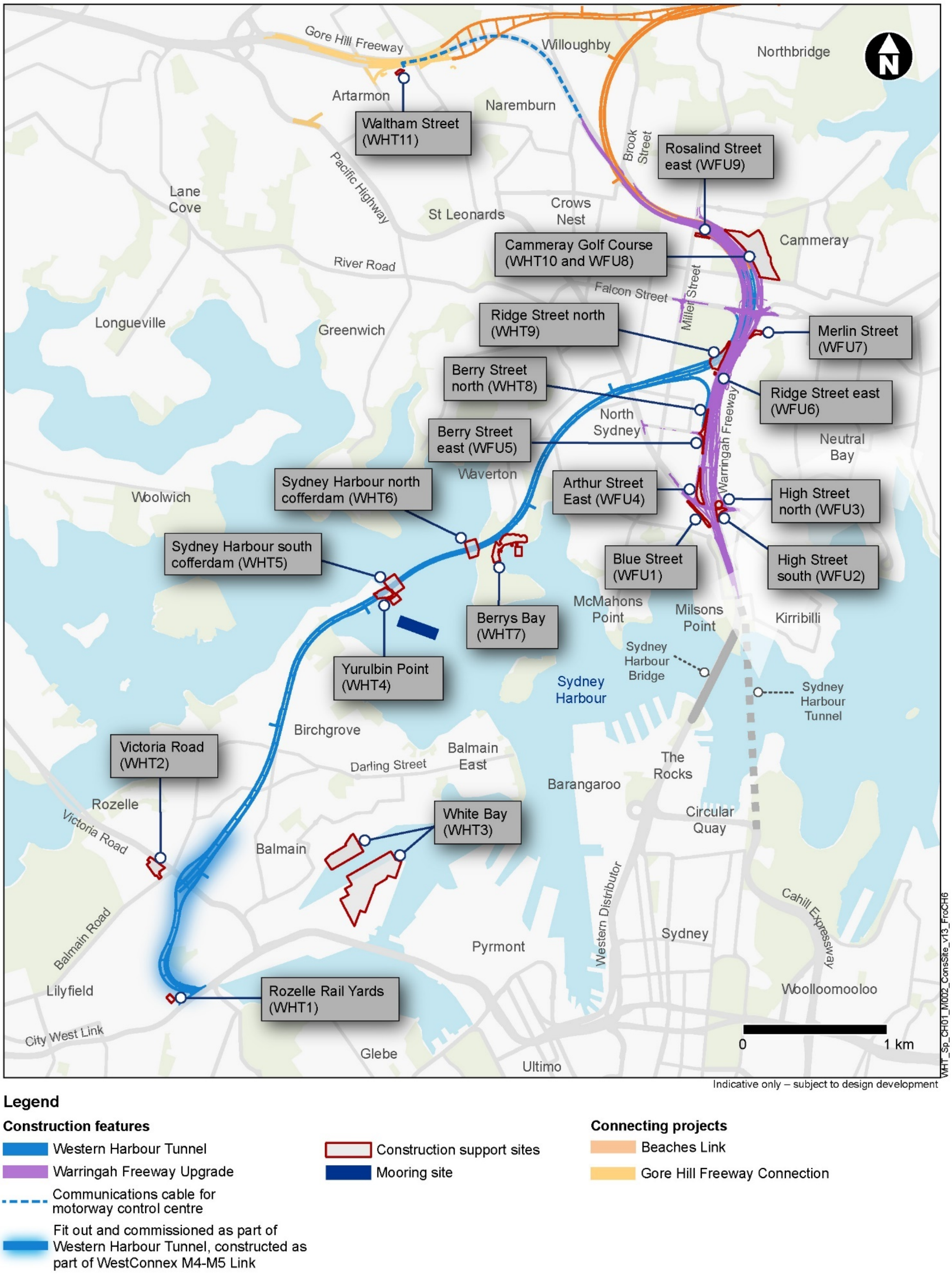


Figure 1-3: Overview of construction support sites

1.4 Project location

The project would be located within the Inner West, North Sydney and Willoughby local government areas, connecting Rozelle in the south with Naremburn in the north.

Commencing at the Rozelle Interchange, the mainline tunnels would pass under Balmain and Birchgrove, then cross Sydney Harbour between Birchgrove and Balls Head. The tunnels would then continue under Waverton and North Sydney, linking directly to the Warringah Freeway to the north of the existing Ernest Street bridge.

The motorway control centre would be located at Waltham Street, Artarmon, with a trenched communications link connecting the motorway control centre to the Western Harbour tunnel along the Gore Hill Freeway and Warringah Freeway road reserves.

The Warringah Freeway Upgrade would be carried out on the Warringah Freeway from around Fitzroy Street at Milsons Point to around Willoughby Road at Naremburn. Upgrade works would include improvements to bridges across the Warringah Freeway, and upgrades to surrounding roads.

1.5 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 Department of Planning, Industry and Environment (Planning and Assessment) (formerly Department of Planning and Environment) ('the Secretary's environmental assessment requirements').

This report documents a hydrodynamic and water quality impact assessment for the Western Harbour Tunnel crossing. 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 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 for the impact assessment for the following items:
    - Impacts of the temporary cofferdams on the hydrodynamics
    - Impacts of the dredging on the water quality for the various stages of dredging.

As the seabed at the Western Harbour Tunnel crossing would be returned to the same level as it was before the crossing, no numerical modelling of this post construction case was required.

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

1.6 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).	Chapter 3
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;	Chapter 6, Chapter 8
(d) direct or indirect increases in erosion, siltation, destruction of riparian vegetation or a reduction in the stability of river banks or watercourses;	Chapter 6, Chapter 8
(f) measures to mitigate the impacts of the proposal and manage the disposal of produced and incidental water.	Chapter 6, Chapter 8
10. Water Quality	
1. The Proponent must:	
(a) describe the background conditions for any surface or groundwater resource likely to be affected by the development;	Chapter 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.	Chapter 7, Chapter 8 Pollutants quality and quantity information and estimations of groundwater inflows to be treated are presented in Technical working paper: Surface water quality and hydrology. Contaminated harbour sediments are presented in Technical working paper: Contamination.

Secretary's environmental assessment requirements	Where addressed
(e) assess the significance of any identified impacts including consideration of the relevant ambient water quality outcomes;	Chapter 7, Chapter 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;	Chapter 7, Chapter 8
(i) identify sensitive receiving environments (which may include estuarine and marine waters downstream including Quarry Creek and its catchment) and develop a strategy to avoid or minimise impacts on these environments.	Chapter 7, Chapter 8

2 Available data

2.1 Introduction

This chapter 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 review of historical data available, were filled so that sufficient data was available to describe the existing environment and calibrate the hydrodynamic model.

2.2 Historical data

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

Bathymetric data is made up of the latest available detailed soundings data provided by Roads and Maritime for areas around the project crossing and the majority of the Parramatta River. For other areas, the detailed bathymetric data was augmented with digitised navigation charts available from C-Map[1].

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 MHL on behalf of OEH 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	This study predominately utilised 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 project crossing with a sufficiently long record.
Currents	Balls Head	Data collected by Sydney Ports (Port Authority of NSW). 6 months of current speed and direction data have been purchased for this study (1 June 2017 – 5 December 2017).
Water quality	Barangaroo	RHDHV have 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).

[1] C-Map is a commercially available source of digitised Admiralty navigational charts

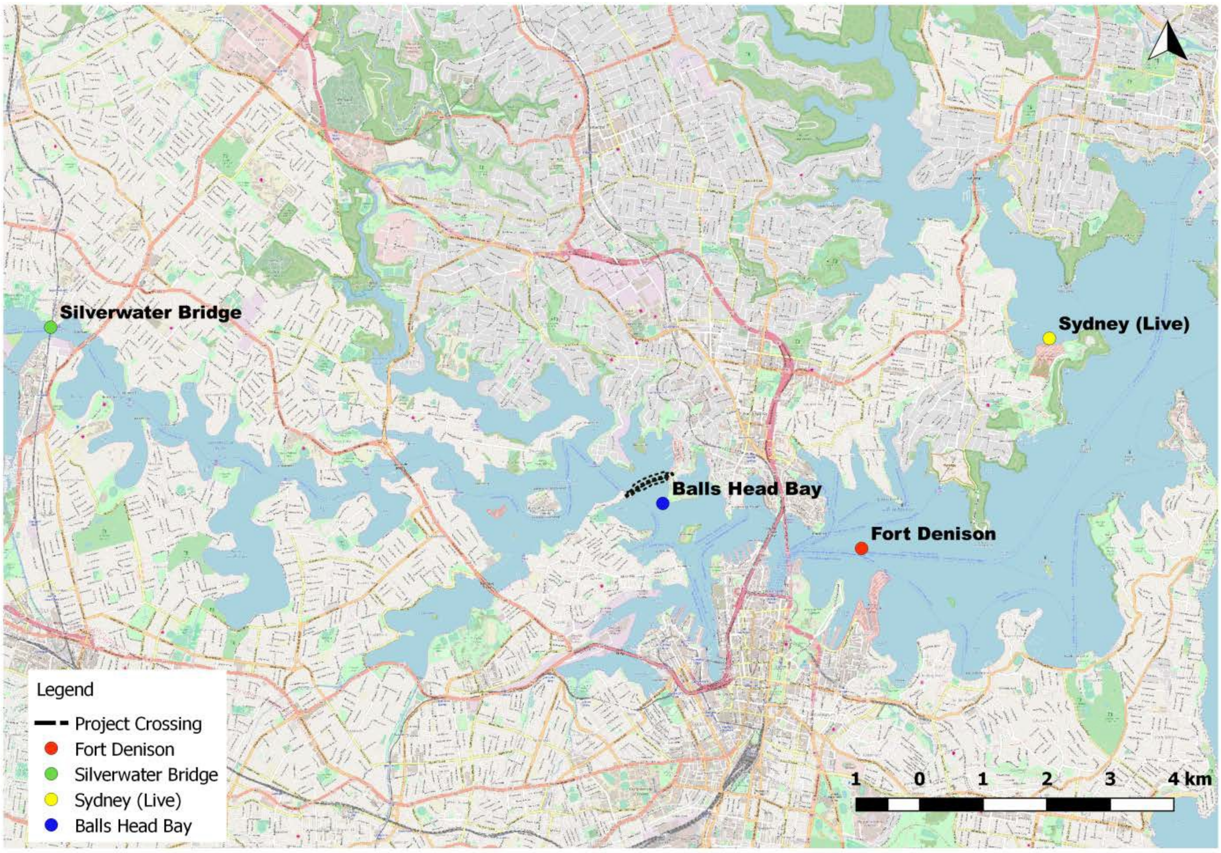


Figure 2-1: Existing data available in the Port Jackson and Parramatta River area.

2.3 Project specific data collection

Project-specific hydrodynamic and water quality data has been collected at the project crossing site. Details regarding the monitoring campaign, including a factual presentation of the data, are provided in Appendix A. The aim of this 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 has been collected at Western Harbour are shown in Figure 2-2. The measured data collected is summarised as:

- Two in situ monitoring sites (WH1 and WH2) located near the project crossing have been used to measure 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 Acoustic Doppler Current Profiler (ADCP) type instrument. At any one time, one of the in situ monitoring sites also measures water quality parameters (primarily turbidity). Water quality monitoring for the project was primarily carried out by Cardno on behalf of Roads and Maritime and reported separately. 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 initial monitoring period for which data was available for the purposes of the 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 the three transects at the project crossing shown in Figure 2-2. Vessel-mounted ADCP transects were carried out during spring tidal

conditions on 21 August 2017 to determine spatial variability in currents and discharge throughout a tidal cycle

- Opportunistic surface seabed sediment samples were collected at each crossing location and analysed for particle size distribution.

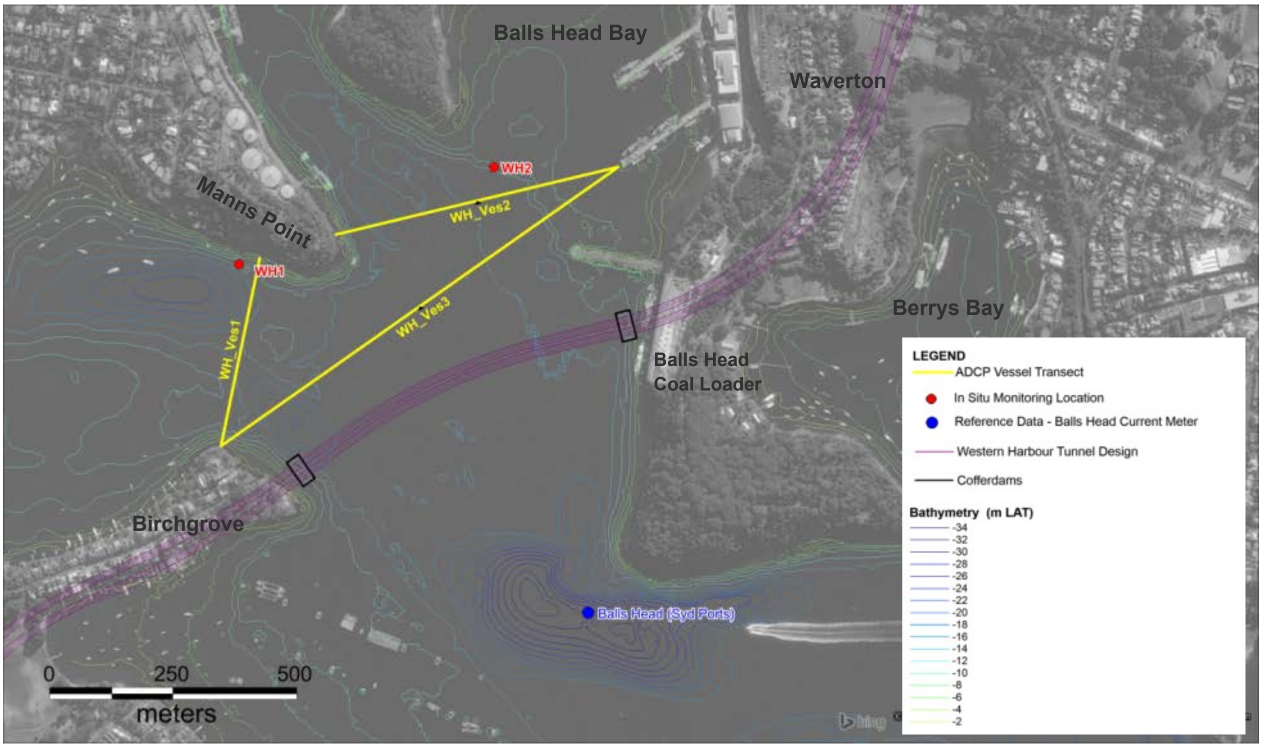


Figure 2-2: Map showing the hydrodynamic and water quality monitoring locations at the project's crossing location.

3 Description of the existing environment

3.1 Site description

The proposed project crossing site is located within Port Jackson around two kilometres to the north-west 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 comprises three harbours: North Harbour, Middle Harbour and Sydney Harbour (the main branch of the estuary). The western region of Port Jackson is the western branch of the estuary, and comprises two of Port Jackson's main tributaries: Lane Cover River and Parramatta River (Western Harbour). The other main tributary is Middle Harbour.

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 mm/day.

The hydrodynamic conditions at the project crossing location are primarily driven by astronomical tides with other influences from barometric effects, wind and freshwater flows from local creeks and rivers being comparatively small. The wave climate is limited to locally generated wind waves and waves from boat wakes (specifically ferries).

3.2 Bathymetry

3.2.1 Level datum

All levels in this report refer to Australian Height Datum (AHD) unless noted otherwise. Chart Datum lies 0.925 m 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 Roads and Maritime. This available bathymetry data is presented in Figure 3-1.

The seabed of Port Jackson comprises many deep holes, shoals, basins, rocky islands and reefs. The bathymetry near the crossing site is complex and irregular with defined channels, shallow embayments and deep holes.

The location of the crossing stretches from Long Nose Point (part of the suburb of Birchgrove) in the south to Balls Head in the north and crosses the main channel of Port Jackson. The bathymetry at the proposed crossing location is best described as a defined channel with relatively steep banks. The depth of the channel in the vicinity of the crossing averages about 15 metres, with a notable hole about 17 metres deep at the Birchgrove end. Other notable features within the vicinity of the site include a deep hole to the north of the proposed tunnel crossing. This hole is about 32 metres deep and is located immediately south-west of Balls Head.

A number of bays are located in close proximity to the location of the proposed crossing. These bays include Balls Head Bay directly to the north, Snails Bay to the south and Berrys Bay on the opposite side of Balls Head. These comparatively shallow bays adjoin the main channel and from a hydrodynamics perspective act as relatively large reservoirs for tidal waters.

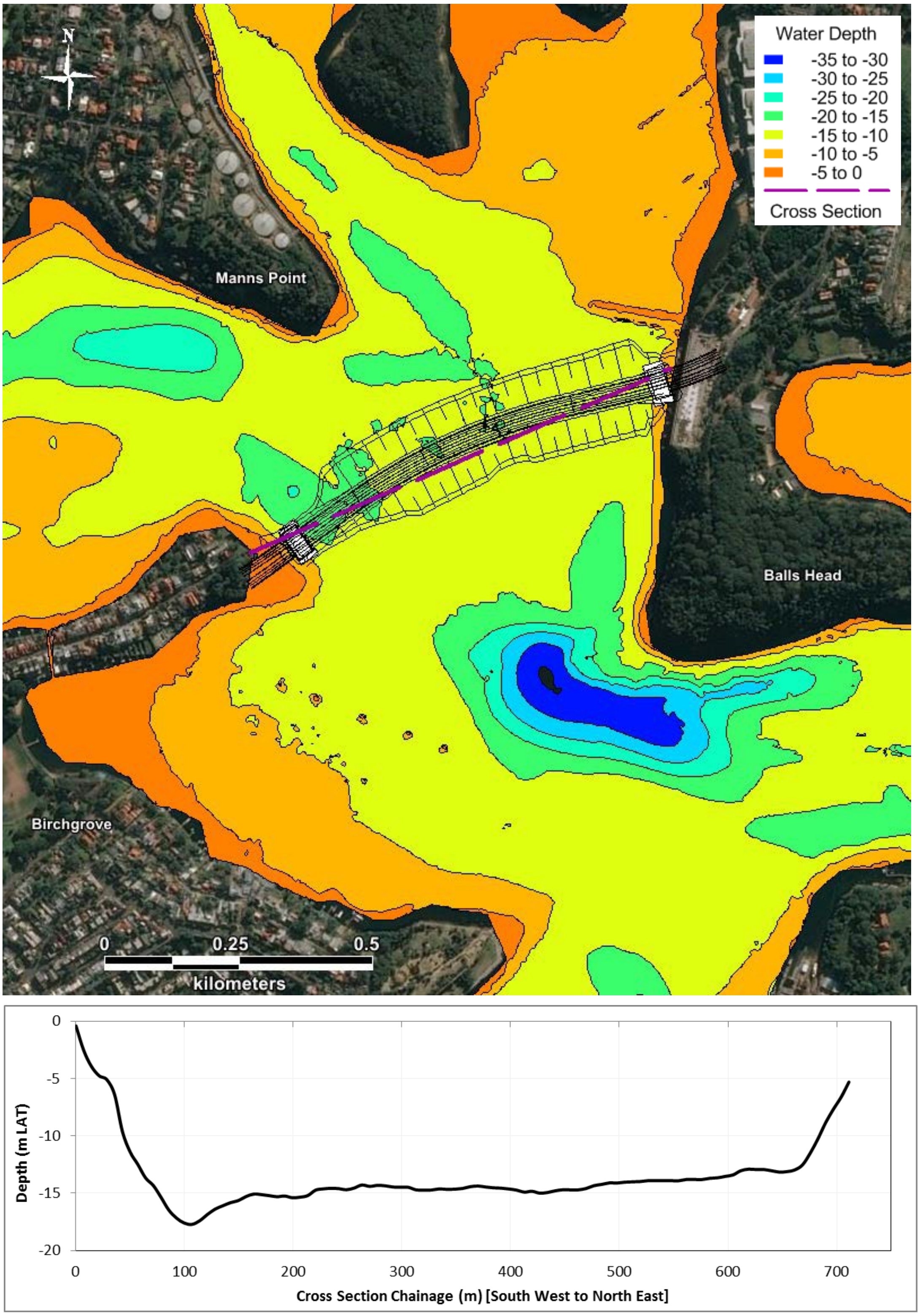


Figure 3-1: Local bathymetry at the project crossing (data source: Roads and Maritime).

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, ebb and flood tidal discharges are the dominant cause of water movement in the harbour.

Tides are propagated through deeper regions and steered by the complex geometry of the harbour, with very little change in tidal amplitude and phase throughout Western Harbour.

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 for Sydney Harbour (Manly Hydraulics Laboratory 2016).

Tidal Plane	Camp Cove (33°50’, 151°17’) (m AHD)
HAT	1.15
MHWS	0.65
MHWN	0.40
MSL	0.03
MLWN	-0.36
MLWS	-0.61
LAT	-0.90

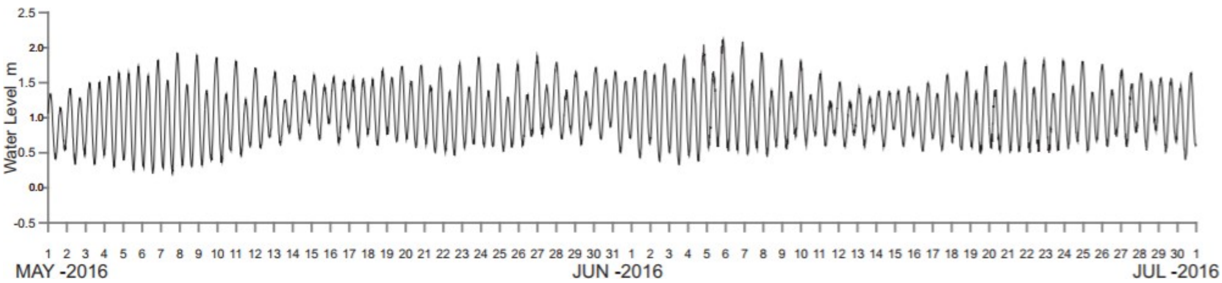


Figure 3-2: Sydney Harbour water level referenced to zero metres (ie Chart Datum) at Fort Denison (Manly Hydraulics Laboratory 2016).

3.3.2 Currents

A summary of current speed statistics taken from the WH1 and WH2 in situ monitoring locations is provided in Table 3–2. Additional information, including time series plot, current roses, U versus V[2] scatter plots and depth profile plots of the currents recorded at WH1 and WH2 are provided in Appendix 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 current coincides with high wind speeds.

Table 3–2: Summary of current speed statistics at RHDHV's two project monitoring sites.

Parameter	Statistic	WH1	WH2
Flood Current Speed (m/s)	Maximum	0.43	0.18
	95th percentile	0.27	0.08
	Mean	0.12	0.04
Ebb Current Speed (m/s)	Maximum	0.41	0.29
	95th percentile	0.26	0.12
	Mean	0.13	0.05

Figure 3-3 presents a current rose for depth averaged current data at the Balls Head location (see Figure 2-2). The current rose highlights the strength of the tidal stream within the estuary as current direction is split along a south-east/west (ebb/flood) axis indicative of the horizontal water movement during ebb and flood tides. The maximum depth averaged current speed recorded in the six month data set obtained from the Port Authority of NSW was 0.73 m/s, the 90th percentile speed was 0.37 m/s and the 50th percentile speed was 0.18 m/s. The 90th percentile surface current speed in the six month data set was 0.4 m/s.

The spatial (two-dimensional (2D) depth averaged) and vertical patterns in tidal currents are shown in Figure 3-4 and Figure 3-5 for peak flood and ebb tidal stages, respectively. It is observed that spatial current patterns in Western Harbour are influenced by the complex shape of the harbour with stronger tidal streams in the main channels, weaker currents outside the main channels along with circulating eddies in some of the embayments (eg Balls Head Bay). It is further observed that spatial measurements, similar to the in situ monitoring, showed little change in speed with depth.

[2] *U and V are the east-west and north-south velocity components, respectively.*

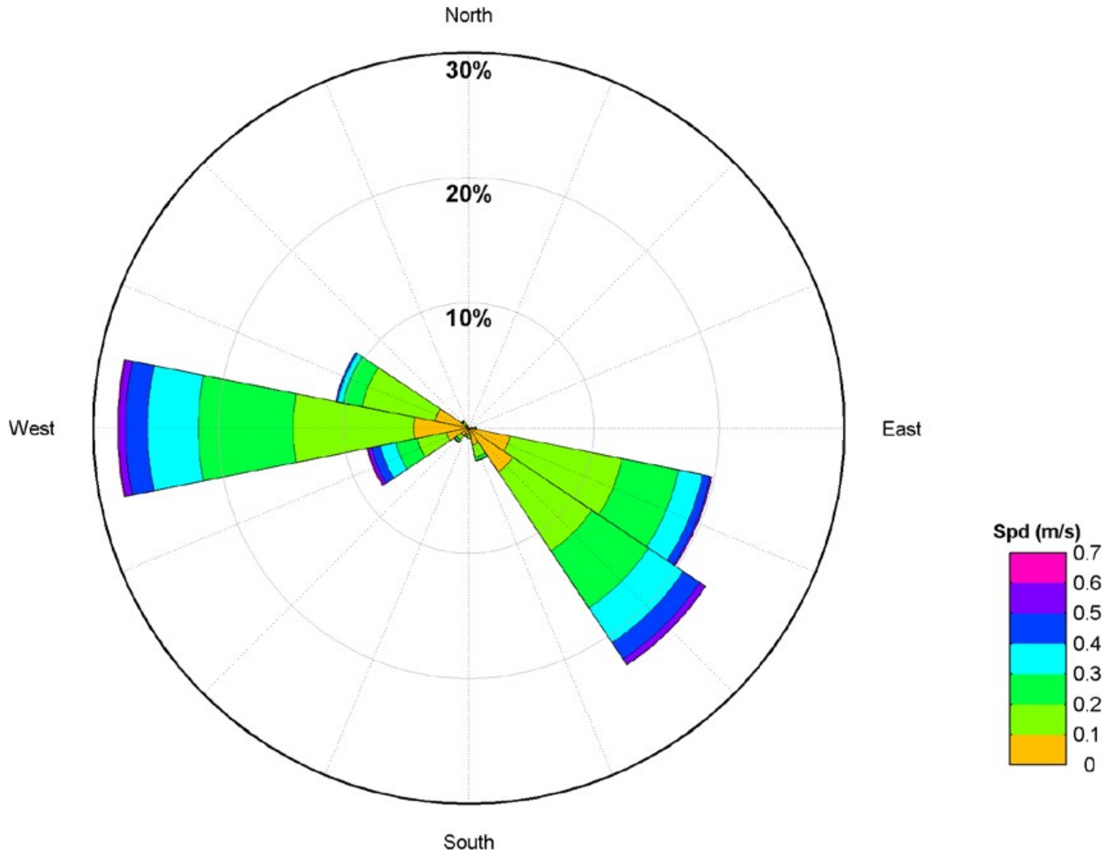


Figure 3-3: Current rose derived from Balls Head current data from 1 June 2017 – 5 December 2017 (data source: Port Authority of NSW).

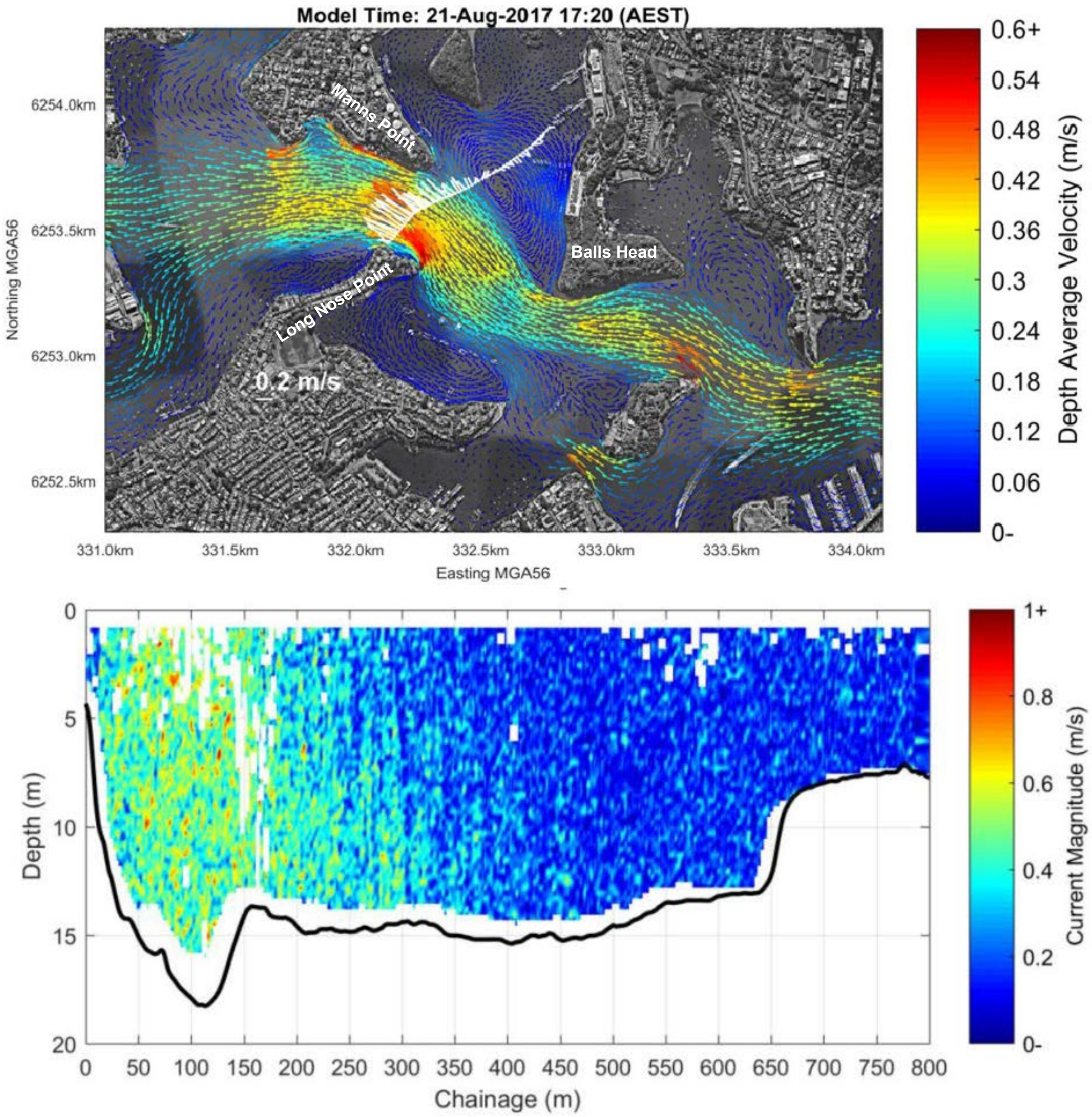


Figure 3-4: Spatial flood tidal current patterns based on measured and modelling data (top) and measured flood current speeds with depth along transect WH-Ves3 (bottom).

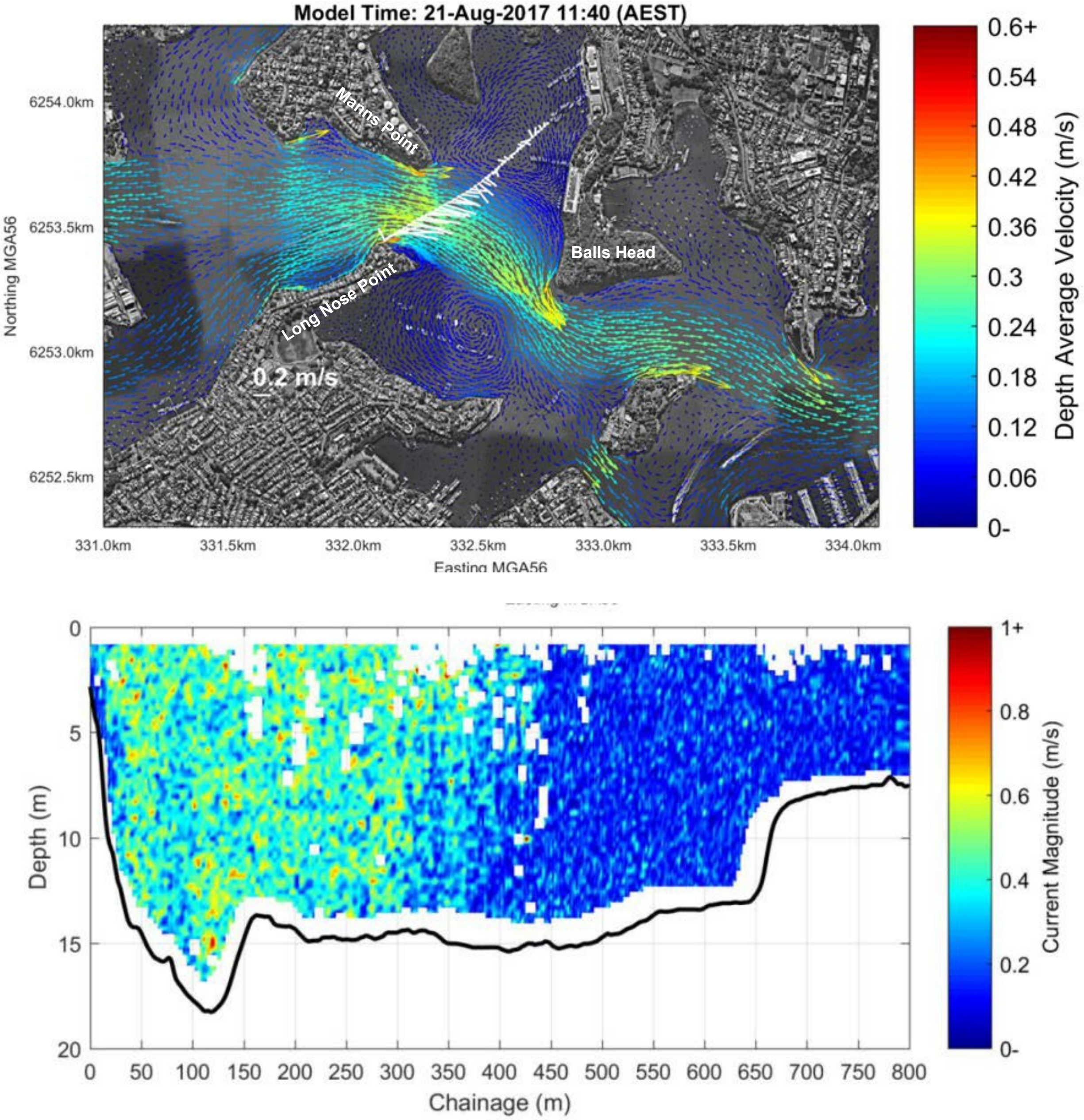


Figure 3-5: Spatial ebb tidal current patterns based on measured and modelling data (top) and measured ebb current speeds with depth along transect WH-Ves3 (bottom).

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 project crossing compared to other available data sources.

