Woollahra Municipal Council

Double Bay - Hydrogeological Geotechnical Impacts

Groundwater and Geotechnical Assessment Report

June 2020

Executive summary

This report describes the findings of a hydrogeological and geotechnical study for assessing short and cumulative long term impacts associated with excavation, subterranean building and dewatering in the Double Bay area on the structural and geological integrity of Double Bay residential and commercial buildings. The study comprised three main components, namely, (i) identification of groundwater study area and its groundwater setting; (ii) assessment of potential cumulative impact of future developments on long-term groundwater change; and (iii) assessment of short term construction dewatering on risks of damage of adjacent buildings.

The identified groundwater study area within Double Bay is situated in the valley between the ridgelines of Edgecliff/Darling Point and Bellevue Hill/Point Piper, occupying the low elevation harbour front area. The normally consolidated sediments within the valley underlying the Double Bay area form a highly productive water table aquifer (Alluvium), which is underlain by the less permeable fractured Bedrock aquifer. The Alluvium, comprising sand with minor silts, clay and peat, has high hydraulic conductivity and is readily replenished by rainfall-derived recharge, resulting in fresh groundwater with salinity of typically less than 400 mg/L. The water table fluctuates in response to seasonal variations in rainfall, with up to 1 m of variation observed in monitoring bores constructed within the Alluvium.

Due to the shallow water table in the Double Bay area, there is high potential for future developments to interact with groundwater. The nature of interaction may be short term, during construction when the water table is lowered to enable dry excavations, or long term when the basements are constructed below the water table and alter the natural flow regime. To assess the latter, a regional groundwater model has been developed and calibrated to available groundwater level data, using hydrogeological parameters that are considered realistic based on prior investigations and conditions observed to date. The modelling of cumulative impacts associated with multiple subterranean structures (basements) has shown that mounding and lowering of the water table could occur over the long term albeit this is generally estimated to be less than 0.3 m assuming full cut-off (basements extending to the Bedrock) and up to 0.2 m assuming partial cut-off, with mounding of <0.2 m in areas of shallow water table.

For the sandy alluvium generally encountered within the Double Bay valley, the impact of construction dewatering is expected to extend far beyond the excavation footprint. The lateral impact can extend up to some 800 m away from the excavation near the recharge point at the sandstone hillside. Further, the severity of the dewatering-induced settlement is strongly related to ground conditions on site. The lowering of groundwater in areas with presence of compressible upper peat soils would cause a much greater settlement than other areas without the peat layers. Consequently, a “Settlement Index Plot” in response to a fixed groundwater drawdown depth was developed based on 271 analysed settlement points, each was assessed based on available site specific geotechnical investigation data. Based on the Settlement Index Plot, a more generalised “Settlement Map” was developed, which shows the different degrees of susceptibility to dewatering-induced ground surface settlements for different sub-divided zones within the Double Bay study area (refer to Figure 27 of this report).

To effectively control the potential damage caused by dewatering, it is essential to assess the likely maximum settlement tolerable by the buildings in the Double Bay area. For the purposes of current assessment of dewatering, we have considered a ground surface settlement of 15 mm as being the limiting value to minimise potential damages of existing buildings. The settlement criteria applicable to the existing buildings, typically one to two storey structures supported on shallow footings, have been developed primarily based on Australian Standard AS2870-2011 and relevant published works by Burland et al. (2002) on building settlements and associated damages. Other considerations including possible past damages of the buildings, flexibility of the structures, pipe drain tolerances and historic groundwater level fluctuation have also been given as part of the assessment process. This threshold surface movement of 15 mm is associated with deflection ratio of 0.075% for a typical wall length of a residential structure. This ratio is commensurate with that of Category 1 damage to walls and concrete floors given in Tables C1 and C2 of AS2870-2011 respectively. The damage Category 1 is described as fine cracks to walls and concrete floors of less than 1 mm which typically do not need repair.

For the different subdivided areas identified in the “Settlement Map”, the allowable drawdown depths associated with proposed settlement limit of 15 mm were assessed to vary between 0.2 m and 1.2 m. A corollary of this finding is that a 0.2 m depth of dewatering can be considered as a relatively safe limit to minimise potential building damages with zone of influence up to some 800 m away from the location of dewatering. From constructability viewpoint, it can be necessary to dewater sufficiently to enable dry excavation during construction. If the abovementioned drawdown limits cannot be achieved, other controls are then needed to effectively reduce groundwater drawdown in the surrounding areas to within the acceptable limit. These controls could include the following:

- Systematic groundwater reinjection/recharge during excavation dewatering
- Sufficient cut-off depth to limit groundwater drawdown outside of the excavations
- Elimination of the need for the dewatering by providing a sealing layer at the excavation base which needs to be adequately designed to resist uplift pressure

Alternative measures can be considered on a case-by-case basis to allow for a review of the drawdown limit. These measures should include the undertaking of sufficient additional geotechnical investigation and subsequent analysis to demonstrate that settlement impacts of surrounding buildings are within acceptable limit.

This report is subject to, and must be read in conjunction with, the limitations set out in Section 2 and the assumptions and qualifications contained throughout the Report.

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Appendix A – List of supplied information

Appendix B – Geotechnical Long Sections and Initial Groundwater Levels

1. Introduction

1.1 General

Urban development is increasingly aiming to maximise the value of land in the Double Bay region. Many developments are considering the construction of basements, underground car parking and other associated below ground structures. Where the water table is intersected temporary dewatering is required to ensure safe and stable construction conditions, and longer term dewatering occurs where drained subsurface structures have been built. The construction of these underground structures can have implications for the groundwater environment in short term and long term, and the magnitude of these implications can be significant when the developments are considered from a cumulative perspective. In terms of the built environment, the depressurisation of compressible sediments can lead to consolidation settlement, and settlement differentials can have significant impacts on the existing buildings. Dewatering can also result in other impacts associated with managing (disposal) of the seepage, reduced access to groundwater by the environment, and activation of acid generating geological materials.

1.2 Project Objective and Scope

1.2.1 Objective

GHD Pty Ltd (GHD) has been engaged by Woollahra Municipal Council (Council) to undertake an assessment of geotechnical and hydrogeological impacts associated with urbanised development of the Double Bay region in the southern edge of Sydney Harbour. The main project objective is to provide Council with a review of the geotechnical and hydrogeological risks associated with latest development plan in their service area which would then inform amendments or further review, where appropriate, to Council’s development guidelines and relevant Local Environmental Plan (LEP), as well as the Development Control Plan (DCP) provisions.

1.2.2 Scope of work

Upon the acceptance of GHD fee proposal dated 31 May 2019, Council has prepared a brief document (Brief) for the geotechnical and hydrogeological study (dated July 2019) which outlines the delivery of the project in 4 stages:

- Stage 1 – Identification of the groundwater catchment and establishment of the project study area.
- Stage 2 – Desktop review. Information from Council and publicly available sources was interrogated to characterise the geological and hydrogeological setting of the study area.
- Stage 3 – Assessment of impacts.
- Stage 4 – Review of the planning framework.

In particular, there are two main components for the Stage 3 work, namely, (i) the impact of long term regional groundwater level change due to future developments and (ii) the impact of surface settlement as a result of groundwater drawdown caused by short term construction dewatering.

At present, the project has advanced to Stage 3 of the scope of works outlined above.

1.2.3 Report structure

This report presents the outcome of our review on relevant information and development of geotechnical and hydrogeological models of the study area (Stage 1 and Stage 2 work), as well as the findings of our geotechnical and hydrogeological impact assessment for Stage 3. The report structure is broadly outlined below:

- Compilation of available information – Section 3
- Regional setting, geological setting and groundwater setting of the study area – Sections 4 to 6
- Regional groundwater modelling – Sections 7 to 8
- Assessment of groundwater induced settlement and discussion – Sections 9 to 10
- Summary – Section 11

2. Limitations

This report has been prepared by GHD for Woollahra Municipal Council and *may only be used and relied on by Woollahra Municipal Council for the purpose agreed between GHD and Woollahra Municipal Council as set out in section 1 of this report.*

GHD otherwise disclaims responsibility to any person other than Woollahra Municipal Council arising in connection with this report. GHD also excludes implied warranties and conditions, to the extent legally permissible.

The services undertaken by GHD in connection with preparing this report were limited to those specifically detailed in the report and are subject to the scope limitations set out in the report.

The opinions, conclusions and any recommendations in this report are based on conditions encountered and information reviewed at the date of preparation of the report. GHD has no responsibility or obligation to update this report to account for events or changes occurring subsequent to the date that the report was prepared.

The opinions, conclusions and any recommendations in this report are based on assumptions made by GHD described in this report. GHD disclaims liability arising from any of the assumptions being incorrect.

GHD has prepared this report on the basis of information provided by Woollahra Municipal Council and others who provided information to GHD (including Government authorities)], which GHD has not independently verified or checked beyond the agreed scope of work. GHD does not accept liability in connection with such unverified information, including errors and omissions in the report which were caused by errors or omissions in that information.

3. Available information

Different sources of information that have been used to assist with the hydrogeological and geotechnical impact assessment are listed below:

- Geotechnical and hydro-geotechnical data provided by Council
- Data from GHD archive
- Data from public domain
- Australian Standards and relevant published technical papers
- Observations from site visit (discussed in Section 5.2.2)

As part of our Stages 1 and 2 work, we have reviewed and used available information relevant to our assessment. We have treated each point, where previous geotechnical investigation was conducted, as data point with factual information relating to the ground conditions. The locations of these data points within the study area are indicated in Figure 1 below. This figure also shows the location of Double Bay commercial centre with several outlines showing future potential developments. Other information in relation to the future potential developments, such as basement depth for some of these developments, were given in the Brief document.

3.1 Data Supplied by Council and from GHD Archive

Council has supplied GHD with information which comprised previous and current Development Control Plan (DCP), geotechnical investigation data and relevant assessment reports as well as drawings related to the Development Application (DA) submissions. These information were supplied in 2 packages. The first package of information was provided at the commencement of the work and throughout Stages 1 and 2. The second package was provided prior to the start of Stage 3 work (end of February 2020).

The information received in the first package included those originally listed in the Brief, which are summarised in Table 1 below. The remaining information from the first package that is not listed in Table 1, as well as information provided in the second package are tabulated in Appendix A. The information provided comprised typically geotechnical investigation reports for residential properties.

Data retrieved from GHD archive as listed in Table 2 has also been used in the present groundwater impact assessment. Together with the data supplied by Council, we have plotted the locations of all relevant geotechnical and hydrogeological data in Double Bay area on Figure 1.

Table 1 Summary of information listed in the Brief

Set of information	Reference ID	Description of information	Issued by
Package 1 information listed in the Brief	R1	Report on Groundwater and Geotechnical Study for Double Bay Commercial Centre.	GHD Longmac Associates Pty Ltd, 2001
	R2	Report on the Geotechnical and Hydrogeological aspects of the draft Double Bay Centre DCP, commissioned by the Double Bay Residents Association.	Douglas Partners Pty Ltd

Set of information	Reference ID	Description of information	Issued by
	R3	Double Bay Catchment Flood Study	Bewsher Consulting Pty Ltd.
	R4	Initial Geotechnical Investigation for Proposed Residential Development at 12-16 William Street, Double Bay	JK Geotechnics, 2015
	R5	Report on Geotechnical and Hydrogeological Investigation, Proposed Multi-Storey Development, 16-18 Cross Street, Double Bay (ref. *Douglas Partners Pty Ltd, 2016b*)	Douglas Partners Pty Ltd
	R6	Report on Preliminary Geotechnical Investigation, Proposed Mixed Use Development, 20-26 Cross Street, Double Bay	Douglas Partners Pty Ltd
	R7	Report on Hydrogeological Assessment, Proposed Residential Development, 4-8 Patterson Street, Double Bay, Project 36739.08 Rev 2 (ref. *Douglas Partners Pty Ltd, 2016d*)	Douglas Partners Pty Ltd
	R8	Report on Preliminary Geotechnical and Hydrogeological Investigation, 4-8 Patterson Street, Double Bay (ref. *Douglas Partners Pty Ltd, 2016c*)	Douglas Partners Pty Ltd
	R9	Letter to Mr John Hall, 14 Forest Road, Double Bay	JK Geotechnics, 2019
	R10	Woollahra Local Environmental Plan 2014, in particular its earthworks and flood planning provisions in sections 6.2 and 6.3.	Woollahra Council
	R11	Woollahra DCP 2015, Part D5.6.7 – Geotechnology and Hydrogeology	Woollahra Council
	R12	Guidelines for Geotechnical and Hydrogeological Reports, Attachment 6 to the Woollahra DA Guide	Woollahra Council
	R13	Standard conditions of consent relating to geotechnical and hydrogeological requirements	Woollahra Council

Table 2 Information retrieved from GHD archive

Set of information	Reference ID (1)	Description of information	Issued by
Information retrieved	R76	Geotechnical Study – 47-53 William Street, Double Bay	GHD Longmac Associates Pty Ltd, 1998

Set of information	Reference ID (1)	Description of information	Issued by
from GHD archive	R77	Supplementary Geotechnical and Groundwater Investigation – Kiaora Ln & Jamberoo Ln, Double Bay	GHD Longmac Associates Pty Ltd, 1998
	R78	Draft Double Bay Centre Development Control Plan Geotechnical ad Hydrogeological Issues	GHD Longmac Associates Pty Ltd, 1998
	R79	Hydrogeological Report – Kiaora Road Development	Coffey Pty Ltd 2003
	R80	Report on Geotechnical Investigation for Proposed Development – Kiaora Place Double Bay	Douglas Partners Pty Ltd, 2010
	R81	GHD's previous advices to Council during the preparation of Development Control Plan which was in place from 2002 to 2015	GHD Longmac Associates Pty Ltd, 2001
	R82	Various letters and memos in relation to the Double Bay DA Assessment for Kiaora Place development	GHD Longmac Associates Pty Ltd
	R83	Initial Geotechnical Investigation for Hotel & Retail Development – New South Head Road, Double Bay	Coffey & Partners Pty Ltd, 1989
	R84	Combined Stage 1 Preliminary and Stage 2 Detailed Site Investigation Report on Kiaora Lane Site, Double Bay	GHD Longmac Associated Pty Ltd, 1990
	R85	Groundwater and Geotechnical Assessment – Double Bay Commercial Centre	GHD Longmac Associated Pty Ltd, 1990
Note to Table 2: (1) References R14 to R75 have been listed in Appendix A.			

Figure 1 Study area showing the data points and locations of potential future developments

3.2 Data from Public Domain

Data from public domain in relation to geological and hydrogeological mapping, topographical information and groundwater base have been referenced, where relevant, throughout the report and a list of references is as follows:

- Bewsher Consulting Pty Ltd, 2008: Double Bay catchment flood study
- Groundwater databases including WaterNSW and the Bureau of Meteorology (Groundwater atlas).
- Herbert C., 1983, Sydney 1:100 000 Geological Sheet 9130, 1st edition. Geological Survey of New South Wales, Sydney
- Department of Environment, Climate Change and Water 2009, Sydney 1:100,000 Soil Landscape Map 9130, 4th edition.
- New South Wales Government 2015, Water Sharing Plan for the Greater Metropolitan Region Groundwater Sources 2011, version dates 1 January 2015, accessed via <https://www.legislation.nsw.gov.au/#/view/regulation/2011/111/full>
- Topographical information provided by the NSW Government Spatial Services

3.3 Australian Standards and Relevant Published Technical Papers

Technical standards and papers pertinent to groundwater flow and building settlement damage have been employed in present assessment, with a list of references as follows:

- AS2870-2011 Residential slabs and footings. Standards Australia.
- Barnett, B, Townley, L.R., Post ,V., Evans, R.E., Hunt, R.J., Peeters, L., Richardson, S., Werner, A.D., Knapton, A., and Boronkay, A, 2012. Australian groundwater modelling guidelines National Water Commission, Waterlines Report Series No. 82 June 2012 ISBN: 978-1-921853-91-3 (online).
- Burland, J.B. 1997. Assessment of risk of damage to buildings due to tunnelling and excavations. Invited special lecture. *IS Tokyp '95: 1 Int Conf on Earthquake Geotechnical Engineering*. Balkema, Rotterdam, 1189 -1201.
- Burland, J.B., Standing, J.R. and Jardine, F.M. 2002. Assessing the risk of building damage due to tunnelling – lessons from the Jubilee Line Extension, London. *Proc. 2nd Int. Conf on Soil Structure Interaction in Urban Civil Engineering*, 11-37.
- Doherty, J 2016, PEST, Model-Independent Parameter Estimation User Manual, v6. Brisbane: Watermark Numerical Computing, 2016.
- Doherty, J, 2017, PEST_HP. PEST for Highly Parallelized Computing Environments. Watermark Numerical Computing, 2017.
- Ladd, C. C. and Foott, R. (1974). New design procedure for stability of soft clays. ASCE Journal of Geotechnical and Geonenvironmental Engineering, Vol. 100, Issue GT7, pp. 763 – 786.
- Lake, L.M., Rankin, W.J., and Hawley, J.,1996. Prediction and effects of ground movements caused by tunnelling in soft ground beneath urban areas. Prepared under contract to CIRIA Project Report 30.
- Mesri, G. and Ajlouni, M. (2007). Engineering properties of fibrous peats. ASCE Journal of Geotechnical and Geoenvironmental Engineering, Vol. 133, No. 7, pp. 850 – 866.

- Panday, S, Langevin, CD, Niswonger, RG, Ibaraki, M & Hughes, J, 2013, MODFLOW–USG Version 1: An Unstructured Grid Version of MODFLOW for Simulating Groundwater Flow and Tightly Coupled Processes Using a Control Volume Finite-Difference Formulation, chapter 45 of Section A, Groundwater Book 6, Modelling Techniques. Techniques and Methods 6–A45.
- Powers, J. P. 1985. Dewatering – avoiding its unwanted side effects. Groundwater Committee of the Underground Technology Research Council of the ASCE technical Council on Research.
- Rau, GC, Acworth, TI, Halloran, LJS, Timms, WA & Cuthbert, MO, 2018, 'Quantifying Compressible Groundwater Storage by Combining Cross-hole Seismic Surveys and Head Response to Atmospheric Tides', Journal of Geophysical Research: Earth Surface 123(8),1910-1930.
- Tammetta, P., and Hawkes, G. 2009, Analysis of aquifer tests in Mesozoic sandstones in western Sydney, Australia. IAH NSW, Groundwater in the Sydney Basin Symposium, Sydney, NSW.
- W.A. Milne-Home (Ed).Tóth, J. 1963. A theoretical analysis of groundwater flow in small drainage basins. Journal of Geophysical Research 68, no. 16:4795-4811.

4. Regional Setting

4.1 Council service area and study area

The Council service area is shown in Figure 2, however, the focus area is within Double Bay. Double Bay sits in the valley between the ridgelines of Edgecliff/Darling Point and Bellevue Hill/Point Piper, occupying the low elevation harbour front area.