Table 3–3 details wind statistics based on monthly percentiles and Figure 3-6 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 tend 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

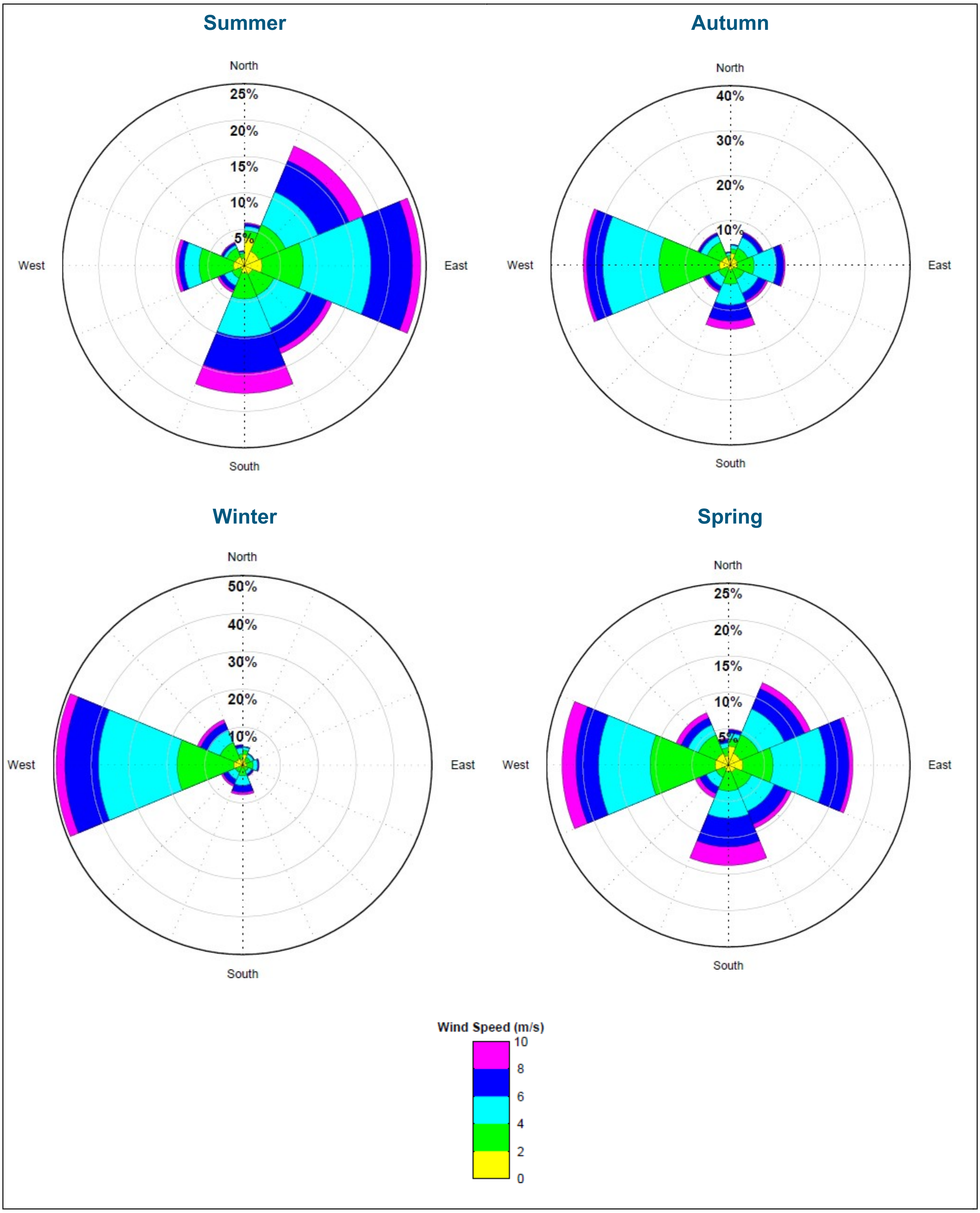


Figure 3-6: Seasonal wind roses derived from data at the Fort Denison weather station.

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 (the latter probably being the greatest energy inside the harbour, particularly within the Western Harbour region where ferries and other vessels are common). The wave climate at the project crossing location 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. Wave periods associated with Rivercat and Harbourcat ferries can exceed four seconds depending on vessel speed.

The bathymetry within the vicinity of the project crossing is relatively deep meaning that the potential effect of waves (either wind waves or boat wakes) on hydrodynamic or sediment plumes at the seabed is significantly reduced.

3.4 Rainfall and freshwater inputs

The mean annual rainfall observed at Observatory Hill, Sydney is 1215 millimetres. Figure 3-7 presents the mean monthly precipitation observed at Sydney's Observatory Hill. The data shows that rainfall is relatively evenly spread throughout the year with only 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 varies significantly from both year-to-year and month-to-month time scales. Much of the variability in precipitation is due to large-scale climate variations, with El Niño – Southern Oscillation being the most important (BoM, 2015).

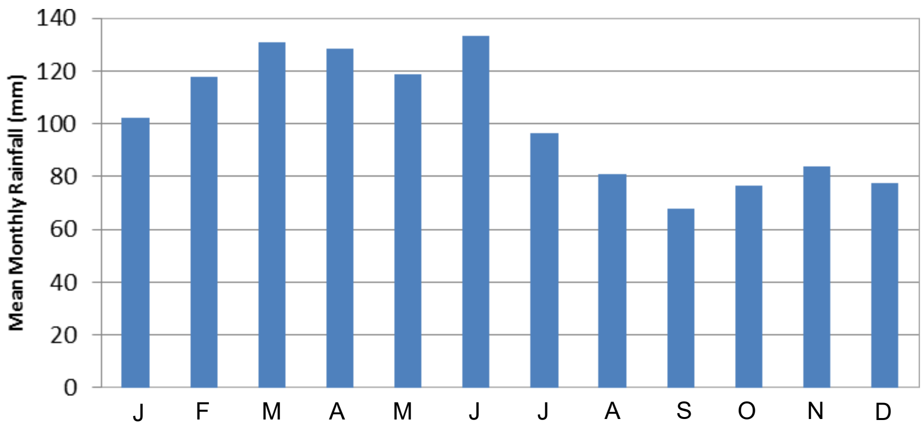


Figure 3-7: Mean monthly rainfall observed at Sydney's Observatory Hill (1859-2017).

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.

During dry weather conditions (rainfall less than one millimetre per day) freshwater discharge from the Parramatta and Lane Cove catchment is minimal and is estimated to be less than 0.1 m^3/s from both catchments (Rochford, 2008; Birch and Rochford, 2010).

A basic empirical calculation using the rational method was used to estimate discharge from the Parramatta and Lane Cove catchments during a design rainfall event. Assuming a 50 millimetre per day rain event (which is considered a relatively rare rainfall event for Sydney) an estimated freshwater discharge from the Parramatta and Lane Cove catchments of 142 m^3/s and 53 m^3/s is predicted.

Due to relatively 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 mobilise 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 upon 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, 2017). Turbidity data for the greater Port Jackson estuary is available from various sources and was reviewed in Cardno (2017). A summary of measured turbidity for the waters around Balls Head have been provided in Table 3–4 below.

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 nearby to the site in Darling Harbour. Generally, ambient turbidity in Darling Harbour is low (<5 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 3-8 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.

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

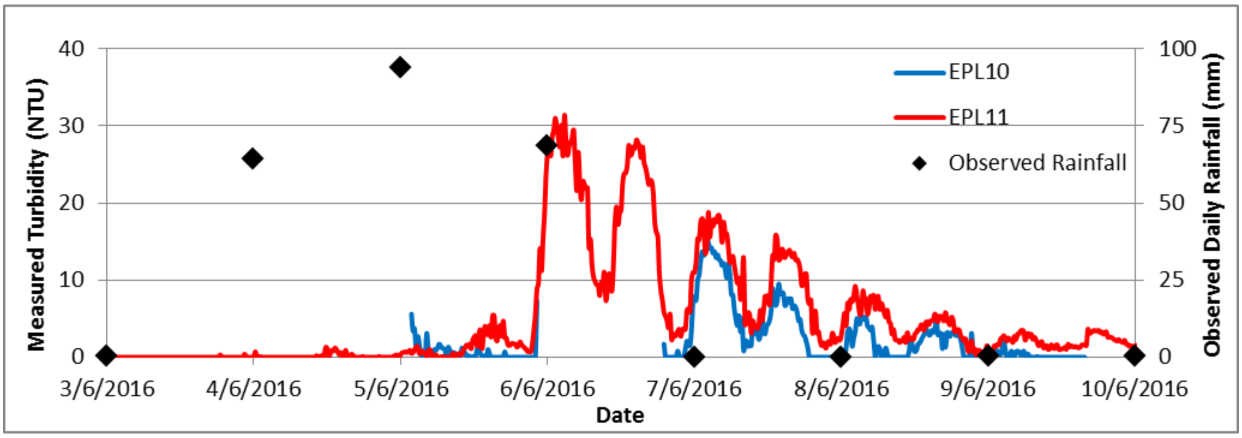


Figure 3-8: Turbidity observed at Darling Harbour during notable catchment rainfall event (data source: water quality data collected by RHDHV at Barangaroo).

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, 2017).

4 Model setup and configuration

4.1 Overview

The existing RHDHV MIKE 21 Flexible Mesh hydrodynamic model of Port Jackson has been updated and refined for this study (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 has been developed further. It has been 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 has been 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 projects. 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 project crossing location) while the resolution in other areas can be coarser to help maintain acceptable model run times.

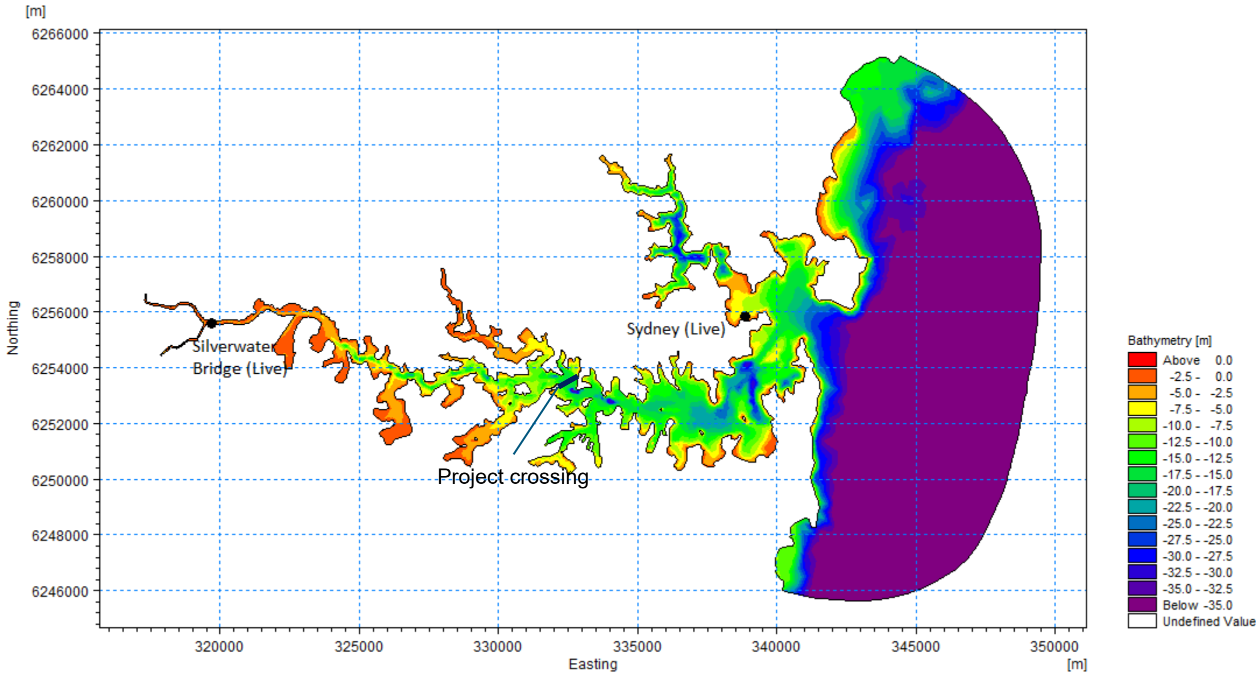


Figure 4-1: Extent of the RHDHV Port Jackson MIKE 21 model domain including bathymetry and location of water level calibration sites.

4.2 Hydrodynamic model

4.2.1 Model bathymetry

The model bathymetry has been defined based on measured data supplied by Roads and Maritime 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 4-2). In addition, the model mesh was refined to ensure adequate representation of the project design including the dredge footprint and temporary cofferdams structures (eg Figure 4-3). The project design was implemented based on drawings (provided as AutoCAD files) of the immersed tube tunnel layouts (received from Roads and Maritime on 22 December 2017). 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
Offshore	450 m
Sydney Harbour Entrance	250 m
Sydney Harbour	140 m
Middle Harbour	70 m
Parramatta River	250 m
Project crossing	30 m

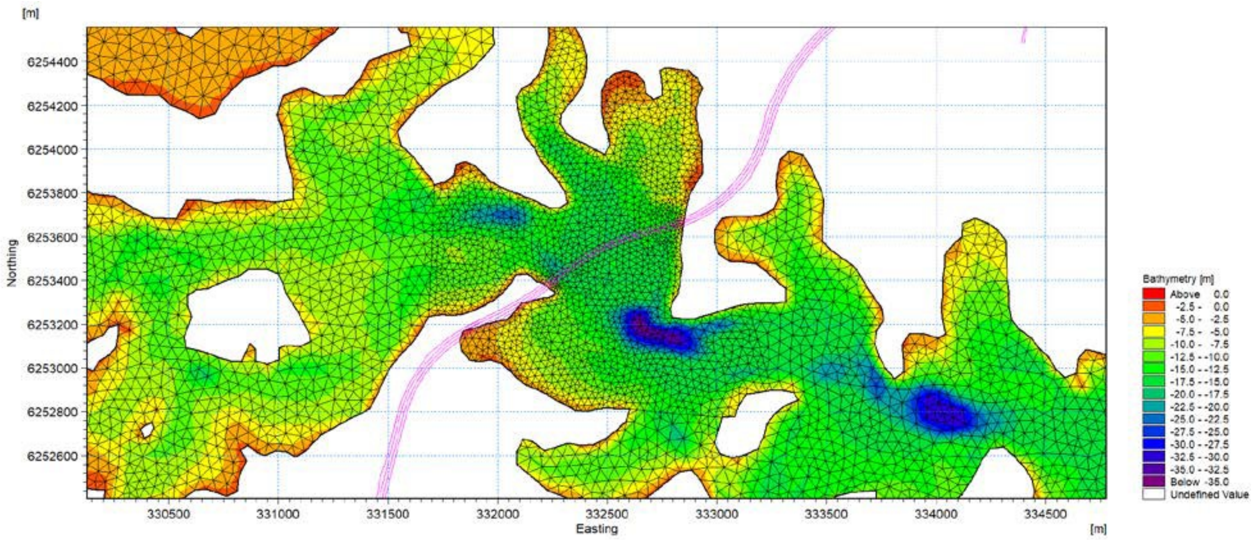


Figure 4-2: Model mesh and bathymetry at the project alignment (pink polylines).

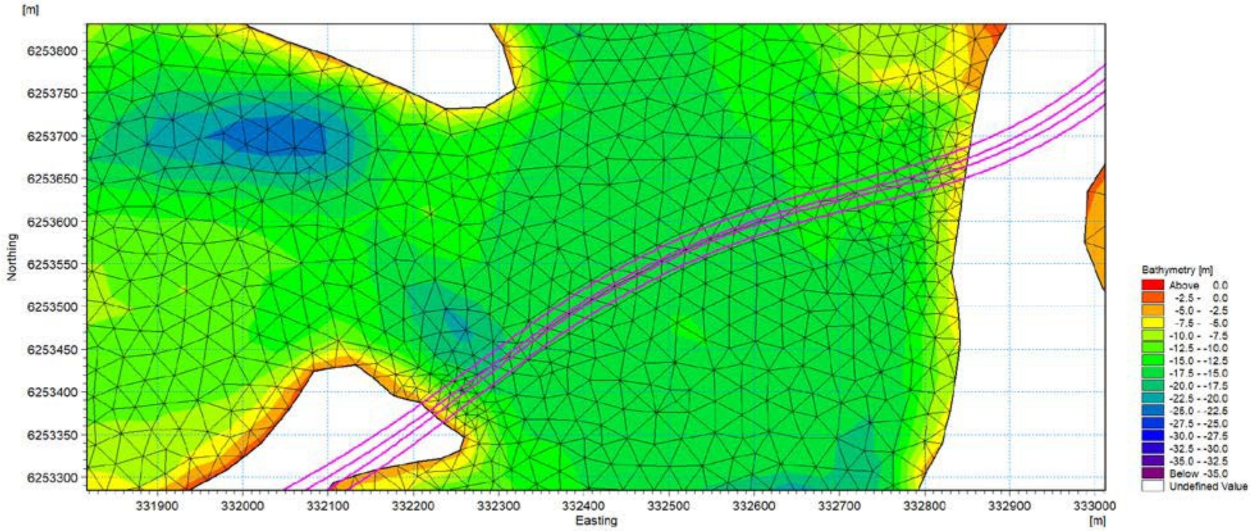


Figure 4-3: Model mesh at the project location (the dashed pink polyline represents the proposed tunnel alignment).

4.2.3 Model boundaries

To define suitable hydrodynamic model boundary conditions, the measured water level data from the Sydney (Live) tide gauge has been 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 model simulation period. A small phase shift was then applied to account for the distance between the model 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) by specifying 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.

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 provides continuous measurements of water level and current velocity which have been used for model calibration. In addition to the two project specific in situ ADCP monitoring sites, the Port Authority of NSW's Balls Head monitoring site has been used for model calibration. The vessel mounted ADCP transects that were carried out at three locations (ie WH_Ves1, WH_Ves2, and WH_Ves3) to measure spatial patterns in current velocities and subsequently calculate discharge have also been used. Figure 2-2 displays the in situ monitoring sites and ADCP transect locations for the Western Harbour region.

Tidal variation is the governing physical process for the hydrodynamics of Port Jackson. The model calibration has thus 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 measure 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 have been 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 per cent of measured level (spring tide), ±15 per cent of measured level (neap tide)
Water level phase	Timing of high/low water to with ±15 min at the mouth of the estuary or ±25 min at the head of the estuary
Average current speed	±20 per cent 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° of measured direction
Discharge (Q m^3/s)	±5 per cent (very good), ±10 per cent (good), ±15 per cent (moderate) or >15 per cent (poor) of measured flows

The statistical standards provided in Table 5–1 are a good basis for assessing model performance, but 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 standards. 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 has also included comparison to spatial current patterns which are not specifically mentioned in Table 5–1. 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 has been calibrated against measured water level, current speed and direction as well as ADCP data transect and discharge data. The model calibration has been carried out over a 35 day period from 16 August 2017 to 20 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 Sydney Harbour.

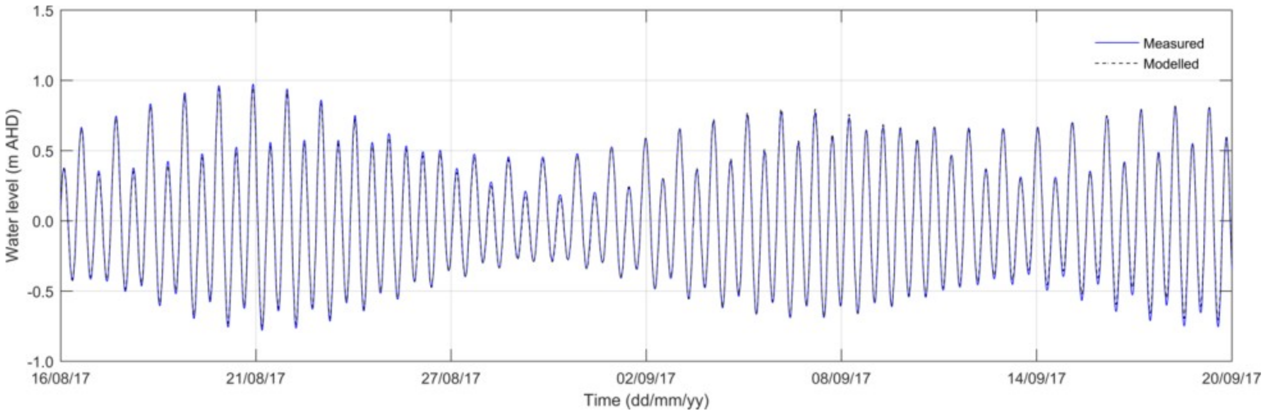


Figure 5-1: Measured and modelled water levels at the Sydney (Live) tide gauge.

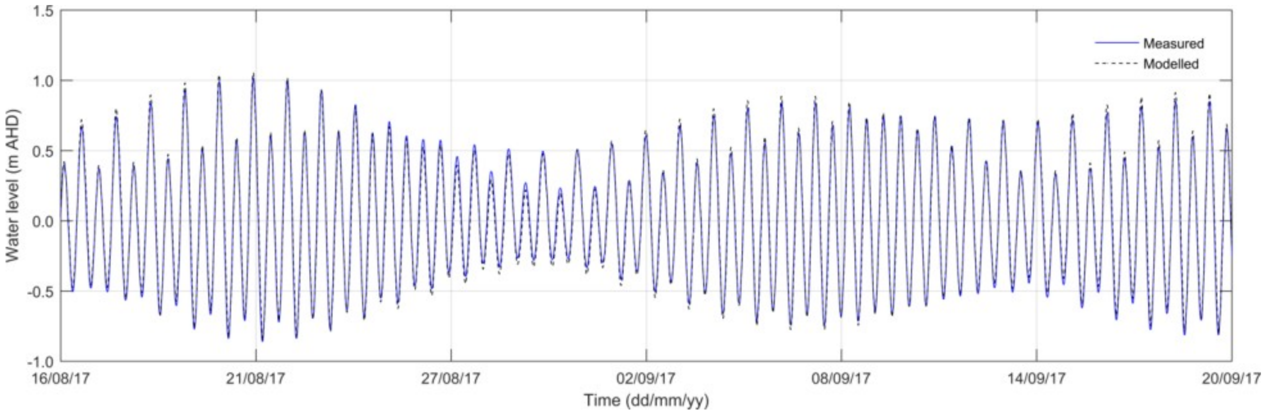


Figure 5-2: Measured and modelled water levels at the Silverwater Bridge tide gauge.

Table 5–2: Water level calibration statistics.

Statistical Description	Sydney Live	Silverwater Bridge
Mean High Water Difference (m)	0.003	-0.005
Mean High Water Difference relative to Tidal Range (%)	0.3	-1.4
Mean Low Water Difference (m)	0.007	-0.033
Mean Low Water Difference relative to Tidal Range (%)	0.6	-0.3
RMSE for High Water (m)	0.02	0.03
RMSE for Low Water (m)	0.02	0.04
Mean High Water Phase Lag (mins)	-0.04	14.0
Mean Low Water Phase Lag (mins)	1.8	-3.6

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 WH1 and WH2) underwent post-processing to determine 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. Water level measurements were not provided for the Balls Head ADCP and have not been included.

Table 5–3: Water level calibration statistics at the WH1 and WH2 in situ monitoring sites.

Statistical Description	WH1	WH2
Mean High Water Difference (m)	0.01	0.13
Mean High Water Difference relative to Tidal Range (%	1.0	-14.0
Mean Low Water Difference (m)	0.01	-0.1
Mean Low Water Difference relative to Tidal Range (%)	1.2	-10.7
RMSE for High Water (m)	0.06	0.14
RMSE for Low Water (m)	0.06	0.10
Mean High Water Phase Lag (mins)	-8.0	-35.5
Mean Low Water Phase Lag (mins)	-6.0	-42.1

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 WH1 with phasing less than eight minutes and a 1.2 per cent difference in tidal range
- The modelled water level at WH2 provides a reasonable representation of the measured water level with only a 14 per cent difference in tidal range. The difference in the measured and modelled tidal phase is outside the recommended standard for variation in water level phasing (15–25 min). These differences in water level and phase are suspected to be a result of a malfunctioning pressure sensor on the ADCP deployed at WH2. Further commentary on the suspected malfunction of the pressure sensor has been included in the data collection report (RHDHV, 2017b).

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 has undergone harmonic analysis. The harmonic analysis allows 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. The modelled and measured current speeds and directions at the ADCP deployment locations WH1, WH2 and Balls Head are shown in Figure 5-3, Figure 5-4 and Figure 5-5 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.

- The average difference between modelled and measured peak current speed at WH1 indicates very good model performance (<0.05 m/s). The comparison does identify some variability between various stages of the tide. The mean difference for ebb and flood speeds ranged from 0.0005 m/s to 0.03 m/s, respectively, indicating the model better predicted current speed better during the ebb tide. The root mean square error (RMSE) for flood tide direction is 9.7°, while the RMSE for ebb tide direction which is 15.4°, which is only slightly outside the recommended calibration standard of <15°. The Port Jackson estuary has a complex planform shape, with large gradients in the direction of the main tidal streams. This makes the calibration of modelled flow directions more difficult when compared to the other estuaries and justifies the relaxation of this particular calibration standard. The slight difference in modelled current direction is not expected to have a significant impact on the results of the impact assessment
- At WH2, the measured data indicates the current speeds are considerably weaker than measurements made at WH1. This is due to the position of WH2 within Balls Head Bay, outside the primary tidal stream. The average difference between modelled and measured peak current speed at WH2 indicates very good model performance (<0.01 m/s). The mean difference between measured and modelled speeds was 0.01 m/s for both ebb and flood tides. The RMSE for both flood and ebb tide direction is 12° and 10°, which is within the recommended calibration standard of <15°. It is worth noting that tidal currents within Balls Head Bay are relatively weak and the spatial distribution of currents considered relatively complex as a result of tidal eddies circulating within the bay. Given the weak current speeds and complex circulation patterns present, the model is considered to realistically represent tidal currents within Balls Head Bay
- The Balls Head ADCP measured the fastest currents of all three sites given its location within the tidal stream and position at the tip of Balls Head. The mean difference between modelled and measured peak current speed at Balls Head indicates good model performance (<0.04 m/s). The mean differences in the speed of flood and ebb tides were less than 0.07 m/s (representing good model performance) and percentage difference up to 14 per cent which falls within the calibration standards (±15 per cent). The RMSE for both flood and ebb tide direction is 12.5° and 7.5°, which is within the recommended calibration standard of <15°.

In summary, the modelled currents for all three calibration sites closely matched the measured ADCP data. The differences in the measured and modelled data all fell within the calibration standards for both speed and direction (noted in Table 5–1), with the exception of the ebb current direction observed at WH1 (difference of 15.4 per cent which is only slightly outside the recommended calibration standard of <15°).

Table 5–4: Current speed and direction calibration statistics at the WH1 and WH2 monitoring sites.

Statistical Description	WH1	WH2	Balls Head
Mean Difference in Speed of Flood (m/s)	0.03	0.01	-0.04
Mean Difference in Flood Speed Relative to Maximum Observed Speed (%)	9.82	11.5	-6.7
Mean Difference in Speed of Ebb (m/s)	-0.0005	0.01	-0.07
Mean Difference in Ebb Speed Relative to Maximum Observed Speed (%)	-0.15	10.1	-14.0
RMSE for Flood Speed (m/s)	0.04	0.01	0.06
RMSE for Ebb Speed (m/s)	0.02	0.02	0.09
RMSE for Direction of Flood (°)	15.4	12.9	12.5
RMSE for Direction of Ebb (°)	9.7	10	7.5

Note: 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.

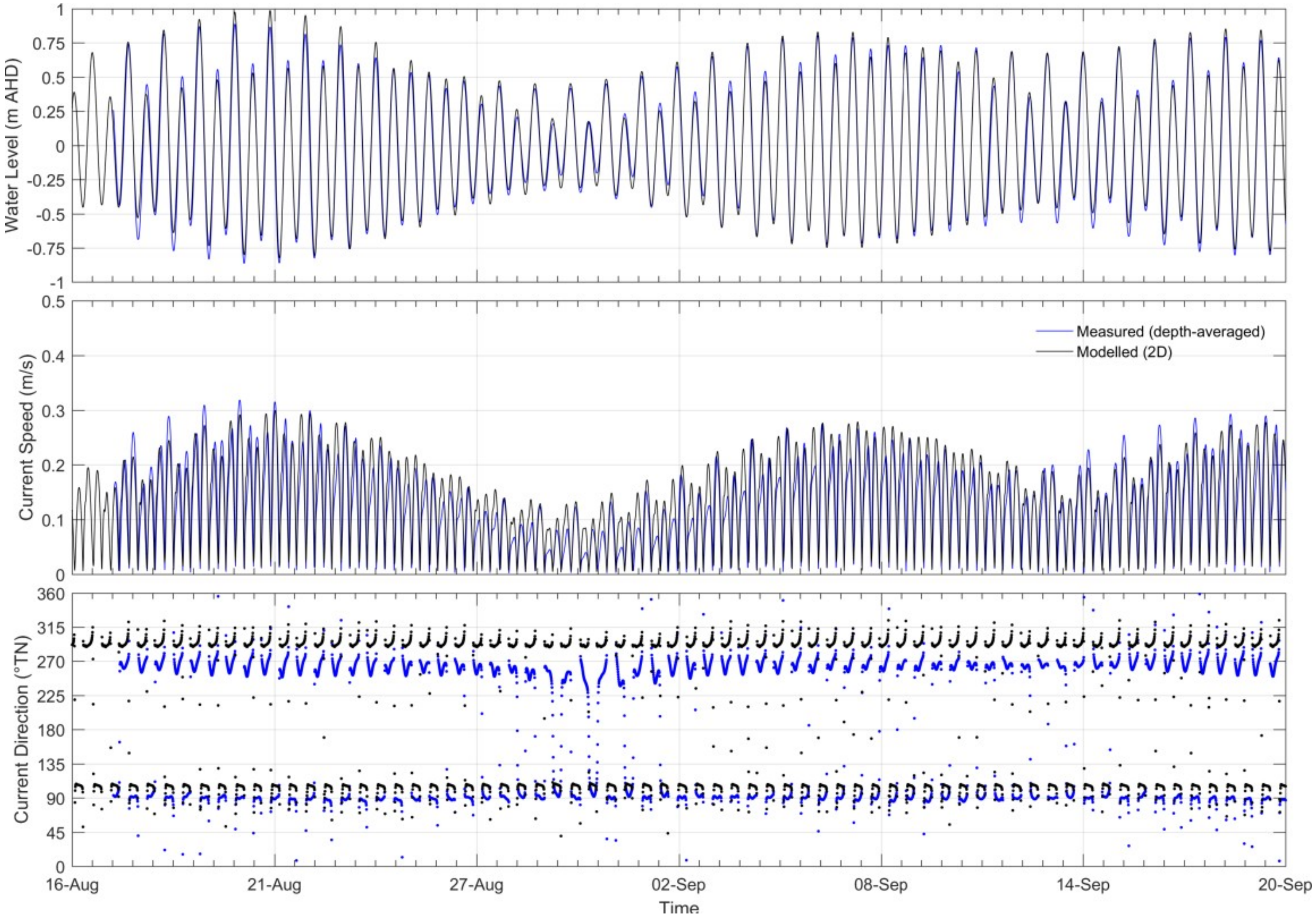


Figure 5-3: Measured and modelled water levels, current speed and direction at WH1.

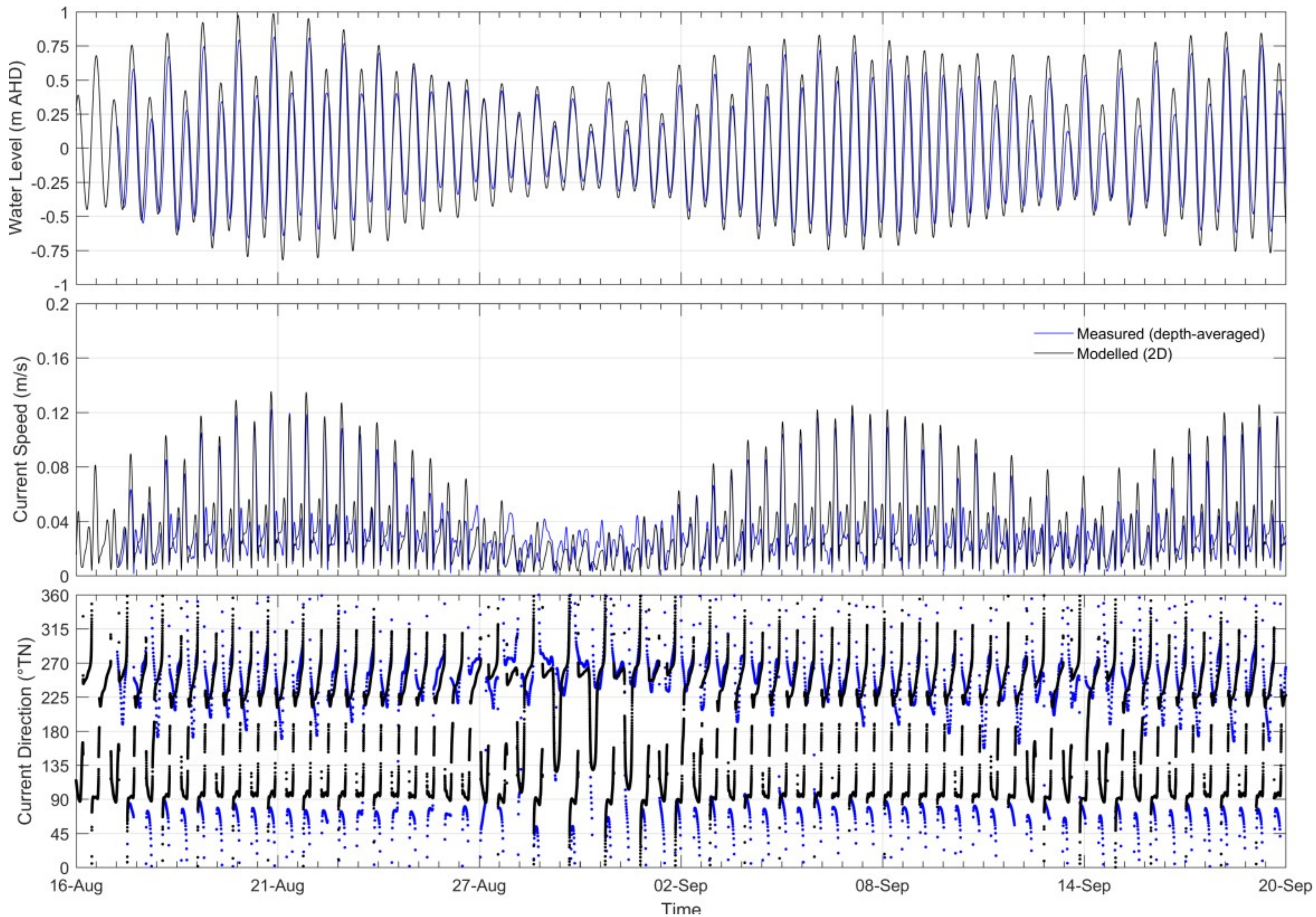


Figure 5-4: Measured and modelled water levels, current speed and direction at WH2.

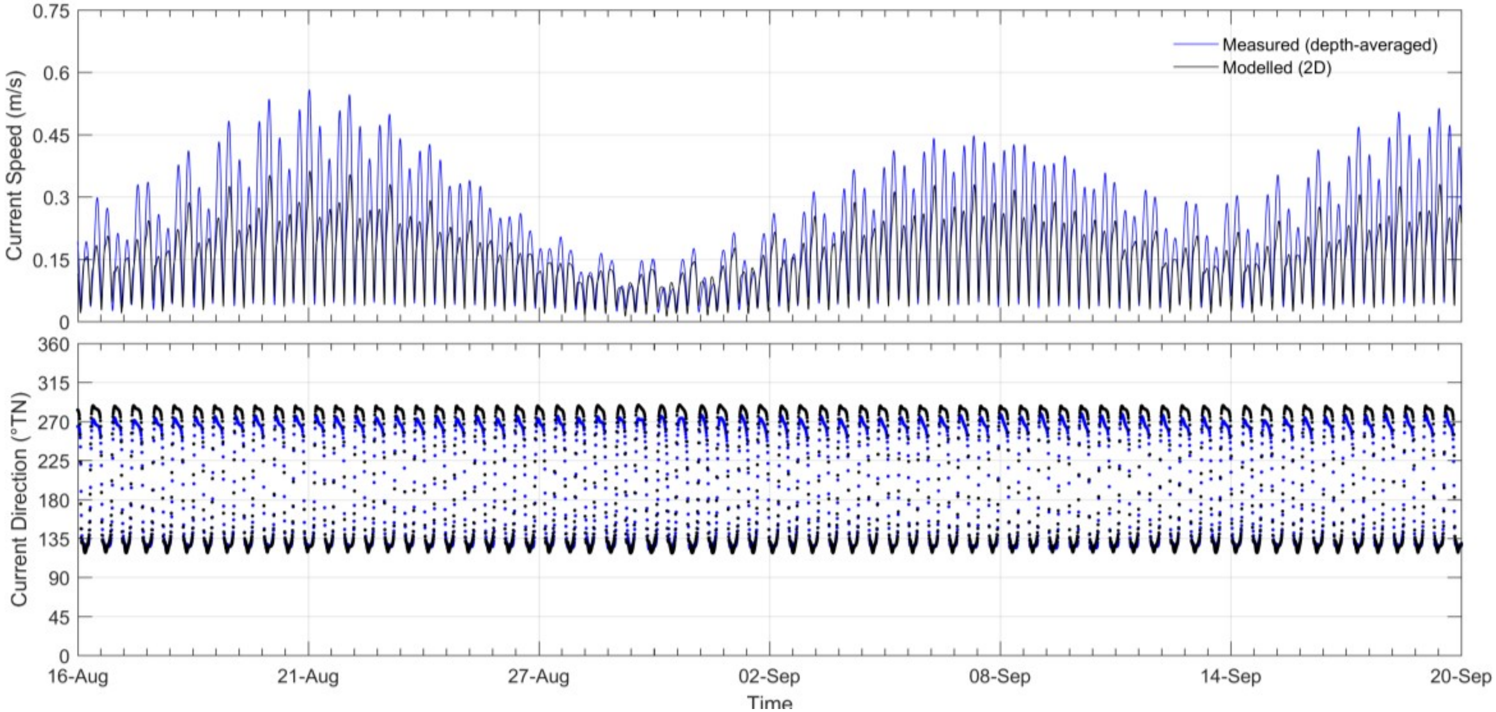


Figure 5-5: Measured and modelled current speed and direction at Balls Head.

Discharges and Velocity Transects (ADCP vessel transects)

In addition to in situ monitoring, three predefined ADCP vessel transects were carried out eight times throughout the tidal cycle on 21 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 WH_Ves1 and WH_Ves3 on 21 August 2017 are shown in Figure 5-6 and Figure 5-7. The modelled discharge (blue dots), agrees well with the measured discharge at WH_Ves1 and WH_Ves3.

Figure 5-8 to Figure 5-11 display the measured and modelled current speed and direction at the vessel transects WH_Ves1 and WH_Ves3 during the ebb and flood, respectively.

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 the models accurate representation of the flood tide eddy within Balls Head Bay.

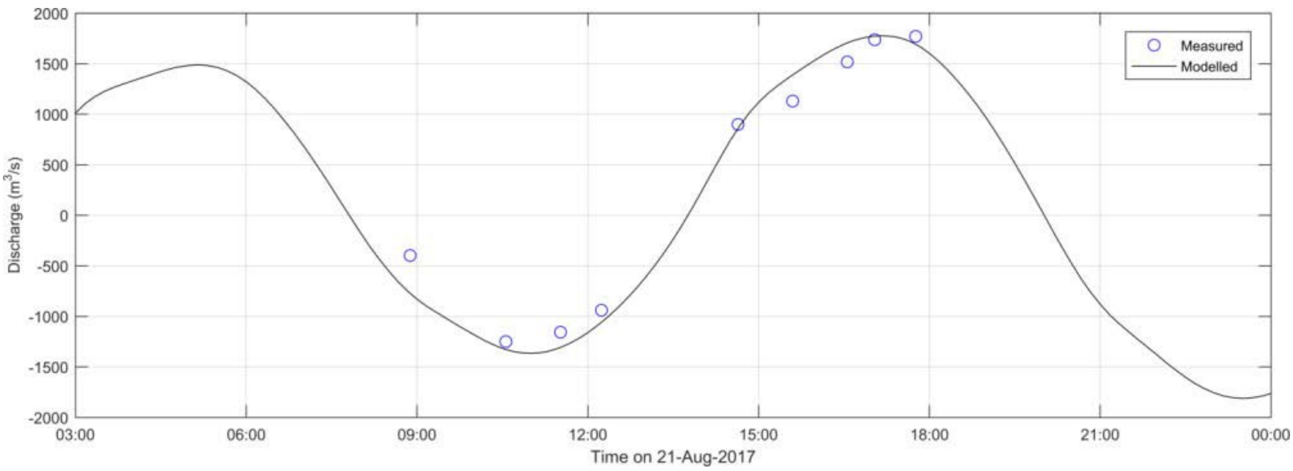


Figure 5-6: Measured and modelled discharge volumes at transect WH_Ves1.

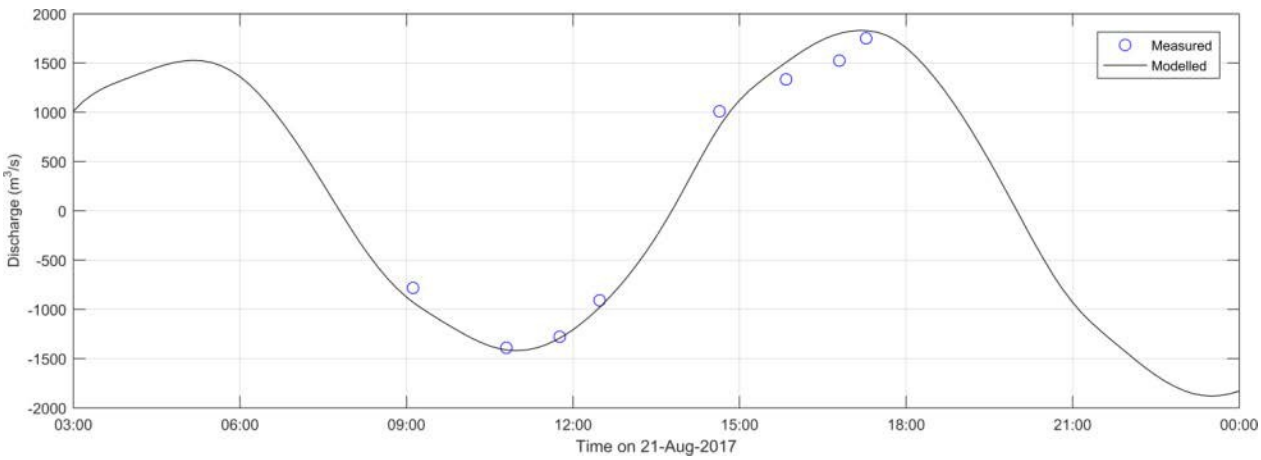


Figure 5-7: Measured and modelled discharge volumes at transect WH_Ves3.

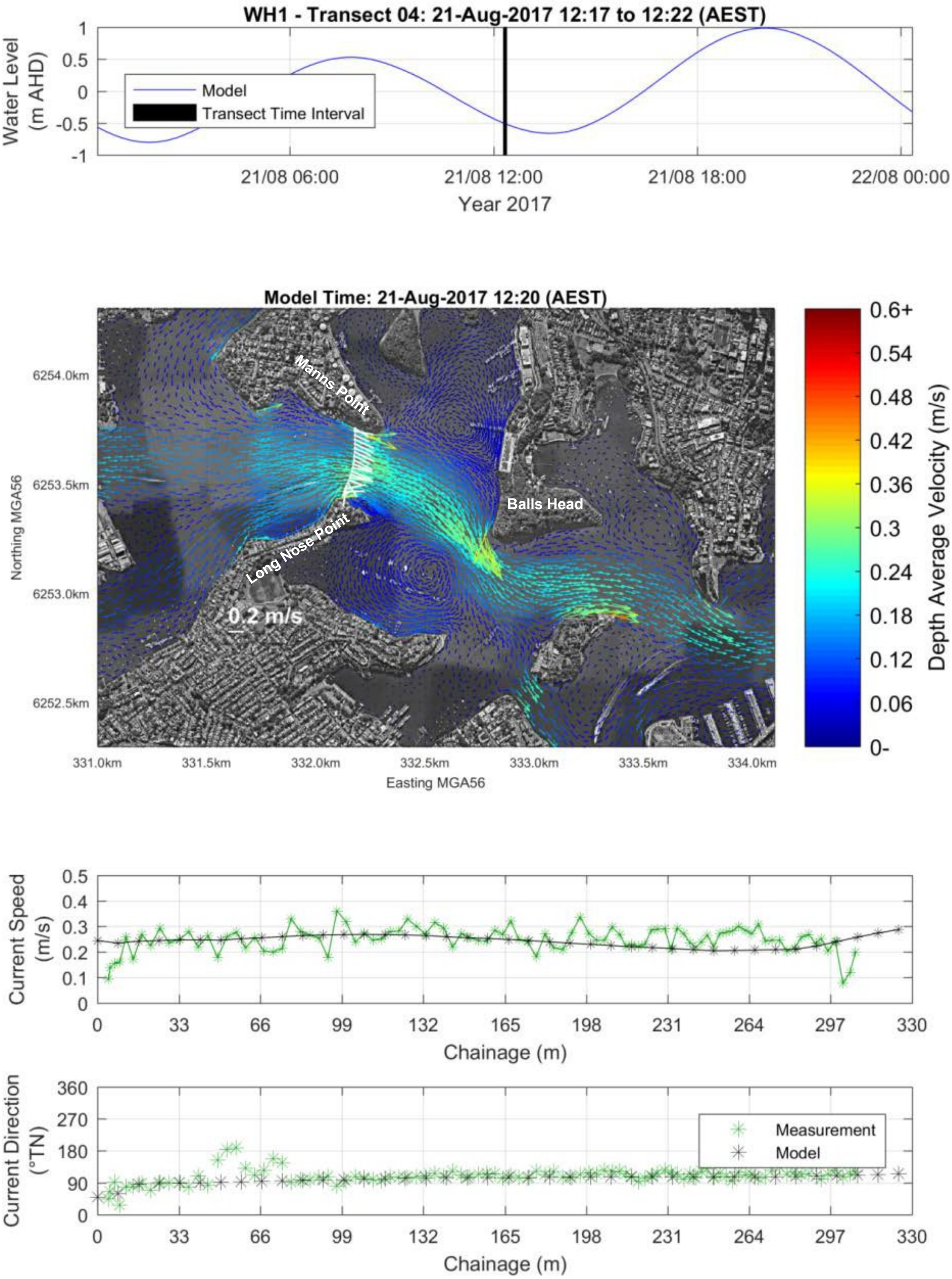


Figure 5-8: Measured and modelled current speed and direction at transect WH_Ves1 during the ebb tide.

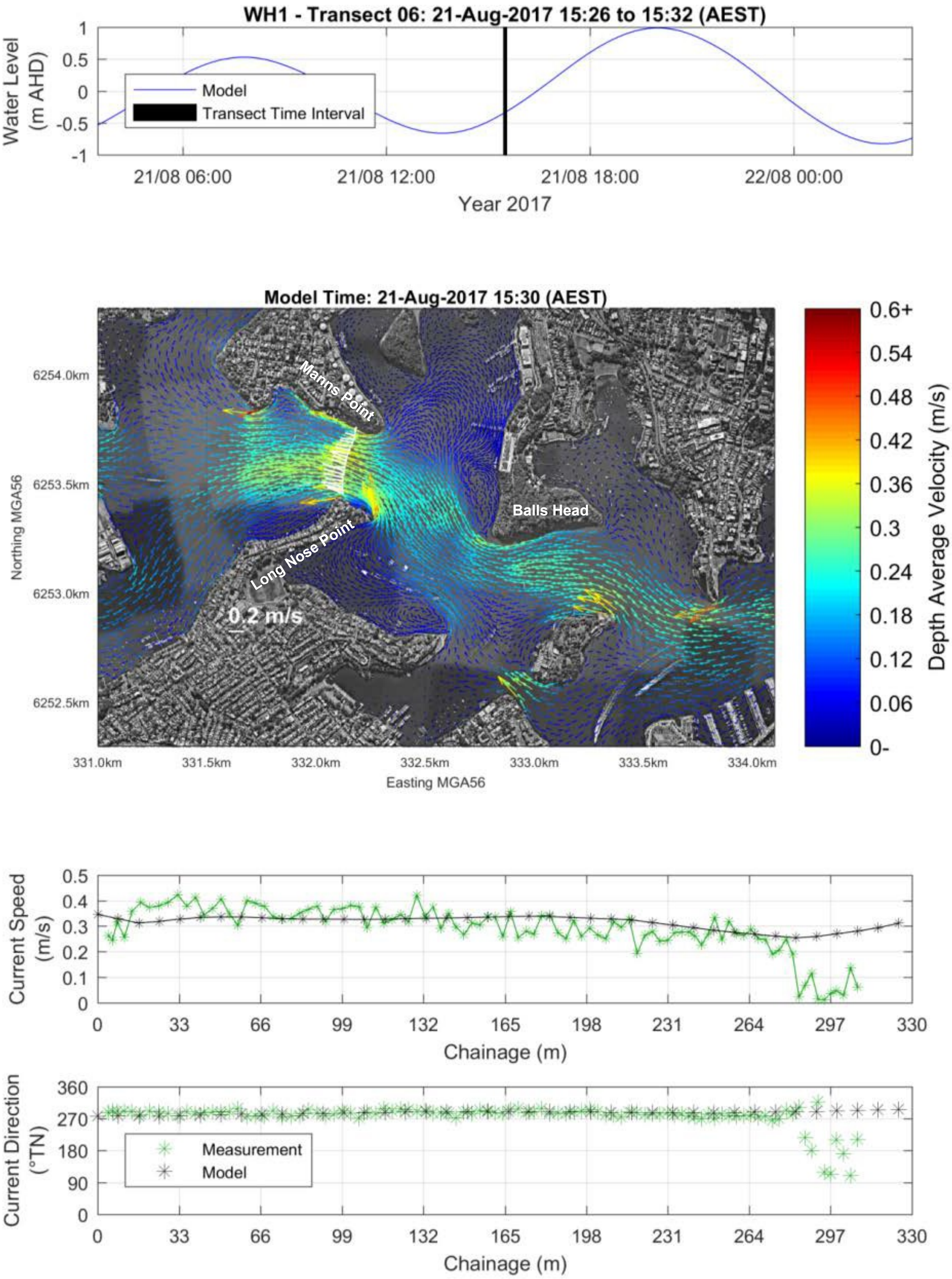


Figure 5-9: Measured and modelled current speed and direction at transect WH_Ves1 during the flood tide.

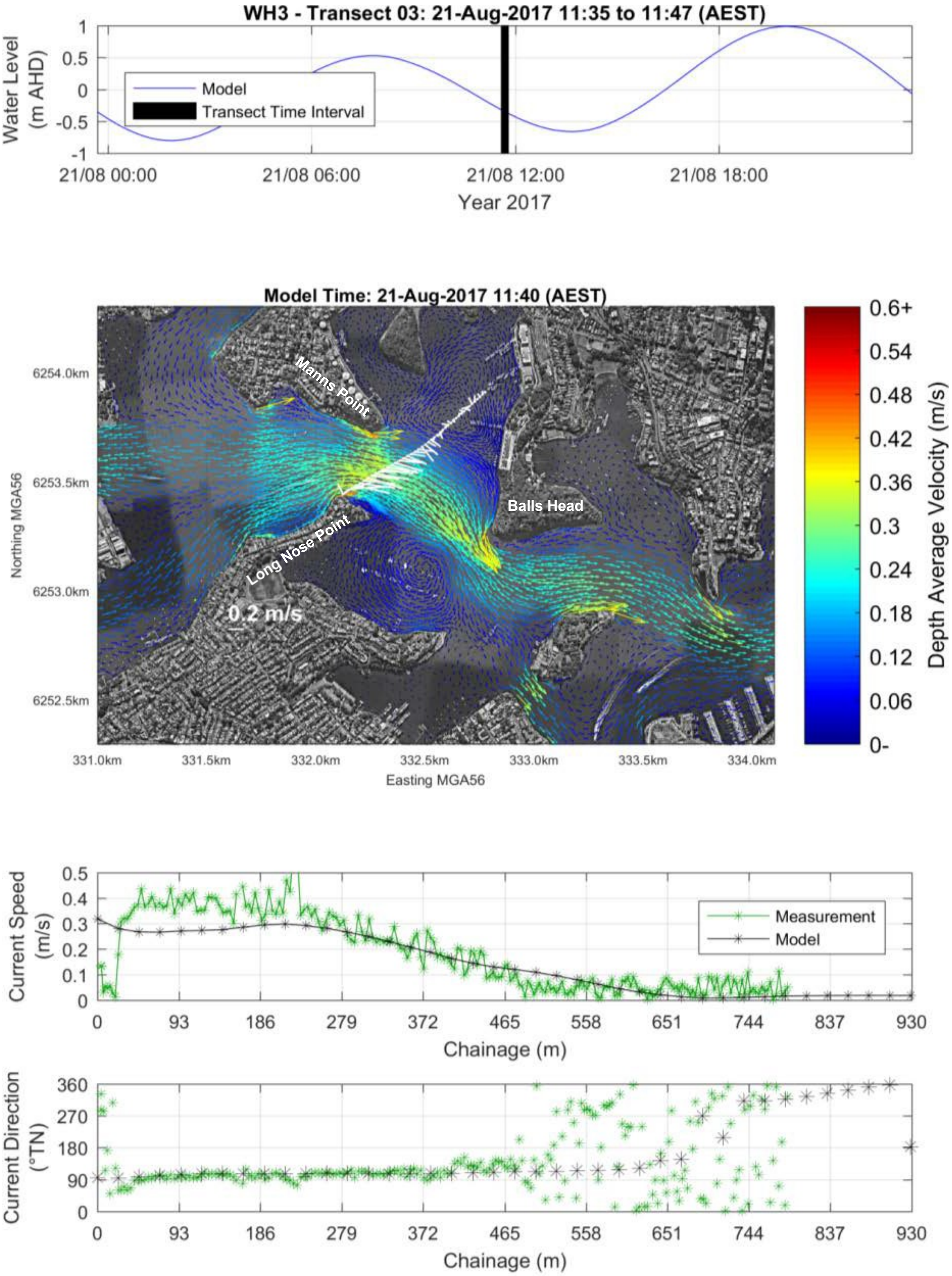


Figure 5-10: Measured and modelled current speed and direction at transect WH_Ves3 during the ebb tide.

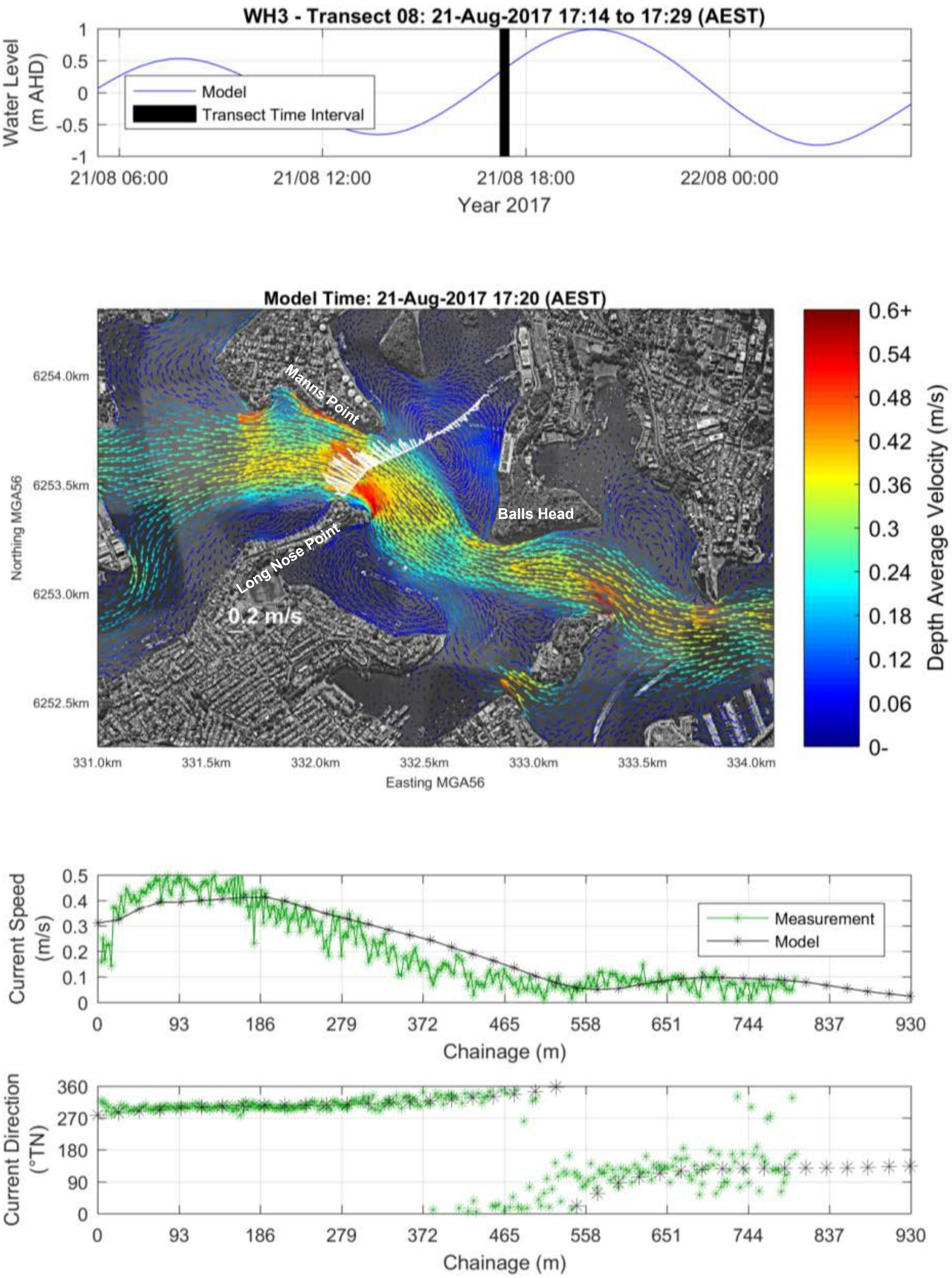


Figure 5-11: Measured and modelled current speed and direction at transect WH_Ves3 during the flood tide.

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 and validated against the in situ monitoring data at WH2**.** Figure 5-12 displays the measured and modelled current speed and direction throughout the water column (surface, middle and bottom) from 15 August 2017 to 21 November 2017.

Generally, there is good agreement observed in current speed and direction for bottom, middle and surface layers. The measured current speeds at the surface are slightly weaker and out of phase with those in the middle and bottom of the water column. This is likely due to wind forcing which was not included in this model simulation, but may influence the measured currents.

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).

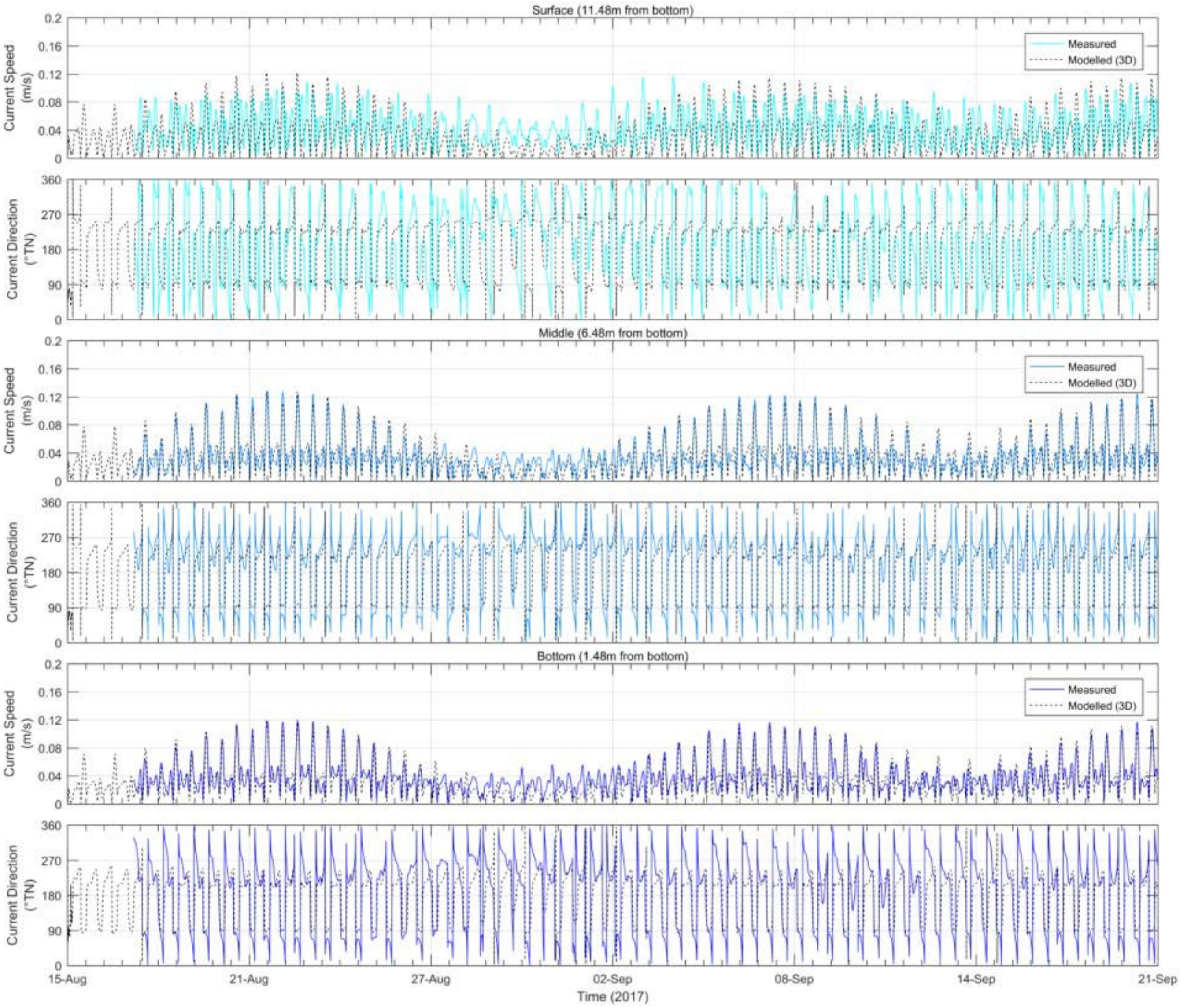


Figure 5-12: Measured and modelled current speed and direction at the surface, middle and bottom of the water column at WH2.

5.4 Wind sensitivity testing

The importance of the effect of wind on current circulation within the model has been assessed. Predominant wind directions (see Figure 3-6) 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. Figure 5-13 to Figure 5-16 displays the modelled current vectors during several of these sensitivity tests (north, east, south and west).

The testing indicated wind driven circulation was most pronounced when winds blow from the south-east to south-west, as well as from the north-east or north-west. The magnitude of the currents in the area of interest can be seen in the current pattern plots.

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 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 speed percentiles (see Table 5–5) and directional distribution. 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.

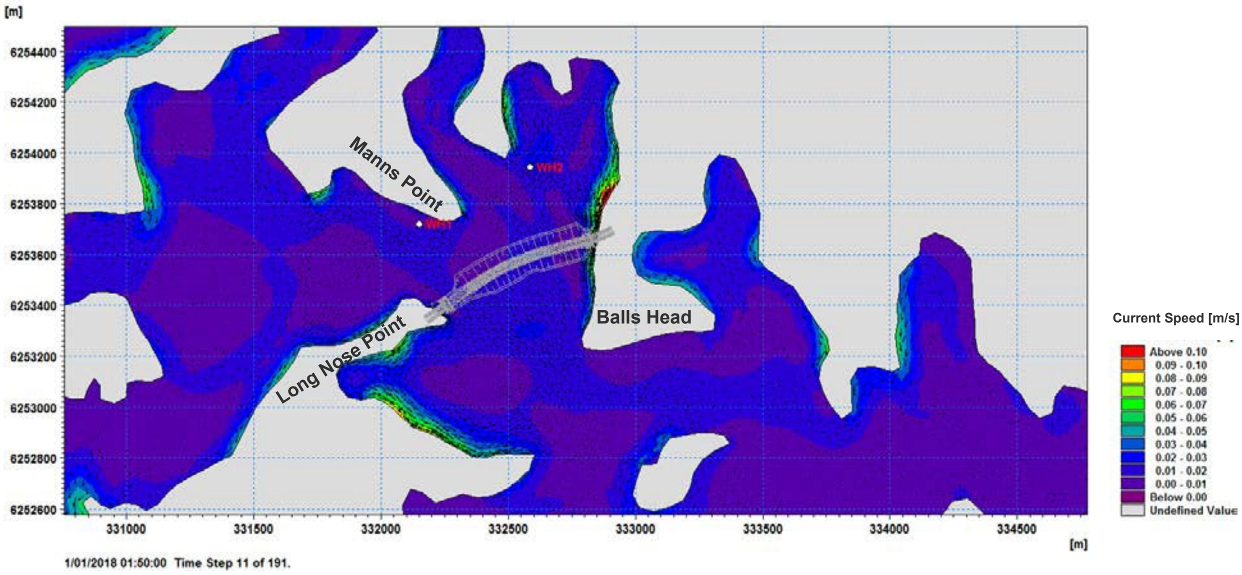


Figure 5-13: Modelled current vectors during sensitivity testing with static winds from the north at 10 m/s.

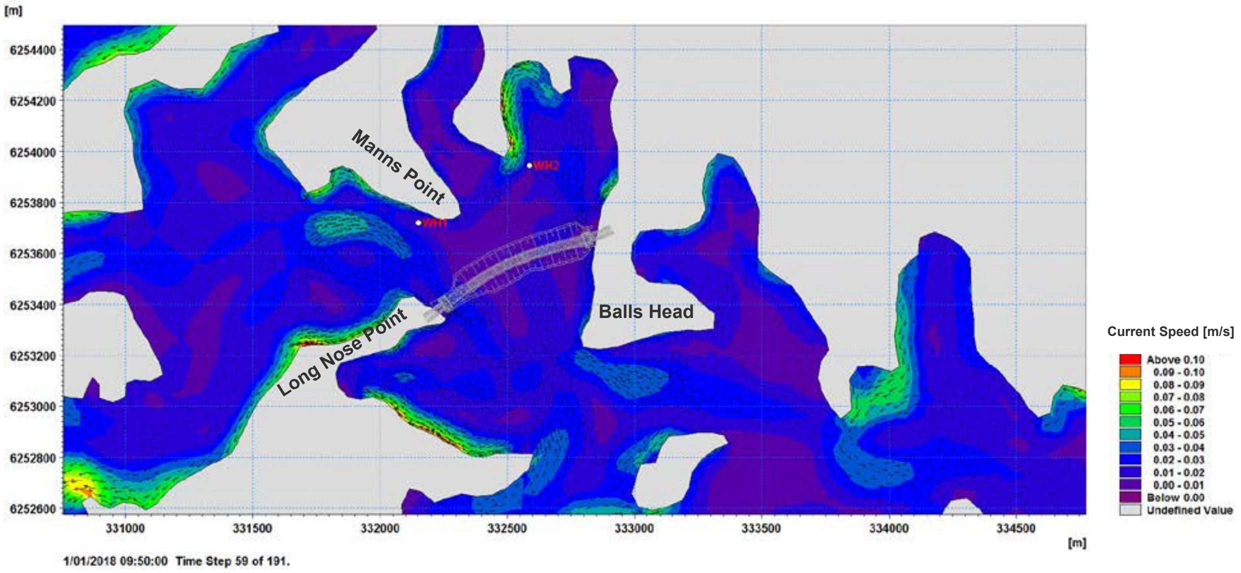


Figure 5-14: Modelled current vectors during sensitivity testing with static winds from the east at 10 m/s.

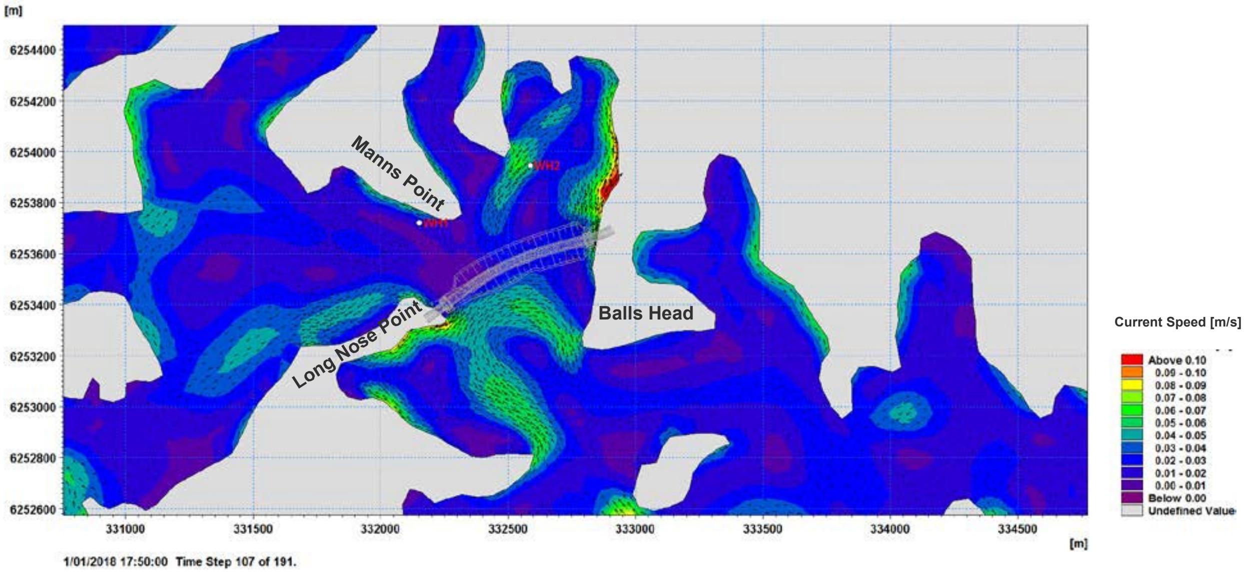


Figure 5-15: Modelled current vectors during sensitivity testing with static winds from the south at 10 m/s.