Elevations along the Edgecliff / Darling Point ridgeline are around 90 m in the south and fall towards the north to around 50 m. The eastern ridgeline in the Bellevue Hill area is approximately 100 m above sea level. South of Syd Einfeld Drive on the margins of the Council service area and towards Bondi Junction the topography rises to between 70 m and 110 m.

In terms of the hydrogeological study area, a broader area has been adopted as there is a need to consider regional groundwater flow systems.

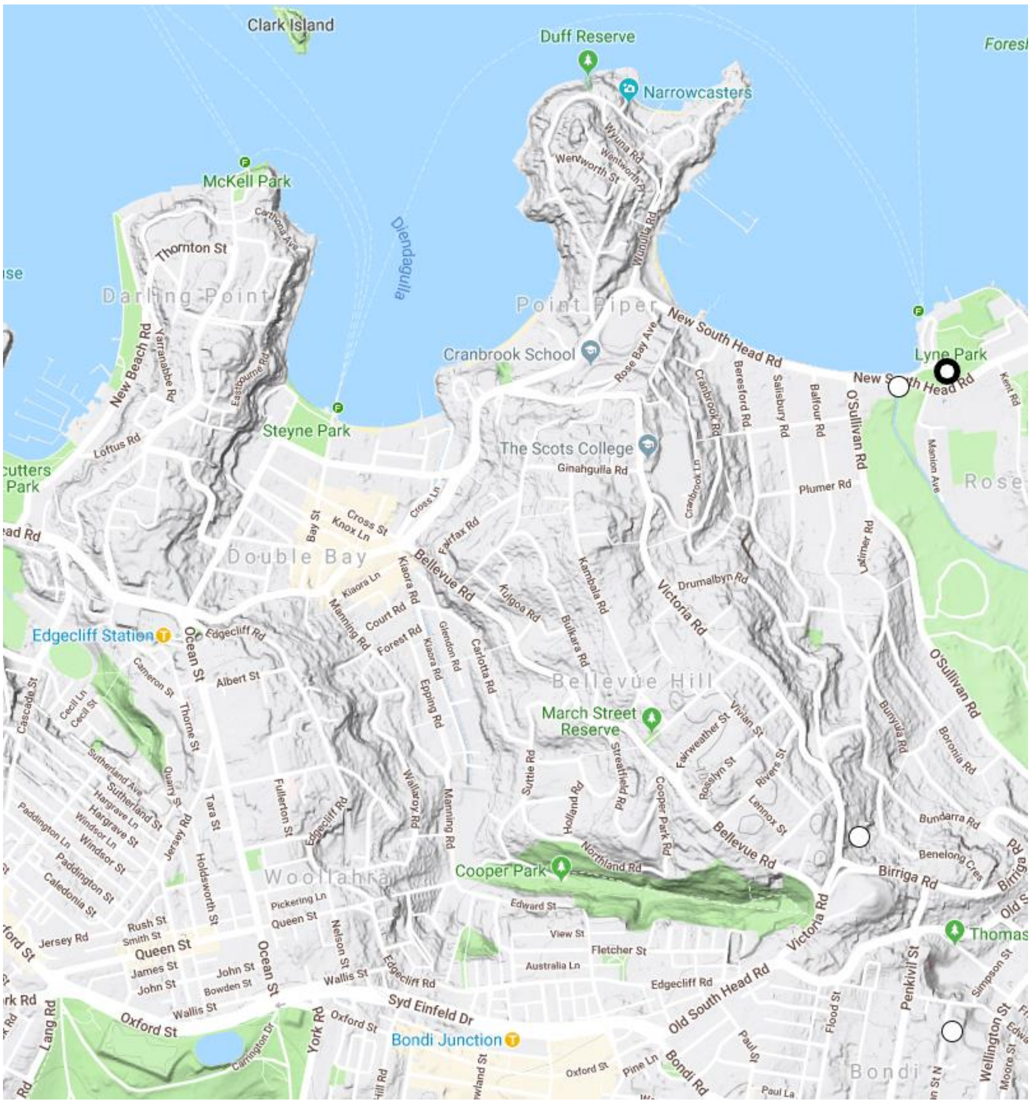


Figure 2 Study area

4.2 Waterways and drainage

The valley follows the former Cooper Creek alignment, which emanates from Cooper Park, running from Bellevue Hill, north to the harbour. The creek, now channelised, generally runs along Kiaora Road, below New South Head Road, to the eastern edge of the bay. Within

Cooper Park the alignment of the creek is interpreted to be influenced by the Jurassic volcanic dyke.

This watercourse, and its entry into the harbour, has resulted in variably deep alluvial sediments within the valley base, with the greatest depth of soils close to the bay, where boreholes have encountered greater than 50 m of mainly coarse grained sediments, occasionally peaty sands with stiff clay basal layers.

4.3 Climate

Climate data was obtained from the Bureau of Meteorology Sydney Royal Botanic Gardens station (66006) and at Rose Bay (Royal Sydney Golf Club – 66098). The mean rainfall data is summarised in Table 3 for these two stations, which indicates an average annual rainfall of around 1230 mm occurs in this region.

Table 3 Summary of rainfall data

Month	Monthly rainfall (mm)			
	Royal Botanic Gardens		Rose Bay (Royal Sydney Golf Club)	
	Since 1950	Since 1990	Since 1950	Since 1990
January	116.0	100.3	114.2	91.5
February	138.5	144.7	134.4	139.3
March	145.7	123.2	139.2	115.3
April	115.6	125.6	124.0	127.8
May	111.8	106.6	115.3	117.3
June	155.3	151.6	156.7	152.4
July	78.8	80.3	87.2	97.7
August	93.7	85.7	89.9	85.2
September	64.6	71.4	64.7	72.3
October	89.2	71.8	85.2	66.0
November	103.1	95.9	96.7	86.4
December	81.2	78.5	84.9	82.7
Annual	1289.6	1227.3	1288.4	1226.2

Note:
1. Site elevation: Botanic Gardens: 15 m, Rose Bay 8 m

The annual rainfall and average annual rainfall (since 1990) for the two stations has been presented in Figure 3. A monthly residual mass curve of rainfall has been prepared to identify long term rainfall trends and has also been presented in Figure 3. This has been undertaken to characterise the influence of climate on groundwater levels.

The absolute value of the residual mass curve is not important, but rather the slope:

- A positive slope indicates a wetter than average period
- A negative slope indicates a drier than average period
- A section of both negative and positive indicates a period of generally average rainfall
- The grade of the slope indicates how much wetter or drier than average the climate is

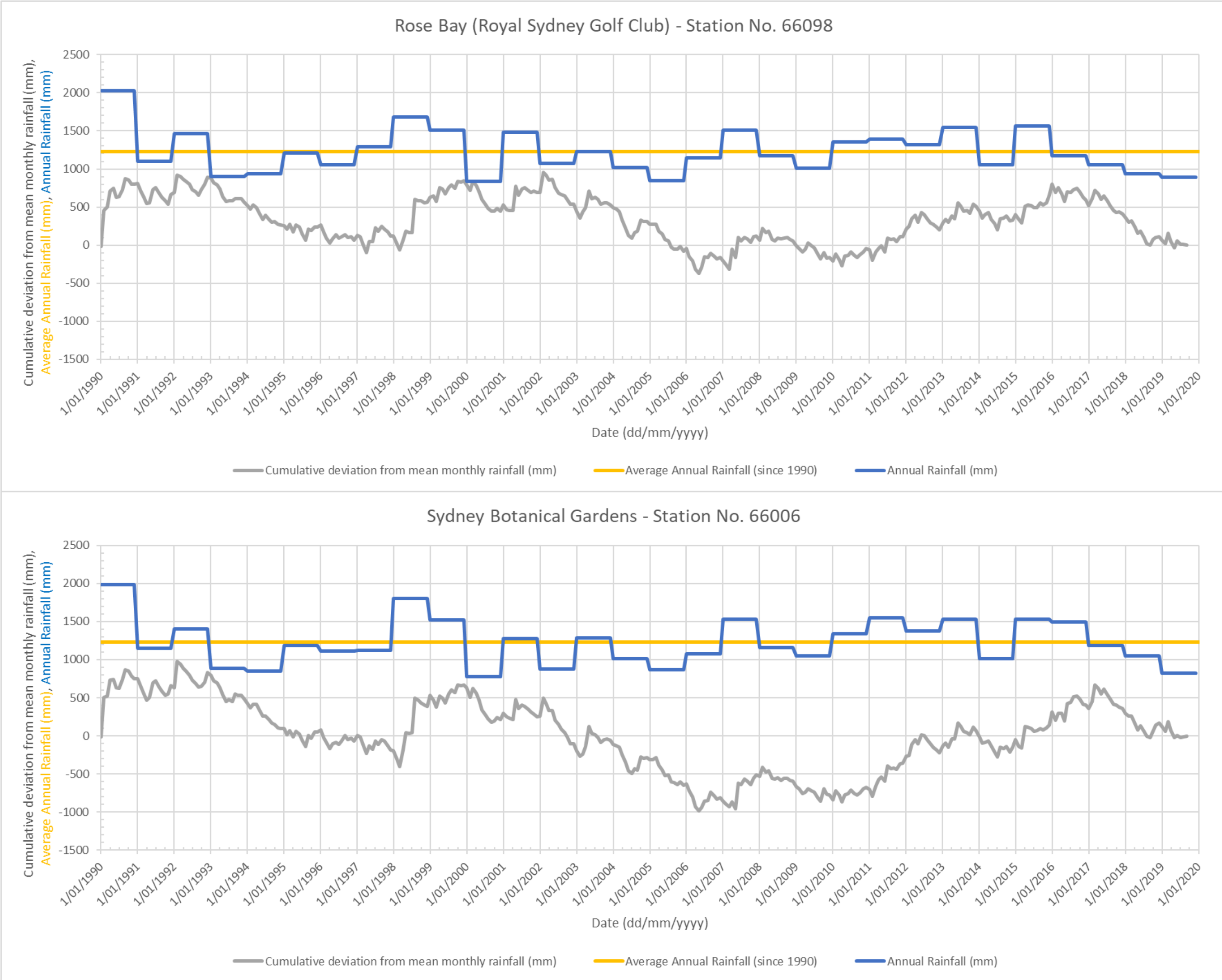


Figure 3 Study area rainfall

5. Geological setting

5.1 Regional Geology

A summary of the stratigraphy has been presented in Table 4 which indicates geology of the study area can be broadly simplified into a two layer system, with Quaternary age unconsolidated sediments overlying Mesozoic age sandstones.

The early Triassic and older geology has been omitted for brevity.

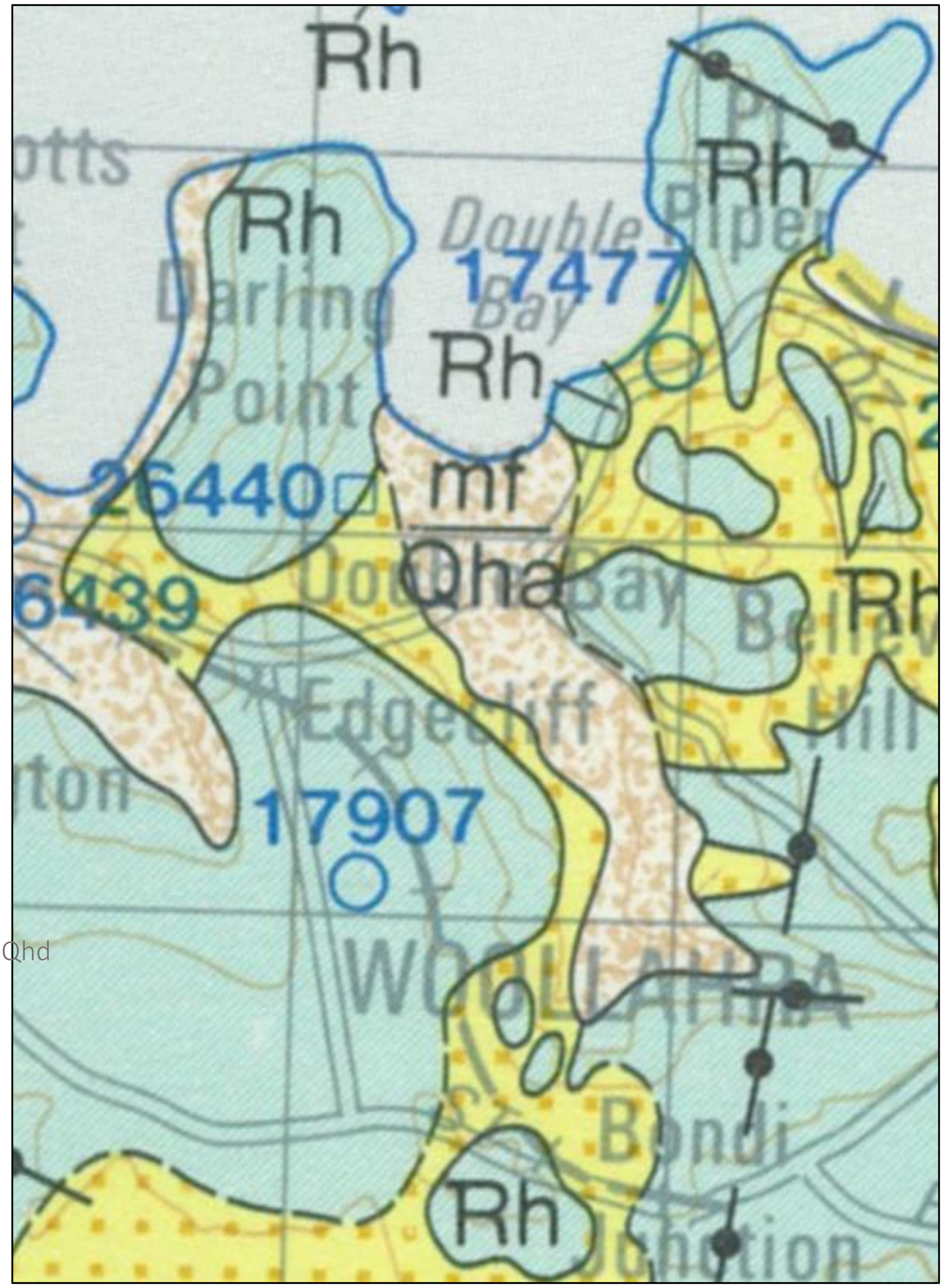
Table 4 Summary of study area stratigraphy

Era	Period	Epoch	Formation
Cainozoic	Quaternary	Holocene	Anthropogenic filling
		Pleistocene	Undifferentiated sands, silts, peaty sands, shell beds.
	Tertiary	Miocene	Absent from Study Area
Mesozoic	Jurassic		Absent from Study Area A period of erosion, forming valleys within the Hawkesbury Sandstone, with some volcanic intrusions.
	Triassic	Middle	Hawkesbury Sandstone

An extract of the 1:100,000 scale geological sheet for Sydney, showing the bedrock zones of Point Piper and Darling Point either side of the fill and valley sediments has been shown in Figure 4. The alluvial region generally follows the shape of the valley, which suggests that the valley was drowned and filled with sediments during the Quaternary (Holocene) period.

Within the incised valley at Cooper Park, there is an east-west trending dyke shown in Figure 4. Another dyke, with a north-south trend, intersects perpendicular to the dyke at Cooper Park. Much of the study area falls within the Hawkesbury Sandstone and soils developed over such terrain.

The 1:100,000 Sydney Soil Landscape Map (Sheet 9130 4th edition) indicates that the majority of the study area (middle and southern portion) is underlain by Deep Creek soil landscape. This is typically described as deep soil on well-drained terraces. The sand in current floodplain typically comprises Siliceous Sand. Such landscape is also characterised by flooding, soil erosion hazard and permanently high water table. The remaining portion of the study area located between the harbour and middle portion consists of Disturbed Terrain. This type of soil landscape is typically associated with the terrain which has been disturbed by human activity which includes the disturbance, removal or burial of original soil materials. The limitation of this soil type comprises the mass movement hazard, low fertility, soil permeability and poor drainage as well as the potential contamination.



mf	Man-made fill. Dredged estuarine sand and mud, demolition rubble, industrial and household waste.
Qha	Silty to peaty quartz sand, silt, and clay. Ferruginous and humic cementation in places. Common shell layers.
Qhs	Peat, sandy peat, and mud.
Qht	Sandy mud and muddy sand.
Qhd } Qd Qpd }	Medium to fine-grained "marine" sand with podsols.
Qhbr	Quartz sand, minor shell content, interdune (swale) silt and fine sand.
Qhf	Medium to fine "marine" sand
Qhb	Coarse quartz sand, varying amounts of shell fragments.

Rh – Hawkesbury Sandstone – Medium-coarse grained, quartz sandstone, very minor shale and laminite lenses

Figure 4 Double Bay Geological Map (Extract of 1:100,000 scale Sydney geological map sheet)

5.2 Geological model

5.2.1 Subsurface conditions

Relevant information summarised in Section 3.1 have been reviewed and used in our assessment to develop a geological model. The construction of the geological model was undertaken using Leapfrog Works 2.3. Leapfrog Works is a commercially available software specifically designed to create dynamic 3D geological models for engineering designs and flow models. The 1-m grid Digital Elevation Model (DEM) gathered from NSW Government Spatial Services was used to develop the topographic surface.