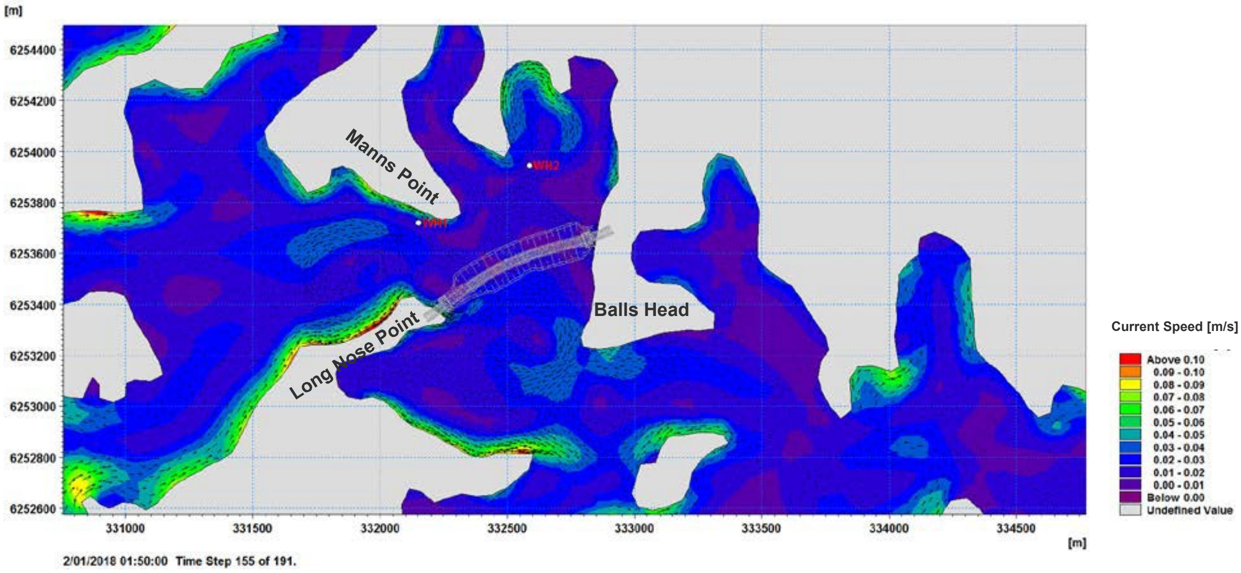


Figure 5-16: Modelled current vectors during sensitivity testing with static winds from the west at 10 m/s.

Table 5–5: Wind statistics for Fort Denison.

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 Hydrodynamic impacts

6.1 Construction impacts

6.1.1 Overview

The immersed tube tunnel would require the formation of trenches by capital dredging into the seabed for the sinking, connection and support of submerged tunnel elements between the terminal joints. Following connection of the immersed tube tunnel the dredged trench would be backfilled with the seabed, returning to pre-project conditions. Therefore, once the project is operational there would be minimal impact on hydrodynamics (see Section 6.2).

However, during the construction period two temporary cofferdams – Sydney Harbour south cofferdam and Sydney Harbour north cofferdam – would be required at the connections points on either side of the crossing (see Figure 6-1). Each cofferdam would be about 25 metres by 50 metres and would be constructed through the full depth of the water. The Long Nose Point (south-west) cofferdam is located in water depths of about 10 to 16 metres. The Balls Head (north-east) cofferdam is located in water depths of about seven to 13 metres. The cofferdams would be constructed using steel circular piles which act as a complete barrier to the flow of water. The cofferdams are expected to be in place for about 19 months. The impact of the cofferdams on the tidal currents at the project site has been assessed using the calibrated hydrodynamic model.

As outlined in Section 4, the model mesh was refined around the project location to ensure that both the cofferdams and the dredged trench could be accurately represented in the model. The 3D hydrodynamic model was run for two scenarios over 16 weeks:

- Base case (existing conditions scenario)
- Cofferdams in place. This is based on the project construction period scenario and incorporates the Sydney Harbour south cofferdam and Sydney Harbour north cofferdam as per the project's design. These structures have been removed from the model mesh with no flow allowed to pass through their respective footprints. All other areas of the seabed are the same as the existing conditions scenario.

Figure 6-1: Cofferdams during construction at the project crossing.

6.1.2 Temporary construction at cofferdams

Tidal current speed and patterns at the surface and seabed during the peak ebb and flood for the existing (base) conditions are shown in Figure 6-2 to Figure 6-7. 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:

- Current speed differences (shown as colours) compare the base case to the project 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 existing scenario model.

Surface currents during the ebb tide are relatively fast with current speeds of >0.4 m/s observed across the channel between Manns Points and Birchgrove Wharf. Slower current speeds of around 0.28 m/s and <0.06 m/s are observed at the proposed location of the south-west and north-east cofferdams, respectively. During the flood tide currents converge around Long Nose Point, creating relatively high current speeds (0.46 m/s at the surface) in the vicinity of the proposed south-west cofferdam. At the north-east cofferdam during the flood tides current speeds are high compared to the ebb, but still remain relatively low (typically <0.15 m/s).

During the ebb tide, the south-west (Long Nose Point) cofferdam caused a reduction in the current speed downstream of the structure. This is offset by a small increase in speeds in the middle of the channel and around Balls Head. The north-east (Ball Head) cofferdam has a very minor impact on current speeds during the ebb tide. This is because near the Coal Loader Wharf ebb current speeds are very low in both

existing and cofferdam scenarios resulting in the structure not significantly impacting on local flow conditions.
During the flood tide, a similar pattern is observed at the south-west cofferdam with currents significantly reduced upstream of the structure and a corresponding increase in the middle of the channel and along the northern bank (near Birchgrove Wharf). At the north-east cofferdam, larger reductions in current speeds surround the cofferdam and Coal Loader Wharf. This is because the structure interacts with the eddy that is set up in the entrance to Balls Head Bay during the flood tide.

During both ebb and flood tide the differences are more pronounced in the surface layer (where existing currents are higher) when compared to bottom layers.

Comparative time series tidal current speeds for the base case and cofferdam scenario were produced to assess the relative impact of the cofferdams. Figure 6-8 and Figure 6-9 display the time series comparisons for three locations; two adjacent to the south west and north east cofferdams and the other adjacent to the Greenwich foreshore at WH1 (about 350 metres downstream of the Greenwich Baths). The locations where results have been extracted from the model are shown in the current speed difference plots. The time series again indicate the higher current speeds at the south-west cofferdam as opposed to north-east, and that the introduction of the cofferdams lead to a reduction in current speeds at both sites. At WH1 there is an increase in the current speeds during the flood tide, which is most noticeable during spring tides. The plots show the relative magnitude of the current speed differences at these three locations. For example, at WH1 the spring flood current speeds increase from 0.36 m/s to around 0.41 m/s, a relative increase of 14 per cent. This small increase in current speeds during spring flood tides is not expected to have any notable impact on recreational amenity at Greenwich Baths.

Overall the main impact on hydrodynamics is from the Long Nose Point (south-west) cofferdam. During the ebb tide there is a reduction in current speeds immediately downstream of the Sydney Harbour south cofferdam and an associated smaller increase in the middle of the channel. During the flood tide, a similar pattern is observed with currents considerably reduced upstream of the structure and a corresponding increase in the middle of the channel and along the northern bank, near the Greenwich Baths for example. There is also a considerable increase in current speeds indicated in the gap between the Sydney Harbour south cofferdam and the foreshore (near Birchgrove Wharf). A review of the project specific geotechnical data, which described the seabed sediment at the project crossing, indicates that the sea bed in the area, being composed of cobbles, boulders, sand and clay, would not expected to be eroded by the higher current speeds. Likewise the foreshore in this area is also protected from erosion by seawalls or rocky shorelines. As such the changes in tidal currents are unlikely to lead to erosion of the seabed or adjacent foreshores.

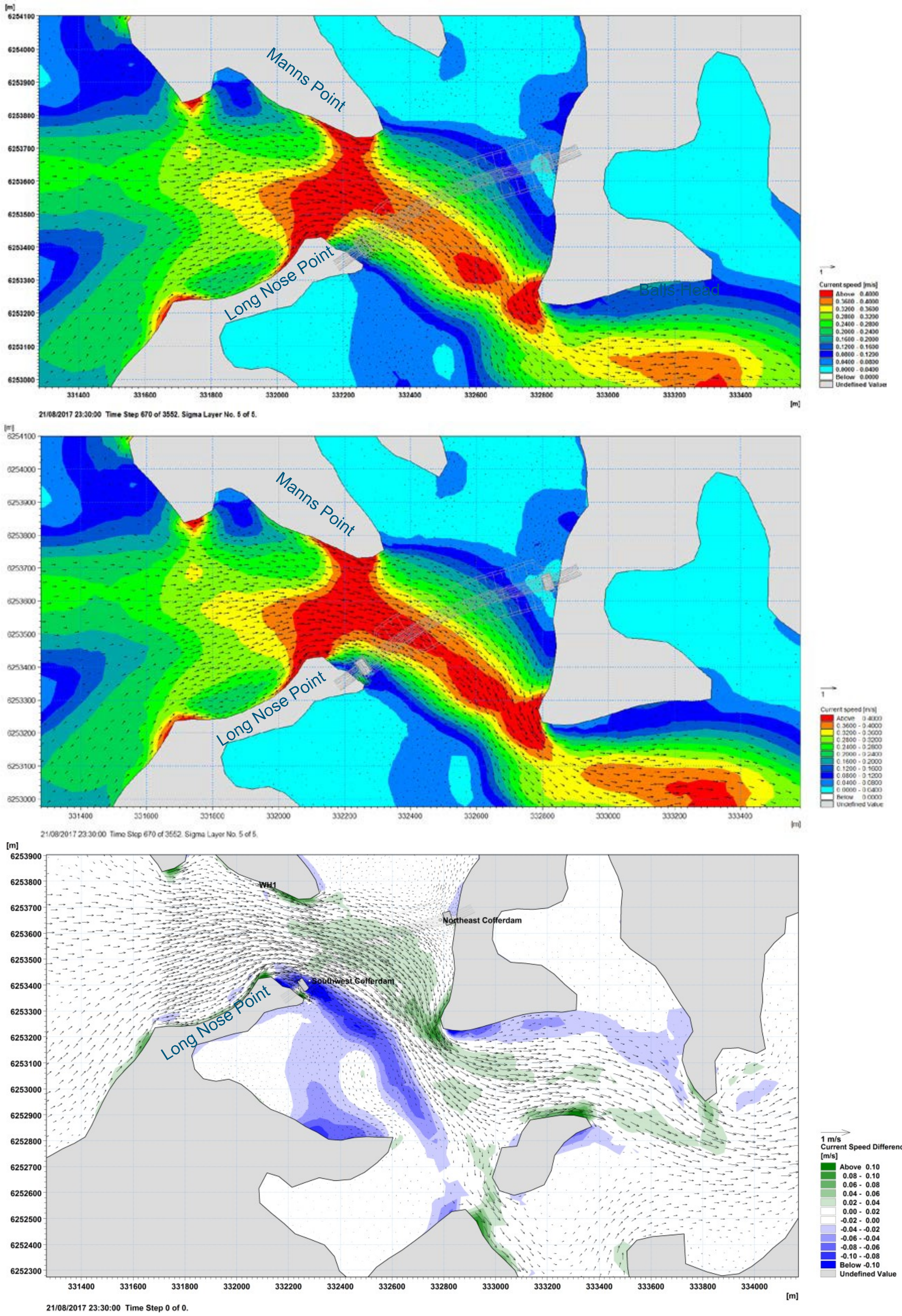


Figure 6-2: Ebb tide hydrodynamic conditions at the surface for existing scenario (top), construction scenario (middle) and current speed difference (bottom).

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

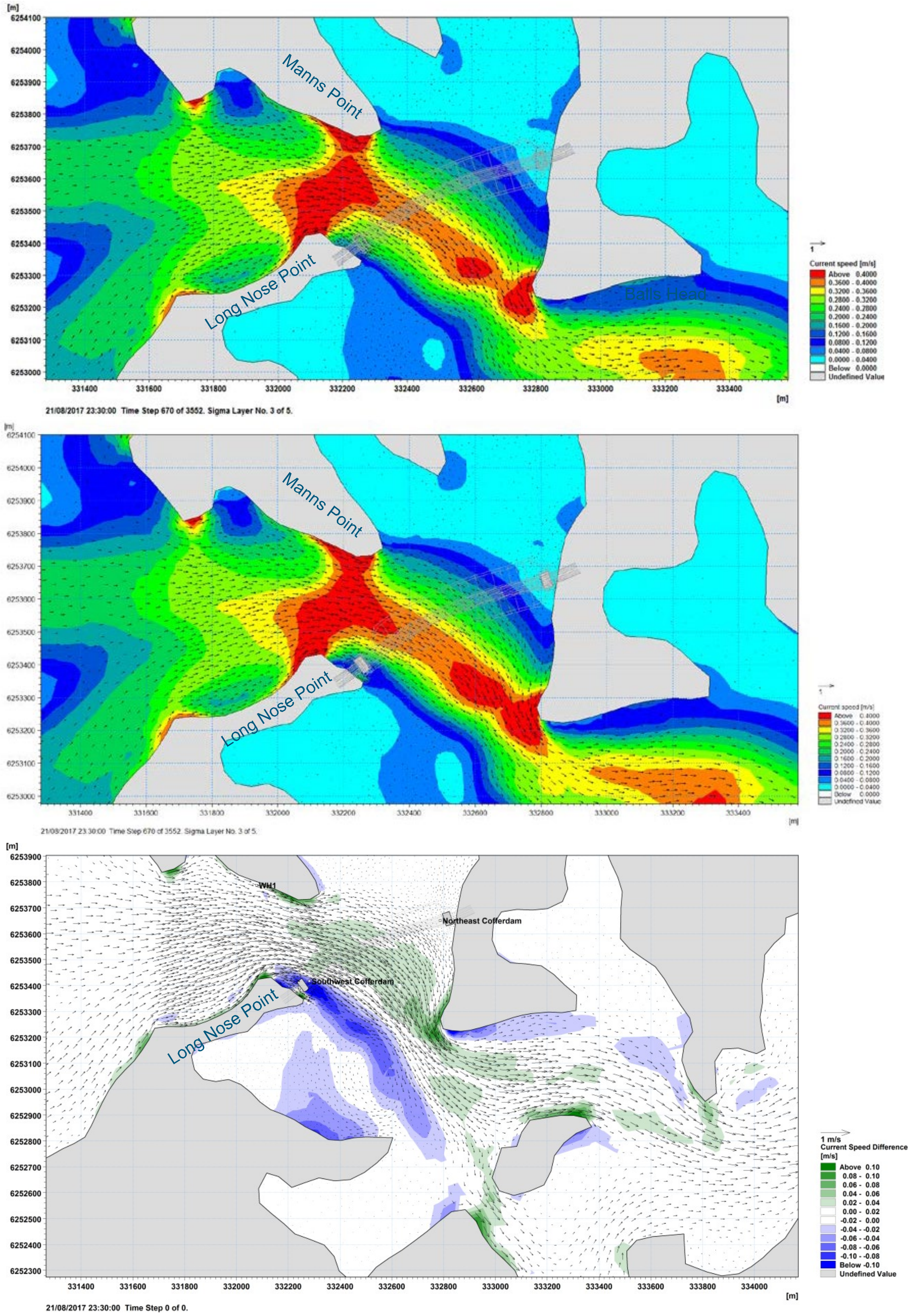


Figure 6-3: Ebb tide hydrodynamic conditions in the middle of the water column for existing scenario (top), construction scenario (middle) and current speed difference (bottom).

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

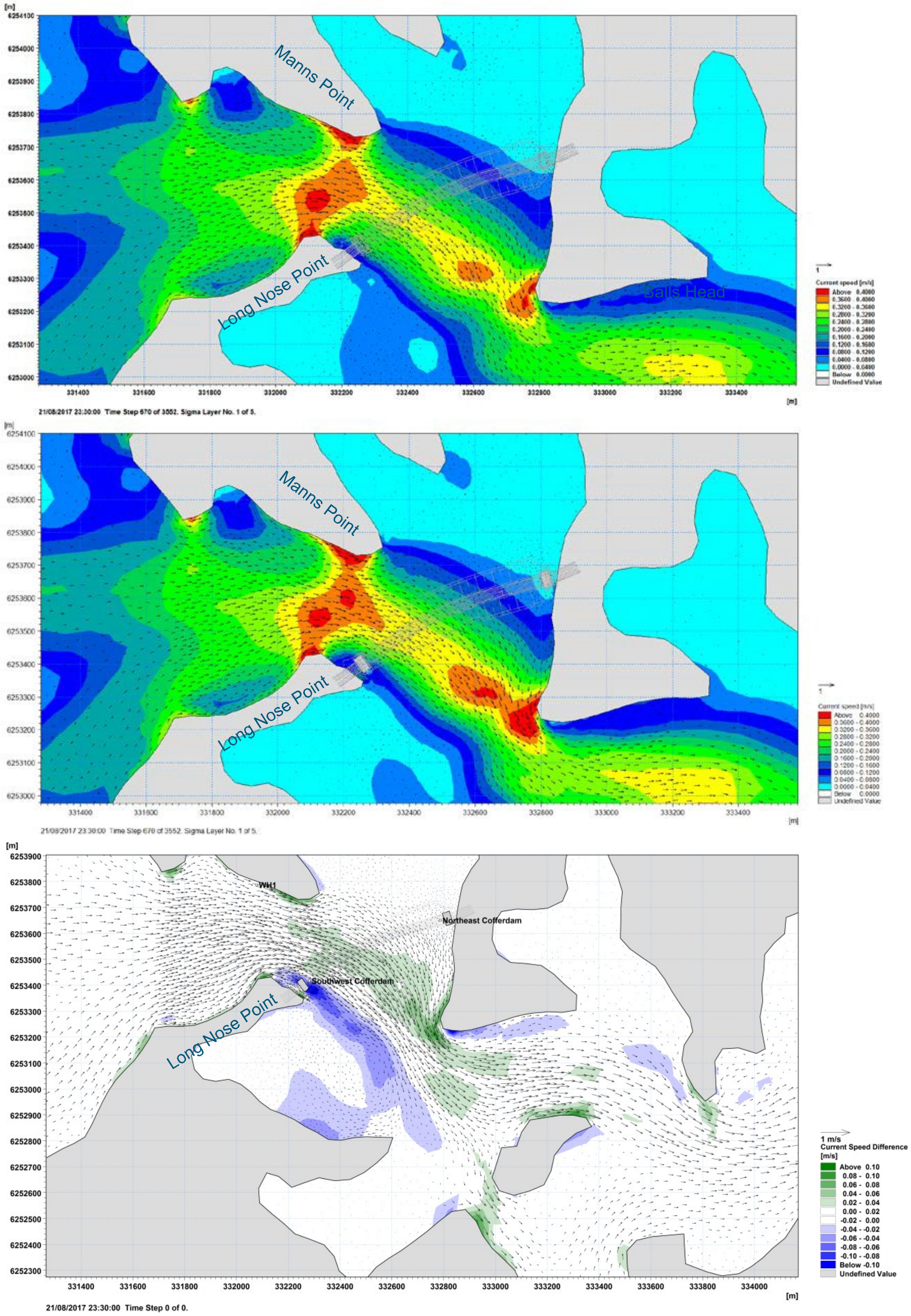


Figure 6-4: Ebb tide hydrodynamic conditions near the seabed for existing scenario (top), construction scenario (middle) and current speed difference (bottom).

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

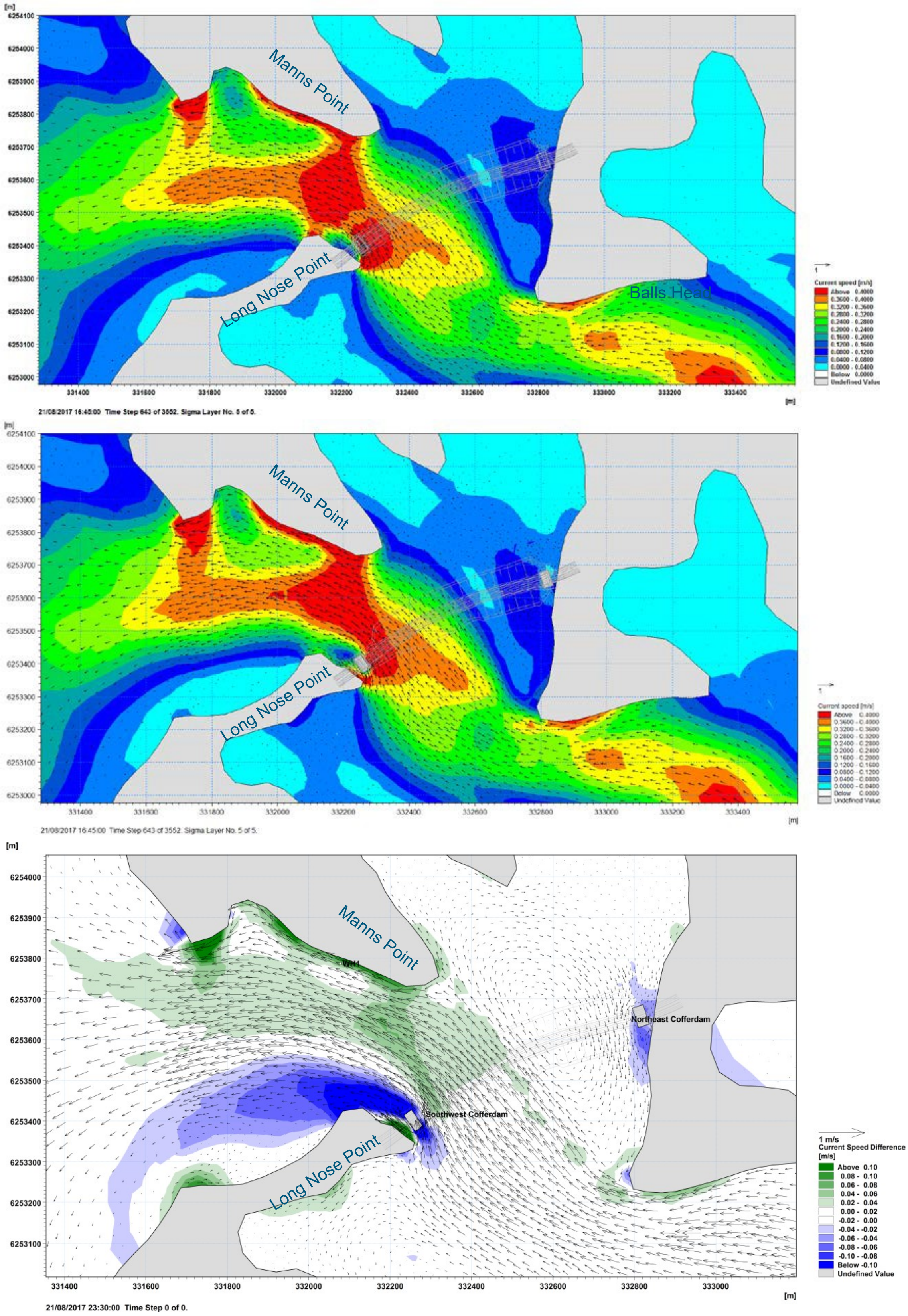


Figure 6-5: Flood tide hydrodynamic conditions at the surface for the existing scenario (top), construction scenario (middle) and difference (bottom).

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

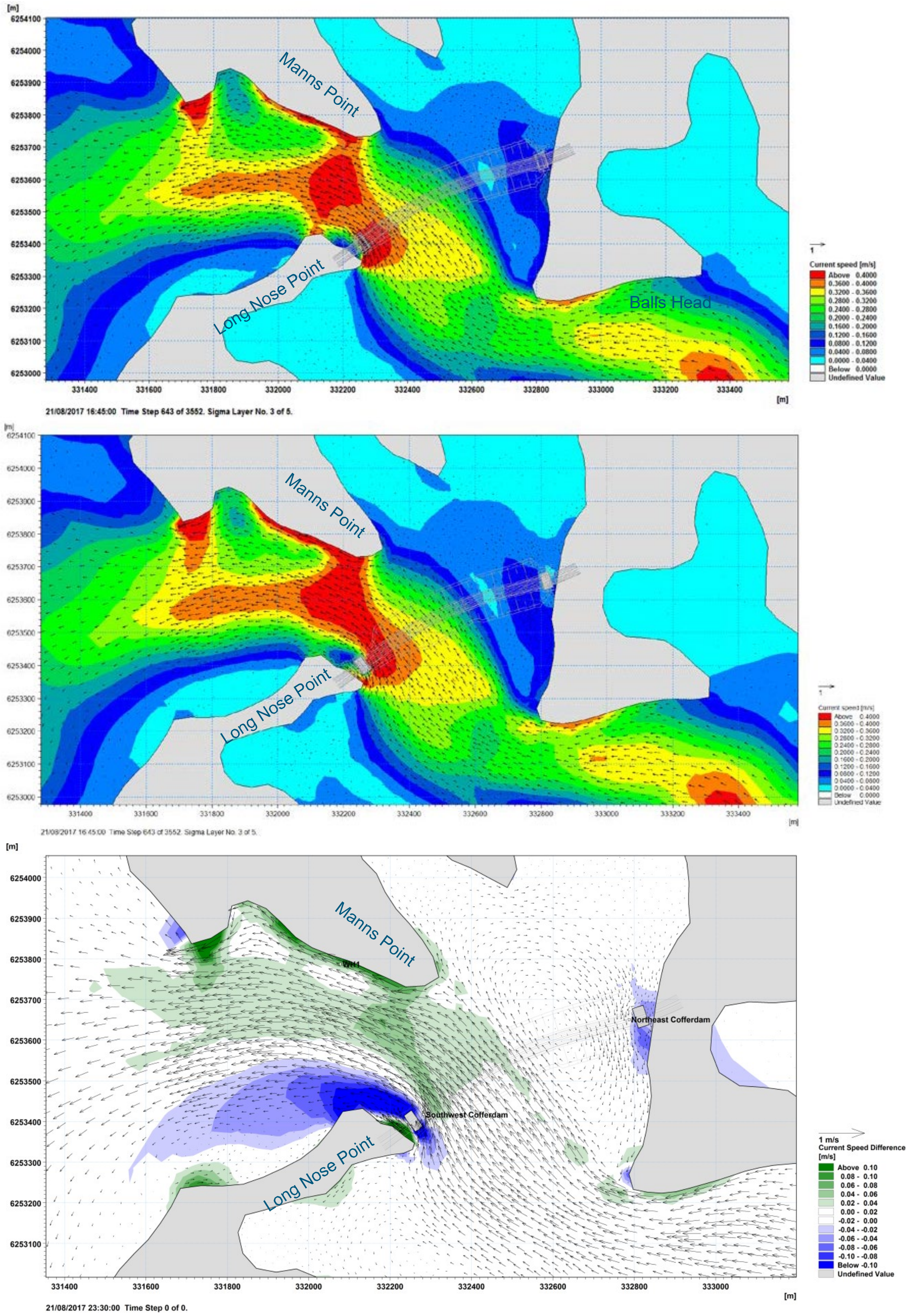


Figure 6-6: Flood tide hydrodynamic conditions in the middle of the water column for the existing scenario (top), construction scenario (middle) and difference (bottom).

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

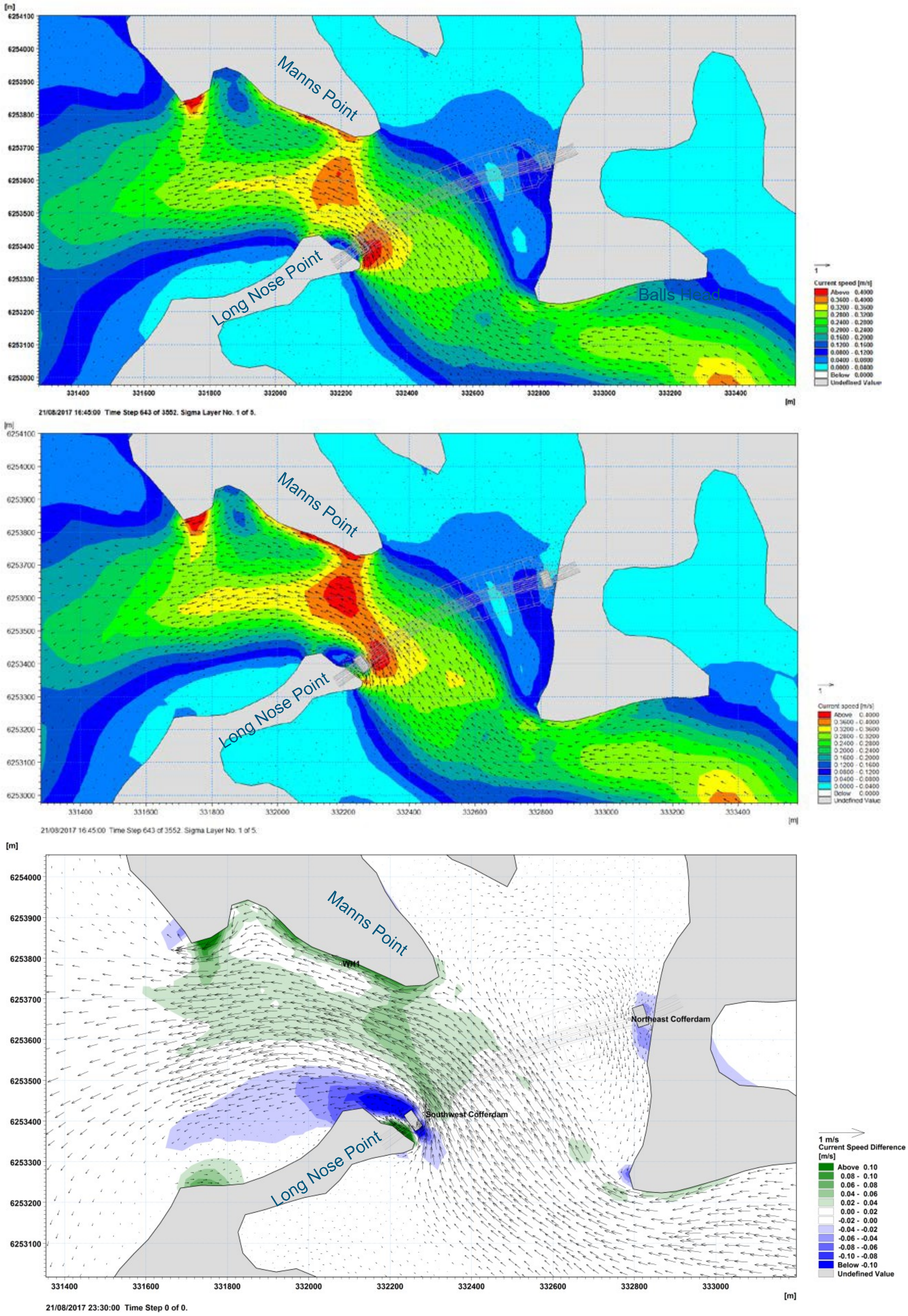


Figure 6-7: Flood tide hydrodynamic conditions near the seabed for the existing scenario (top), construction scenario (middle) and difference (bottom).

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

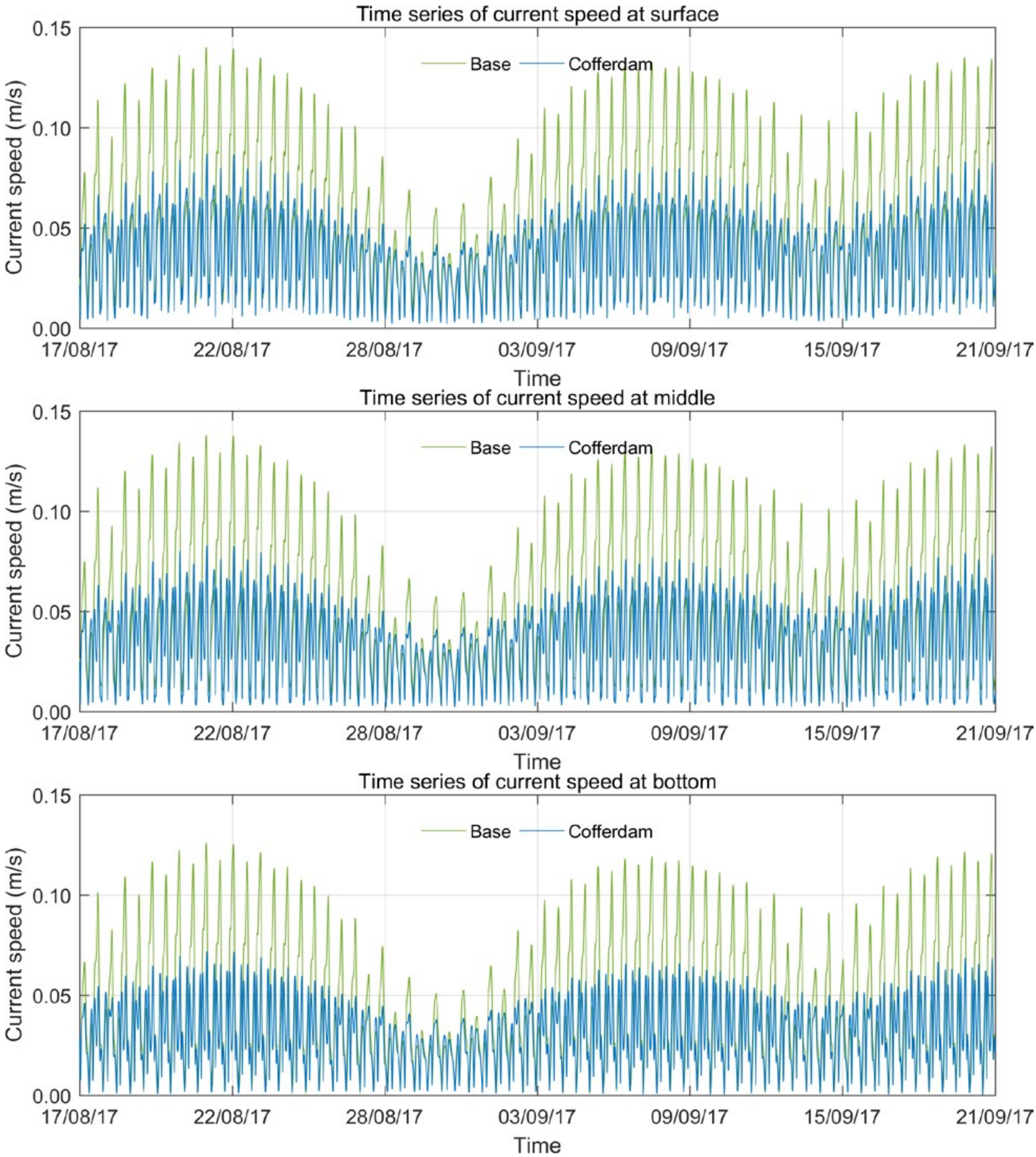


Figure 6-8: Time series plots of modelled current speed adjacent to the north-east cofferdam.