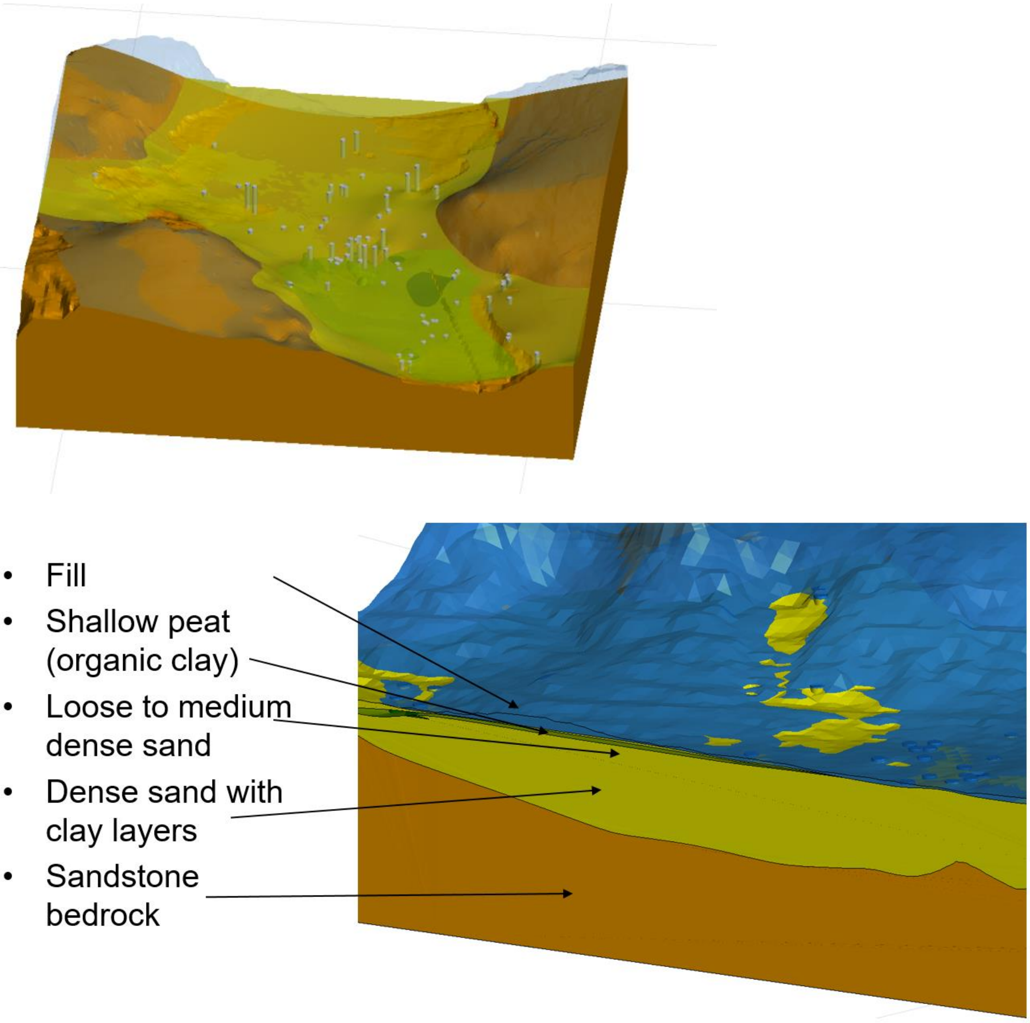


Figure 5 3D geological model generated using Leapfrog

Figure 5 shows the 3D geological model generated using Leapfrog. Five geotechnical sections were prepared for the study area and the locations of these sections are shown in Figure 6. The main geotechnical Section AA was developed in north-south direction roughly parallel to the direction of groundwater flow.

Geotechnical section AA is shown in Figure 7 . Plots of Geotechnical Sections BB, CC, DD and EE are presented as Figures B1, B3, B5 and B7, respectively, in Appendix B.

Description of subsurface conditions by material types

The subsurface profile encountered in the Double Bay study area and delineated in our geological model can be broadly categorised into fill, sand, peat and bedrock as follows:

- *Fill* - The fill profile is generally consisted of concrete, topsoils and/or sand composites. The fill extends across the majority of the study area associated with commercial and residential developments.
- *Upper Peat* – The upper peat layers were considered to be the most compressible deposits and are generally encountered at shallow depth of 0.5 – 2.5 m. Previous investigations indicated that the dark grey peat lenses are of high plasticity with high moisture content organic clay materials. The presence of peat has been observed intermittently although it was consistently noted in the area located to the south of Forest Rd (see Figure 8). The upper peat layers are considered to have significant influence on dewatering induced footing settlements and further discussion of this material is given in Section 5.2.2 below.
- *Alluvial Sand* – The underlying alluvial sand is generally clean and medium to fine grained. It varies in consistency from loose at shallow depth to very dense at depth. Interlayered sandy clays, clays and lower peats of typically stiff to very stiff consistency are also encountered. It appears that these bands are found at lower depths and encountered mainly at the southern Double Bay study area to the south of Kiaora Lane (see Figure 6). The alluvial sands generally fill the incised valley and in topographic depressions and extend to a maximum depth of about 35 m.
- *Bedrock* - Hawkesbury Sandstone underlies the Quaternary deposits. Hawkesbury Sandstone generally comprises medium to coarse grained quartz sandstone with minor shale and laminate lenses. It is typically extremely to highly weathered and fractured at the top and becomes moderately to slightly weathered and only slightly fractured with depth. Collation of available data suggests that the weathered sandstone bedrock surface follows the general shape of ground surface. An assessed contour of bedrock level is presented in Figure 6.

The general site geology within Double Bay study area has been subdivided into geological units based on the available geotechnical investigation data. A summary is presented in Table 5.

<u>Description of subsurface conditions by areas</u>

The subsurface conditions in the areas of Double Bay North, Central commercial development and Double Bay South can be described as follows:

- *Double Bay North* – With reference to the geological sections BB and CC given in Figures B3 and B5, the subsurface profile comprises fill overlying aeolian sands and sandstone bedrock. Aeolian sand extends up the valley terraces. Few peat lenses have been identified in this area. Aeolian sand occupies the western depressions in topography. Little to no Aeolian sand is interpreted along eastern side of valley.
- *Central commercial development (Development Area)* – With reference to the geological section AA given in Figure 7, the bedrock contact is relatively deep around Kiaora Lane. Bedrock was observed at surface along the incised valley terraces. Lower peat lenses have been identified at depths of 20 m to 30 m depth. Sparse shallow peat lenses have also been identified in this area.
- *Double Bay South* – With reference to the geological section AA given in Figure 6, a distinct peat lens at shallow depths of 0.5 - 2.5 m is observed from Court Road to the southern end of Epping Road and sparsely present at Kiaora Lane. Aeolian sand deposits are observed to occupy the incised valley and extends to topographic depressions towards the west and south east area of Double Bay.

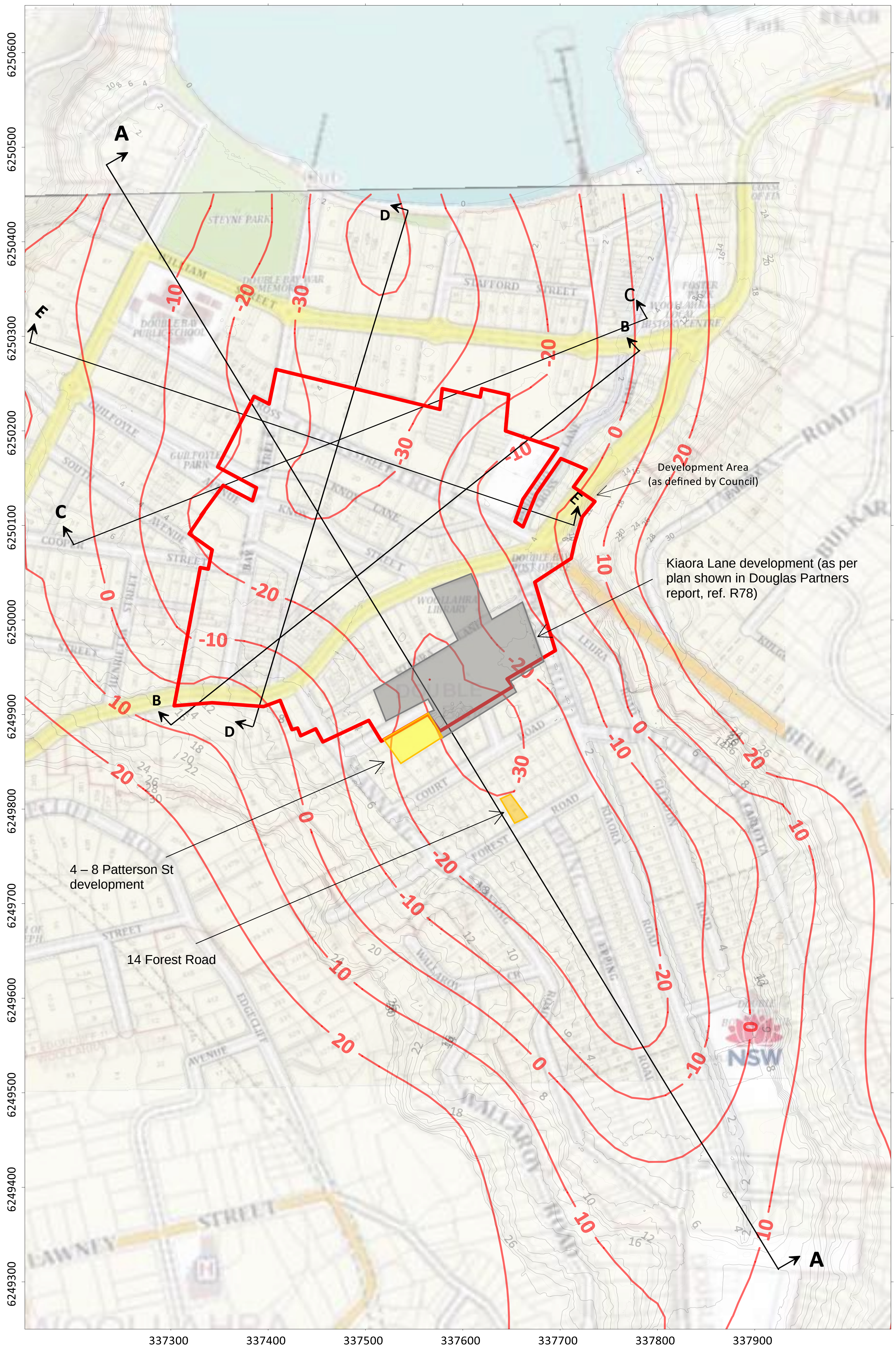


Figure 6 Contours of Top Elevation of Bedrock with Geotechnical Sections

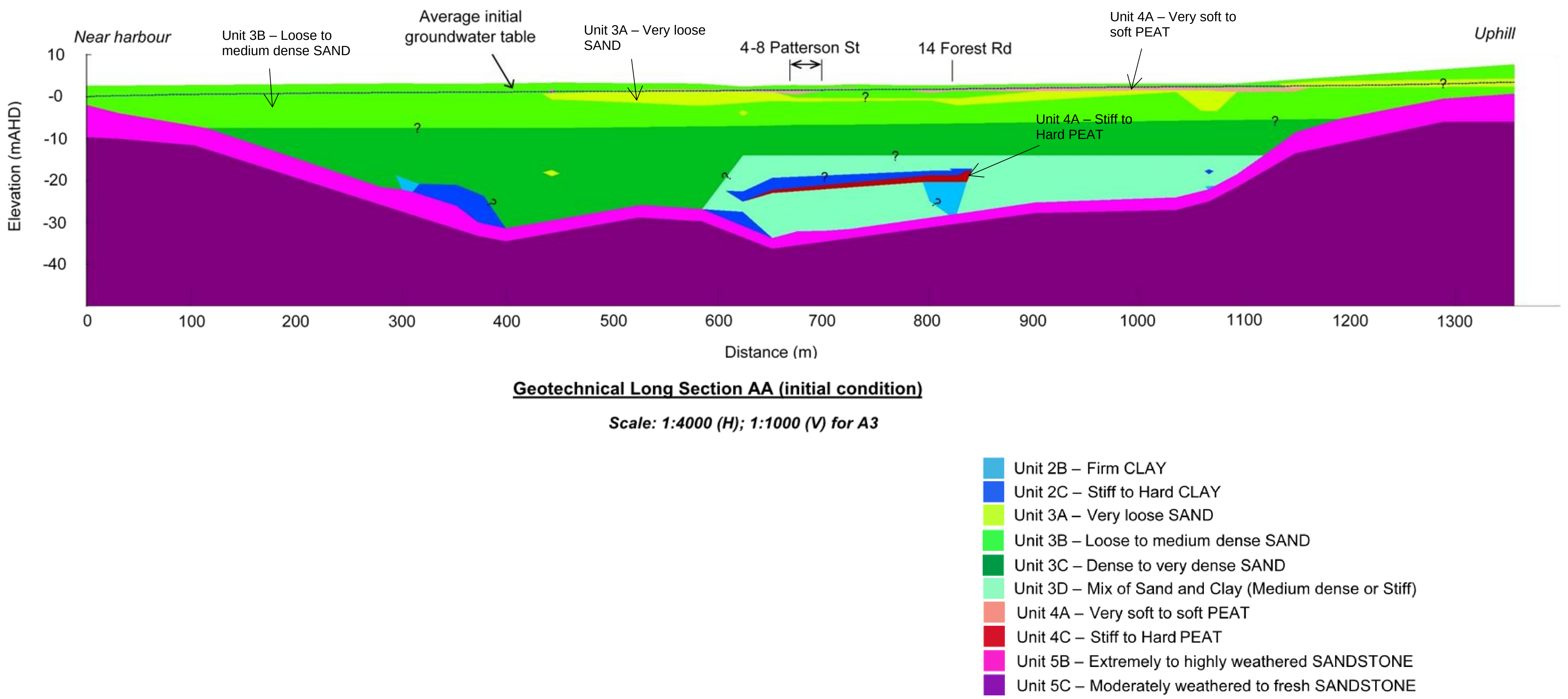


Figure 7 Geotechnical Long Section AA

Table 5 Geotechnical units identified with Double Bay area

Unit	Typical Depth (m bgl) to the top of layer	Unit thickness (m)	Description and Comments[3]
1 – Fill	0 – 2.4	0.1 – 2.4	Concrete, topsoil and/or sand, dry to moist
2A – Very soft to soft Clay	*Note 1*	0.2 – 5	CLAY or silty CLAY, medium to high plasticity, very soft to soft consistency
2B – Firm Clay	*Note 2*	0.2 – 5	CLAY or silty CLAY or sandy CLAY, medium to high plasticity, firm consistency
2C – Stiff to Hard Clay	13.8 – 28.9	0.3 – 16.9	CLAY or silty CLAY or sandy CLAY, medium to high plasticity, stiff to hard consistency
3A – Very Loose Sand	0.1 – 7	0.2 – 8.4	SAND or silty (clayey) SAND, fine to medium, dry to wet, very loose
3B – Loose to Medium Dense Sand	2 – 12	0 – 8	SAND or silty (clayey) SAND, fine to medium, dry to wet, loose to medium dense
3C – Dense to Very Dense Sand	8 – 12	0.5 – 10	SAND or silty (clayey) SAND, fine to medium, wet, dense and very dense
3D – Mix of Sand and Clay	17 – 28.9	1 - 14	Sandy CLAY or clayey SAND, fine grained, low plasticity clay, typically wet, medium dense
4A – Very soft to soft Peat	0.2 – 1.5	0.3 – 5.2	PEAT or Clayey PEAT, moisture content (MC) of above 145, very soft to soft, with organic odour and materials
4B – Very loose Peaty Sand/very soft Sandy Peat			Peaty SAND or sandy PEAT, fine to medium grained, typically wet, very loose sand or very soft peat, with organic odour and materials
4C – Firm Peat	7.5 – 21.6 [(3)]	0.4 – 0.7	PEAT or Clayey PEAT or Peaty CLAY, natural MC of about 110, firm, with organic odour and materials

Unit	Typical Depth (m bgl) to the top of layer	Unit thickness (m)	Description and Comments[3]
4D – Stiff to Hard Peat	1.7 – 28.9 (3)	0.3 – 6	PEAT or Clayey PEAT or Peaty CLAY, natural MC of about 100, stiff to hard, with organic odour and materials
5A – Residual Soil	27 – 40.31	0.5	Sandy CLAY or Clayey SAND, medium to high plasticity clay, fine to medium grained sand, typically dense to very dense sand, very stiff clay
5B/5C Bedrock	0.5 – 42.5	Not proven	Fine to medium grained SANDSTONE, extremely low to medium (estimated) strength, defect partings 0-5° planar, crushed seams, clay seams and joints (variable angles)

Notes to Table 5:

1. Unit 2A was rarely encountered in the data points and can be considered as isolated and localised layers.
2. Unit 2B was encountered at various depths
3. Units 4C and 4D occurred intermittently across the data points, at varying depths and thicknesses
4. Soil type in capital letters indicates primary constituent material

5.2.2 Upper peats and our observations during site visit

The upper peat layers were considered to be the most compressible deposits compared to other soil units identified. The isopach map shown in Figure 8 illustrates the assessed upper peat layer thicknesses within Double Bay study area. The upper peat lenses have been observed at shallow depth (about 0.2 – 1.5 m below ground surface) along the incised valley terraces. These lenses are observed to distribute intermittently, but mainly to the south of Forrest Road and towards the southern end of Epping Road.

During the site visit undertaken by Mr. Kim Chan and Mr. Mark George from GHD and accompanied by Mr Allan Coker from the Council on 29 August 2019, substantial cracking was observed within a residential property located at 14 Forest Road. It was understood that cracking began to occur in about October 2018 and has worsened in the following months. This coincided with the period of construction occurred downslope at 4-8 Patterson Street where substantial dewatering has been carried out to allow basement construction. Several properties located to the south of Court Road were understood to also have experienced some damages. However, cracking or building damages were not reported in some of the buildings immediately to the north of Court Road, albeit in close proximity to the development at 4-8 Patterson Street. It is not clear if the observed cracking/ damages to the existing residences were associated with the construction activities undertaken at Patterson Street.

Figure 8 Isopach map of upper peat layer thickness

6. Groundwater Setting

6.1 Groundwater management and use

6.1.1 Groundwater management

The study area is subject to a Groundwater Management Plan, the Water Sharing Plan for the Greater Metropolitan Region Groundwater Sources. The plan has multiple objectives to protect groundwater as a resource and ecosystems that rely on groundwater. It also sets the long-term average annual extraction limits, performance indicators and water management and licensing rules. The study area sits within the Sydney Basin Central Groundwater Source which has a long-term average annual extraction limit of 45,915 ML/year.

6.1.2 Groundwater use

Groundwater use within the study area is based on the data extracted from the Bureau of Meteorology's Australian Groundwater Explorer. The bores on the Explorer are based on bore information collected by State and Territory lead water agencies which have fed into the National Groundwater Information System (NGIS) as shown in Figure 9. The limitations associated with this dataset include the following:

- Older bores may not be identified where such bores were installed prior to there being any mandatory requirements to license bores.
- Information regarding the operational status of groundwater bores is not known.
- Many bores have not been surveyed for location. Final locations often have a positional accuracy greater than ± 250 m.
- The information registered on the database is subject to the accuracy of bore completion reports submitted by drilling contractors.
- Information registered on the database is subject to change since the completion of the bore e.g. groundwater level information, pump setting depth and groundwater quality.
- Some information is not available on the database, e.g. pump setting depth, bore ownership.

A search of BoM's Australian Groundwater Explorer identified 40 bores in the approximate Double Bay area. The uses of these bores were identified as following:

- Water Supply (28 bores)
- Monitoring (6 bores)
- Irrigation (1 bore)
- Dewatering (1 bore)
- Other (4 bores)

The depths of these bores range from 2.75 m to 52 m, with an average depth of 9.6 m.

A search of WaterNSW also identified 48 bores in the approximate Double Bay area, however, this dataset did not identify the use of each bore. The data obtained from the review of existing geotechnical and hydrogeological investigation reports indicate several bores constructed within the Double Bay area specifically for monitoring purposes. The data available from these bores provide the basis for interpreting the groundwater flow directions and trends, which are described in detail in Section 6.4.

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0 50 100 150 200

Metres

Map Projection: Transverse Mercator
Horizontal Datum: GDA 1994
Grid: GDA 1994 MGA Zone 56

N

GHD

Woollahra Municipal Council
Double Bay Hydrogeological Impacts

Project No. 12512436
Revision No. A
Date 22/04/2020

NGIS groundwater bores

FIGURE 9

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Print date: 22 Apr 2020 - 14:33

Data source: WPC, roads, 2019; BOM, NGIS bores, 2019; GEPro, Imagery, captured 22/08/2017, extracted 07/10/2019; . Created by: bsmyth

6.2 Hydrostratigraphy and aquifer types

Hydrostratigraphic units (HSUs) are zones within the groundwater system that have similar hydrogeological properties and behave in a similar manner from the point of view of groundwater flow. For the study area, the hydrostratigraphy is broadly divided into the Alluvium, comprising unconsolidated sediments, and the Bedrock, which underlies the Alluvium and forms a fractured rock aquifer. The Alluvium forms an unconfined aquifer, within which the water table (upper surface of the shallow groundwater system) is located. The Bedrock is confined beneath the Alluvium within the Double Bay area, and becomes unconfined where it outcrops outside of the valley and forms a regional aquifer. The hydrostratigraphy of the study area is summarised in Table 6.

Table 6 Study area hydrostratigraphy

HSU	Period	Lithology	Aquifer type
Alluvium	Quaternary	Undifferentiated sands, silts, clay, peaty sands, shell beds.	Unconfined
Bedrock	Triassic	Hawkesbury Sandstone	Unconfined / confined

6.3 Groundwater quality

6.3.1 NGIS data

A search of BoM's Australian Groundwater Explorer identified that one (GW107539) of the 40 bores in the Double Bay area had a groundwater salinity record. The groundwater salinity at this location is indicated to be 193 mg/L, with a bore depth of 13 m. The bore is listed as being used for water supply.

As the groundwater quality data from the BoM database was limited within Double Bay, the search area was expanded by approximately 2 km to provide indications of typical range of groundwater salinity. This identified 23 additional bores with salinity data. Table 7 summarised the salinity data based on the lithologies encountered, with sand and peat representing the Alluvium and sandstone representing the Bedrock.

Table 7 Salinity summary

Lithology	Salinity range (mg/L TDS)	Number of bores with TDS information	Salinity range (µS/cm EC) (Number of bores)	Number of bores with EC information
Sand	90 to 646	10	N/A	0
Sand and peat	140 to 160	2	222 to 320	9
Sandstone	150 to 360	5	N/A	0
Unknown	181 to 385	3	255	1

Note: N/A – Not available
TDS – Total Dissolved Solids
EC – Electrical Conductivity

6.3.2 Existing investigation reports

The review of existing geotechnical and hydrogeological investigation reports within the Double Bay area indicate the following additional information on water quality:

- A groundwater sample collected from a monitoring bore constructed at a site referred to as the Kiaora Lane Car Park recorded a field EC measurement of 359 µS/cm (GHD, 1999).
- Groundwater samples collected from 8 monitoring bores constructed for the Kiaora Road Development project indicated Total Dissolved Solids (TDS) concentrations ranging from 172 to 424 mg/L, with an average of 247 mg/L based on the laboratory analysis (Coffey, 2003).

The salinity data from these bores are generally consistent with the salinity data available from the BoM database, indicating that groundwater in the Double Bay area is fresh with a TDS of typically below 400 mg/L.

6.4 Groundwater flow system

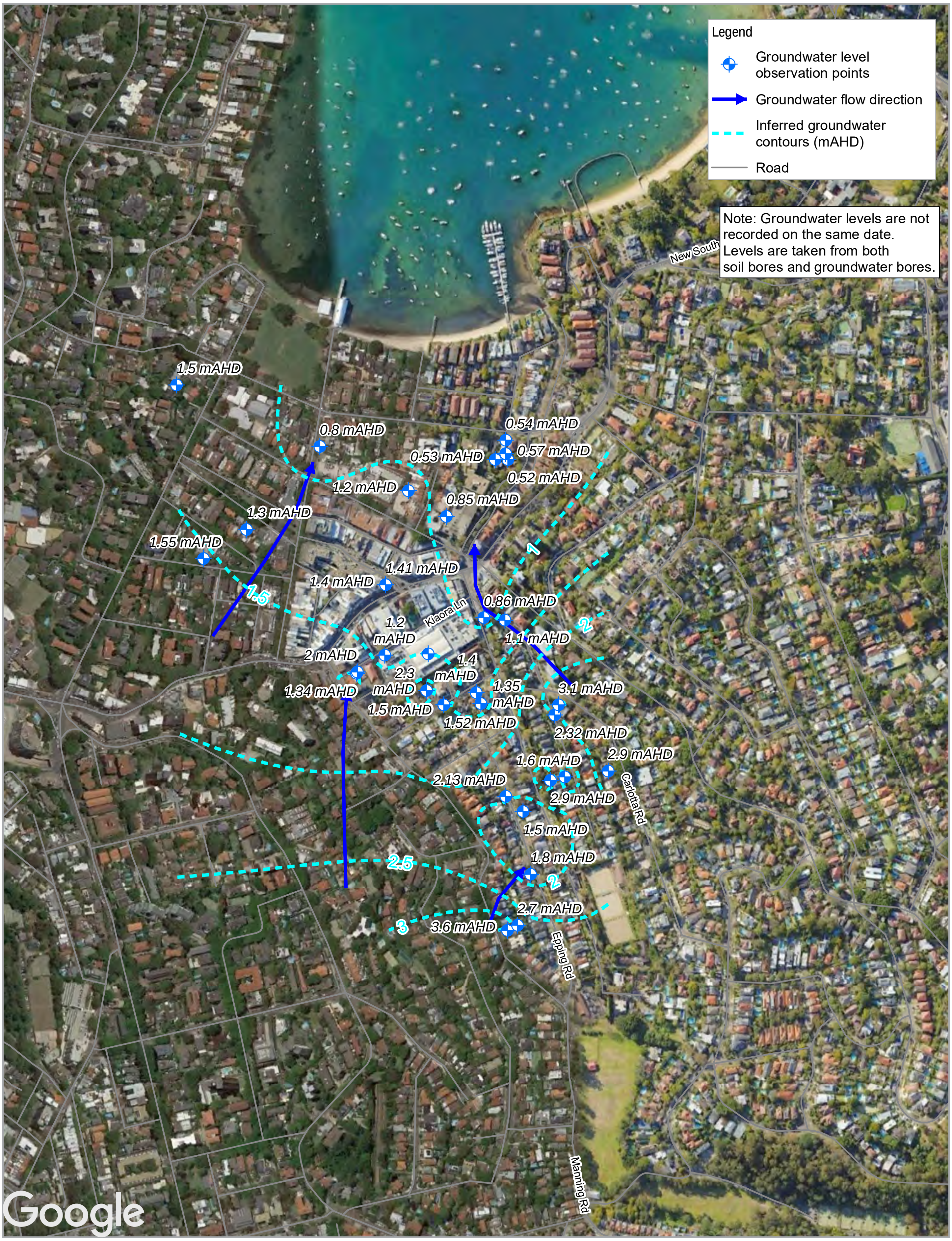
6.4.1 Groundwater flow directions

Groundwater is derived from rainwater that percolates through cracks and pores in rocks and sediments. Groundwater discharges at surface in low-lying areas and along coastal boundary to the north, whereas in topographically elevated areas the water table rises to higher elevations. The difference in the elevation of hydraulic heads resulting from these recharge and discharge mechanisms drives the flow of groundwater from topographically higher levels to topographically lower levels. This results in the water table typically being a subdued reflection of the ground surface, with shallow groundwater potentially interacting with surface watercourses along drainage lines and vegetation (via evapotranspiration). In the deeper part of the system, within the regionally extensive Bedrock aquifer, groundwater flows via longer flow paths driven by regional difference in hydraulic heads associated with regional differences in topography (Tóth, 1963).

Groundwater contour maps previously developed by Longmac Associates (1990) indicate northerly flow of groundwater, with a gentle hydraulic gradient. The contours also indicate a component of flow from west to east, from a topographically elevated area to a low-lying area in the valley, with a hydraulic gradient of around 0.08. The contours are consistent with a topographically controlled flow system, which is maintained by rainfall-derived recharge and discharge along the costal boundary.

To undertake a further analysis of groundwater flow directions, contours of water table have been prepared using groundwater level data extracted from the existing geotechnical and hydrogeological investigation reports (see Figure 10). The contours are interpreted from groundwater levels taken at different points in time, many of which are opportunistic measurements collected from open-holes at the time of field investigations. As such, there are some local variability and the contours should be considered indicative only. Despite these limitations, the interpreted contours provide useful indications of groundwater flow directions, confirming the northerly groundwater flow towards the coastal boundary along the centreline of the valley and flow from topographically elevated areas along the valley edges towards the valley centre. The hydraulic gradient is around 0.003 along the valley centreline, indicating a gentle hydraulic gradient across the Alluvium comprising permeable valley-filled sediments.

The data currently available is insufficient to ascertain local variability in the water table due to anthropogenic influences such as groundwater pumping and existing basement structures.



Paper Size ISO A4

0 100 200 300 400

Metres

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Map Projection: Transverse Mercator
Horizontal Datum: GDA 1994
Grid: GDA 1994 MGA Zone 56

Woollahra Municipal Council
Double Bay Hydrogeological Impacts

Interpreted groundwater contours and flow directions

Project No. 12512436
Revision No. 0
Date 22/04/2020

FIGURE 10

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Print date: 22 Apr 2020 - 14:16

Data source: WPC, roads, 2019; GHD, groundwater contours, 2020; Various geotechnical reports, groundwater levels; GEPro, Imagery, captured 22/08/2017, extracted 07/10/2019; . Created by: bsmyth

6.4.2 Groundwater trends

The most comprehensive record of groundwater level measurements over time are available from 8 monitoring bores constructed for the Kiaora Lane Development project by Coffey (2003). The continuous monitoring record is available over two time periods, from December 2002 to August 2003 (Coffey, 2003) and from December 2004 to August 2005 (Douglas Partners, 2010). Although the raw data have not been provided, the hydrographs included in the reports show seasonal variations ranging from around 0.5 to 1 m over the long term with clear correlation with rainfall. This indicates that the water table within the Alluvium is sensitive to rainfall-derived recharge, consistent with low salinity (the Alluvium is readily replenished by recharge). At some locations the water table reaches close to ground surface (see BH1 in Figure 11, located on the corner of Anderson Street and Court Road).

In general, the range of seasonal fluctuation is smaller at bores closer to the coastal boundary where the groundwater level is constrained at mean sea level. This can be seen in Figure 11, where BH6 is approximately 200 m from the coastal boundary and shows much smaller seasonal variations than BH1 located approximately 400 m farther inland. Hydrographs from December 2004 to August 2005 indicate that some bores during this period displayed trends that appear to be inconsistent with the rainfall-trend. For example, groundwater levels at BH6 in early 2004 were close to sea level until rapid recovery commenced in May 2005, potentially reflecting the influence of groundwater pumping or temporary dewatering.

Douglas Partners (2016e) present groundwater levels recorded in three monitoring bores at 4 – 8 Patterson Street from December 2004 to November 2014. Although the record is not continuous (only up to 5 readings per bore), the data indicates a seasonal range of around 0.6 m and groundwater levels are similar to those of the nearby bores constructed by Coffey (2003)

Jeffrey and Katauskas (2007) recorded groundwater levels over a period of about 2 months (May 2005 to July 2005), prior to the commencement of a dewatering trial at 59 William Street. During this period, the groundwater levels were reported to vary due to daily tidal effects and rainfall, and the average groundwater levels were around 0.6 mAHD. Jeffrey and Katauskas (2011) also recorded groundwater levels at a property between December 2010 and March 2011. The groundwater levels showed little variations during this period, with a general declining trend. While these monitoring periods were brief, small seasonal variations and tidal influence are consistent with the proximity of the site to the coastal boundary.

The BoM Australian Groundwater Explorer did not identify any bores within Double Bay with time series of groundwater levels. However, several bores were identified to the south of Double Bay, in a similar alluvial aquifer system, with time series groundwater level data. Figure 12 presents hydrographs of bores with more than 1,000 groundwater level recordings (and where the lithology is known), showing long term trends dating back to the late 1990’s. Most of these bores show seasonal range that is broadly consistent with that observed in Double Bay with the exception of bores GW075020.1.1 and GW075025.1.1, which may be influenced by their proximity to water supply bores and other groundwater interfering activities.

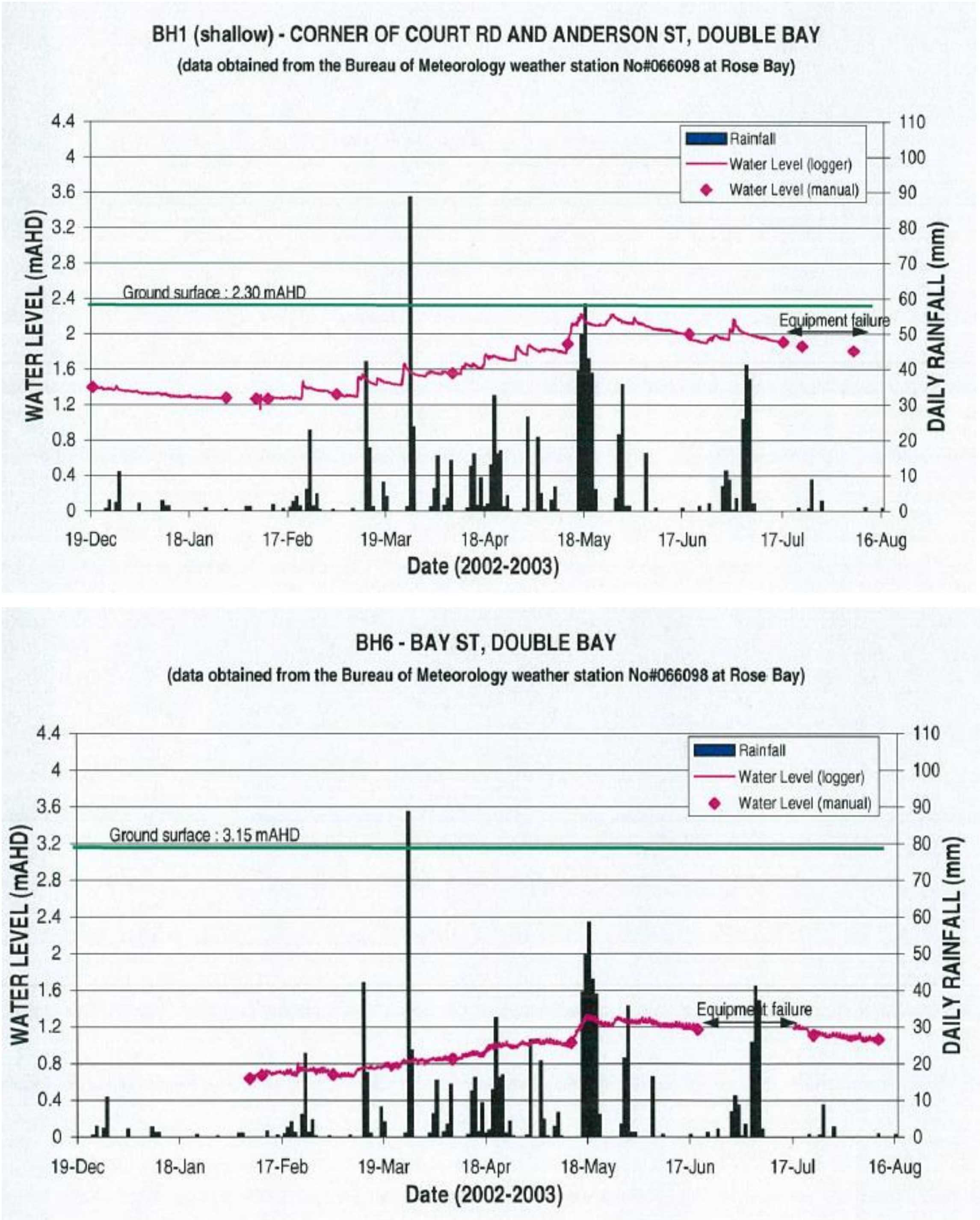


Figure 11 Seasonal trends (after Coffey, 2003)

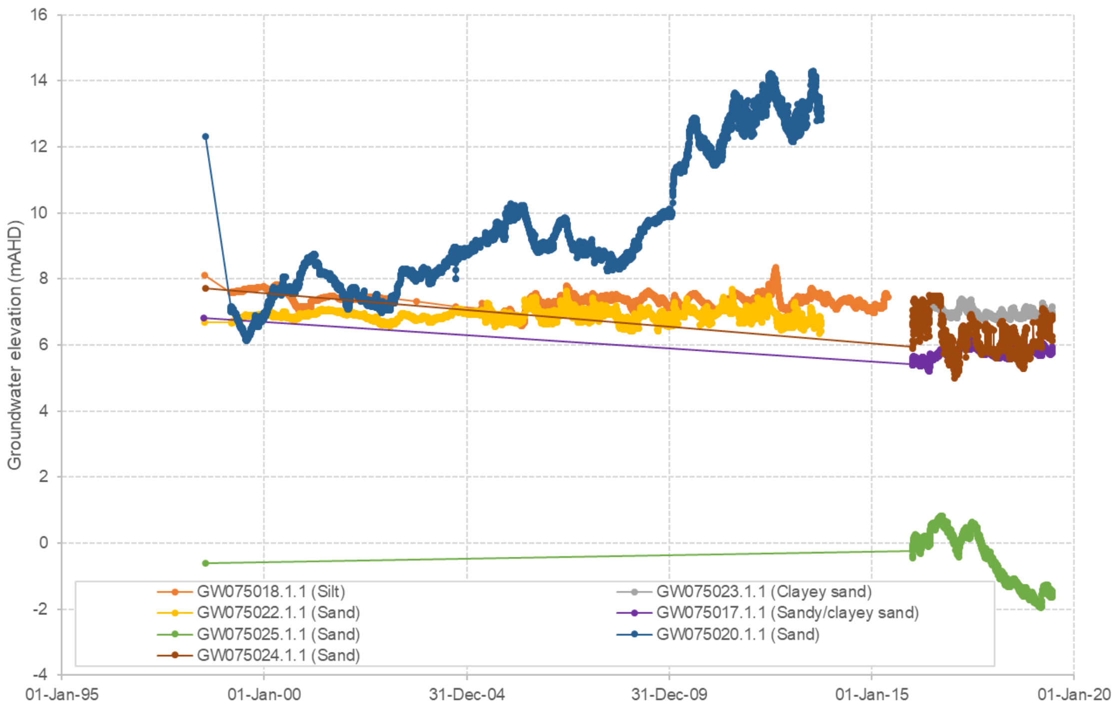


Figure 12 Seasonal trends – NGIS bores outside of study area

6.5 Groundwater dependent ecosystem

6.5.1 Definitions

A groundwater dependent ecosystem (GDE) is an ecosystem which has its species composition and natural ecological processes determined by groundwater. That is, GDEs are natural ecosystems that require access to groundwater to meet all or some of their water requirements so as to maintain their communities of plants and animals, ecological processes and ecosystem services. If the availability of groundwater to GDEs is reduced, or if the quality is allowed to deteriorate, these ecosystems are impacted.