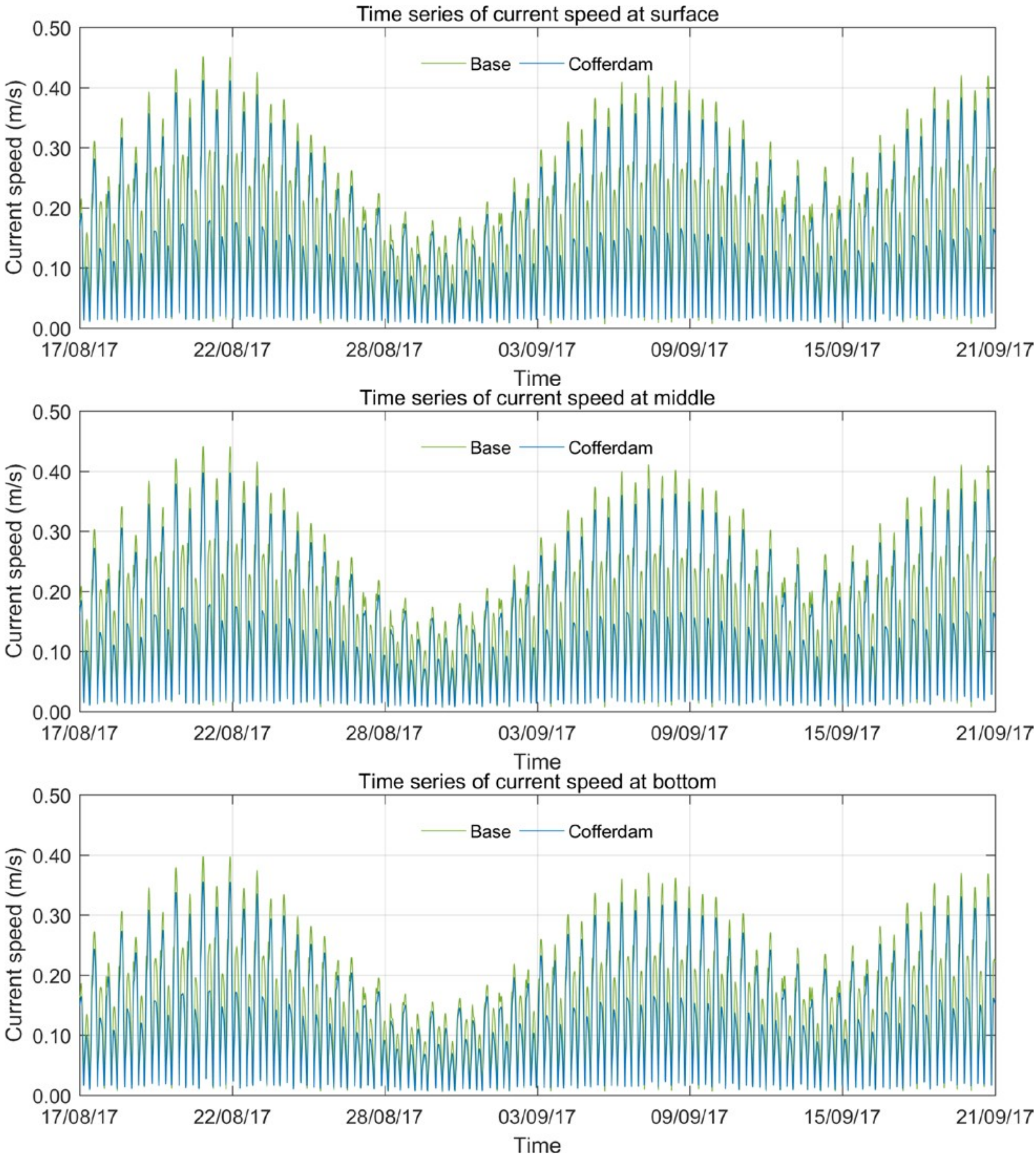


Figure 6-9: Time series plots of modelled current speed adjacent to the south-west cofferdam.

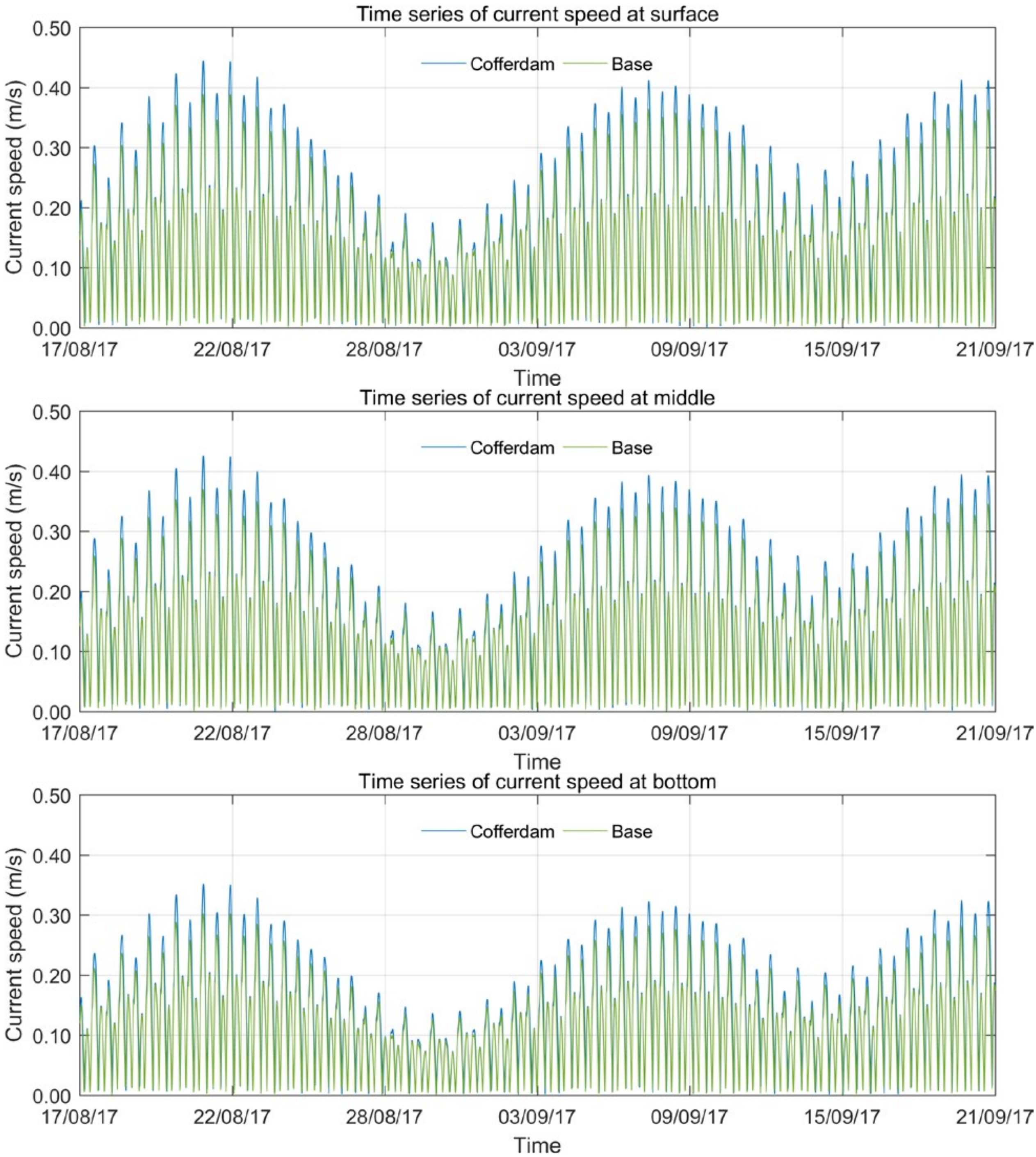


Figure 6-10: Time series plots of modelled current speed at the WH1 location (downstream of Greenwich Baths).

6.2 Operational impacts

Following construction the seabed would be returned to pre-project conditions. Therefore, operational impacts on hydrodynamics were not assessed as they are expected to be minimal.

7 Dredge plume modelling

7.1 Overview

Construction of the immersed tube tunnel requires dredging activities to be carried out. To assess the potential impacts resulting from these dredging activities, numerical modelling has been carried out to represent the potential fate of sediments released into the water column through dredging. The modelling simulates the dispersal of suspended sediments by ambient currents as well as the subsequent deposition of sediment suspended by dredging (dredge SSC), showing the impact dredging has on water quality. Ambient SSC and sedimentation are not simulated. The disposal of the dredged material is not a relevant consideration in the dredge plume modelling presented here. Dredged material would be disposed of either offshore at an approved offshore disposal site or on land, for further details refer to Section 2.2.2 of Chapter 2 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, have 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 dredging activities is referred to as a 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 the type of dredging plant, 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 are based on information obtained from detailed geotechnical investigations as provided by Roads and Maritime.

It is noted that the volume of soft silty clay and silty sand indicated as unsuitable for offshore disposal (142,500 m^3) is preliminary and is subject to the outcome of further sampling and testing.

Table 7–1: Estimated in situ dredging quantities and properties.

	Dredge Material	In situ volume (m^3)	Dry density (kg/m^3)	In situ fines (per cent)*
Sediment unsuitable for offshore disposal	Soft silty clay	118,750	900	69
	Silty sand	23,750	1280	15
Sediment suitable for offshore disposal	Clayey silt	30,000	920	75
	Sandy silty clay	275,000	700	73
	Silty sand	275,000	1280	15
	Silty clay	30,000	900	69
	Soft rock (weak sandstone)	3000	2300	3
	Hard rock (sandstone)	105,000	2300	3
	Rehandling (crushed gravel)	105,000	1600	3

*Coarse silts and finer (<63 μm).

7.2 Dredge description

RHDHV engaged a dredging expert to assist in developing a potential 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 where 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)
- Medium-sized trailer suction hopper dredge (TSHD) with approximately 11,000 m^3 hopper capacity
- Very large cutter suction dredge (CSD)
- 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 controls to ensure best environmental practice would be achieved. These controls included but are not limited to: the installation of silt curtains around some dredging plant, ensuring no overflow from the receiving hoppers and use of specialist dredge equipment (ie enclosed environmental buckets).

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

1. All materials unsuitable for offshore disposal would be dredged by a very large BHD working in conjunction with hopper barges (refer Figure 7-1 and Figure 7-2). The BHD would be fitted with a closed bucket attachment (refer Figure 7-1) and would load dredged material directly into hopper barges positioned immediately adjacent to the dredge (with no overflow). These materials would be transported to shore 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 significantly reduced.

2. Following validation that all material unsuitable for offshore disposal has been removed, dredging of underlying non-cohesive material that is considered to be suitable for offshore disposal would be completed by a medium-sized TSHD (refer Figure 7-4). The TSHD would progressively remove the majority of non-cohesive materials present within the dredge footprint suitable for offshore disposal. This material exists above the denser and harder layers of sands and clays that are unable to be dredged by a TSHD. The TSHD would move in long passes over the dredging footprint trailing its draghead(s) until its hopper is full (no overflow from hopper), after which it would transport the dredged material through the Harbour to the offshore disposal site.

3. The dredging of hard and cohesive materials that are unable to be removed by the medium-sized TSHD would be completed by the very large BHD, which would also remove some of the underlying soft rock (1–5 MPa strength). The BHD would be fitted with a standard open bucket (refer Figure 7-2) and would load material into a fleet of split hopper barges (with no overflow) with a suitable capacity and number to match its production rate. The hopper barges (refer Figure 7-3) would be used to transport the dredged material overwater to the offshore disposal site.

4. Hard rock (>5 MPa strength) would be crushed with a very large CSD (refer Figure 7-5). The CSD would use its cutter head operating in ‘no pumping’ mode to crush the rock into small fragments. The crushed rock would be left within the tunnel trench in a layer immediately behind the cutter head for subsequent removal by the very large BHD. Due to the limited cutting depth of the CSD, the BHD would need to work in close coordination with the CSD to remove crushed rock before the CSD returns to perform another cut. The BHD would load the crushed rock into self-propelled split hopper barges (with no overflow) for offshore disposal. The self-propelled barges would have an expected capacity of 1800 m^3.

Figure 7-1: Example BHD loading hopper barge with closed 'bucket' (clamshell) attachment.

Figure 7-2: Example very large BHD operating with standard open bucket attachment.

Figure 7-3: Example self-propelled Split Hopper Barge.

Figure 7-4: Example medium-sized TSHD.

Figure 7-5: Example very large Cutter Suction Dredge (CSD).

7.2.1 Dredge schedule

A schedule has been prepared for completion of dredging. This proposed schedule has been 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 has been assumed that dredging would occur Monday to Friday between the hours of 7am and 6pm, with the exception of dredging with the TSHD, which would occur seven days per week on a 24-hour basis. It is noted that dredging may potentially occur outside these hours, provided it is consistent with the overall planning approval and environmental protection licences.

Stage 1 – Sediment and soft (weathered) rock removal

- Dredging operations of material with the very large BHD would occur five days per week (Monday to Friday) for a total of 10 hours a day during daylight hours (7am to 6pm)
- Actual dredging hours utilising the very large BHD 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 west to east direction)
- Once all materials unsuitable for offshore disposal have been dredged, a medium-sized TSHD would be employed to dredge the majority of the sediments and soft rock suitable for offshore disposal. The TSHD would operate continuously (24/7, typically 156 hours per week) with the dredger disposing of the material offshore. Due to the cycle time of the dredging and disposal activity, and the restriction on overflow from the hopper, the TSHD would only dredge for about 25 minutes every five to six hours, the remainder of the time is spent moving to and from the offshore disposal site and bottom dumping.

Stage 2 – Hard rock removal

- The very large CSD would be used to crush regions of rock within the dredge footprint. The CSD would operate five days per week (Monday to Friday) for a total of 11 hours a day during daylight hours. Actual dredging hours utilising the CSD is 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 a BHD or grab dredge would handle crushed sandstone through placement into barges for offshore disposal.

The rate at which the material would be dredged is dependent on the properties of material being dredged and the type of equipment used.

7.2.2 Material release to the water column by dredging

As previously discussed, the dredging methodology involves no overflow of material from barges or from the TSHD 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, outside of Port Jackson. Therefore the only potential source for material to be suspended into the water column within Port Jackson would be through the direct dredging/loading process.

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 into the water column as a proportion of the total production rate.

The source terms applied for the various dredging activities and soil 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 Loses (per cent)
BHD – environmental bucket[1]	Silty clay	1.5
	Silty sand	1.5
TSHD	Clayey silt	1.75
	Sandy silty clay	1.75
	Silty sand	1.75
BHD – closed/standard bucket	Silty sand	1.5
	Sandy silty clay	1.5
	Silty clay	1.5
BHD – standard bucket	Sandstone (soft, weathered)	2.0
CSD	Sandstone – low strength	3.0
	Sandstone – medium strength	2.75
	Sandstone – medium to high strength	2.5
BHD (re-handle crushed sandstone)	Crushed gravel	1.5

[1] Used for the dredging of surface 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 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 Roads and Maritime as well as information from previous RHDHV projects
- Particle size distribution information from the geotechnical site investigations (which have been carried out using freshwater hydrometer), surface sediment samples collected on behalf of Roads and Maritime and analysed using laser analysis methods as well as information reported from other nearby projects (eg Sydney Metro).

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 with the TSHD and CSD. 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 surface silt curtains (ie shallow draft silt curtains about two to three metres deep) would be used for the BHD to limit the surface sediment plume when the bucket is lifted above the water surface. Inclusion of shallow draft silt curtains has been assumed when deriving the source terms (presented in

Table 7–2) resulting from the dredging (as opposed to the silt curtains being included as structures in the hydrodynamic model).

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 have been simulated for a 16 week period
- The models bathymetry is based on survey provided by Roads and Maritime
- Five sigma layers are used
- Two temporary cofferdams (Sydney Harbour south cofferdam and Sydney Harbour north cofferdam).

The MIKE 3 FM Mud Transport (MT) module is 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 (<63 μm) are included in the model, which was setup to include four sediment fractions: clay (<2 μm), fine silt (2–6.3 μm), medium silt (6.3–20 μm) and coarse silt (20–63 μm). The sediment fractions used in the model are based on ISO 14688-1:2002 size classes. 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) for the dredging is provided in Figure 7-6.

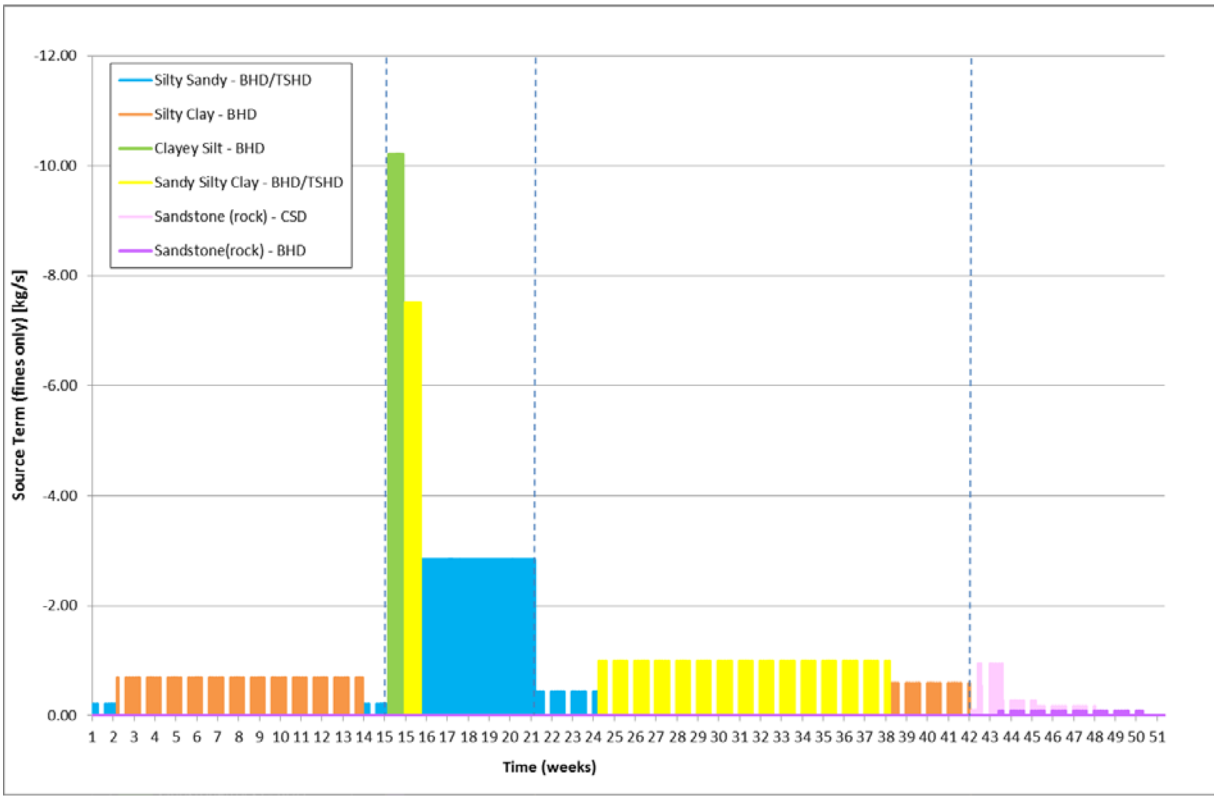


Figure 7-6: Summary of source terms for fine sediment released by dredging at project crossing.

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

The following assumptions have been made as part of this assessment:

- The dredge plume model does not include any background ambient SSC
- Suspended sediment is released into the bottom water layer for the TSHD and CSD for all soil types. The BHD model releases sediment uniformly throughout the water column
- No overflow of any barges used for 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 N/m^2 applied
- Surface silt curtains (ie shallow draft silt curtains) would be used for the BHD to limit the surface sediment plume when the bucket is lifted above the water surface. Inclusion of silt curtains has been assumed when deriving the source term of sediment resulting from the dredging (as opposed to being included as structures in the hydrodynamic model)
- The plume resulting from the bedding and backfilling activities would be relatively small compared to the dredging activity and these are not included in the modelling
- Any clay particles released by the dredging activity would be subject to flocculation. The amount of flocculation has been assumed to be low, only allowing relatively small flocs to form. The settling velocity of the clay sized particles in the model has been calculated assuming the size of flocs which would be expected to form with a SSC of approximately 10 mg/L.

7.4 Water quality modelling results

7.4.1 Suspended sediment

The SSC due to dredging (ie dredge plume) model results have been 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 90thpercentile 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 are processed for specific periods that related back to the varying dredging activities (refer to Section 7.3). These periods are indicated on Figure 7-6 and listed as:

- The entire 51 weeks of the dredging program
- Weeks one to 13 of dredging when the BHD is working in soft sediments with moderate source terms
- From weeks 15 to 20 when the TSHD is operational with high source terms
- From weeks 21 to 41 when the BHD is working in soft sediments with moderate source terms
- From weeks 41 to 51 when CSD and BHD work to remove rock on either side of the crossing with low source terms.

Appendix 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 all three vertical layers. Example plots showing the 95th percentile over the entire dredging duration are shown in Figure 7-7. 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 is 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 Long Nose Point, WH1, WH2 and Berrys Bay (Figure 7-8). These locations are also shown on the dredge plume SSC plots (see Figure 7-7). These locations 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 (eg WH1 site is one of RHDHV’s monitored sites and is located approximately 320 metres from Greenwich Baths), or are locations directly adjacent to the dredging (ie Long Nose Point). The time series at Long Nose Point demonstrates the relatively local and short duration increase in SSC predicted in these areas. 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 comparison to ambient monitoring data which is typically collected at the surface.

Table 7–3: Summary of SSC statistics in the surface layer at the four model extraction locations.

Location	Percentile SSC (mg/L)				
	20th percentile	50th percentile	90th percentile	95th percentile	99th percentile
Long Nose Point	0.03	0.37	0.72	0.90	1.77
WH1	0.03	0.40	0.95	1.19	2.12
WH2	0.03	0.36	1.04	1.30	2.83
Berrys Bay	0.02	0.18	0.45	0.54	0.70

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 > 2 mg/L) are limited to a relatively small extent, concentrated at the north eastern end of the dredging footprint near Balls Head Bay and around Balls Head. There are two main factors that influence the plume statistical distribution:
    - In this north eastern part of the dredge footprint, current speeds are low, particularly during the ebb tide (see Figure 3-4 to Figure 3-5), allowing the sediment suspended by dredging to accumulate. In other zones of the dredge footprint, stronger tidal currents disperse the sediment suspended by dredging
    - The BHD spends a considerable amount of time in this area (weeks 24 to 38) removing the deposit of sandy silty clay located adjacent to Balls Head
- For the lower 90th percentile the dredge plume extents (SSC >2 mg/L) are limited to the area adjacent to the shoreline along the western side of Balls Head, within the inner and eastern areas of Balls Head Bay and extend in front of Gore Cove to the tip of Green Island
- The extent of the plume that is above SSC >20 mg/L[3], when analysed over the entire dredge duration and viewed at the 95th percentile (surface layer), is very small and contained in the dredging footprint immediately adjacent to the north-east cofferdam
- The suspended sediment released during the dredging activity is transported both in an upstream and downstream direction, with a slight downstream dominance. This is a result of the tidal currents that are predominately aligned with the main longitudinal axis of the estuary. Suspended sediment is also transported into Balls Head Bay when dredging activity is being carried out on the eastern part of the dredge footprint
- For 50 per cent of the time of the dredging campaign the SSC is less than 1 mg/L, even in the dredging footprint. This shows that for half the time the dredging occurs, the increases in SSC are expected to be low everywhere
- The dredge plume extents are greater in the bottom layer (layer one) than at the surface (layer five) (see Figure 7-7)
- Short duration (less than one per cent of the time) increases in SSC above 5 mg/L occur, but only in the area adjacent to the Coal Loader Wharf.

[3] The exact SSC value of what can be considered a 'visible' plume is uncertain. However, 20 mg/L provides an indication as to what may be considered to be a visible plume assuming an ambient SSC of about 5 mg/L.

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

- At Long Nose Point, a site immediately adjacent to the dredging footprint, dredging creates a distinct increase in SSC for short bursts. This is most evident at the start of the dredging period when the BHD is working to remove soft sediment near the south-west cofferdam and at the end when the CSD and BHD are working to dredge and remove rock. While values of up to 59 mg/L[4] were modelled, these increases are very short lived (ie observed as spikes in SSC)
- Within Balls Head Bay at site WH2, the 95th percentile SSC in the surface layer is predicted to be 1.3 mg/L, which is slightly higher than 1.2 mg/L at the nearby WH1 site. WH1 is located in the main channel and the time series plot shows elevated dredging SSC through the period of dredging. Within Balls Head Bay, dredging SSC is most evident when the BHD or the TSHD is working in the north-east of the dredging footprint
- At the Berrys Bay site, dredging SSC only exceeds 1 mg/L for a very short period during the TSHD dredging. For 99 per cent of the time, dredging SSC in the surface layer is less than 0.7 mg/L.

[4] This value is not shown in the figure due to comparative scale used; it is an isolated event that occurs in week 43 of dredging activity.

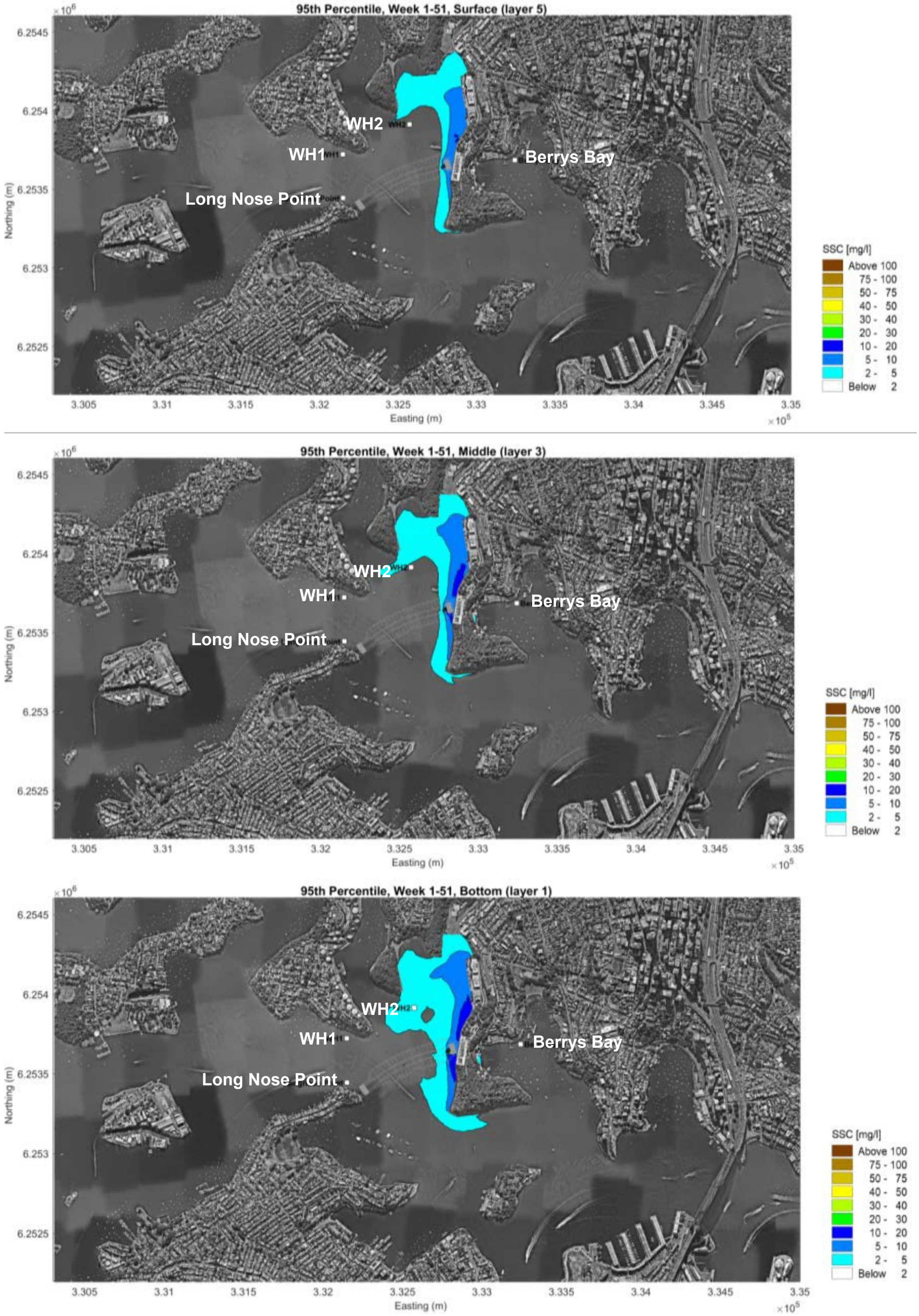


Figure 7-7: 95th percentile, for surface (top), mid-depth (middle) and bottom (bottom) for the entire project dredging period (weeks 1 to 51).

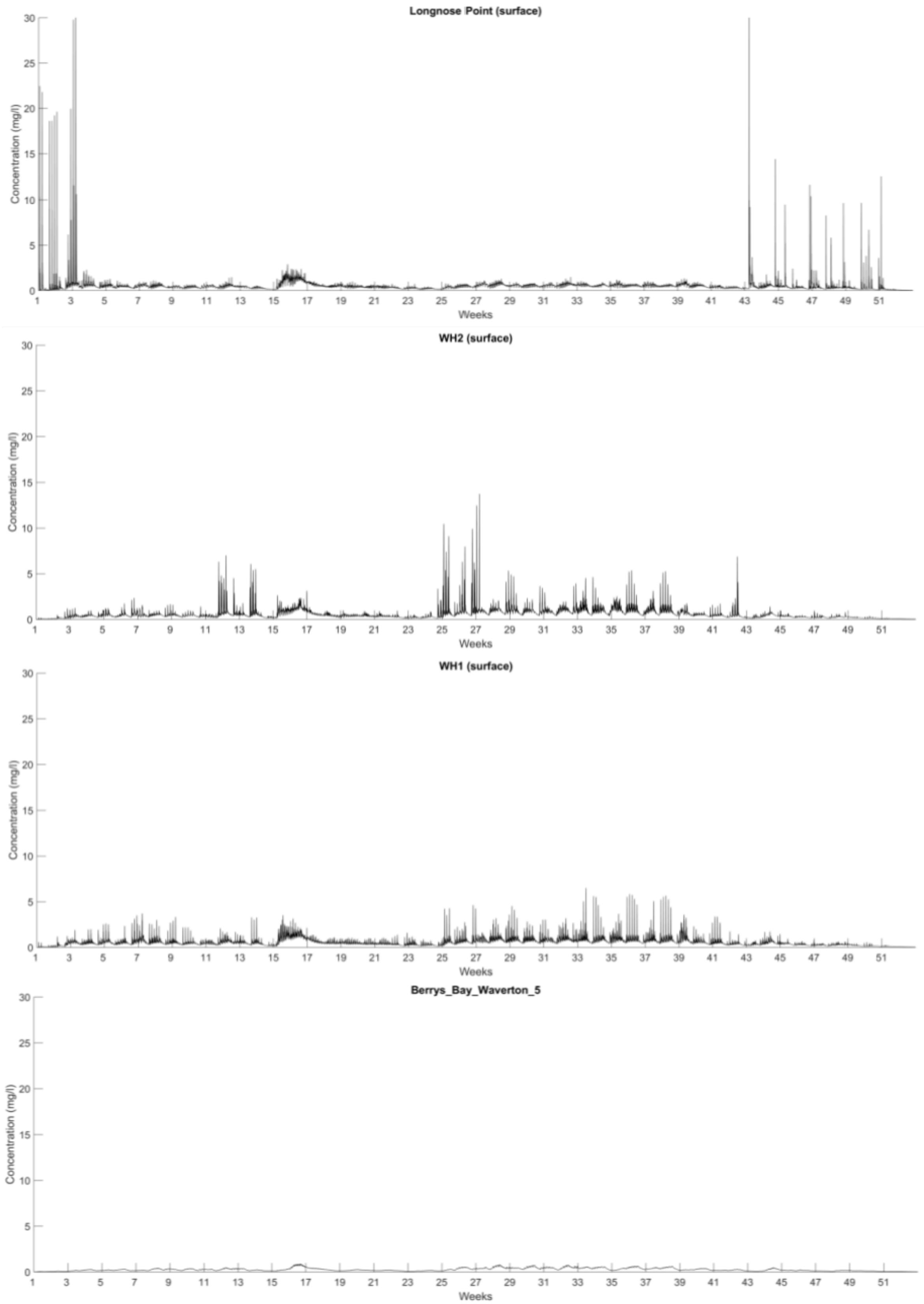


Figure 7-8: Time series of dredging SSC at five sites across the Western Harbour region (surface layer).

7.4.2 Sensitivity of dredge plumes to winds

The relative influence of the dredge SSC to wind driven circulation was tested. A hydrodynamic simulation was completed which included wind forcing to the 16 week tidal hydrodynamics (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 16 weeks of the dredging activity.

Table 7–4 presents a summary of SSC statistics at WH1 and WH2 for the tide only and tide + wind hydrodynamic simulations. Time series results of SSC at the WH1 and WH2 locations are presented in Figure 7-9. The WH1 and WH2 locations were selected for this comparison as one site (WH1) is located in the main tidal channel, where the sensitivity of dredge SSC would be expected to be low. The other (WH2) is located within Balls Head Bay, an off-channel embayment, where the sensitivity of dredge SSC to wind is expected to be more significant due to weaker tidal currents and shallower bathymetry. Results are provided for the surface layer.

The results show very little wind sensitivity at WH1 with a more significant difference at WH2. The most noticeable differences occur during weeks 11 to 12 where peak SSC are amplified in the tide + wind simulation. Overall the results indicate sensitivity to wind is limited to brief periods at selected locations in the off-channel embayments.

Table 7–4: Summary of SSC statistics in the surface layer for tide only and tide + wind over the 16 week simulation.

Percentile	WH1		WH2	
	Tide only SSC (mg/L)	Tide + wind SSC (mg/L)	Tide only SSC (mg/L)	Tide + wind SSC (mg/L))
20th percentile	0.29	0.29	0.26	0.29
50th percentile	0.47	0.46	0.43	0.47
90th percentile	1.10	1.14	1.05	1.20
95th percentile	1.41	1.46	1.27	1.50
99th percentile	2.22	2.35	2.69	3.58

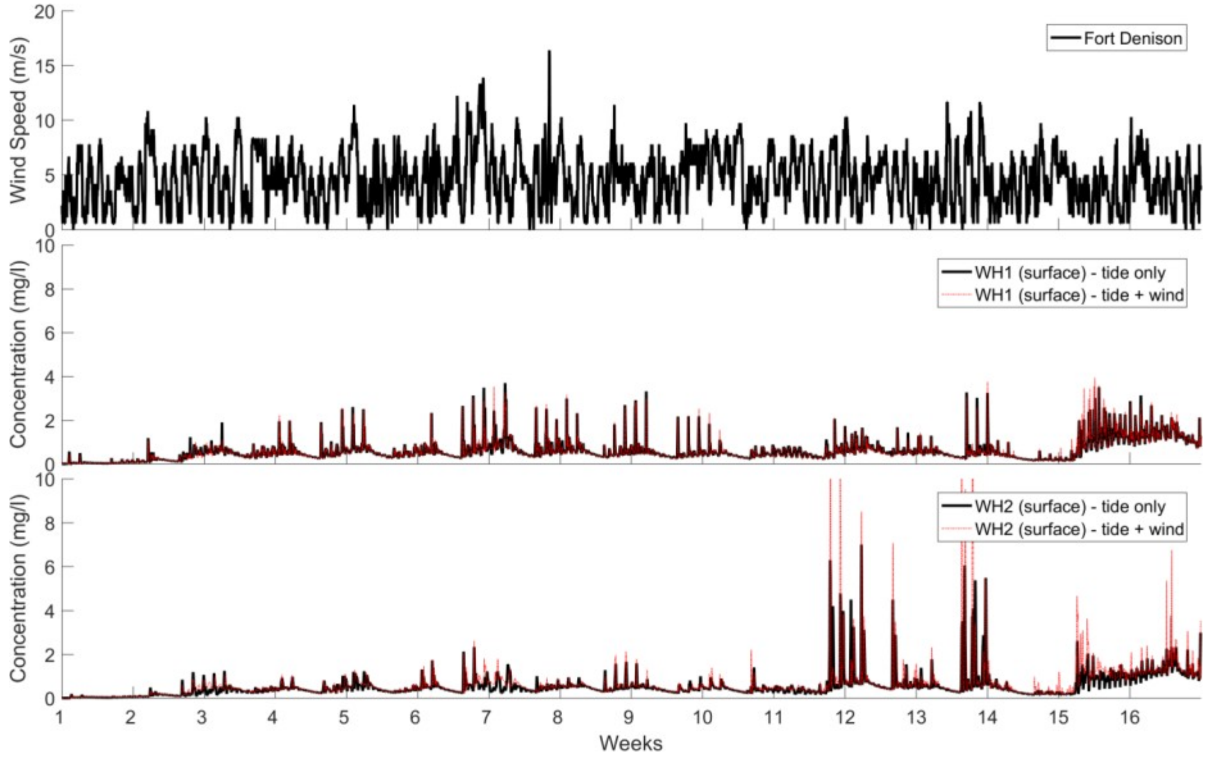


Figure 7-9: Time series of dredging SSC comparing the tide only and tide + wind plume simulations.