It is widely acknowledged that a poor understanding exists in recognising GDEs, or understanding the hydrogeological processes affecting GDEs, or their environmental water requirements. Common types of GDEs include:

- Ecosystems that depend on the surface expression of groundwater:
  - Swamps and wetlands can be sites of groundwater discharge and may represent GDEs. The sites may be permanent or ephemeral systems that receive seasonal or continuous groundwater contribution to water ponding or shallow water tables. Tidal flats and inshore waters may also be sites of groundwater discharge. Wetlands can include ecosystems on potential acid sulphate soils and in these cases maintenance of high groundwater levels may be required to prevent water from becoming acidic.
  - Permanent or ephemeral stream systems may receive seasonal or continuous groundwater contribution to flow as baseflow. Interaction would depend upon the nature of stream bed and underlying aquifer material and the relative groundwater level heads in the aquifer and the stream.
- Ecosystems that depend on the subsurface presence of groundwater. Terrestrial vegetation such as trees and woodlands may be supported either seasonally or permanently by groundwater. These may comprise shallow or deep rooted communities that use groundwater to meet some or all of their water requirements. Animals may

depend upon such vegetation and therefore indirectly depend upon groundwater. Groundwater quality generally needs to be high to sustain vegetation growth.

6.5.2 GDEs in study area

A review of regional mapping (BoM GDE Atlas) was undertaken as a preliminary means of identifying potential GDEs at a broad scale. Aquatic GDEs were not identified in the study area. Terrestrial GDEs were identified either on the margins, or outside of the study area in the following areas:

- Coastal sandstone gully forest and littoral thickets at Vaucluse (north east)
- Coastal sand swamp forest in Centennial Park (south).

While broad scale mapping did not identify GDEs, it is possible that some of the trees within the Double Bay area intercept the water table due to the shallow depth to groundwater. However, no information is currently available on the environmental water requirements of these trees and whether or not some of these are sourced from groundwater.

6.6 Acid generating materials

6.6.1 Definitions

Acid sulfate soils are soils, sediments, unconsolidated geological material or disturbed consolidated rock mass that contain elevated concentrations of the metal sulfide. It occurs principally in the form of pyrite (iron sulfide). These soils can be rich in organics and were formed in low oxygen or anaerobic depositional environments.

The soils are stable when undisturbed or located below the water table. However, when oxygen is introduced, the sulfides oxidise to sulfate, with resultant soils having low pH and potentially high concentrations of the heavy metals.

Groundwater levels may rise as a result of recovery from construction dewatering activities, or leaching of infiltrating rainfall through the sulfate rich zones. This can result in oxidisation of materials and the mobilisation of pH and heavy metals into the environment where they can potentially impact deep-rooted vegetation, aquatic flora and fauna, and can be aggressive to reactive materials (such as concrete, steel) of foundations, underground structures (such as piles, pipes, basements) or buried services in contact with groundwater. It can also result in the discharge of acid groundwater to receiving surface water systems.

The occurrence of acid sulfate soil can be present in the form of:

- Potential Acid Sulfate Soil (PASS) – Soil that contains unoxidised metal (iron) sulfides. This is usually in oxygen free or waterlogged conditions. When exposed to oxygen through drainage or disturbance, these soils produce sulfuric acid.
- Actual Acid Sulfate Soil (AASS) – Potential acid sulfate soil that has been exposed to oxygen and water, and has generated acidity.

There are two main pathways for the activation of acid sulfate soil to form groundwater impacts:

- Excavation of PASS soils above the water table and their management, such as acid run-off from stockpiles and treatment areas.
- Dewatering required as part of construction of features below the water table, such as for the excavation of basements.

6.6.2 Occurrence within study area

Acid-generating materials in Sydney are commonly found in a number of broad settings:

- Typically geologically young sediments (Holocene age) near sea level.
- Sediments and tidal lakes of marine origin, and estuarine sediments.
- Coastal wetlands, mangroves and swamps.
- Ligneous rich deposits.
- Indurated sediments that may contain elevated concentrations of metal sulphides (Cambrian to Middle Devonian age).

A review of regional mapping (SEED NSW Government) has been undertaken, which is presented in Figure 13. This suggests that the bulk of the study area has a low probability of acid sulfate soils.

Figure 13 Study area acid sulfate soil risk (after SEED NSW)

7. Hydrogeological Parameters

7.1 Overview

From the point of view of groundwater flow, the critical in-situ material properties are the hydraulic conductivity and storage coefficients (specific yield and specific storage). These properties control the resistance of the subsurface material to flow and the rate in which it is drained and/or re-saturated in response to stresses (and the rate in which aquifer pressure is propagated in a fully confined system at depth). Components of inflow and outflow, such as recharge and evapotranspiration, are also important although these are rarely measured in the field and more commonly inferred through other means (such as model calibration), using field-derived estimates of in-situ properties as constraints.

This section provides a summary of prior estimates of hydrogeological properties derived from field testing and modelling undertaken in the Double Bay area. These estimates provide the basis for parameterising and calibrating the regional numerical groundwater model described in Section 8.

7.2 Aquifer testing

Aquifer testing completed as part of geotechnical and hydrogeological investigations in the Double Bay area include CPTU dissipation testing and in-situ permeability testing (such as falling and rising head tests). Table 8 summarises the horizontal hydraulic conductivity values collected during field investigations. The majority of these tests have targeted discrete horizons within the Alluvium, as flow into shallow excavations are controlled by the properties of this shallow aquifer. While low hydraulic conductivity values have been derived from discrete clay lenses, the abundance of sand within the Alluvium and high hydraulic conductivities associated with the sand intervals indicate that the aquifer as a whole behaves as a high transmissivity system.

Limited information is available from the Bedrock. Testing undertaken by Longmac Associates (1990, 1998) indicates low hydraulic conductivity values although Longmac Associates (1998) note that the hydraulic conductivity of the weathered sandstone bedrock could be variable depending upon the weathering profile and presence of jointing in the rock. Information available from other parts of the Sydney area indicate that the mean horizontal hydraulic conductivity in the upper 100 m of the Hawkesbury Sandstone ranges from around 0.01 to 0.1 m/d (around $1x10^{-5}$ to $1\ x10^{-4}$ cm/sec) (Tammetta and Hawkes, 2009).

There are no estimates of vertical hydraulic conductivity although a horizontal to vertical permeability ratio of 10:1 has been reported (Longmac Associates, 1990), which is common in layered sedimentary aquifer systems.

There are no site specific estimates of storage coefficients. Specific yield of 0.1 to 0.3 is commonly assumed for the Alluvium comprising fine sands and specific storage of $1x10^{-6}$ to $1x10^{-4}$ /m is reported in the literature for the confined Hawkesbury Sandstone (GHD, 2015). For most lithologies, specific storage of $1x10^{-6}$ to $1x10^{-5}$ /m is considered realistic, with recent work by Rau et al (2018) suggesting a plausible upper threshold of around $1.3x10^{-5}$ /m for specific storage in confined aquifers.

Table 8 Aquifer test data

Lithology	Method	Reference	Number of tests	Horizontal permeability (cm/sec)
Sand with silt	In-situ permeability	Longmac Associates (1998)	3	4.9×10^{-4}* to 2.3×10^{-3}
Sand with silt	In-situ permeability	Coffey (2003)	1	$<1 \times 10^{-3}$
Sand	In-situ permeability	Longmac Associates (1990)**	-	6×10^{-4} to 2×10^{-2}
Sand	In-situ permeability	Coffey (2003)	7	1×10^{-3} to 1×10^{-2}
Sand	In-situ permeability	Douglas Partners (2016b)	1	1.2×10^{-2} to 2.3×10^{-2}
Clay	CPTU	Longmac Associates (1998)	10	2.5×10^{-5} to 2×10^{-4}
Clay bands	Laboratory testing	Coffey (1989)	2	7.1×10^{-9} to 5.8×10^{-8}
Clay/peat	In-situ permeability	Longmac Associates (1990)**	-	1×10^{-7} to 6×10^{-4}
Hawkesbury Sandstone	In-situ permeability	Longmac Associates (1998)	1	9.4×10^{-6}^
Hawkesbury Sandstone	In-situ permeability	Longmac Associates (1990)**	-	Negligible small to 9×10^{-4}
Notes: *Based on falling and rising head tests and Hvorslev analytical solution ^Based on packer testing **Desk study – values inferred from other reports provided by Council and test numbers are not specified				

Note: 1 cm/sec = 864 m/day

7.3 Groundwater modelling

Several local scale groundwater models have been developed previously at individual sites to estimate the potential impact of temporary construction dewatering activities. Most of the models have adopted parameter values that are considered plausible based on field data available at the time or literature derived values for representative lithologies. The modelled parameter values are summarised as follows:

- Longmac Associates (1990) assumed horizontal hydraulic conductivity of 5 m/d (6 x 10^{-3} cm/sec) for the Alluvium and 0.05 m/d (6 x 10^{-5} cm/sec) for the Bedrock with the horizontal to vertical hydraulic conductivity ratio of 10:1.
- Coffey (2003) assumed horizontal hydraulic conductivity of 5 m/d (6 x 10^{-3} cm/sec) and vertical hydraulic conductivity of 1 m/d for the Alluvium (1.16 x 10^{-3} cm/sec), with a recharge rate of 120 mm/year and evapotranspiration rate of 1200 mm/year (with an extinction depth of 1.5 m).
- Douglas Partners (2016b) assumed horizontal hydraulic conductivity of 10 to 20 m/d (1.2 x 10^{-2} to 2.3 x 10^{-2}cm/sec) based on in-situ testing at one bore.
- Douglas Partners (2016d) assumed horizontal hydraulic conductivity of 5 to 20 m/d (6 x 10^{-3} to 2.3 x 10^{-2}cm/sec) and vertical hydraulic conductivity equal to 20 % of the horizontal hydraulic conductivity.

8. Regional Groundwater Modelling

8.1 Modelling objectives

Due to the shallow water table in the Double Bay area, there is high potential for future developments to interact with groundwater. The nature of interaction may be short term, during construction when the water table is lowered to enable dry excavations, or long term when the basements are constructed below the water table and alter the natural flow regime.

The purpose of regional groundwater modelling is to provide outputs that would assist with the quantification of potential impacts and risks, and ultimately the planning framework. Specifically, the modelling is undertaken to provide:

- Spatial distribution of piezometric heads, depth to groundwater and associated seasonal range across the study area, such that the likely level of groundwater interference at future development sites could be understood.
- Potential cumulative long-term impacts of multiple subterranean structures (basements), including the magnitude and spatial extent of changes to the water table.

In order to achieve this intended use, the model must be appropriately designed and calibrated, using the available geological and hydrogeological data. The modelling described in this section is undertaken at a regional scale, to provide outputs across the study area. Local scale impacts associated with individual sites, such as during dewatering, are examined separately using models appropriate for that scale (Section 10). The outputs from the regional scale modelling, such as the distribution of piezometric heads and calibrated hydrogeological parameters, provide inputs to the local scale modelling.

The local scale modelling is presented in Section 10.3 of this report.

8.2 Model design and construction

8.2.1 Modelling software

An unstructured grid version of the industry standard MODFLOW code, called MODFLOW-USG (Panday et al., 2013), has been selected as the most appropriate groundwater modelling software for this study. Features of MODFLOW-USG that are particularly suited to address the modelling needs and objectives include efficient local mesh refinement around areas of interest within a regional model domain while retaining larger cells elsewhere, minimising model size (total cell count) and run times without compromising resolution in critical areas. The model layers can also ‘pinch out’ where hydrostratigraphic units (HSUs) are not present and cells are not required throughout the model domain, reducing the total cell counts and improving numerical stability.

The unstructured mesh of the MODFLOW-USG model has been generated using a graphical user interface GMS10.4.4. Although the model was originally designed to be fully contained within GMS, not all aspects of the modelling could be addressed efficiently using the features available within this interface. This meant the model input files needed to be prepared using a combination of GMS, Geographic Information Systems (GIS) and a range of in-house and third-party utilities.

8.2.2 Model domain and mesh

Figure 14 shows the model domain and model mesh. The model domain is based on the local groundwater catchment delineated using the Digital Elevation Model (DEM), with the coastal boundary representing the zone of groundwater discharge in the north and no-flow boundary elsewhere along topographical ridges where a natural groundwater divide is expected. The domain is large enough to fully enclose the extent of the Alluvium and capture the influence of key hydrological stresses.

The model mesh uses a layered quadtree-mesh and the model cells are progressively refined in areas of interest to provide greater numerical accuracy. The minimum cell size is 3 m by 3 m over the footprint of the future development areas, which is small for the size of the model and allows the influence of subterranean structures to be readily incorporated into the regional domain.

8.2.3 Model layers

The model layers are based on the Leapfrog geological model and includes the Alluvium, Peat and Bedrock. Although the Peat lenses are generally thin or localised, they have been incorporated into the model for consistency with the geological and geotechnical modelling. Once incorporated, the model can also be used to examine the sensitivity of model outputs to the properties of Peat. For the purpose of groundwater modelling, only the thin (but laterally extensive) upper Peat layer and two Peat lenses at depth have been incorporated.

Table 9 summarises the model layers and Figure 15 presents a cross-section through the model, showing the relationship between model layers and HSUs. With the exception of the Bedrock layer (layer 7), each model layer is discontinuous and pinched out against the adjacent unit. This means there are areas where some model layers are absent e.g. layer 1 locally overlies and connected to layer 4. In order to accommodate the future basements of different depths, an additional layer (layer 4) has been incorporated into the Alluvium.

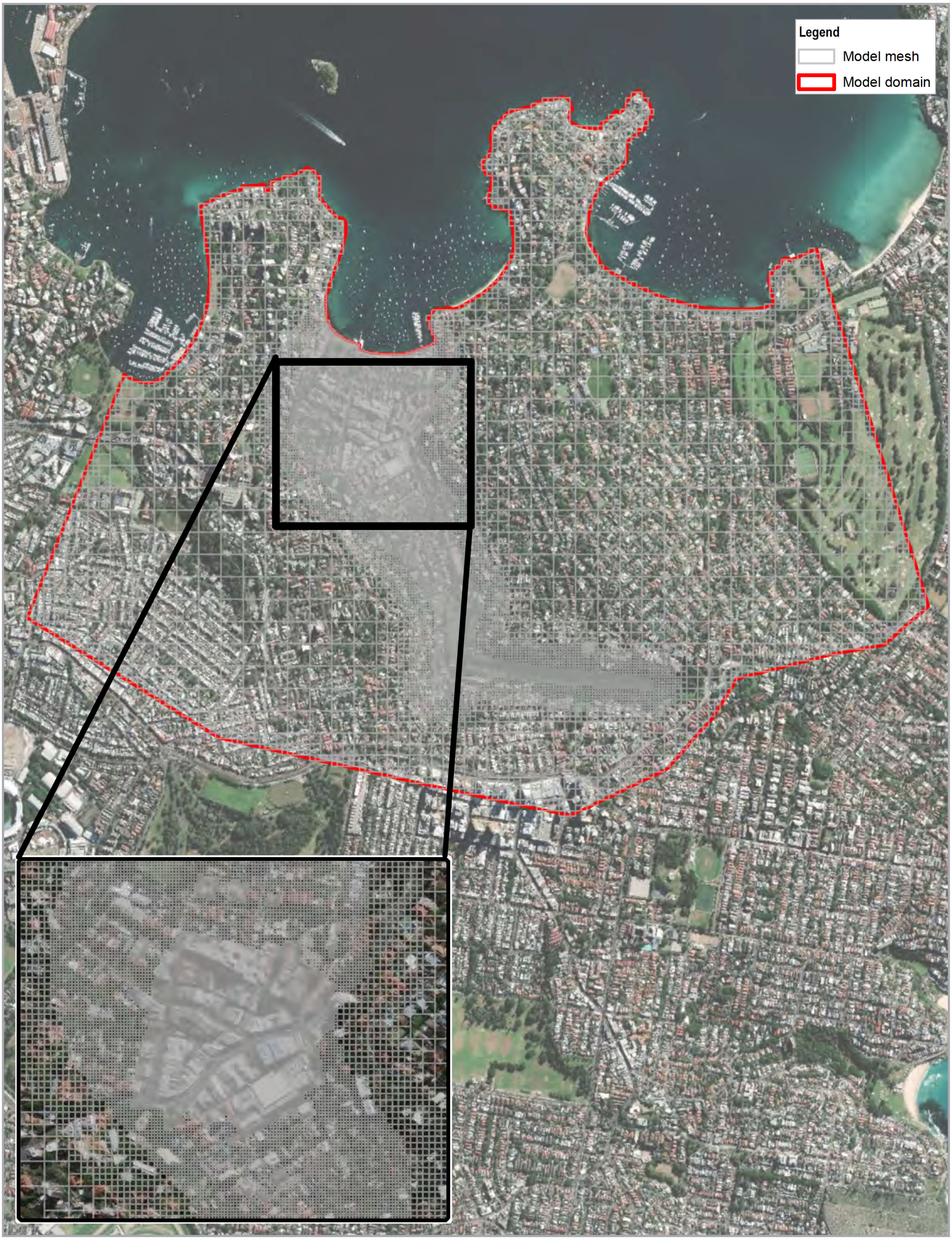
The model top is based on the DEM and the bottom of the Bedrock is set at -100 mAHD. The model has 98,236 cells in total.

Table 9 Model layers

Layer	Cells	Continuity	HSU
1	27,226	Pinch out	Alluvium
2	4,290	Pinch out	Peat
3	4,331	Pinch out	Alluvium
4	24,744	Pinch out	Alluvium
5	871	Pinch out	Peat
6	855	Pinch out	Alluvium
7	35,919	Continuous	Bedrock

8.2.4 Model boundary conditions

Along the coastal boundary, a constant head boundary condition is assigned with a head value of 0.1 mAHD. Elsewhere, a no-flow boundary condition is assumed along the model boundary. Recharge and evapotranspiration are prescribed to the uppermost nodes (the highest node in a 2-d array, using option 2 of MODFLOW-USG’s recharge and evapotranspiration packages). Two recharge zones have been defined based on the modelled extent of the Alluvium and outcropping Bedrock, to account for different recharge rates expected in these units of different properties. Both recharge and evapotranspiration rates have been adjusted during model calibration and are described further in Section 8.3.2.



Paper Size ISO A4

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Metres

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Map Projection: Transverse Mercator
Horizontal Datum: GDA 1994
Grid: GDA 1994 MGA Zone 56

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Model domain and model mesh

FIGURE 14

G:\31\12512436\GIS\Maps\Working\12512436_11_ModelExtent_A4P_RevB.mxd
Print date: 22 Apr 2020 - 14:21

Data source: GHD, groundwater model data, 2020; Source: Esri, DigitalGlobe, GeoEye, Earthstar Geographics, CNES/Airbus DS, USDA, USGS, AeroGRID, IGN, and the GIS User Community. Created by: bsmyth

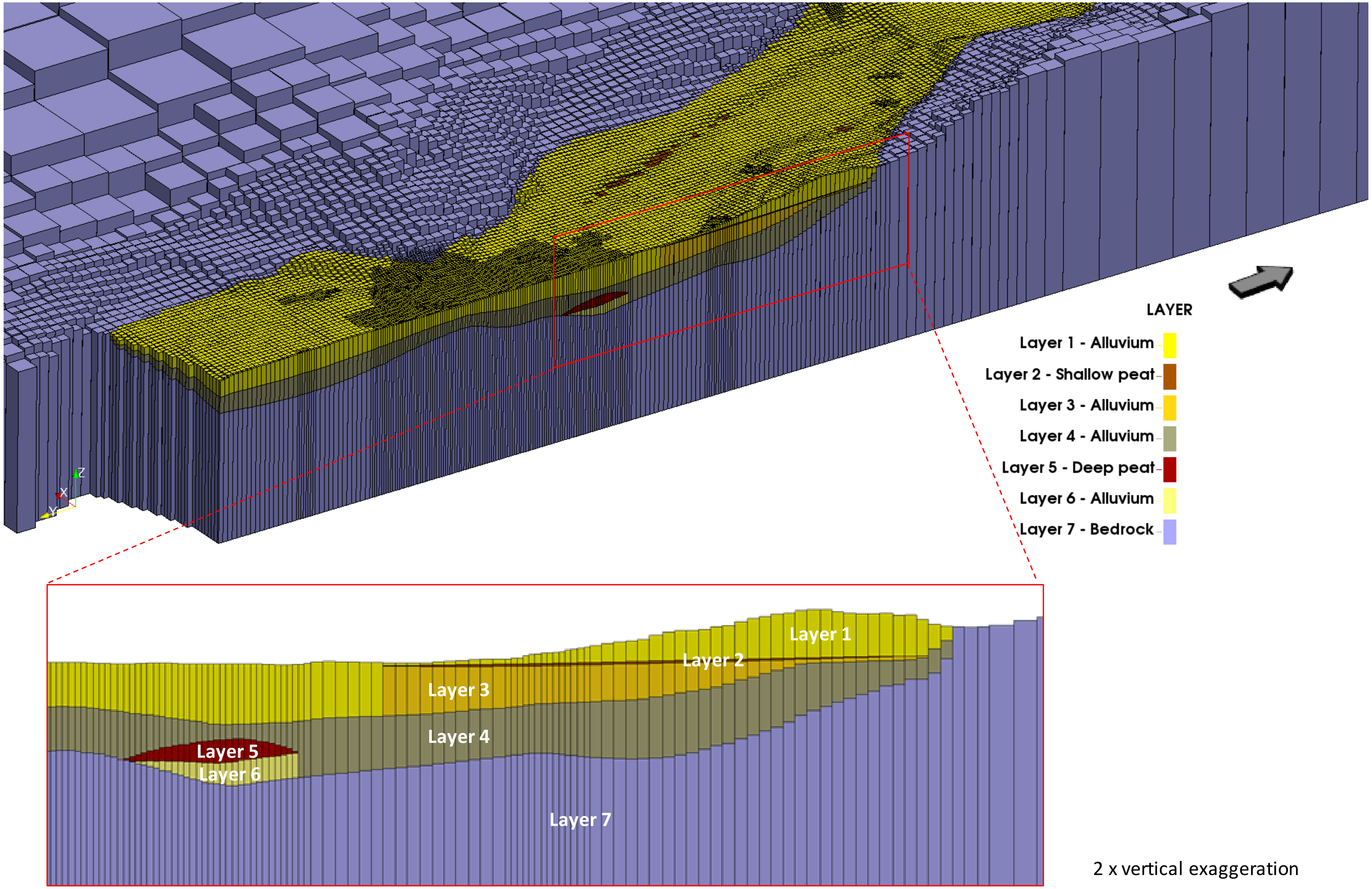


Figure 15 Model cross-section

8.3 Model calibration

8.3.1 Calibration methodology

Model calibration is a process by which model parameter values are altered within realistic bounds until the model outputs fit historical measurements, so that the model can be accepted as a reasonable representation of the physical system of interest (Barnett et al., 2012).

In order to make use of all available groundwater level data, the model has been calibrated transiently using a combination of single groundwater level measurements collected from 25 bores at different times and time series of groundwater level measurements obtained from 8 monitoring bores constructed by Coffey (2003). As the raw data from Coffey (2003) and Douglas Partners (2010) were not available, the groundwater levels from hydrographs were extracted manually to provide sufficient data points to enable meaningful transient calibration. The model calibration period starts in January 2002 and finishes at the end of 2019, capturing 18-years of climate data. The model uses a combination of quarterly and monthly stress periods to capture seasonal variability, with monthly stress periods used from December 2002 to August 2003 and from December 2004 to August 2005, when the time series data are available.

The model parameters have been adjusted during calibration on a HSU-basis, to derive representative hydraulic conductivity (horizontal and vertical), specific yield and specific storage for each HSU. The exception is the Alluvium, where the hydraulic conductivity has been varied spatially via interpolation of parameter values assigned to pilot points located on a 300 m by 300 m grid (a total of 10 adjustable pilot points). The spatial variability enables the model to better account for spatial differences in the measured groundwater levels. The vertical hydraulic conductivity has been estimated by calibrating the horizontal to vertical hydraulic conductivity ratio (anisotropy factor).

Recharge is calculated as a percentage of average daily rainfall for each stress period. Rainfall is first converted to recharge using a factor and applied over the Alluvium. This Alluvium recharge is then converted to Bedrock recharge using another factor. This two-stage approach maintains a sensible ratio between the two recharge rates throughout the calibration process, ensuring that recharge applied over less permeable Bedrock is no greater than recharge over more permeable Alluvium. Evapotranspiration rate and extinction depth are adjusted as single model-wide values.

The calibration has been undertaken rigorously using the automated parameter estimation code PEST(Doherty, 2016) and PEST_HP in a parallelized computing environment (Doherty, 2017). The minimum and maximum parameter values permitted during calibration are derived from relevant prior studies, as discussed in Section 7, and those that are considered appropriate based on the conditions observed at the site.

8.3.2 Calibration performance

Table 10 summarises the calibrated model parameters. These parameter values are generally consistent with the parameter values derived from field studies and previous modelling. Recharge applied over the Alluvium is higher than that used previously by Coffey (2003) and this is likely to reflect the rigorous nature of calibration to transient groundwater levels (as opposed to steady state calibration), where the influence of recharge, and its relationship to specific yield and hydraulic conductivity, can be better estimated.

Table 10 Calibrated model parameters

Parameter	Calibrated values
Alluvium horizontal hydraulic conductivity (K_H)	0.5 to 10 m/d (average 3 m/d)
Alluvium hydraulic conductivity anisotropy (K_H/K_V)	10.47
Peat horizontal hydraulic conductivity (K_H)	0.035 m/d
Peat hydraulic conductivity anisotropy (K_H/K_V)	6.36
Bedrock horizontal hydraulic conductivity (K_H)	0.044 m/d
Bedrock hydraulic conductivity anisotropy (K_H/K_V)	11.05
Alluvium specific yield (Sy)	0.08
Alluvium specific storage (Ss)	1.2×10^{-5} /m
Peat specific yield (Sy)	0.085
Peat specific storage (Ss)	2.7×10^{-6} /m
Bedrock specific yield (Sy)	0.022
Bedrock specific storage (Ss)	5×10^{-6} /m
Alluvium recharge	20% rainfall (average 237 mm/yr)
Bedrock recharge	4.4% rainfall (average 52 mm/yr)
Evapotranspiration	1200 mm/yr
Evapotranspiration extinction depth	2.5 m

Figure 16 presents hydrographs from several monitoring bores from Coffey (2003), roughly along a north to south transect across the study area to demonstrate the modelled responses to climate variability, and how these compare against the observed data. The hydrographs show that the modelled heads match the observed heads reasonably well, with seasonal fluctuations appropriately replicated. In particular, smaller seasonal fluctuations observed closer to the coastal boundary are also simulated by the model consistent with the expected groundwater behaviour. The only exception is at BH6, where low groundwater levels were observed in 2005 (effectively reaching sea level), which may reflect the influence of localised pumping/dewatering that has not been accounted for in the model.

Figure 17 and Figure 18 show the modelled groundwater contours within the Alluvium for the wettest and driest periods within the 18-year simulation period, respectively. The contours indicate that the model simulates overall flow to the north, towards the coastal boundary, with components of flow from topographically elevated areas along the valley edges to valley centre. The contour intervals are narrower towards the south during the wet periods when the water table is raised by higher recharge and the hydraulic gradient becomes steeper.

The Root Mean Squared (RMS) error between the simulated and observed heads is 0.3 m and the modelled groundwater levels are generally accurate to within this range where data is available.

The model currently simulates some flooded cells in the very southern end of the valley (further to the south of the extent shown in Figure 17), where there is uncertainty in the thickness of the Alluvium/depth to Bedrock due to absence of data. The Leapfrog model (and hence the groundwater model) currently assumes thinning of the Alluvium in this area and there is insufficient transmissivity for shallow groundwater to laterally drain following recharge events. This occurs some distance away from the proposed development areas and have no effect on model’s performance in areas critical for this assessment.

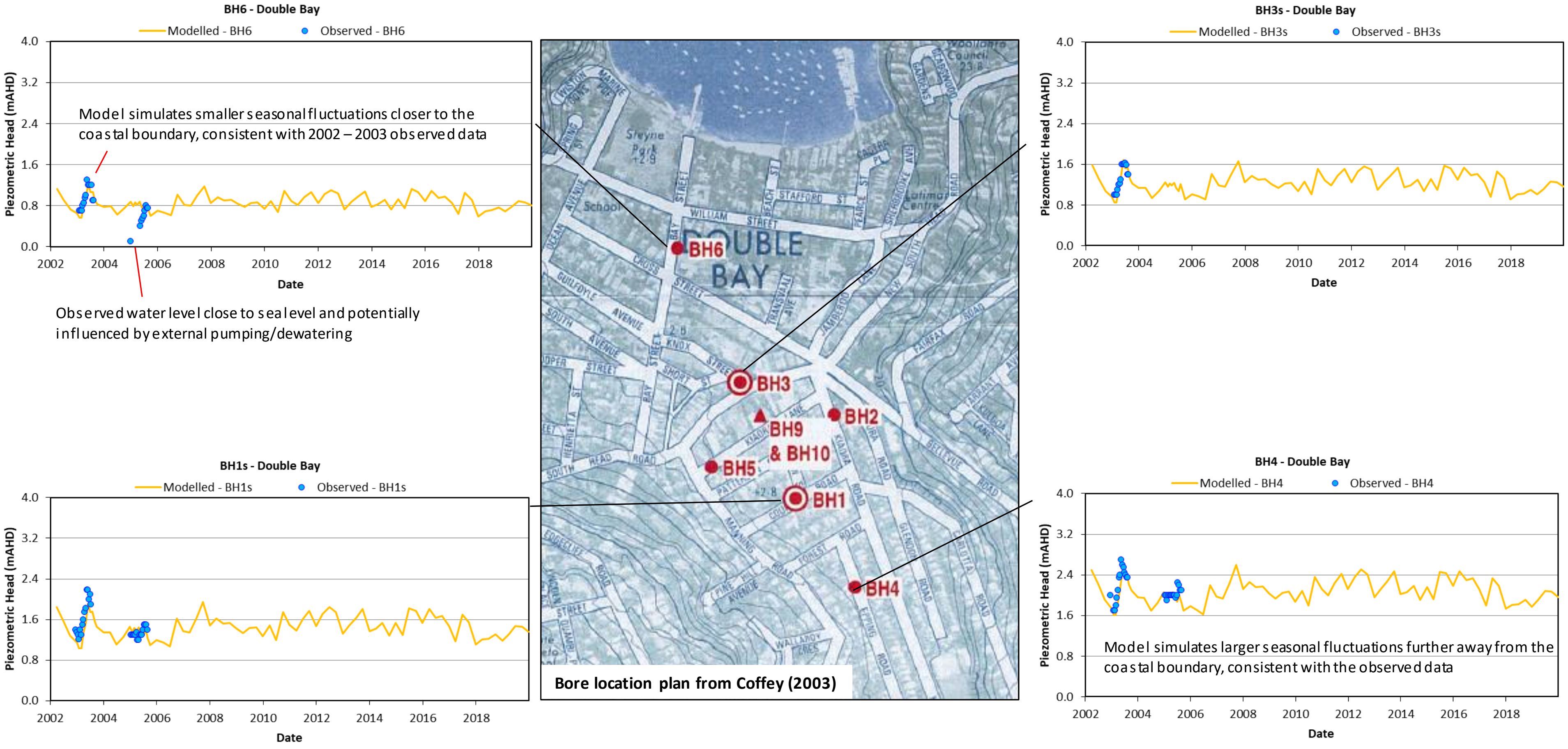
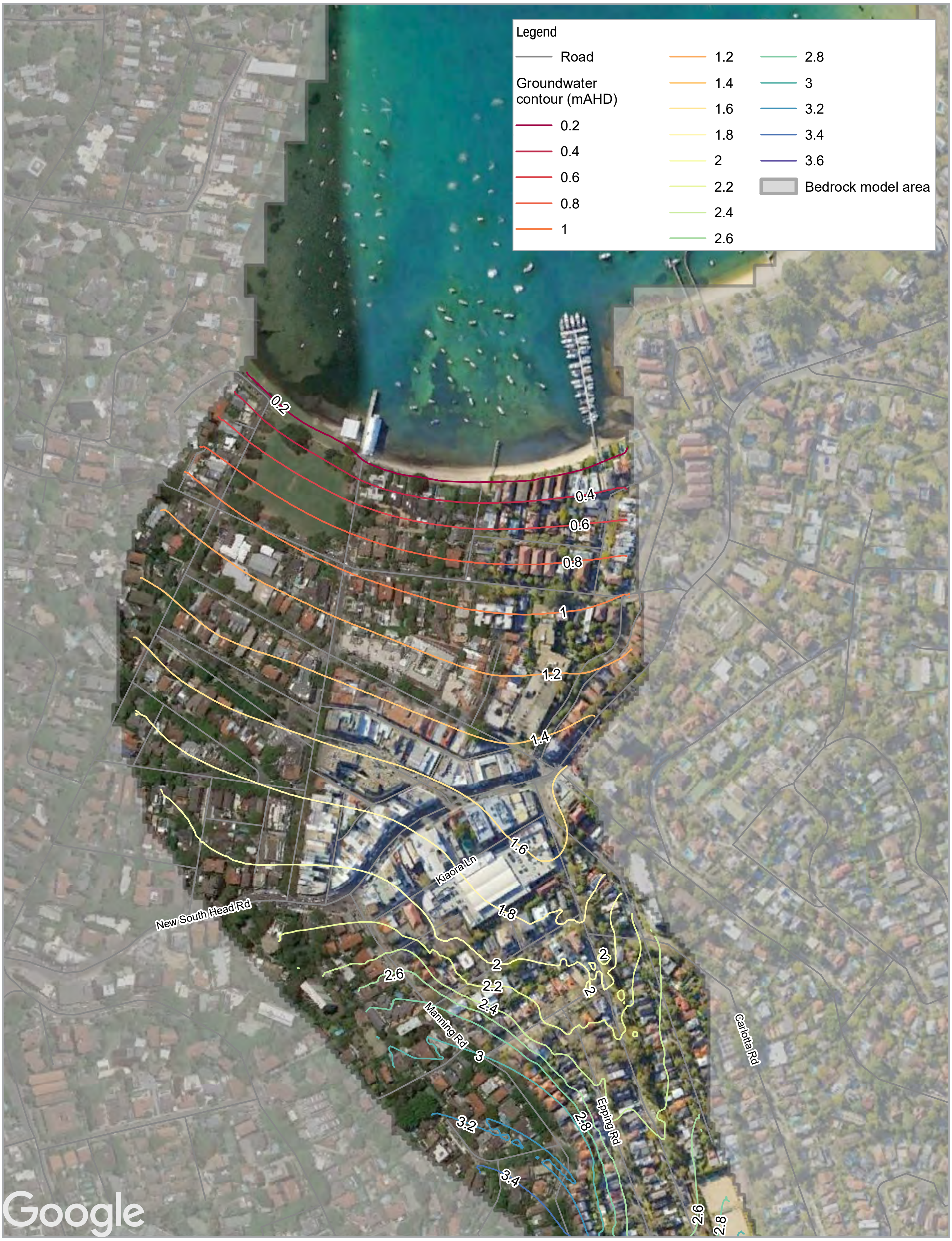


Figure 16 Modelled hydrographs



Paper Size ISO A4

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Map Projection: Transverse Mercator
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Grid: GDA 1994 MGA Zone 56

N

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Modelled groundwater contours - wet period