7.4.3 Sediment deposition

Sediments suspended by dredging and dispersed by ambient currents deposit back on the bed in suitable environments. A map of deposition on the seabed two weeks after the cessation of dredging campaign is presented in Figure 7-10. 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 seabed.

Time series plots of deposition at the two model extraction locations, Coal Loader Wharf and Gore Cove, are provided in Figure 7-12 and Figure 7-13. The plots present the cumulative deposition (mm) as well as the rate of sedimentation. The average daily rate of sedimentation and maximum daily sedimentation rates ($mg/cm^2/day$) were calculated over the entire dredge duration (51 weeks).

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. The deposition is concentrated at the north-eastern end of the dredging footprint, within the north-eastern portion of the dredging footprint and along the shoreline adjacent to the Coal Loader Wharf. At this location sedimentation rates of greater than 35 $mg/cm^2/day$ occur (or just over 1 mm/day)
- Lower levels of sedimentation (one to five millimetres) also occur in Balls Head Bay and within the other embayments that line the Western Harbour region. This is expected given the lower tidal current speeds in these embayments (ie they are depositional environments). This is further illustrated in Figure 7-11, which shows the pattern of bed shear stress. In this plot depositional environments are those with low bed shear stress (or white areas). Any deposition that occurs in areas with high bed shear stress (ie during slack water) is resuspended by tidal currents and reworked into nearby embayments

- The highest rate of deposition occurs during the period when the BHD is working in the sandy silty clay deposit adjacent to Balls Head
- The maximum and daily average deposition rates for the dredging within Balls Head Bay were less than the lowest thresholds noted in the literature.

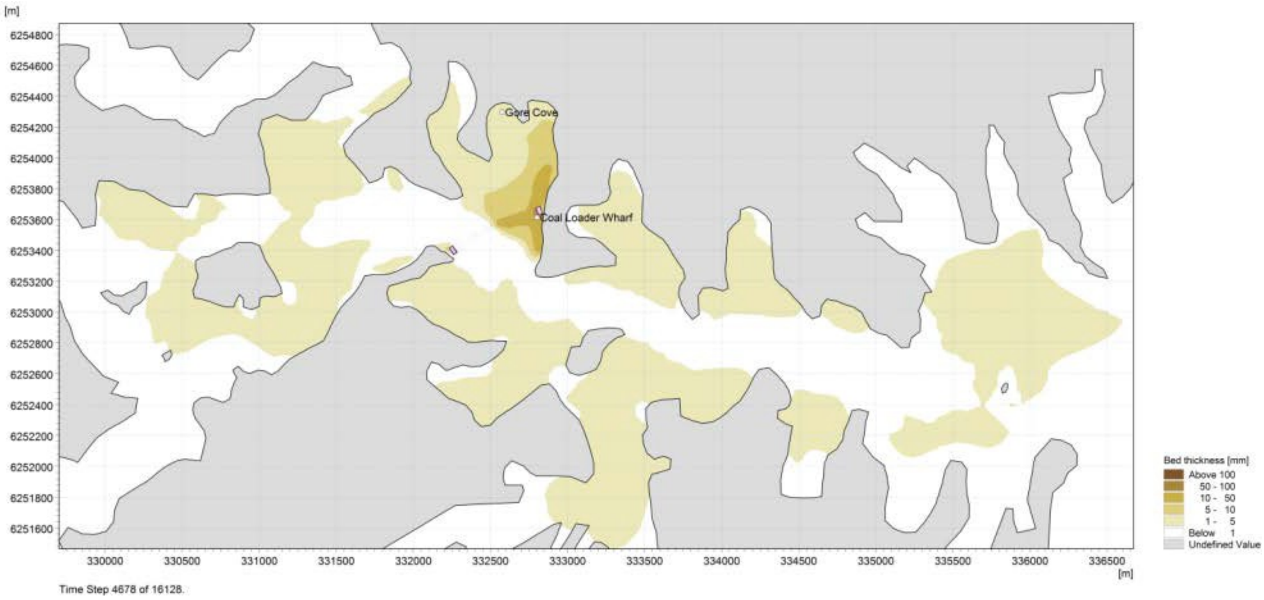


Figure 7-10: Deposition (mm) two weeks after the cessation of dredging.

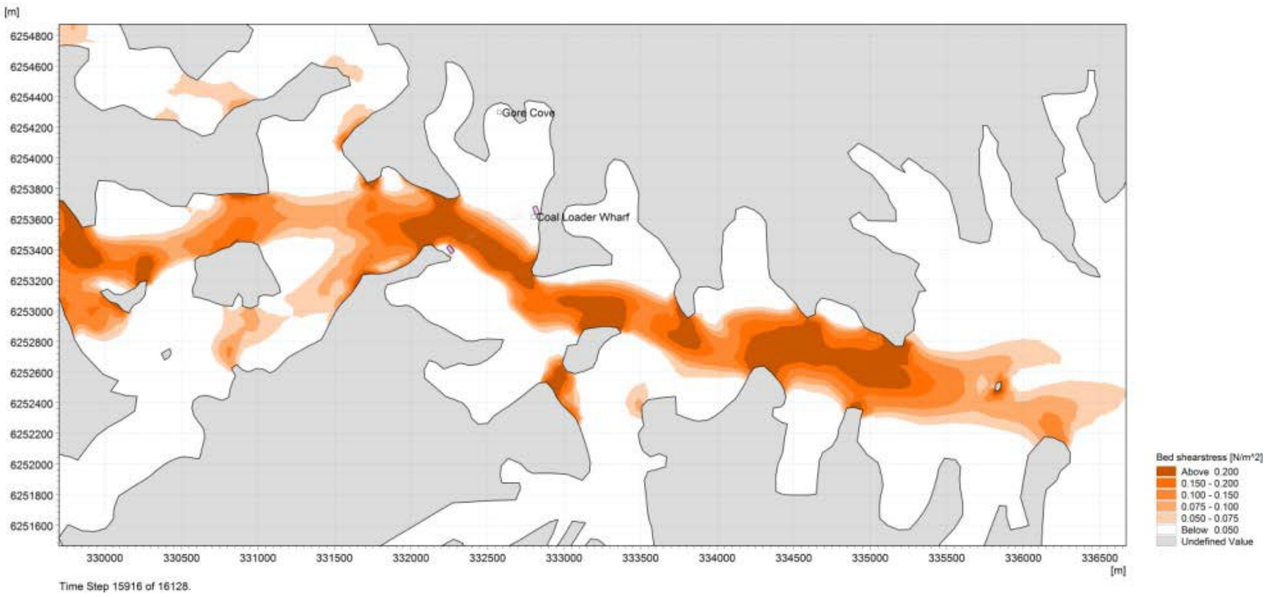


Figure 7-11: Bed shear stress during spring tides (peak of ebb tidal stage).

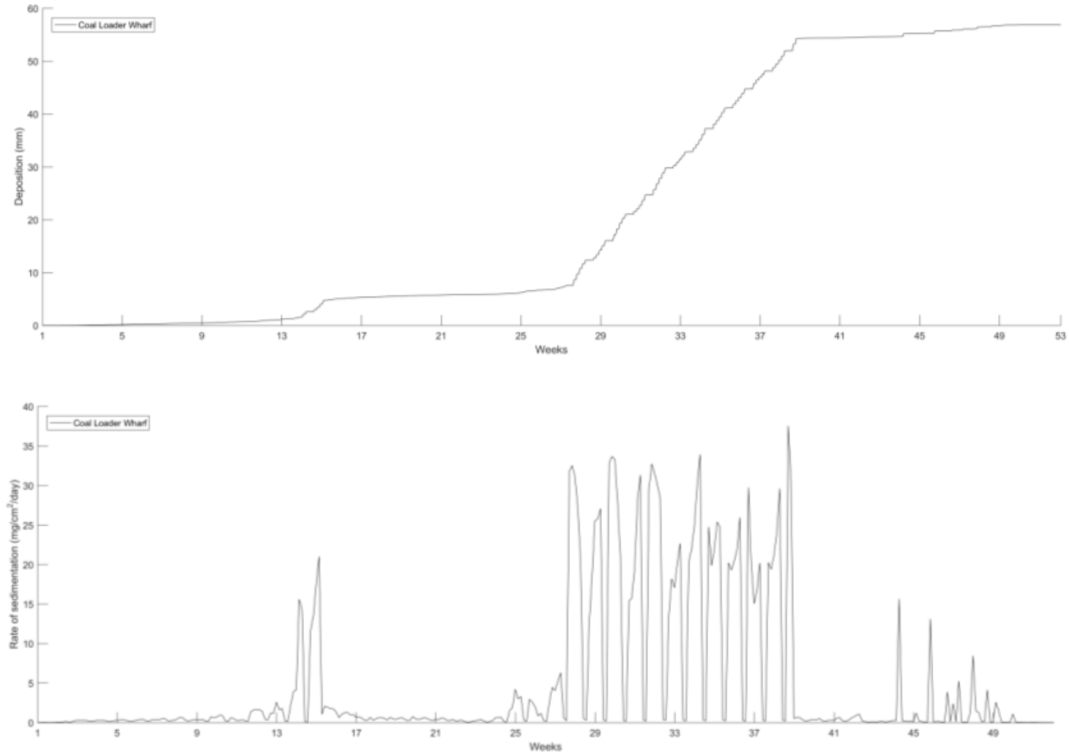


Figure 7-12: Time series of accumulative deposition (top) and rates of sedimentation (bottom) caused by dredging at the Coal Loader Wharf model extraction location.

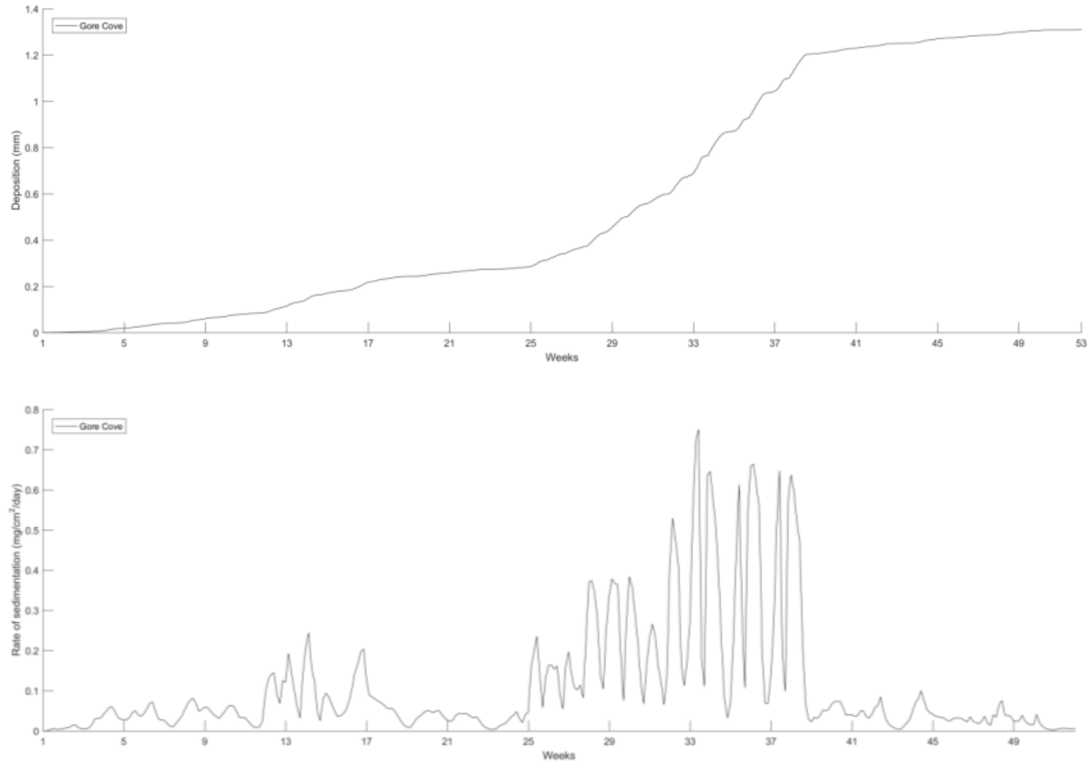


Figure 7-13: Time series of accumulative deposition (top) and rates of sedimentation (bottom) caused by dredging at Gore Cove model extraction location.

8 Summary and discussion

8.1 Summary

This assessment has detailed the findings of numerical modelling to better understand the potential impact construction activities may have 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. This also informed a description of the existing environment of Sydney Harbour and the wider Port Jackson estuary. The project specific hydrodynamic data was then used to calibrate a 3D model that has been established for this 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 focuses on the construction of two temporary cofferdams for the project crossing (located at Long Nose Point and Balls Head) with an immersed tube. The modelling of hydrodynamic impacts during the projects construction phase has shown the following:

- During the ebb tide, the south-west (Long Nose Point) cofferdam would cause a reduction in the current speed downstream of the structure. This would be offset by a small increase in speeds in the middle of the channel and around Balls Head. During the flood tide, a similar pattern is observed with currents significantly reduced upstream of the structure and a corresponding increase in the middle of the channel and along the northern bank (near Birchgrove Wharf)
- The north-east (Balls Head) cofferdam would have a very minor impact on current speeds during the ebb tide. This is because near the Coal Loader Wharf ebb current speeds are relatively low in both existing and cofferdam scenarios resulting in the structure not significantly affecting flow conditions. At the north-east cofferdam larger reductions in current speeds surround the cofferdam and the Coal Loader Wharf. This is because the structure interacts with the eddy that is set up in the entrance to Balls Head Bay during the flood tide
- During both ebb and flood tide the differences are more pronounced in the surface layer when compared to bottom layers
- At a location downstream of the Greenwich Baths the modelled increase in current speeds would occur during the flood tide only and would be most prominent during spring tides. At this location spring flood current speeds would increase from 0.36 m/s to around 0.41 m/s, a relative increase of 14 per cent.

8.1.2 Water quality impacts

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

- 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 >2 mg/L) are limited to a relatively small extent, concentrated at the north-eastern end of the dredging footprint near Balls Head Bay and around Balls Head. There are two main factors that influence the plume statistical distribution:
  - In this north-eastern part of the dredge footprint, current speeds would be low, particularly during the ebb tide allowing the sediment suspended by dredging to accumulate. In other

  - zones of the dredge footprint, stronger tidal currents disperse the sediment suspended by dredging
  - The BHD spends a considerable amount of time in this area (weeks 24 to 38) removing the deposit of sandy silty clay located adjacent to Balls Head.
- For the lower 90th percentile the dredge plume extents (SSC >2 mg/L) are limited to the area next to the shoreline along the western side of Balls Head, within the inner and eastern areas of Balls Head Bay and extend in front of Gore Cove to the tip of Green Island
- The extent of the visible plume (SSC >20 mg/L), when analysed over the entire dredge duration and viewed at the 95th percentile (surface layer), is very small and contained in the dredging footprint immediately adjacent to the north-east cofferdam
- The suspended sediment released during the dredging activity is transported both in an upstream and downstream direction, with a slight downstream dominance. This is a result of the tidal currents that are predominately aligned with the main longitudinal axis of the estuary. Suspended sediment is also transported into Balls Head Bay when dredging activity is being carried out on the eastern part of the dredge footprint
- For 50 per cent of the time of the dredging campaign the SSC would be less than 1 mg/L, even in the dredging footprint. This shows that for half the time the dredging occurs the increases in SSC are expected to be low everywhere
- The dredge plume extents would be greater in the bottom layer (layer one) than at the surface (layer five)
- Short duration (less than one per cent of the time) increases in SSC above 5 mg/L occur, but only in the area adjacent to the Coal Loader Wharf
- The relative influence of the dredge SSC to wind driven circulation was tested using the calibrated model. The results indicate sensitivity to wind is limited to brief periods at selected locations in the off-channel embayments
- The majority of the deposition due to the dredging activity occurs in the dredging footprint and adjacent to the dredging footprint. The deposition is concentrated at the north-eastern end of the dredging footprint, within the north-eastern portion of the dredging footprint and along the shoreline next to the Coal Loader Wharf. At this location sedimentation rates of greater than 35 mg/cm^2/day (or just over 1 mm/day)
- Lower levels of sedimentation also occur in Balls Head Bay and within the other embayments that line Sydney Harbour. This is expected given the lower tidal current speeds in these embayments (ie as demonstrated by they are depositional environments)
- Any deposition that occurs in areas with high bed shear stress (ie during slack water) is resuspended by tidal currents and reworked into nearby embayments
- The highest rate of deposition occurs during the period when the BHD is working in the sandy silty clay deposit adjacent to Balls Head
- The maximum and daily average deposition rates for the dredging within Balls Head Bay were less than the lowest thresholds noted in the literature.

8.2 Discussion

A number of environmental controls are proposed to be implemented as part of dredging operations. These controls 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 controls. Proposed environmental controls for mitigation of potential turbidity impacts are summarised below.

In regard to the dredging by the BHD:

- Actual dredging production hours are 7.5 hours per day, five days per week
- An environmental and/or closed bucket would be used for all suitable material (ie all other than rock (OTR) 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 while 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 rock crushed by the CSD, 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 and leads to lower production rates
- BHD dredging operations would be completed within a floating silt curtain enclosure (or 'moon pool') that is secured to the dredge barge. This would comprise a fixed or floating boom upon which a shallow-draft (two to three metres deep) silt curtain is attached to provide a controlled area for the dredge operator to work within
- A BHD has been assumed to dredge the sandy silty clay sediment unit located next to the bank on the Balls Head side of the dredge footprint. If a TSHD is used here the production rates would likely be higher. However, while a BHD increases the time needed for dredging, it lowers the overall quantity of fines released to the water column as well as the rate of release. Hence the BHD is considered appropriate from an environmental perspective.

In regard to the dredging by the TSHD:

- Medium-sized TSHD used for OTR material considered suitable for offshore disposal
- No overflow when loading hopper. As well as reducing losses to the water column this also leads to lower production rates with only 25 minutes of dredging in every five to six hour cycle
- The draghead on a TSHD can use water jets to loosen sediment to be dredged. However, the use of this technique is not permissible as it leads to additional release of sediment to the water column
- TSHD would work continuously for 6.5 days (or 156 hours) per week
- Use of silt curtains is not possible for this type of equipment.

In regard to the dredging by the CSD:

- Actual dredging hours of 6.5 hours a day, five days per week
- Crushing of rock by CSD would be completed in 'no pumping' mode to minimise dispersal of rock fines into the water column that may be caused by hydraulic dredging techniques (eg side casting). This means that the BHD would need to remove the material left by CSD before the next CSD layer can be dredged. This dredging approach has been adopted as it is considered an appropriate environmental approach. This is because it eliminates a second source point under or

near the CSD where all fines are brought into suspension near the seabed at a distance of 20 to 150 metres from the CSD cutterhead

- Frequent shifting from northern to southern rock area after or during crushing of rock layers has been taken in account to reduce local turbidity
- Use of deep draft (12 metre) silt curtains is not possible, due to tidal currents and navigation traffic
- Removal of crushed rock by BHD to take place while CSD operates at other rock areas (ie north, south or middle of trench).

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 protection would be provided around sensitive areas of aquatic vegetation (ie seagrass beds) adjacent to the dredging footprint. These would be further protected with localised floating silt curtains. These measures have not been included in the hydrodynamic or dredge plume modelling.

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.

Birch, G. F., and Rochford, L. (2010) S*tormwater metal loading to a well mixed / stratified estuary (Sydney Estuary, Australia) and management implications.* Environmental Monitoring and Assessment 169(1–4), 531–551.

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.

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.

Rochford, L., 2008. Stormwater heavy metal loading to Port Jackson estuary, NSW, Australia. M.Sc. Thesis (unpubl) School of Geosciences, University of Sydney.

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.

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

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

Appendix 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 type instrument. Additionally one of the in situ monitoring site 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 approximately 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 seabed sediment samples 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 was between 17 August 2017 and 1 November 2017, totalling a monitoring period of 76 days. Servicing of the instruments took place mid-term on 21 September and 6 October 2017. RHDHV have further serviced and redeployed all monitoring sites on 1 November 2017.

Details of the coordinates, 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 Aug 2017	Deployment of WH1, and WH2	Polaris Marine	H. Loehr
21 Aug 2017	ADCP transects/sediment sampling	Geochemical Assessments	H. Loehr
21 Sep 2017	Servicing and redeployment of WH1	Polaris Marine	H. Loehr
6 Oct 2017	Servicing and redeployment of WH2	Polaris Marine	H. Loehr
1 Nov 2017	Servicing and redeployment of WH1 and WH2	Polaris Marine	H. Loehr

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

Table 9–2: Coordinates of in situ monitoring locations during the first deployment period.

Site name	Coordinates (WGS84)		Instrument	Parameters
	Marker buoy	Instrument		
WH1	-33.84325, 151.18584	-33.84344, 151.18617	ADCP V50	Currents, water level, temperature
WH2	-33.84175, 151.19039	-33.84168, 151.19058	ADCP V20/ Hydrolab	Currents, water level, temperature, water quality

Table 9–3: Coordinates of in situ monitoring locations during the second deployment period.

Site name	Coordinates (WGS84)		Instrument	Parameters
	Marker buoy	Instrument		
WH1	-33.84325, 151.18583	-33.84346, 151.18587	ADCP V20/ Hydrolab	Currents, water level, temperature, water quality
WH2	-33.84167, 151.19045	-33.8415, 151.19061	AWAC	Currents, water level, temperature

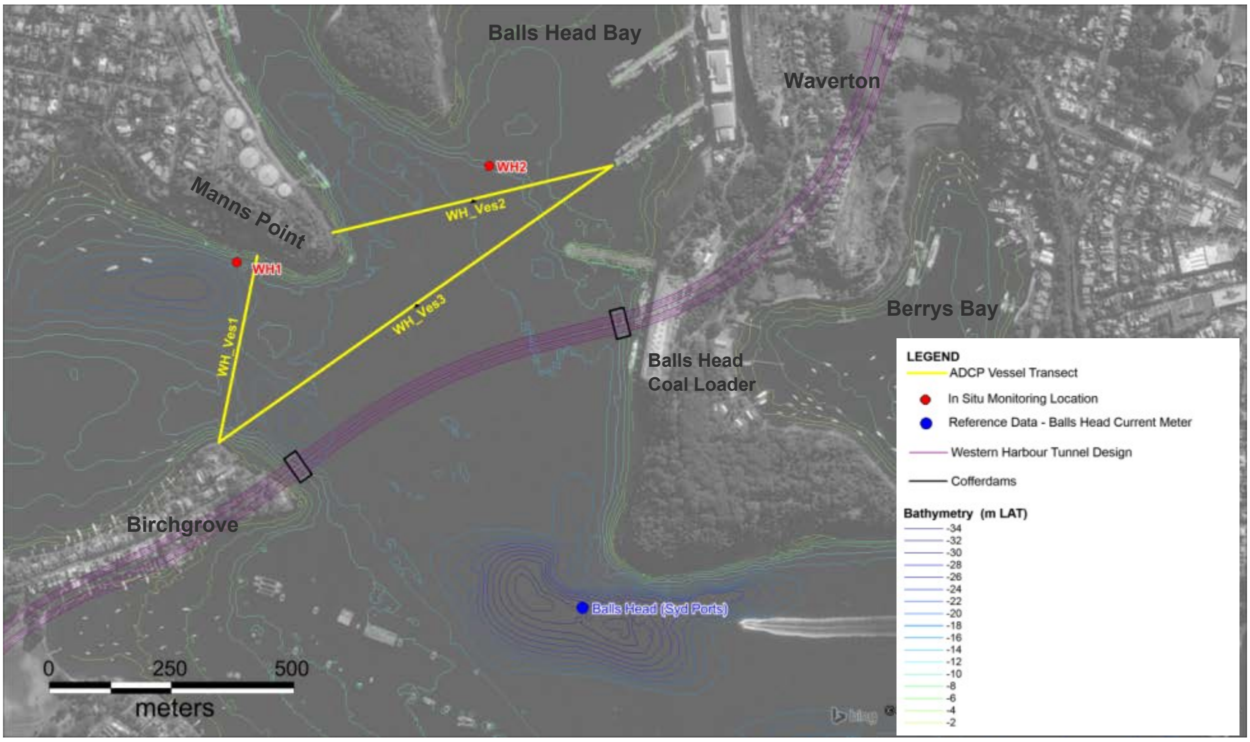
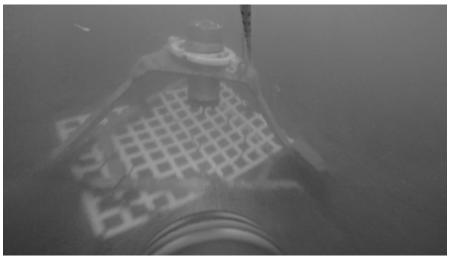


Figure 9-1: Map showing the hydrodynamic and water quality monitoring locations at the project crossing location.



Underwater photograph of bottom-mounted ADCP at WH1

Water quality sonde during deployment at WH2

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	WH1 and WH2
Operation frequency	500 kHz/1000 kHz
Mounting (see Appendix B for typical diagrams)	Upward looking from seabed
Approx. depth (relative to AHD)	13, 11 and 11 m
Vertical resolution (bin size)	1 m
Blanking distance	0.4 m
Current measurement interval	10 minutes
Current averaging interval	120 seconds

Turbidity sonde setup

Turbidity measurements are 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 seabed. All parameters are 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.

Parameter description	Hydrolab DS5
Mounting	Sensor approximately 2 m above seabed
Measurement interval	30 minutes

ADCP vessel transects

A vessel mounted ADCP was used to map current velocities along predefined transect lines at the Western harbour region on 21 August 2017. The three predefined transects (see Figure 9-1 and Table 9–6) was 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 21 and 22 August 2017.

Transect ID	Length (m)	Date
WH_Ves1	320	21 August 2017
WH_Ves2	814	21 August 2017
WH_Ves3	487	21 August 2017

Sediment sampling

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

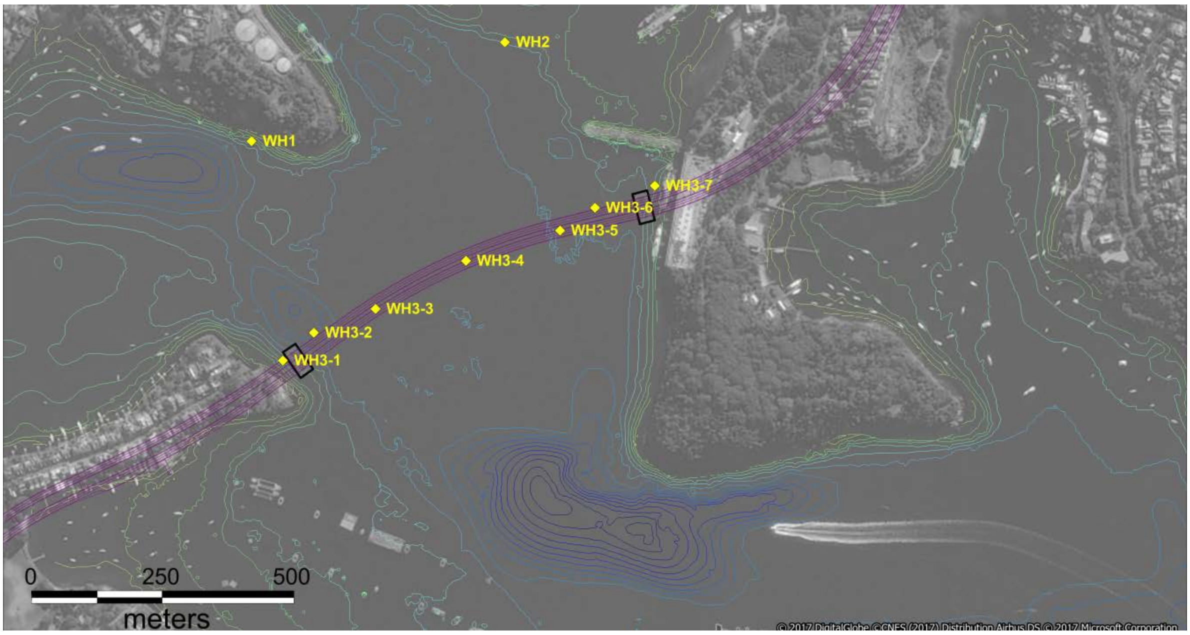


Figure 9-3: Location of sediment samples.

Quality Assurance and Data Processing

All data has 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 84.9 per cent while the average data return of good data was 84.6 per cent (Table 9–7). The ADCP instrument at site WH1 malfunctioned during the first deployment period due to a corroded temperature sensor and was later replaced with a Nortek AWAC Doppler profiler. Replacing the instrument with a spare took two weeks, which is reflected in WH2 data capture rate.

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

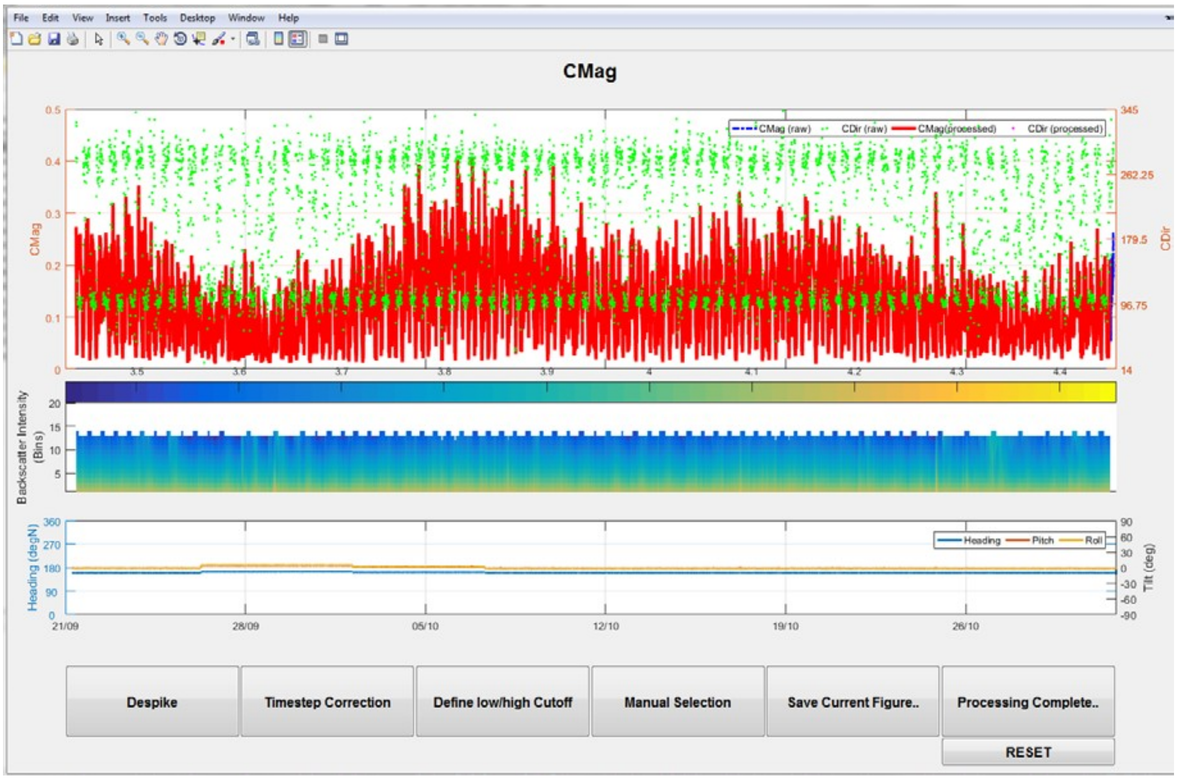


Figure 9-4: RHDHV in situ data QA/QC processing tool used to remove erroneous field data.

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 (%)
WH1	Currents	76	59.2	59.0
	Turbidity	41	100.0	99.7
WH2	Currents	76	80.3	80.0
	Turbidity	35	100.0	99.8
Overall data return		Total:	84.9	84.6

In situ measurements

Time series plots

Time series plots of water level, current speed and current direction for the in situ sites at WH1 and WH2, can be seen on Figure 9-5 and Figure 9-6. Concurrent wind data from the Bureau of Meteorology (Sydney Observatory Hill site) is also presented in these plots.

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

Currents 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 provided 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. An example is provided in Figure 9-10 for site WH1 for three periods:

- Period 1: Peak flood during spring tide conditions on 6 October 2017
- Period 2: Peak ebb during spring tide conditions on 6 October 2017
- Period 3: Low flood currents during neap tide conditions coinciding with a moderate (9 m/s) westerly wind on 27 September 2017.

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.

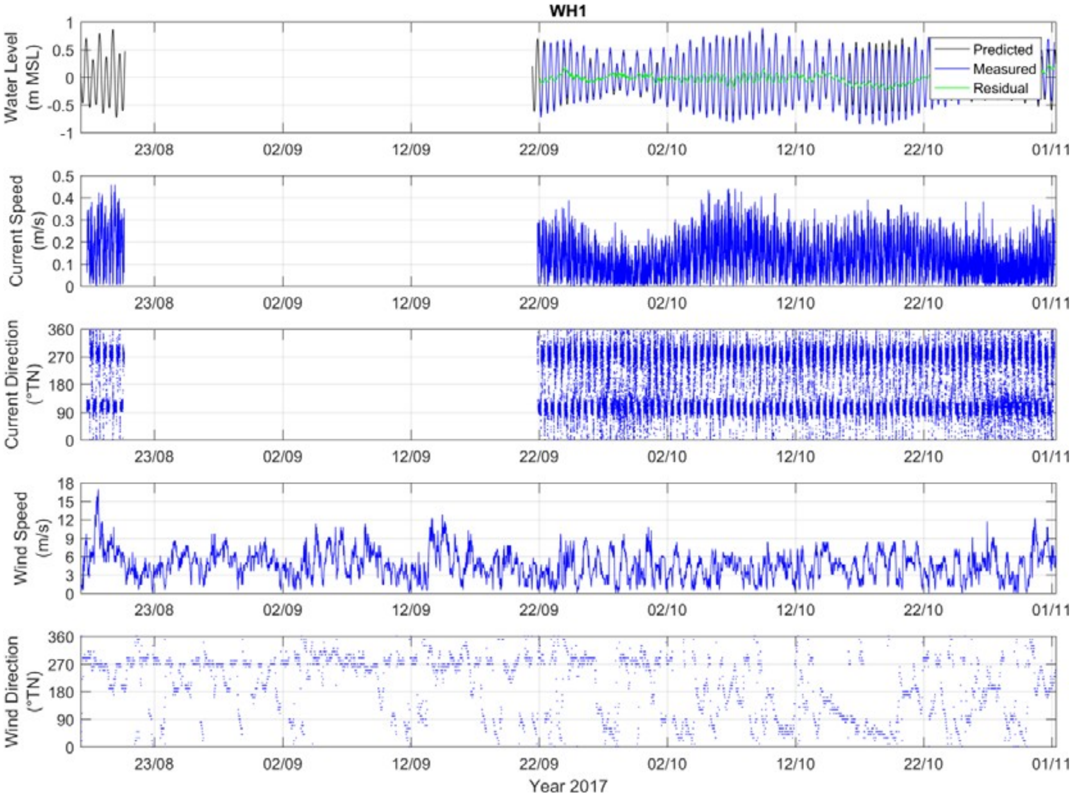


Figure 9-5: Time-series of measured metocean data at site WH1 complemented with measured wind data from BoM.