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FIGURE 17

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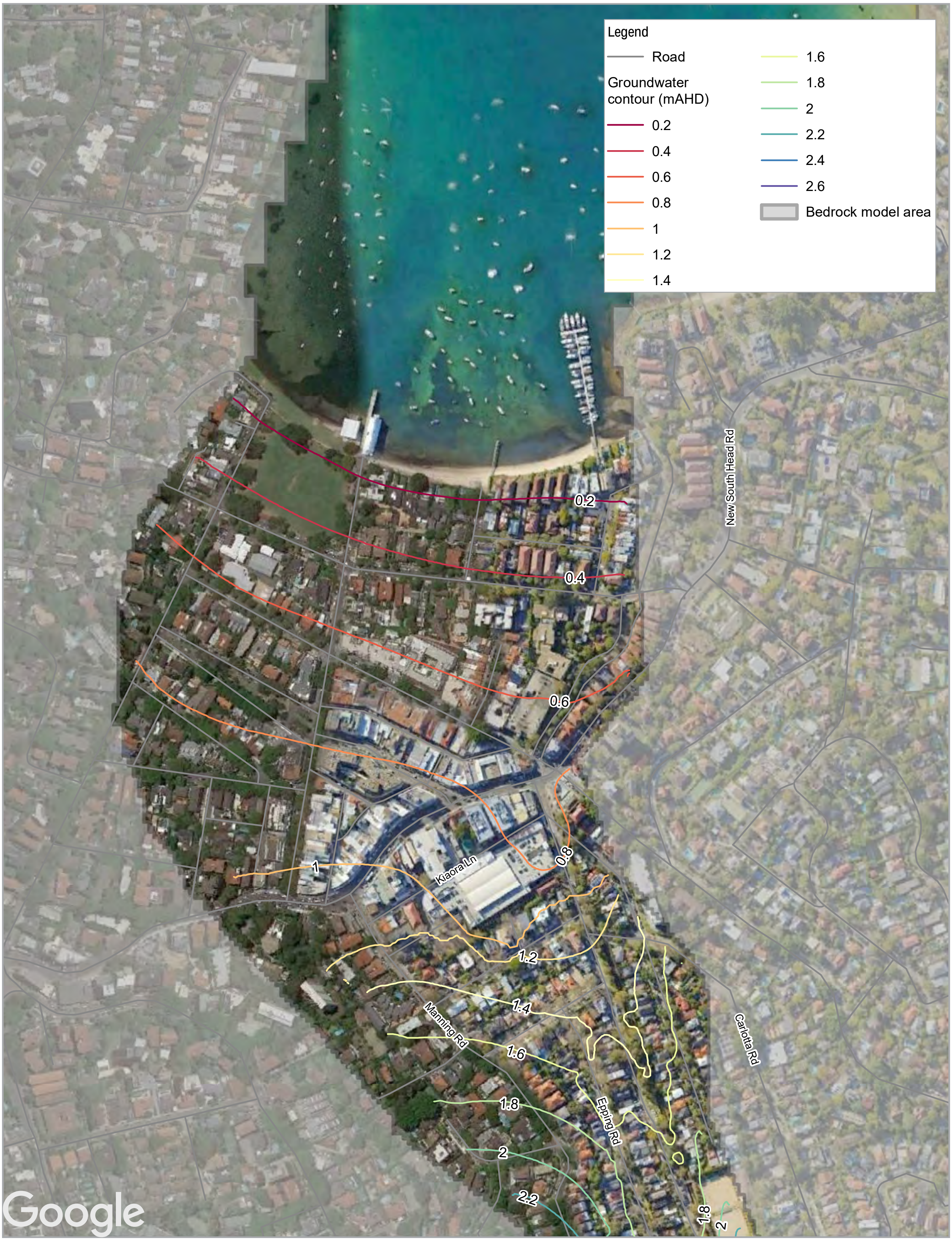

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Metres

Map Projection: Transverse Mercator
Horizontal Datum: GDA 1994
Grid: GDA 1994 MGA Zone 56

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Modelled groundwater contours - dry period

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FIGURE 18

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Print date: 22 Apr 2020 - 14:26

Data source: WPC, roads, 2019; GHD, groundwater model data, 2020; GEPro, Imagery, captured 22/08/2017, extracted 07/10/2019; . Created by: bsmyth

8.4 Model outputs to inform future developments

8.4.1 Depth to groundwater and groundwater interference risks

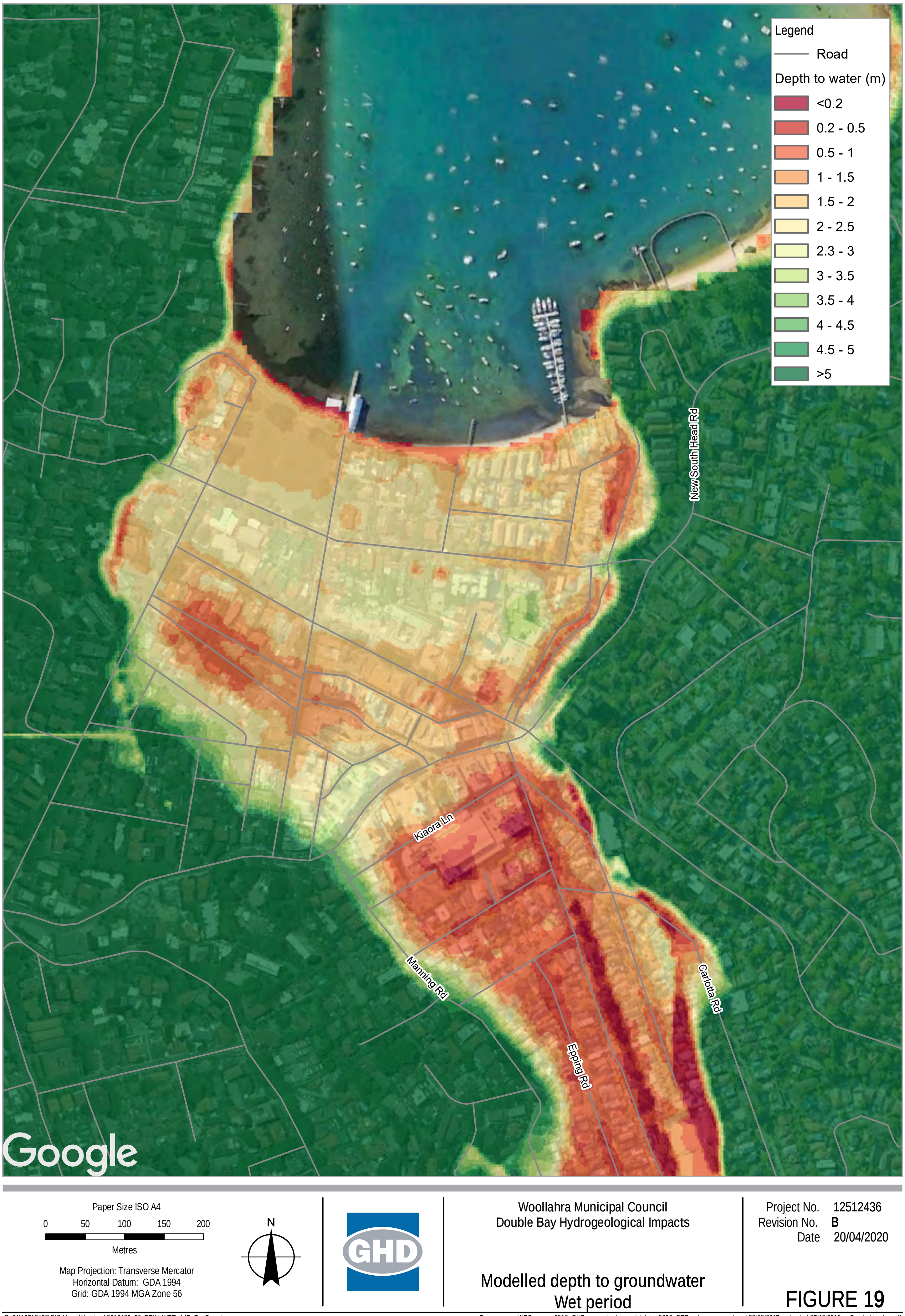
Figure 19 and Figure 20 present the maps of depth to groundwater for the wet and dry periods respectively. These maps have been generated by subtracting the modelled surface of the water table from the DEM. The maps provide indications of areas where the water table is shallow and the expected seasonal range. For example, Figure 19 indicates areas of very shallow water table along Patterson Street and Kioara Road during wet period, consistent with high groundwater levels measured in a monitoring bore located in this area (refer to BH1 in Figure 11). Similarly, a relatively narrow area of shallow water table is simulated along the drain parallel to Kiaora Road, which forms a local low point that is potentially penetrating the water table.

Within the context of potential future developments, the maps provide useful indications of the risk of groundwater interference. For example, where the depth to groundwater is shallow and is less than the proposed depth of excavation, the maps would indicate the need to consider dewatering during temporary construction works. Greater the depth of excavation relative to depth to groundwater, greater the temporary drawdown of the water table required to maintain dry/safe construction conditions. Similarly, where the peak water table is shallow, there may be the need to consider management of groundwater levels over the long term to minimise the potential for subterranean structures (basements) to exacerbate conditions of shallow water table.

Figure 21 presents an example of a groundwater interference risk map based on the wet period depth to groundwater. The map delineates areas of low to very high risk of groundwater interference based on the following classifications:

- Very high risk – the wet period depth to groundwater of <0.5 m
- High risk - the wet period depth to groundwater of 0.5 – 1 m
- Moderate risk - the wet period depth to groundwater of 1 – 2 m
- Low risk - the wet period depth to groundwater of > 2 m

The risk map is intended to assist with the preliminary screening of risks associated with long-term impacts of subterranean structures (basements), where incremental changes in the water table depth could be problematic depending on the natural range of water table. Section 8.4.2 presents outputs from predictive modelling to provide indications of the potential cumulative impact of future developments.



Map Projection: Transverse Mercator
Horizontal Datum: GDA 1994
Grid: GDA 1994 MGA Zone 56

GHD

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Modelled depth to groundwater
Wet period

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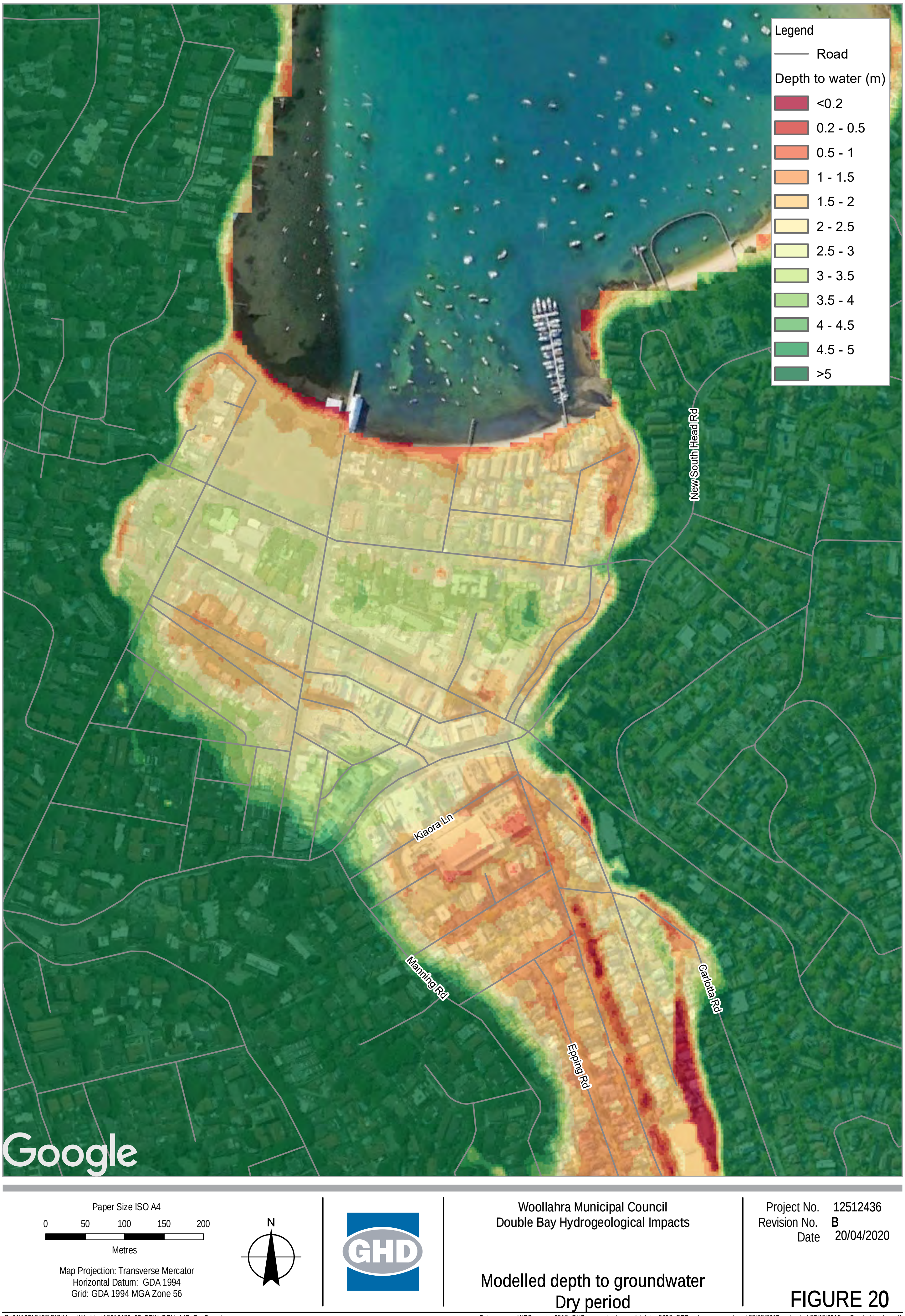
FIGURE 19

G:\31\12512436\GIS\Maps\Working\12512436_06_DTW_WET_A4P_RevB.mxd
Print date: 20 Apr 2020 - 15:55

Data source: WPC, roads, 2019; GHD, groundwater model data, 2020; GEPro, Imagery, captured 22/08/2017, extracted 07/10/2019; . Created by: bsmyth



Map Projection: Transverse Mercator
Horizontal Datum: GDA 1994
Grid: GDA 1994 MGA Zone 56

GHD

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Modelled depth to groundwater
Dry period

FIGURE 20

G:\31\12512436\GIS\Maps\Working\12512436_07_DTW_DRY_A4P_RevB.mxd
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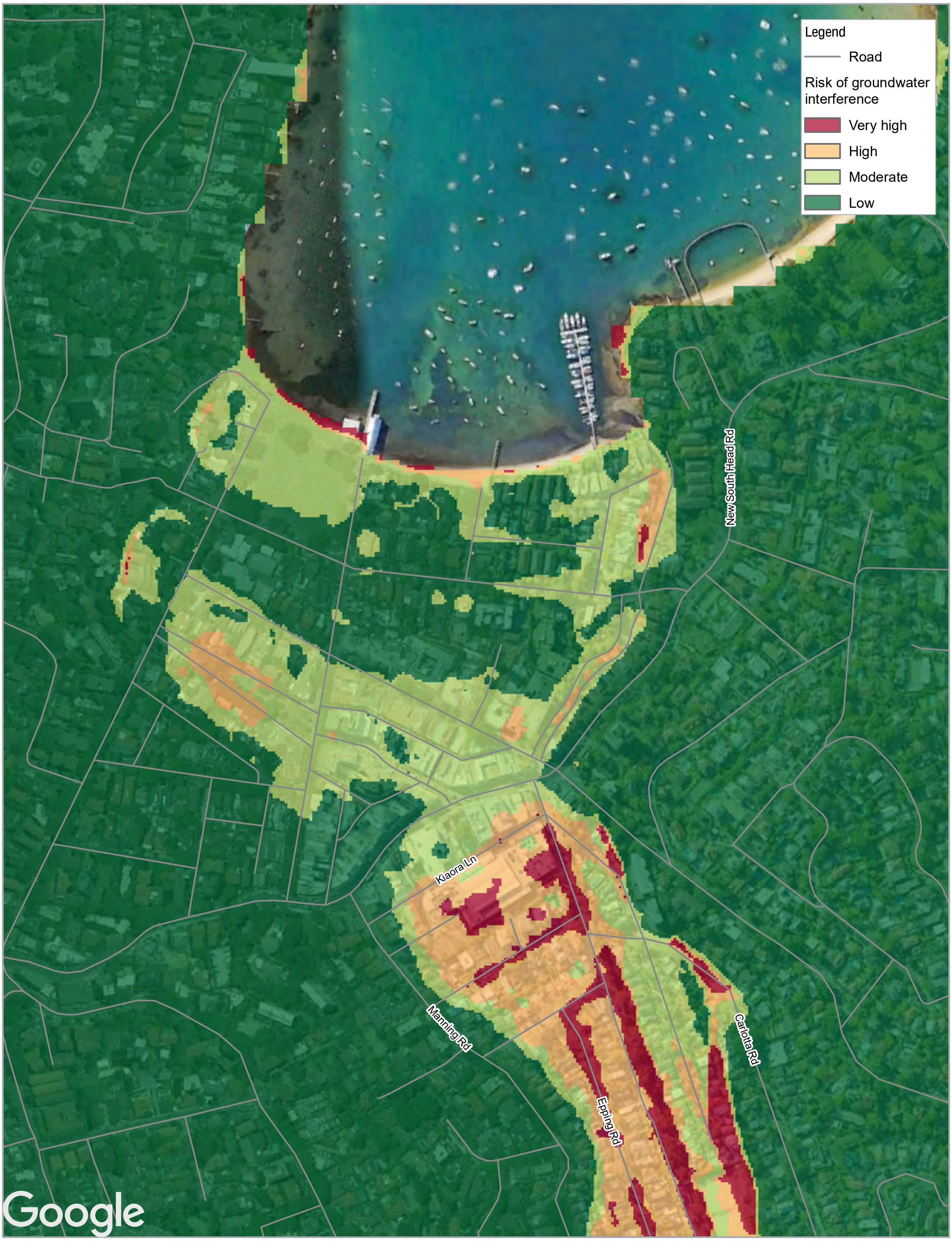

Paper Size ISO A4

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Metres

Map Projection: Transverse Mercator
Horizontal Datum: GDA 1994
Grid: GDA 1994 MGA Zone 56

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Groundwater interference risk map

FIGURE 21

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Print date: 22 Apr 2020 - 14:28

Data source: WPC, roads, 2019; GHD, groundwater model data, 2020; GEPro, Imagery, captured 22/08/2017, extracted 07/10/2019; . Created by: bsmyth

8.4.2 Potential long term changes to water table

The potential long term cumulative impacts of future basements have been assessed by incorporating these features into the calibrated model as zones of low permeability material, based on the information provided by Council. Two scenarios have been modelled:

1. A full cut-off scenario where the basements/low permeability zones are assumed to fully extend to the base of the Alluvium/ top of Bedrock.
2. A partial cut-off scenario where the basements are assumed to terminate 12 m below ground surface, allowing groundwater to flow below them.

Brief explanation regarding the cut-off scenario and its impact on the groundwater flow is provided in Section 9.1.

For both scenarios, the predictive models have been run for the same 18-year period used in the calibration so that the change in groundwater levels under a range of climatic conditions can be simulated. This is important because the effects of groundwater flow impedance are typically greater during wetter periods with steeper hydraulic gradients. The basements are represented using a low hydraulic conductivity value of 1×10^{-5} m/d with zero recharge applied over their footprint.

Figure 22 and Figure 23 present the modelled maximum change in the water table for the full and partial cut-off scenarios, respectively. Also indicated in the figures are the assumed location of basements considered in the predictive modelling and bores from the NGIS. These figures are composite maps based on the maximum change in water table simulated at every location in the model, which can occur at different times at different locations. It provides a snapshot of the maximum extent of impact. The positive change indicates drawdown (lowering) of the water table and negative change indicates mounding (raising) of the water table.