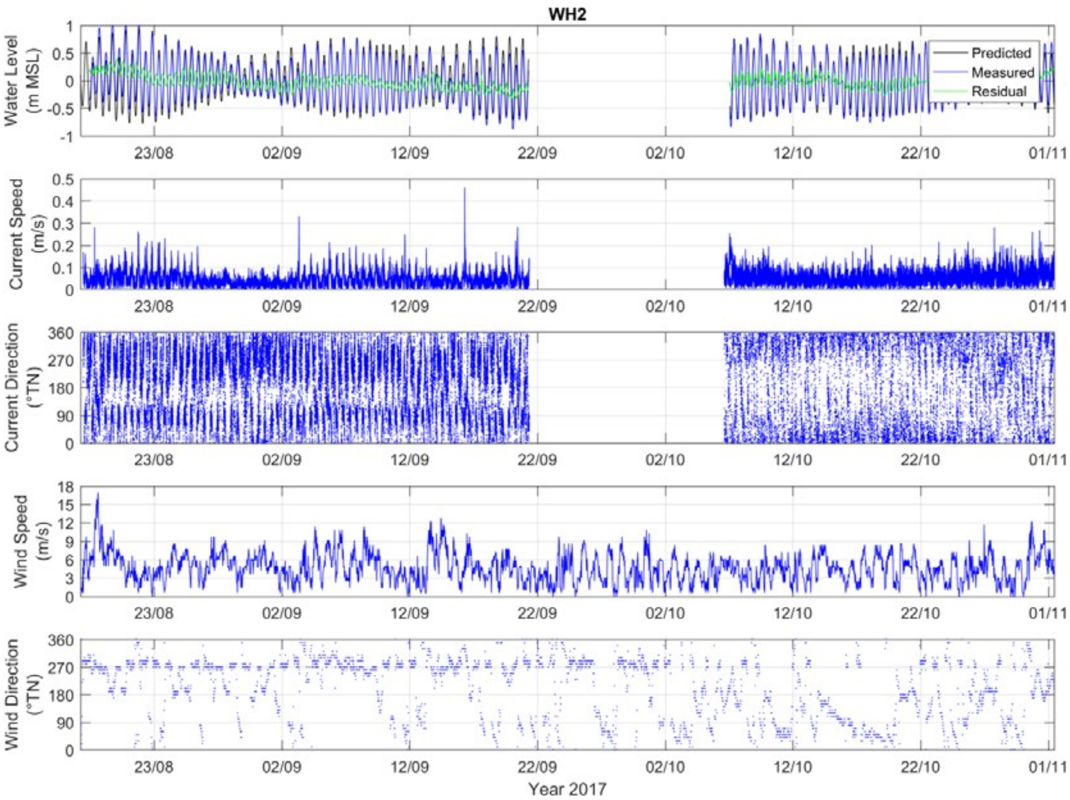


Figure 9-6: Time-series of measured metocean data at site WH2 complemented with measured wind data from BoM.

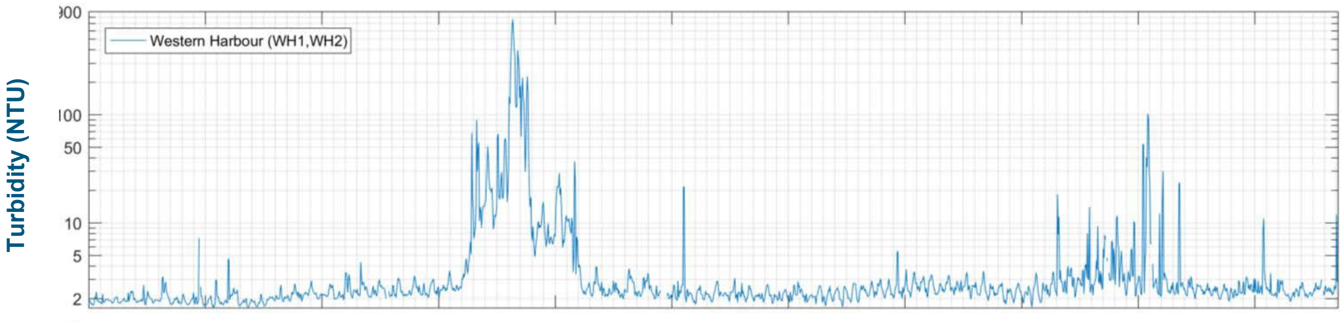


Figure 9-7: Time series of measured turbidity at the Western Harbour region.

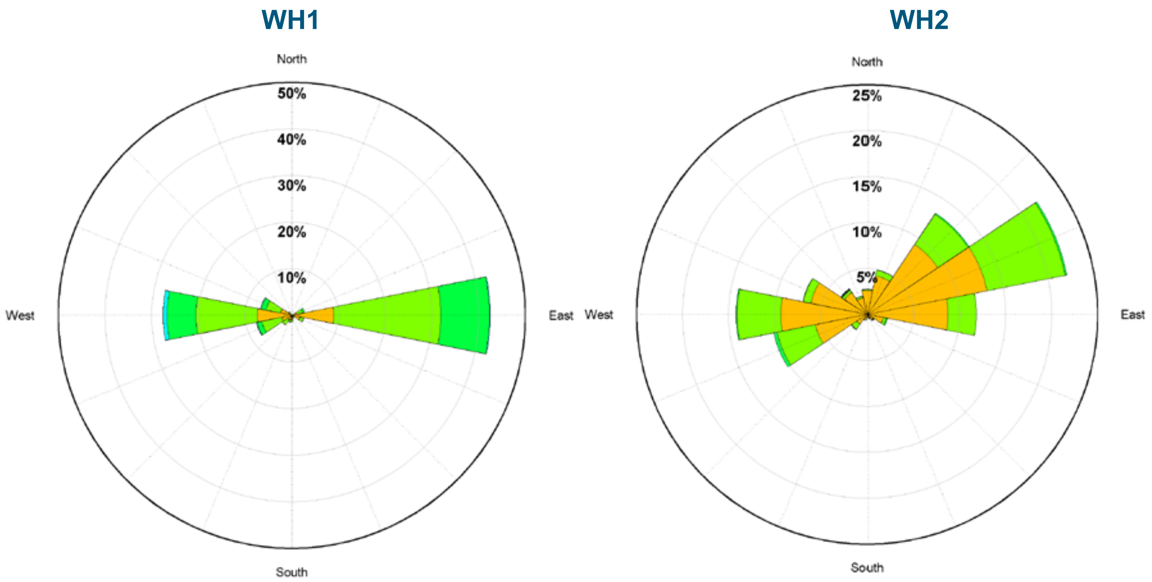


Figure 9-8: Rose plots showing current speed and direction during the deployment period at the two in situ sites.

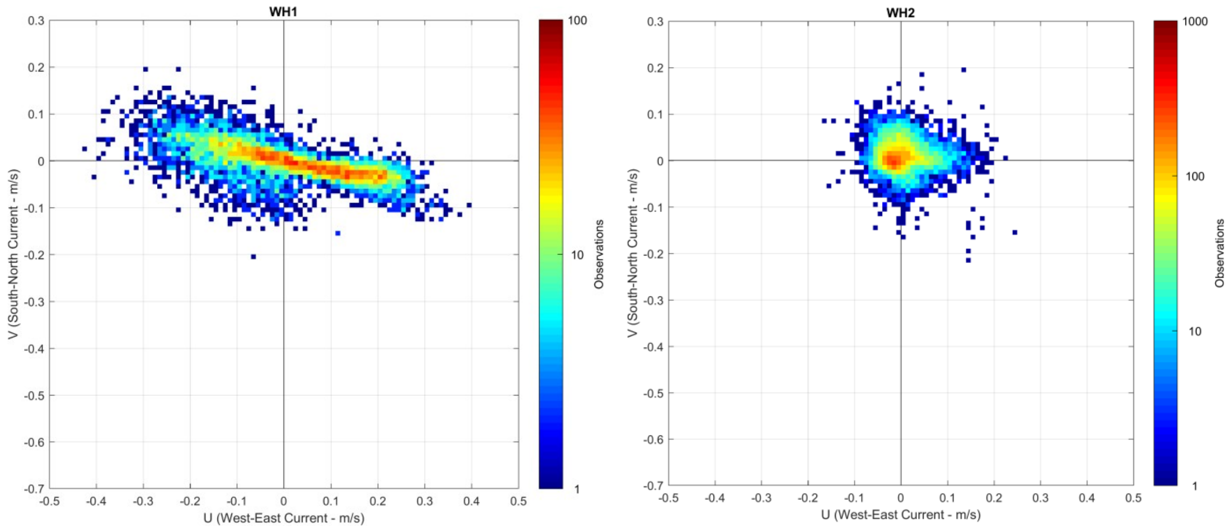


Figure 9-9: Current speed vs current direction at (clockwise from top left) WH1 and WH2.The colour code demonstrates the number of observations.

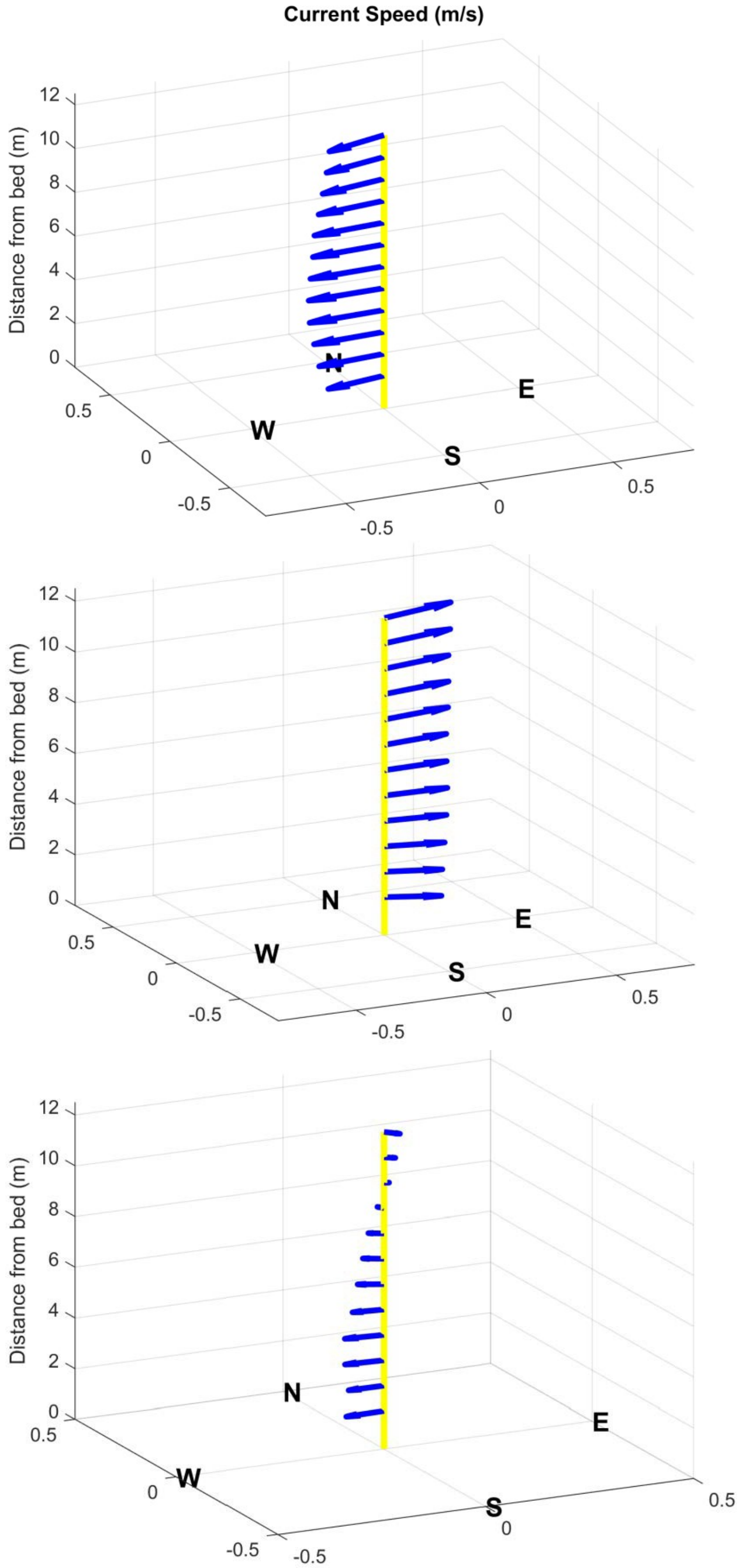


Figure 9-10: Velocity profile at site WH1 during peak flood (top), peak ebb (middle) and during a period of low tidal current coinciding with high wind speeds.

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 undertaking 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	WH1	WH2
Flood current speed (m/s)	Maximum	0.43	0.18
	95th percentile	0.27	0.08
	Mean	0.12	0.04
Ebb current speed (m/s)	Maximum	0.41	0.29
	95th percentile	0.26	0.12
	Mean	0.13	0.05

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[5].

Tidal plane	Fort Denison water level (m AHD)*	WH1 water level (m MSL)	WH2 water level (m MSL)
Mean high water springs (MHWS)	0.69	0.66	0.59
Mean high water (MHW)	0.56	0.51	0.46
Mean high water neaps (MHWN)	0.44	0.37	0.32
Mean sea level (MSL)	0.01	-0.01	0.00
Mean low water neaps (MLWN)	-0.39	-0.39	-0.31
Mean low water (MLW)	-0.51**	-0.54	-0.45
Mean low water springs (MLWS)	-0.64	-0.69	-0.58

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

[5] *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 measured at WH1 and WH2 is provided in Table 9–10 and Table 9–11. Table 9–11 accounts for the turbidity event that occurred at WH2 throughout 9–16 September 2017. The turbidity values during this period were abnormally high but the trend looks like a natural response. No high rainfall or wind occurred over this period, however shipping movements have not been checked. These readings have been included here as there was no compelling evidence of fouling or instrument error observed. However, it is noted that the possibility that the high readings are erroneous cannot be excluded. They have been conservatively retained in the quality controlled dataset. It is recommended that they be compared to other concurrent turbidity data from adjacent areas should such data be available.

Additionally, a time series plot of turbidity was presented in Figure 9-7.

Table 9–10: Summary of measured turbidity statistics at WH1 and WH2.

Statistic	WH1	WH2
Minimum	1.7	1.6
Maximum	101.5	760.6
Mean	3.2	12.7
Median	2.3	2.2
Standard deviation	5.6	55.3
5th percentile	1.9	1.8
99th percentile	23.0	306.3
Data length (days)	41	35

Table 9–11: Summary of measured turbidity data factoring the turbidity event that occurred from 9–16 September 2017.

Statistic	WH2*	
	Event only	Excluding event
Mean	50.3	2.2
25th percentile	6.9	1.9
50th percentile	11.9	2.1
75th percentile	29.3	2.4
90th percentile	143.8	2.8
95th percentile	286.8	3.0
99th percentile	627.0	3.7
99.5th percentile	718.3	4.4
Maximum	760.6	7.2
Data length (days)	7.5	27.2

*No data was recorded at WH1 during the specified event

ADCP vessel transects

Current velocities

The ADCP vessel transects results are presented on 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 WH_Ves1 and WH_Ves3.

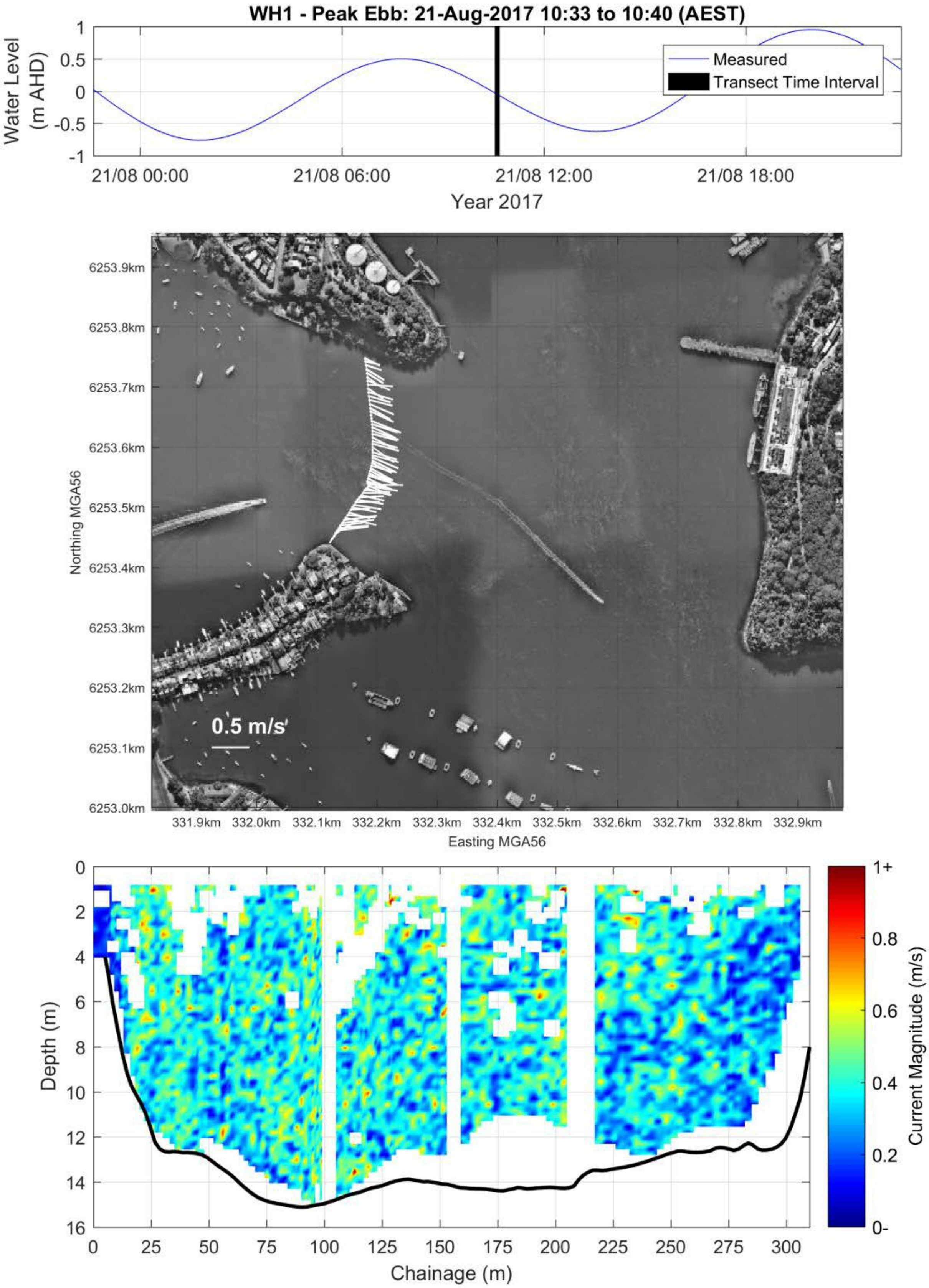


Figure 9-11: Measured spatial currents along WH-Ves1 transect during peak ebb conditions on 21 August 2017.

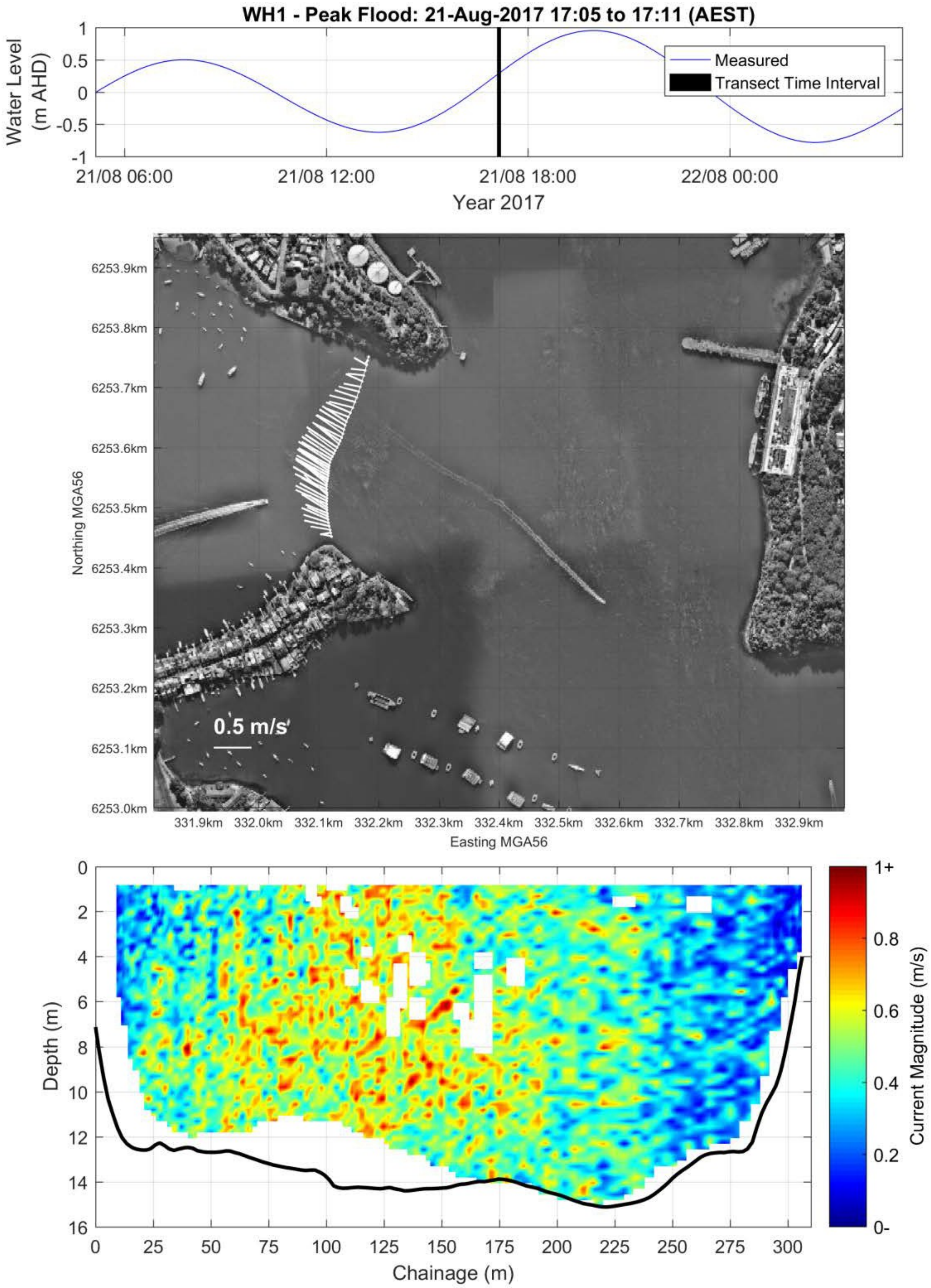


Figure 9-12: Measured spatial currents along WH-Ves1 transect during peak flood conditions on 21 August 2017.

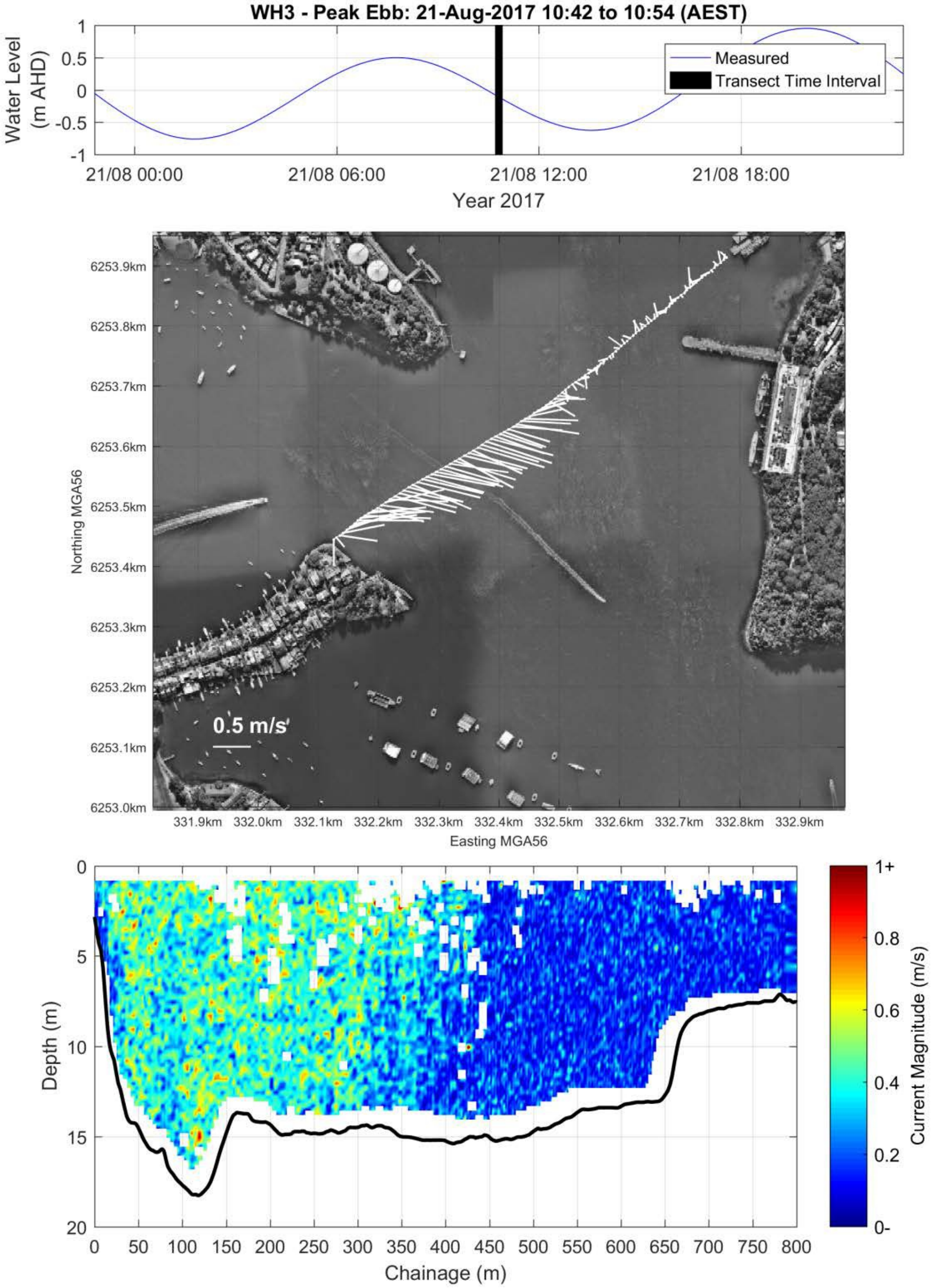


Figure 9-13: Measured spatial currents along WH-Ves3 transect during peak ebb conditions on 21 August 2017.

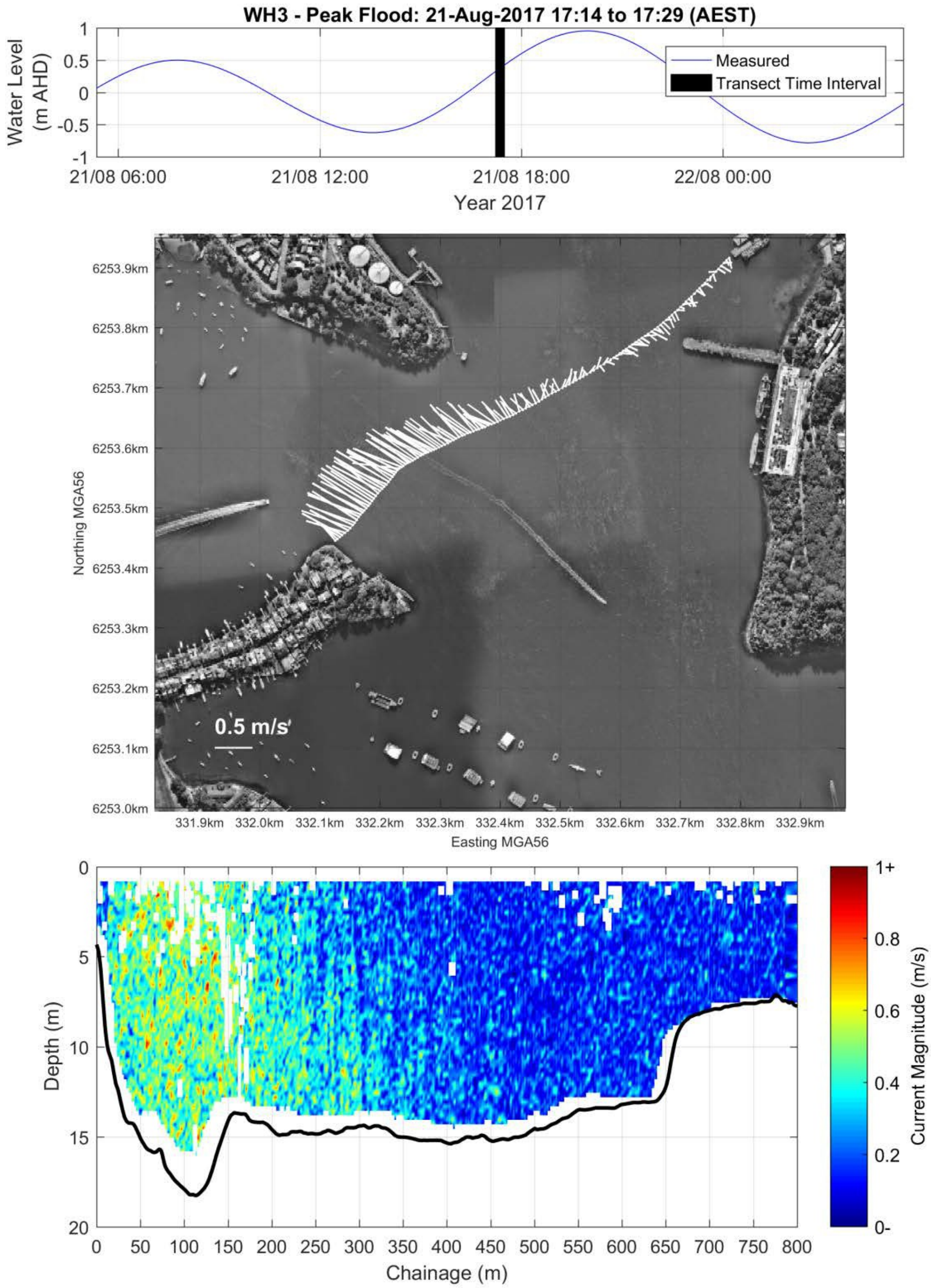


Figure 9-14: Measured spatial currents along WH-Ves3 transect during peak flood conditions on 21 August 2017.

Discharge

The measured discharge at the predefined transect locations (locations shown on Figure 9-1) is presented on 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 9–12 .

Table 9–12: Maximum ebb and flood discharges from the ADCP vessel transects completed on 21 August 2017.

Transect ID	Max. measured discharge (m^3/s)	
	Ebb (+ive)	Flood (−ive)
WH_Ves1	1250	1770
WH_Ves2	51	87
WH_Ves3	1390	1751

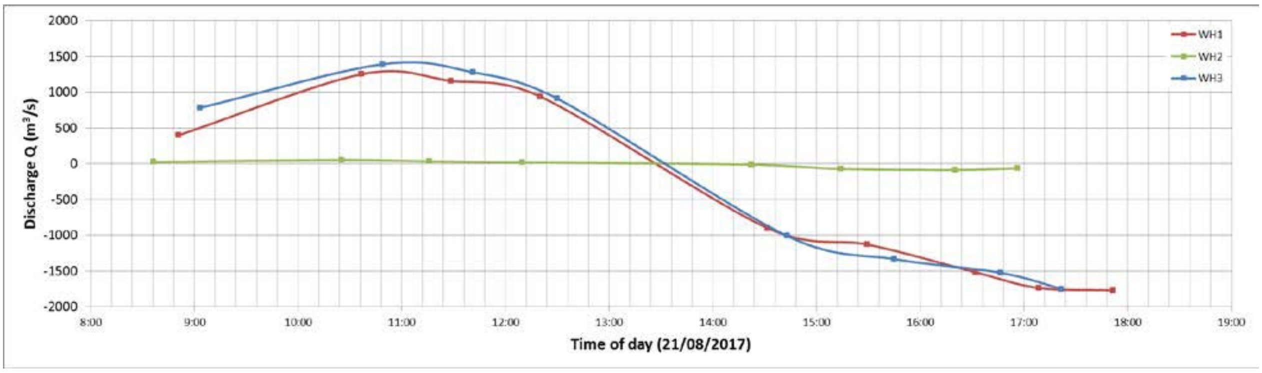


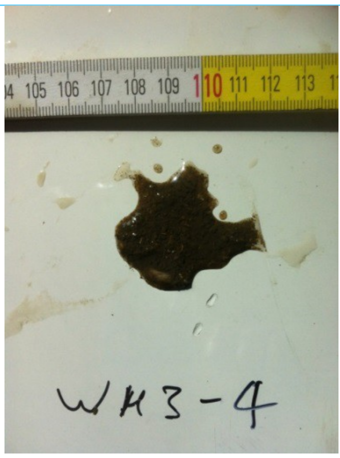
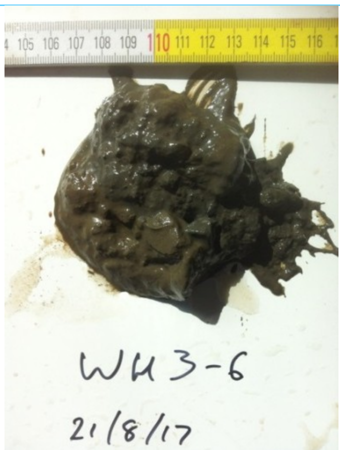
Figure 9-15: Measured discharge at transect WH1, WH2 and WH3 on 21 August 2017.

Sediment Samples

Geochemical Assessments took a photo and characterised each sample (see Table 9–13). Based on this characterisation a subset 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 9–14).

Table 9–13: Sediment samples and description

Sample ID	Photo	Description
WH 2		(Gravelly), sandy mud. Grey - green. Gravel component consists of shell fragments.
WH3-1		Gravel/pebbles/cobbles and trace sand. Lithic fragments to 7 cm long. Shell and shell fragments.
WH3-2		Gravelly, (muddy) sand. Grey-green. Gravel component consists of rock and shell fragments. Medium grained quartzose sand.
WH3-2	No photo.	Effectively no sample (no photograph). About 1 g of medium grained sand.

WH3-4		Very small sample. Muddy, sandy gravel. Gravel component consists of shells and shell fragments.
WH3-5		Sandy mud with trace gravel (<20% sand). Grey-green, but brown when oxidised. Very soft.
WH3-6		Sandy mud (<10% sand). Grey-green. Soft, with firm patches.

WH3-7		Gravelly, sandy mud. ~20% gravel and 30% mud. Grey-green. Gravel consists of coal fragments to 2 cm long.

Table 9–14: Detailed grain size analysis for the four representative samples identified by Geotechnical 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 (%)
WH2	40.4	7.9	32.1	16.1	1.7	0.7	0.9	0.2
WH 3-2	4.3	1.1	15.2	50.9	21.6	1.4	1.2	4.3
WH 3-5	65.1	6.4	17.6	9.5	0.8	0.3	0.1	0.2
WH 3-6	88.8	4.5	4.8	1.3	0.2	0.1	0.3	0.0
WH 3-6 Dup	88.5	4.7	4.8	1.5	0.3	0.1	0.1	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 Leichhardt 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.

Appendix B – Dredge Plume Results

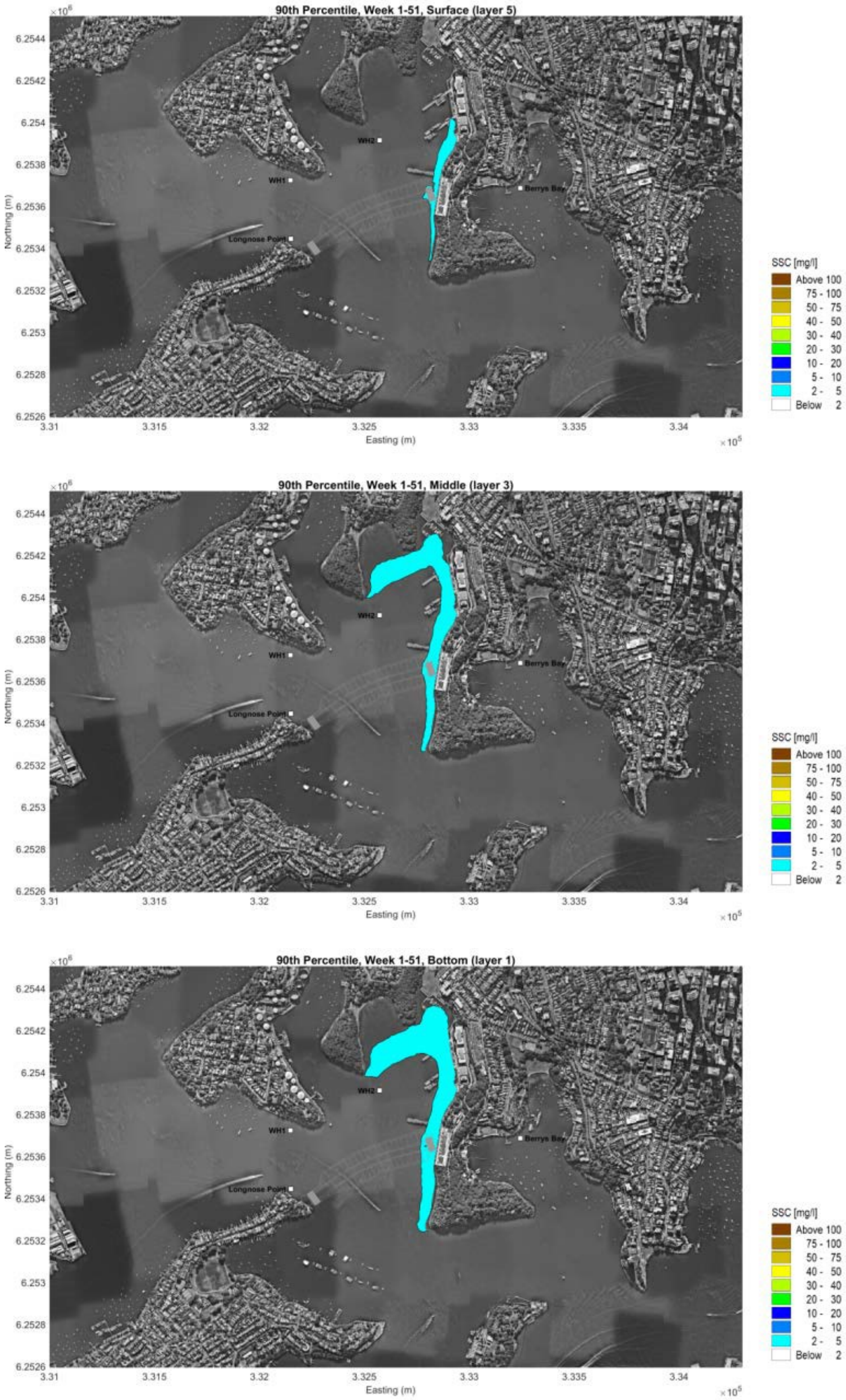


Figure 9-16: 90th percentile, for surface (top), mid-water column (middle) and near the seabed (bottom) for the entire WHT dredging period (weeks 1 to 51).

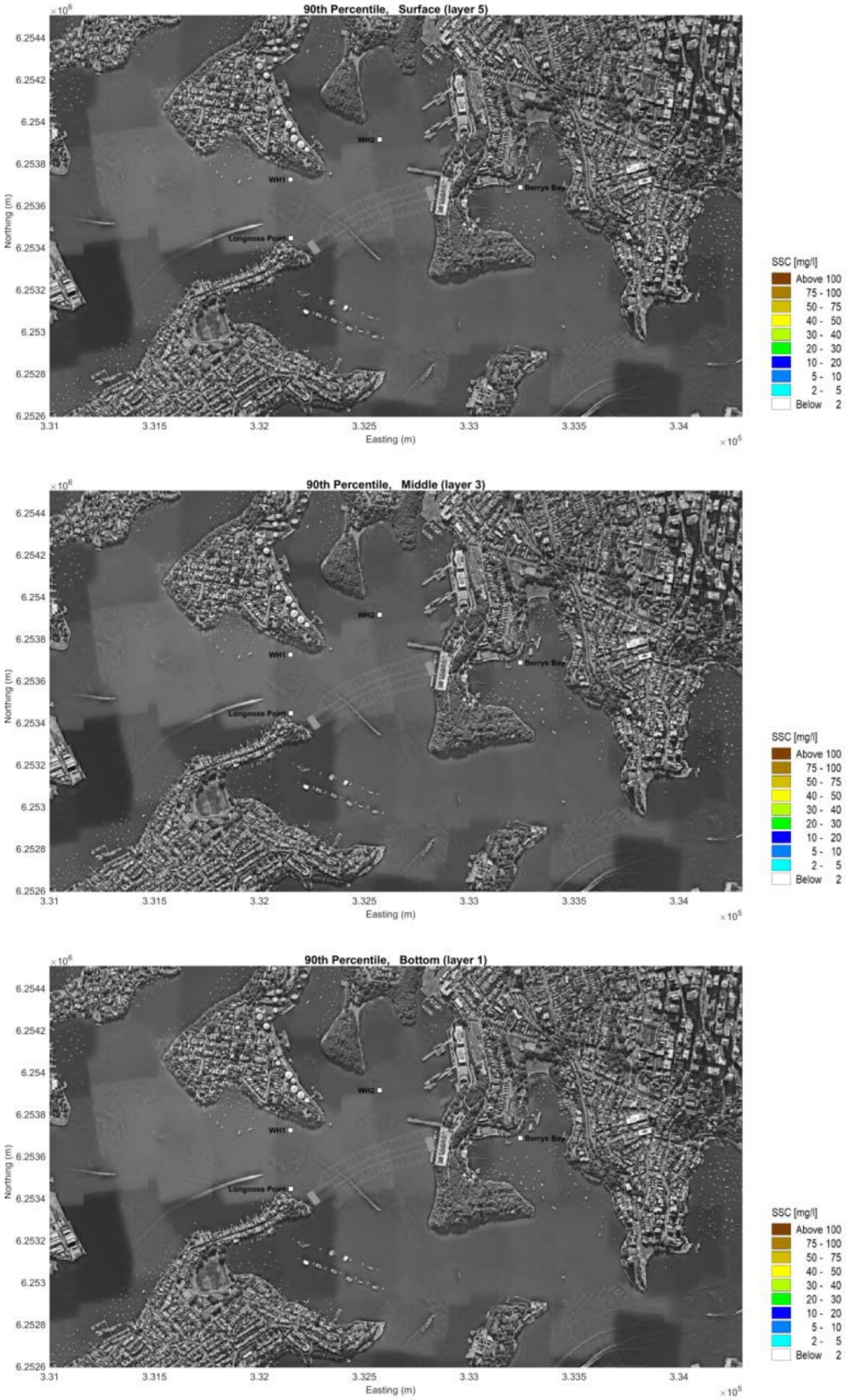


Figure 9-17: 90th percentile, for surface (top), mid-water column (middle) and near the seabed (bottom) for the entire WHT dredging period (weeks 1 to 15).

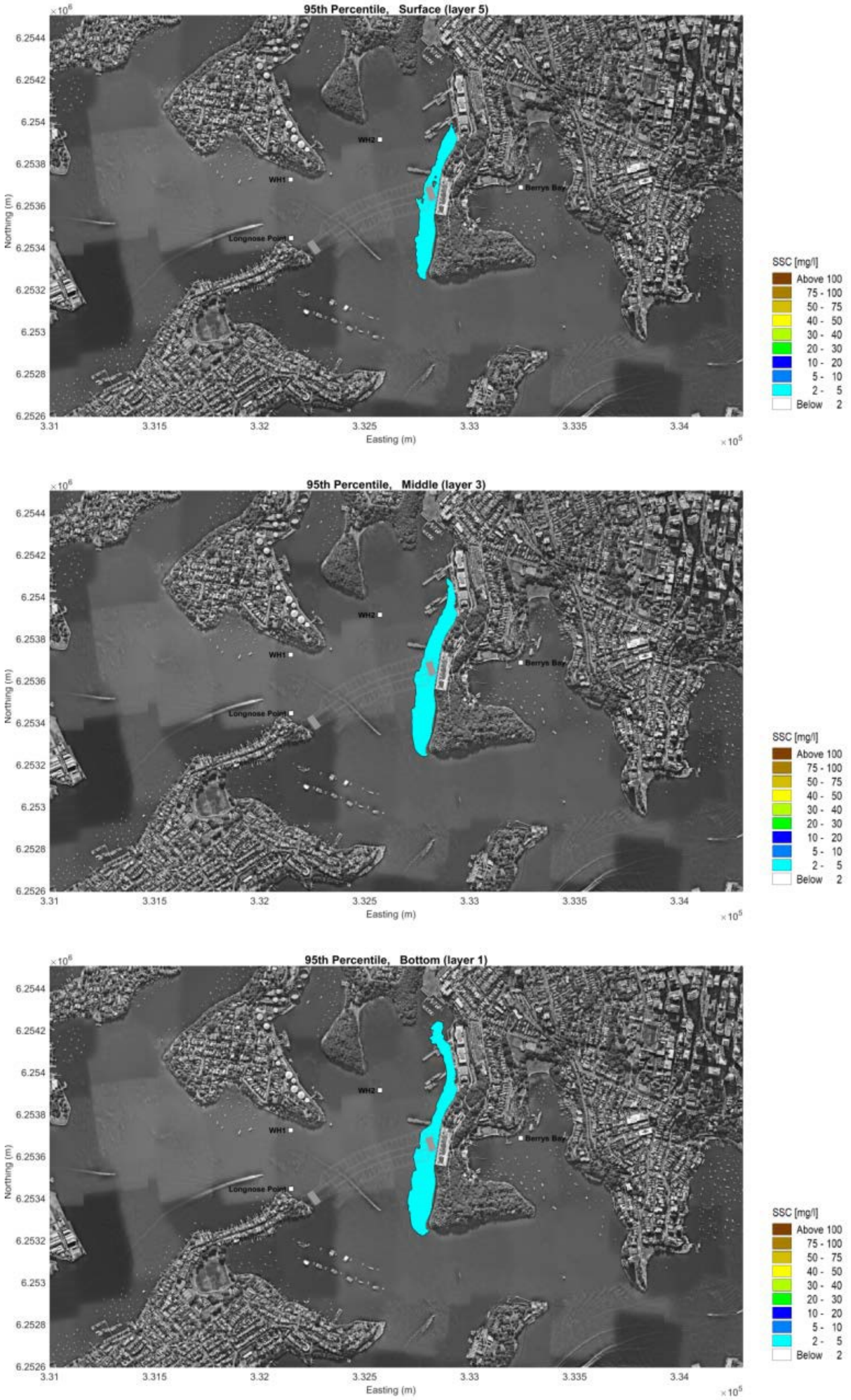


Figure 9-18: 90th percentile, for surface (top), mid-water column (middle) and near the seabed (bottom) for the entire project dredging period (weeks 1 to 15).

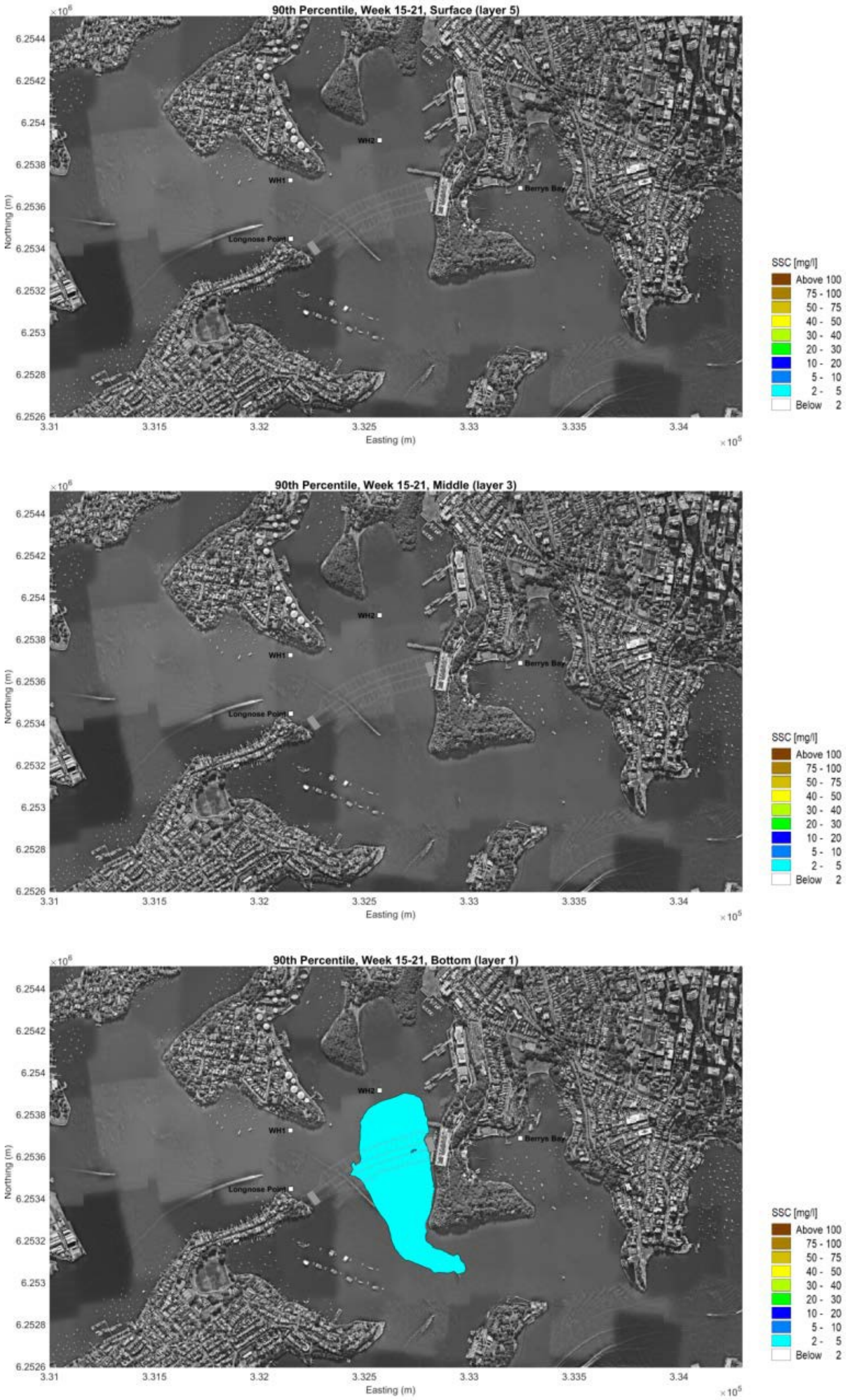


Figure 9-19: 90th percentile, for surface (top), mid-water column (middle) and near the seabed (bottom) for the entire project dredging period (weeks 15 to 21).

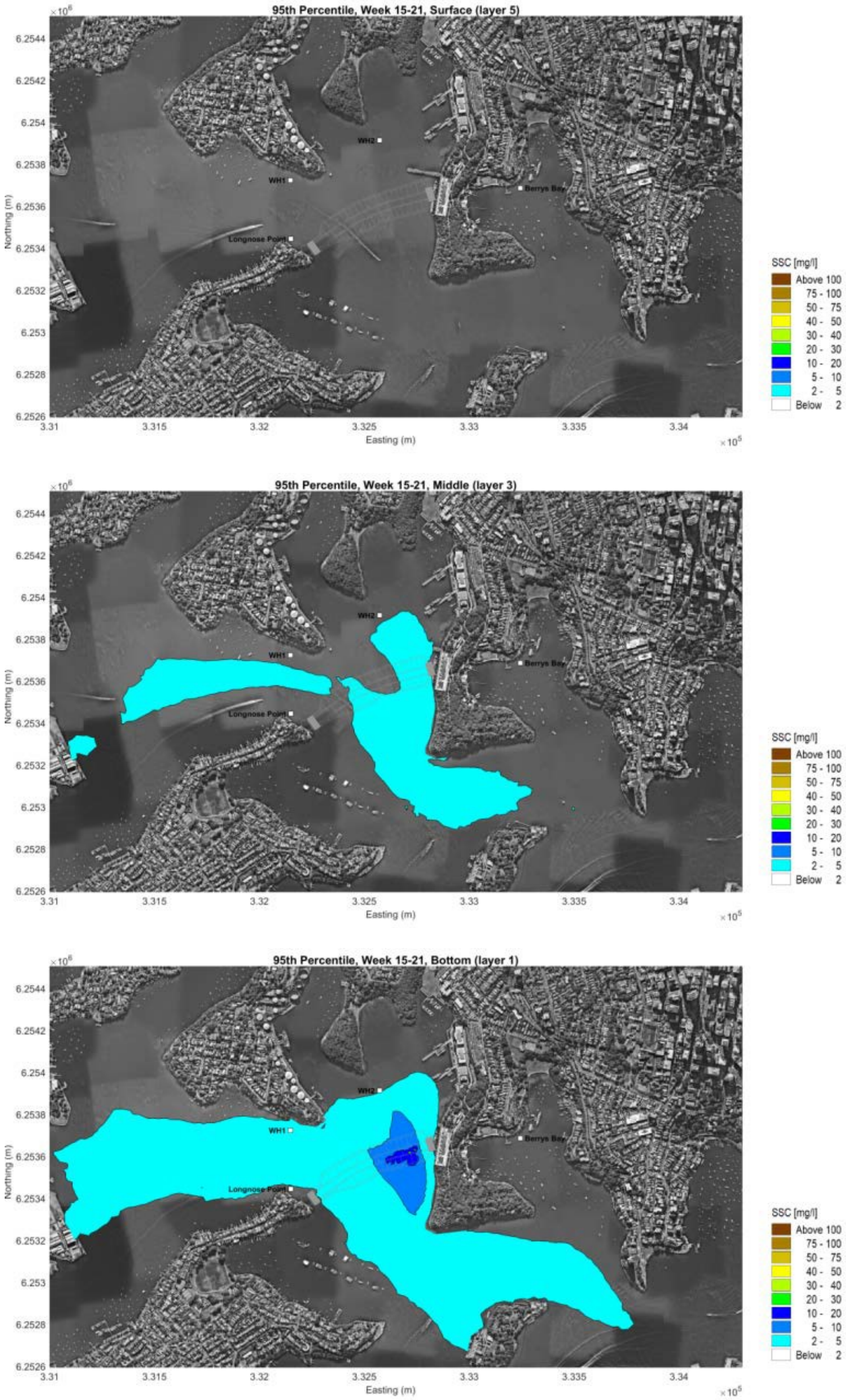


Figure 9-20: 95th percentile, for surface (top), mid-water column (middle) and near the seabed (bottom) for the entire project dredging period (weeks 15 to 21).

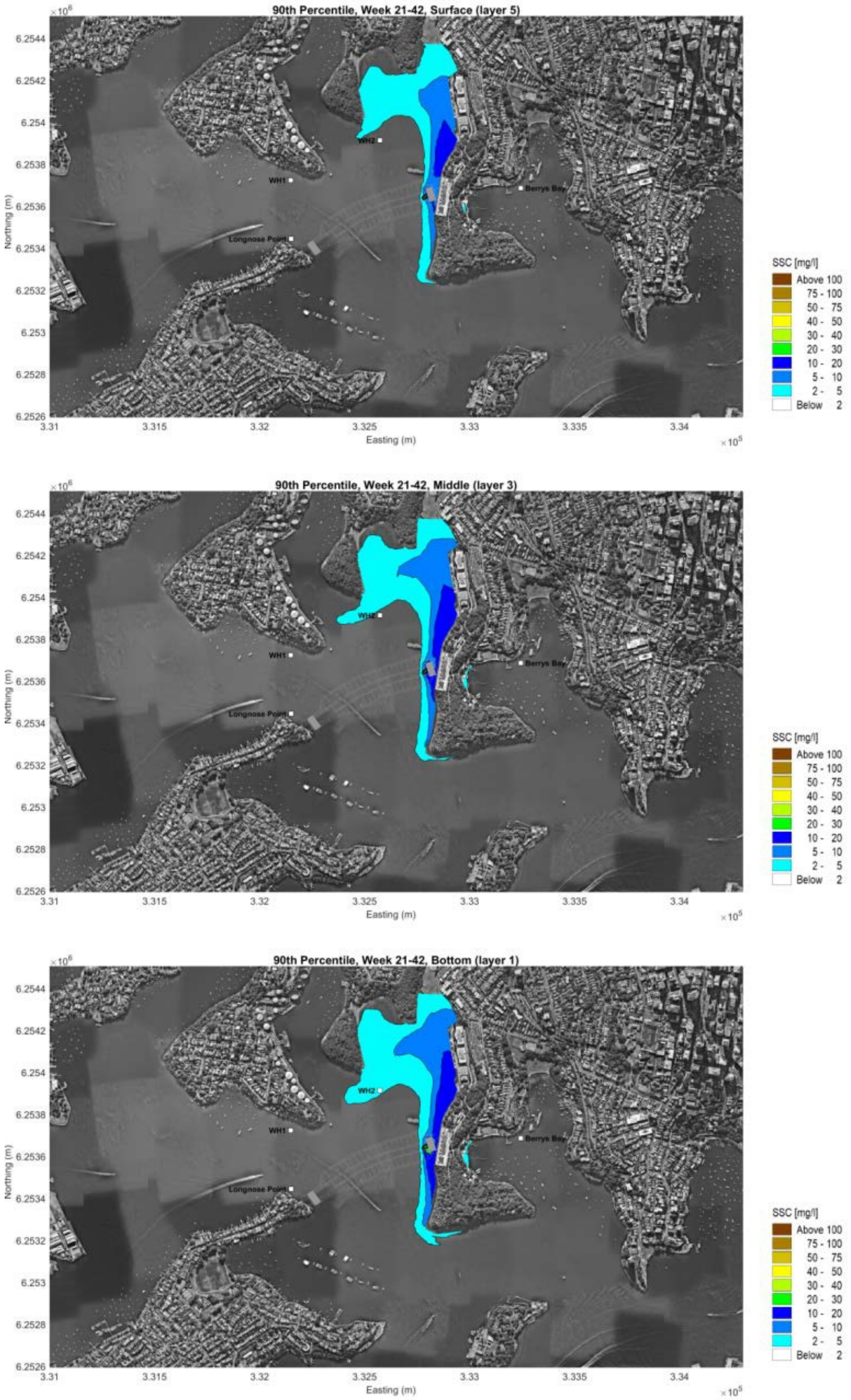


Figure 9-21: 90th percentile, for surface (top), mid-water column (middle) and near the seabed (bottom) for the entire project dredging period (weeks 21 to 42).

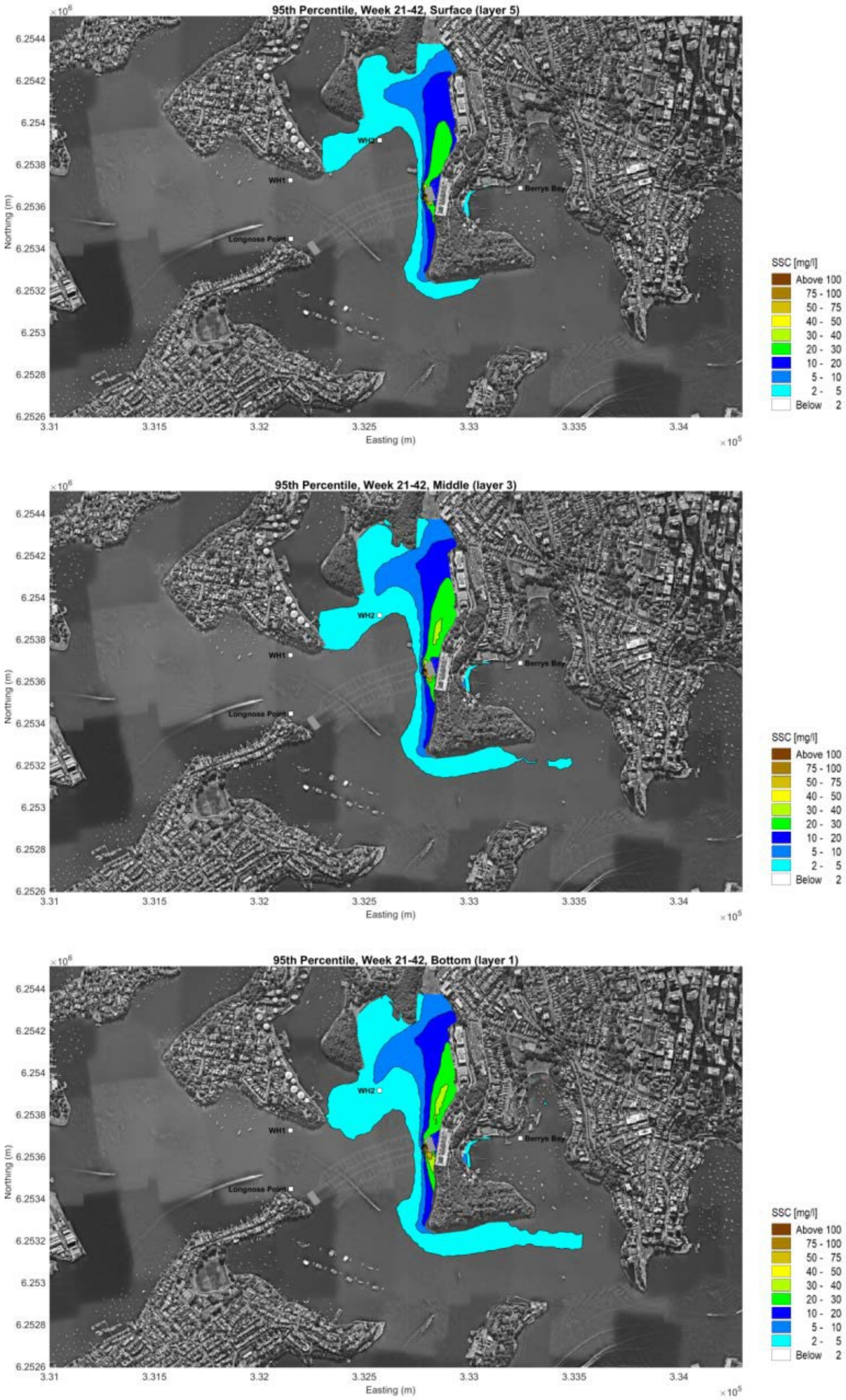


Figure 9-22: 95th percentile, for surface (top), mid-water column (middle) and near the seabed (bottom) for the entire project dredging period (weeks 21 to 42).

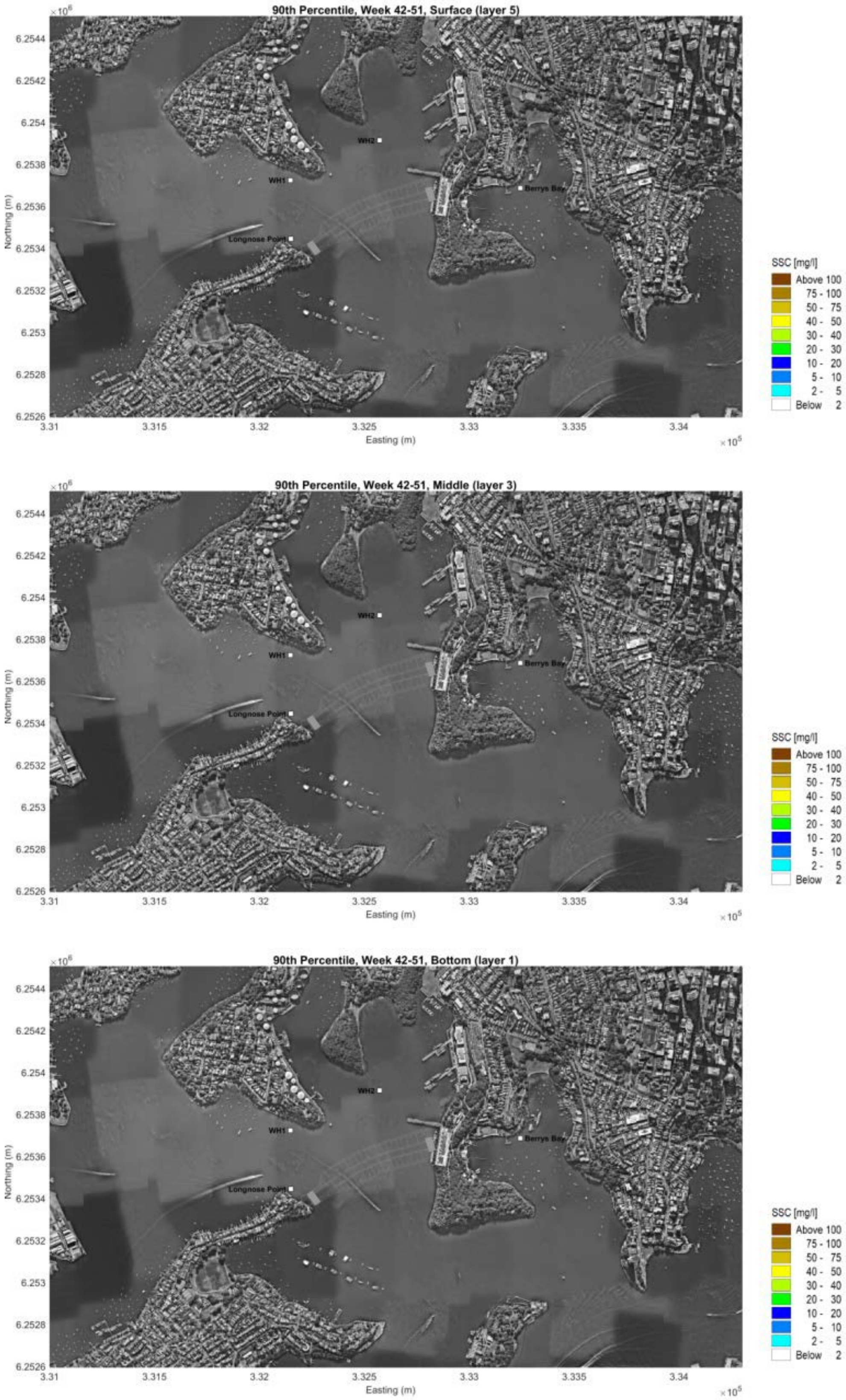


Figure 9-23: 90th percentile, for surface (top), mid-water column (middle) and near the seabed (bottom) for the entire project dredging period (weeks 42 to 51).

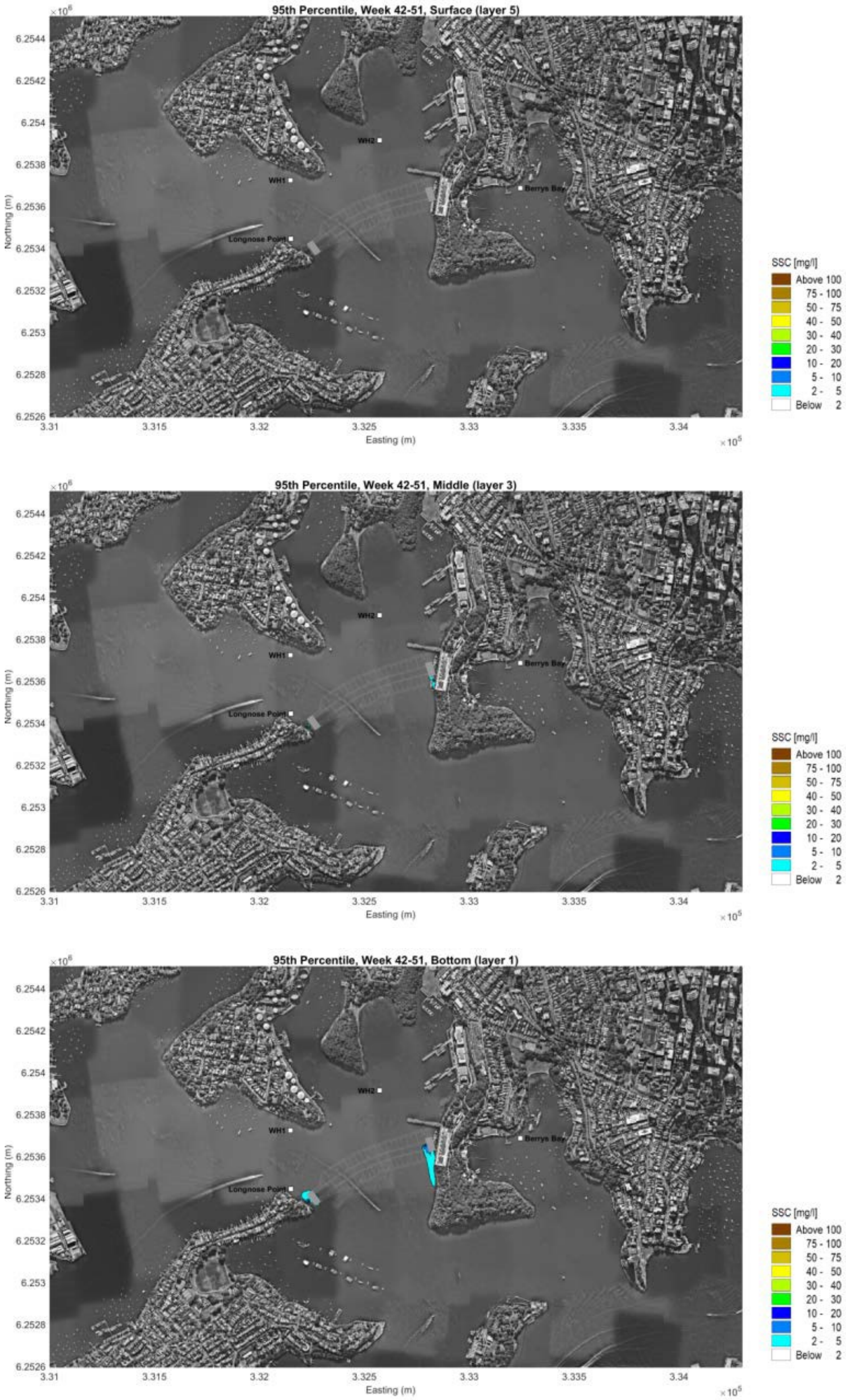


Figure 9-24: 95th percentile, for surface (top), mid-water column (middle) and near the seabed (bottom) for the entire project dredging period (weeks 42 to 51).

RMS 19.1441
ISBN 978-1-922338-08-2