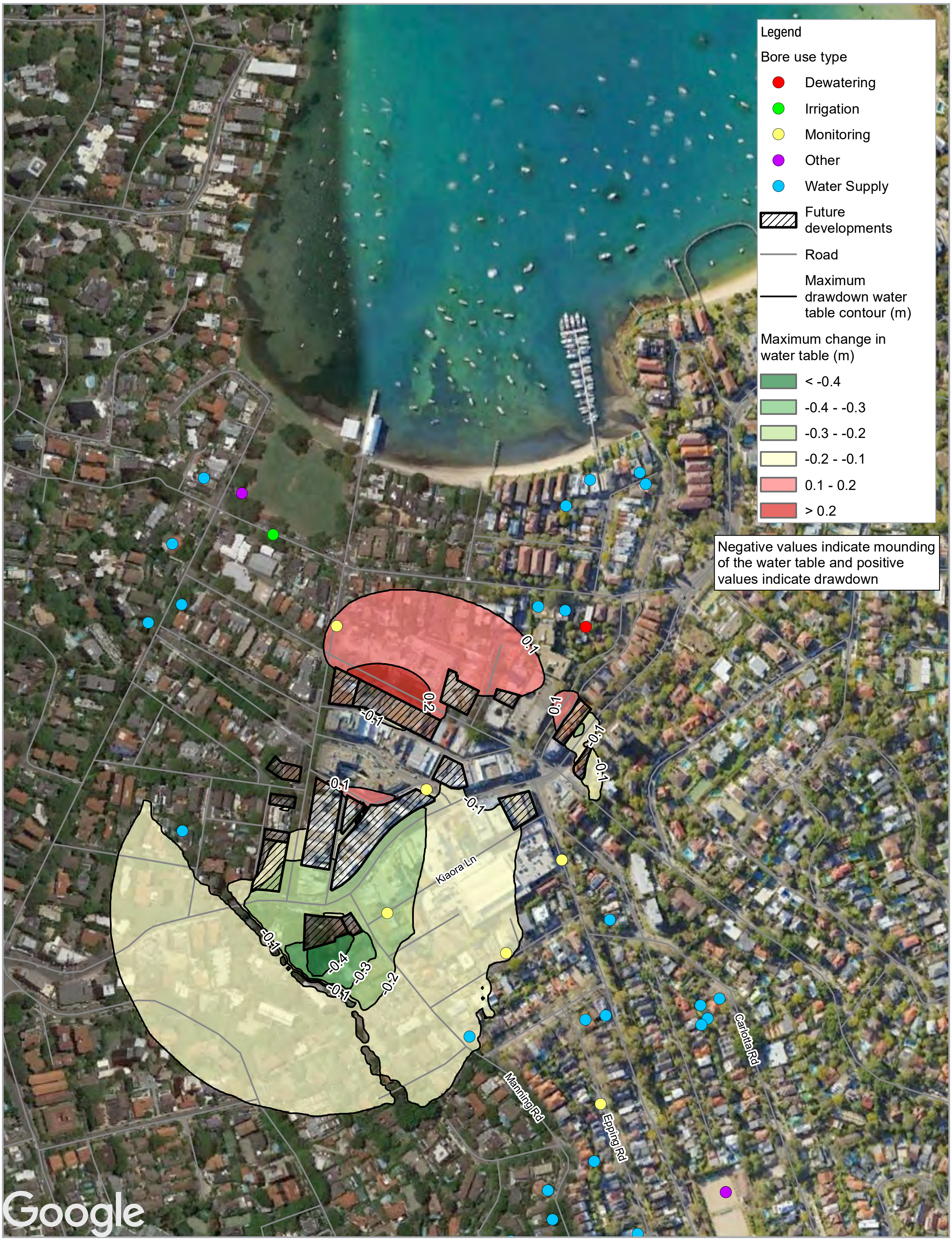
The figures indicate the following:

- The area of influence of the full cut-off scenario is larger than that of the partial cut –off scenario, as expected.
- The full cut-off results in mounding of the water table on the up gradient side and drawdown on the down gradient side due to impedance of groundwater flow. The partial cut-off results in very localised drawdown and mounding.
- The maximum drawdown and mounding simulated by the model are generally less than 0.3 m. Up to around 0.4 m of mounding is simulated in the southwest, where groundwater flows from the valley edge; however, this occurs in an area of low risk of groundwater interference where the depth to groundwater is greater than 2 m.

The cumulative effect may result in mounding of 0.1 to 0.2 m in high risk areas between Kiaora Road and Court Road where the water table is known to reach close to ground surface. This has the potential to increase the extent of shallow groundwater albeit an incremental change of <0.2 m would be difficult to quantify in practice. Only one water supply bore from the NGIS is located within the modelled area of influence, on the boundary of the 0.1 m mounding contour.

8.4.3 Limitations

Hydrogeological systems are complex natural systems whose properties cannot be measured at all spatial and temporal scales. While the regional model has been appropriately calibrated, reliable long term data are not available at all locations within the study area and uncertainty is inherent in model outputs. As additional data become available, the model can be updated progressively and confidence in model outputs would increase over time.



Paper Size ISO A4

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Map Projection: Transverse Mercator
Horizontal Datum: GDA 1994
Grid: GDA 1994 MGA Zone 56

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Contours of maximum change in water table - full cut-off

FIGURE 22

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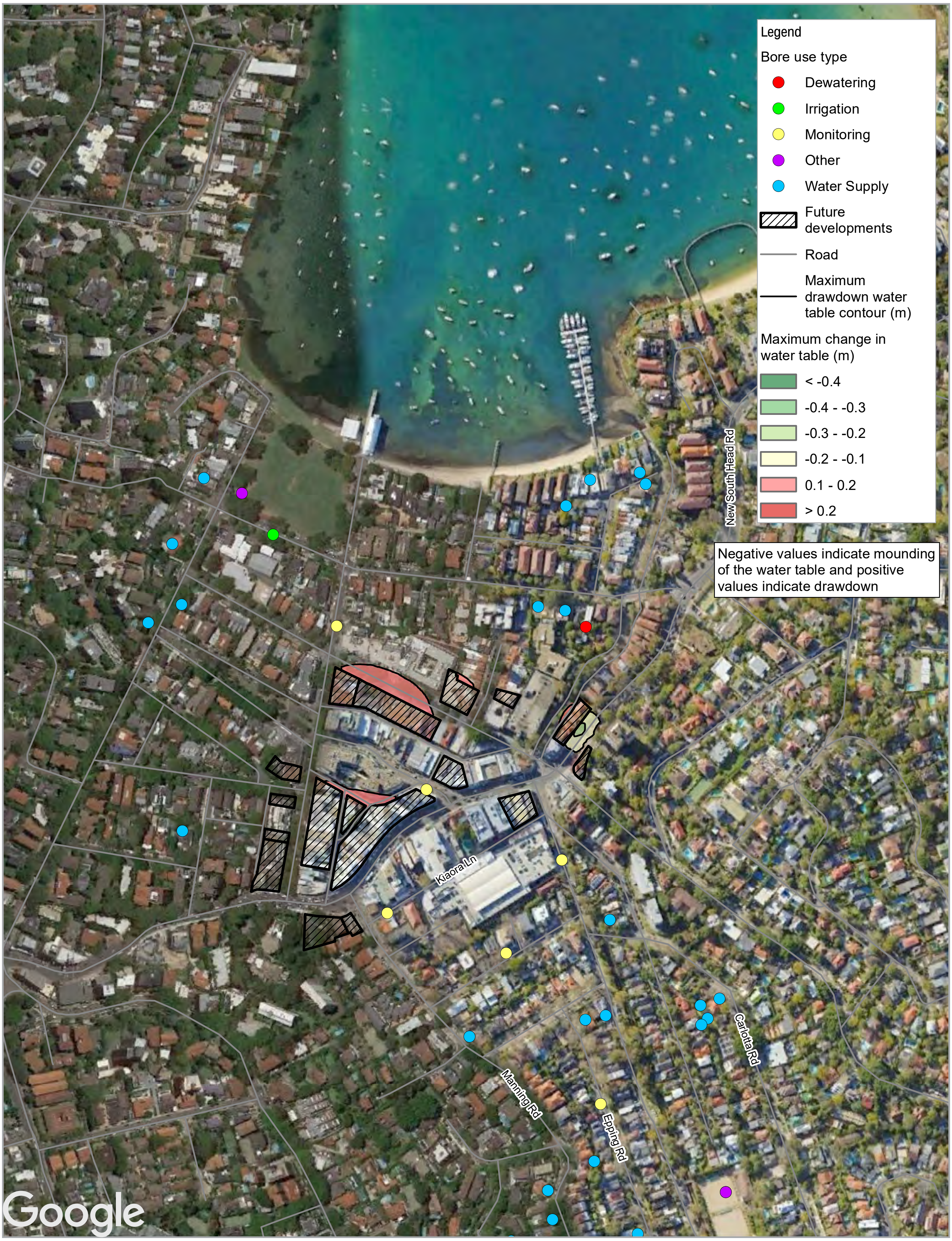
Data source: WPC, roads, 2019; GHD, groundwater model data, 2020; BOM, NGIS bore data, 2019; GEPro, Imagery, captured 22/08/2017, extracted 07/10/2019; . Created by: bsmyth



Paper Size ISO A4

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Metres

Map Projection: Transverse Mercator
Horizontal Datum: GDA 1994
Grid: GDA 1994 MGA Zone 56

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GHD

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Contours of maximum change in water table - partial cut-off

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Date 22/04/2020

FIGURE 23

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Data source: WPC, roads, 2019; GHD, groundwater model data, 2020; BOM, NGIS data, 2019; GEPro, Imagery, captured 22/08/2017, extracted 07/10/2019; . Created by: bsmyth

9. Impact of groundwater lowering (construction dewatering)

9.1 Why buildings settle upon dewatering

When new developments involves basement construction, excavation into existing ground is required. Such excavation when carried out below the water table can be difficult to execute and the excavation side walls often become unstable due to the presence of groundwater within the construction site. In that instance, side wall retention and dewatering can be adopted to facilitate the excavation work and to allow the construction to proceed in dry soil conditions.

There are many forms of retention system for the support of the excavation side walls. In areas where groundwater flow rate is high, vertical cut-off walls are often adopted to act as both the retention system and flow barrier to control groundwater seepage. However, the cut-off walls need to be properly designed to minimise the groundwater flow into the excavation site effectively. Such cut-off walls could be constructed as full depth penetration by extending the walls to the relatively impermeable bedrock or as partial cut-off system. In situations where the groundwater seepage into the excavation is relatively high and the cut-off system does not extend to full depth, dewatering within the construction site is employed to supplement the cut-off system.

However, such dewatering method could cause the lowering of water table not only within the site and in the vicinity of the excavation footprint, but also extend to a certain distance away from the excavation. The extent or zone of influence of dewatering depends on a number of factors including the cut-off system, initial groundwater conditions, and ground conditions, etc.

The lowering of water table by dewatering can induce soil settlement which is detrimental to buildings and structures located above the affected water table. When the water table is lowered, the effective load on the underlying soil is increased by amount equal to the difference between the drained and submerged weights of the entire soil mass between the original and lowered water table. If the underlying soil is compressible, the increased overburden pressure will cause compression of the soil, inducing settlement of the ground.

Such phenomena could occur in most soil types. However, in situations involving weak compressible soils, dewatering can cause more substantial settlements. When there are spatial variability in ground conditions within a given region, it is clear that difference in settlement (i.e. differential settlement) can be expected.

Such total settlement and differential settlement will then impact the structures bearing on the ground surface including residential and commercial buildings, leading to movement and distortion of the structures.

9.2 Adopted settlement criteria

To effectively control the potential damage caused by dewatering, it is essential to assess the maximum acceptable settlement for the buildings in the Double Bay area. The settlement criteria applicable to the existing buildings, typically one to two storeys constructed on shallow footings, have been developed primarily based on Australian Standards AS2870-2011 and relevant published literature by Burland *et al*. (2002) on building settlements and associated damages. Other considerations including possible past damages of the buildings, flexibility of the structures, pipe drain tolerances and groundwater fluctuation have also been given as part of the assessment process.

9.2.1 Assessment of settlement criteria

The Australian Standard AS2870-2011 has been developed for the purpose of site classification, design and construction of foundation systems associated with typical residential buildings. This standard also provides typical surface movements for various site classifications along with the related damage category.

Consistent with the works presented by Burland *et al*. (2002), AS2870-2011 provides five categories of damage with reference to walls, numbered 0 to 4 in increasing severity. Normally categories 0, 1 and 2 relate to 'aesthetic' damage, 3 relates to 'serviceability' damage and 4 represents damage affecting 'stability'. Burland *et al*. (2002) have indicated that the dividing line between categories 2 and 3 damage is particularly important. If the damage exceeds Category 2 the cause is usually much easier to identify and is frequently associated with ground movement. To minimise the residual risks of property damages in Double Bay, the design settlement criterion should be selected based on a more cautious Category of 1 or better.

Cracking in masonry walls is usually, but not always, caused by differential settlement. With reference to the schematic representation shown in Figure 24 regarding the deflection ratio Δ/L at which cracking is initiated, Burland (1997) provided the limiting Δ/L values in percentage for the different categories of damage for masonry wall with zero horizontal strain (see Table 11). With a clear notion of minimising the risks of property damages in the Double Bay area, the threshold for a cautious damage Category 1 was considered. Then using Δ/L of 0.075% (maximum value for category 1) and for a building comprising full masonry construction with a typical wall length of 20 m, a differential wall settlement of 15 mm could be adopted as the maximum tolerable value before cracking become visible and is classified as being at risk of Category 2 damage.

In relation to pipe drain tolerances, the acceptance criteria of 0.1 degree for joint rotation of relatively rigid pipes such as cast iron pipe can be adopted based on consultation with Sydney Water for past projects, as well as CIRIA (1996) publication titled "Prediction and effects of ground movements caused by tunnelling in soft ground beneath urban areas". The aforementioned threshold deflection ratio of 0.075% corresponds to a rotation of about 0.043 degrees, which is deemed to be satisfactory for the allowable joint rotation of rigid pipes.

Theoretically correct and simple as it may seem, the evaluation of differential wall settlement is not always straight forward. Alternatively, total ground (surface) settlement limits could be used as an ultimate measure to control damage of buildings caused by dewatering. Table 2.2 of AS2870-2011 indicates that damage categories 0 to 1 for masonry (veneer or full) are normally present in Class S site, where the site classifications are defined in Table 2.1 of AS2870-2011. Further, the characteristic surface movements (y_s) for Site S is 0 - 20 mm in accordance with Table 2.3 of AS2870-2011. This threshold surface movement is commensurate with the above differential wall settlement of 15 mm for category 1 damage. If the building is conservatively assumed to have no stiffness so that it conforms to the 'greenfield site' subsidence trough, then it is possible to consider y_s to be conservatively the same as the differential wall settlement. The adoption of this conservative assumption is reasonable because the surface settlement limit that is applicable to existing buildings will have to be assessed in light of possible past damage and flexibility of the buildings. Relatively rigid and damaged structures now are likely to be more sensitive to increased surface movement due to loss of stiffness, and therefore some reduction in the settlement limit might be appropriate. The above differential wall settlement of 15 mm occurs within the conditions of Class S Sites, where damage Category 1 ('aesthetic' damage) is applicable. In Tables C1 and C2 of AS2870-2011, damage Category 1 is described as fine cracks to walls and concrete floors of less than 1 mm which typically do not need repair.

For the purposes of current assessment of dewatering, we have considered a total ground surface settlement of 15 mm as being the limiting value to control potential damage of existing buildings.

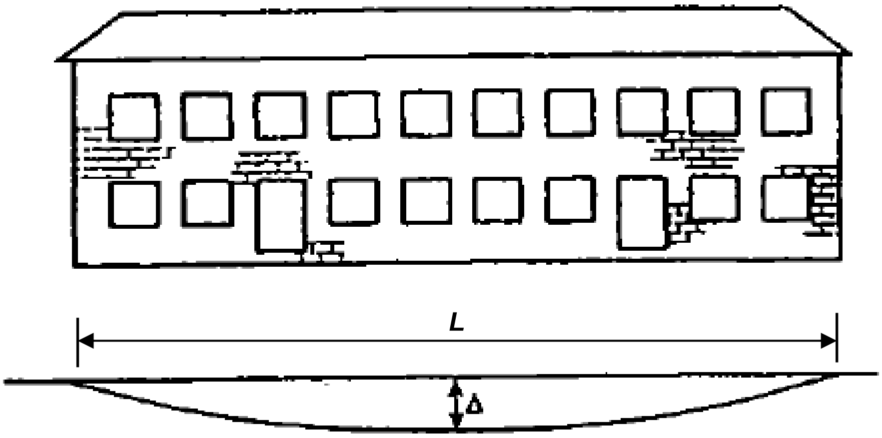

Figure 24 Schematic representation of wall deflection

Table 11 Relationship between category of damage and limiting Δ/L for zero horizontal strain in accordance with Burland *et al.* (2002)

Category of damage	Normal degree of severity	Limiting Δ/L (%)
0	Negligible	0.05
1	Very slight	0.075
2	Slight	0.15
3	Moderate	0.3
4	Severe to very severe	> 0.3

9.2.2 Surface settlement and water table fluctuation

The amount of settlement which could be induced into the existing buildings in the vicinity of a construction site will depend upon the extent of external water table lowering caused by the dewatering and the intrinsic soil properties.

For a water table fluctuation of typically within 1 m, the surface settlement caused by the initial loading (i.e. the initial drop of groundwater level) would be the greatest. The settlement induced by the subsequent groundwater variation of the same magnitude would be only about one-tenth to one-half of that experienced under initial loading. Therefore, if the drawdown of the groundwater level is within the range of the water table fluctuation, then the induced surface settlement is anticipated to be small and should be similar to that observed due to groundwater variation. However, a further drop in water table beyond the historic groundwater fluctuation range would lead to settlements of increased magnitude rapidly approach the estimate for the initial loading. For the assessment of dewatering induced settlement presented in the following sections, our adopted initial groundwater level has generally been based on the relatively low side of the fluctuation range in accordance with the available groundwater records.

10. Geotechnical Assessment of dewatering-induced settlement

10.1 Methodology

10.1.1 Overview of assessment approach

The general methodology for the geotechnical assessment of settlement can be described as per the flowchart given in Figure 25 below. In essence, the severity of the dewatering-induced settlement is strongly related to ground conditions present on site. For example, the lowering of groundwater in areas with presence of highly compressible peaty soils would cause a much greater settlement than other areas without peat. It is essential to examine the variability of ground conditions and to identify areas susceptible to ground movements upon dewatering. Therefore, the “first part” of the settlement assessment was to develop site specific geotechnical models and to assess and compare the ground settlement responses upon dewatering for the different sub-divided areas within the Double Bay study area. These results were presented on ‘settlement index plots’ to provide a visual identification of areas with different degrees of ground settlement response to groundwater drawdown due to construction dewatering. The “second part” of the settlement assessment was to establish the relationship between dewatering of the developments and the groundwater lowering for the different sub-divided areas identified in the first part of the assessment. The ultimate goal of the assessment is to establish groundwater drawdown limit that can be used to develop recommendations in relation of dewatering controls.

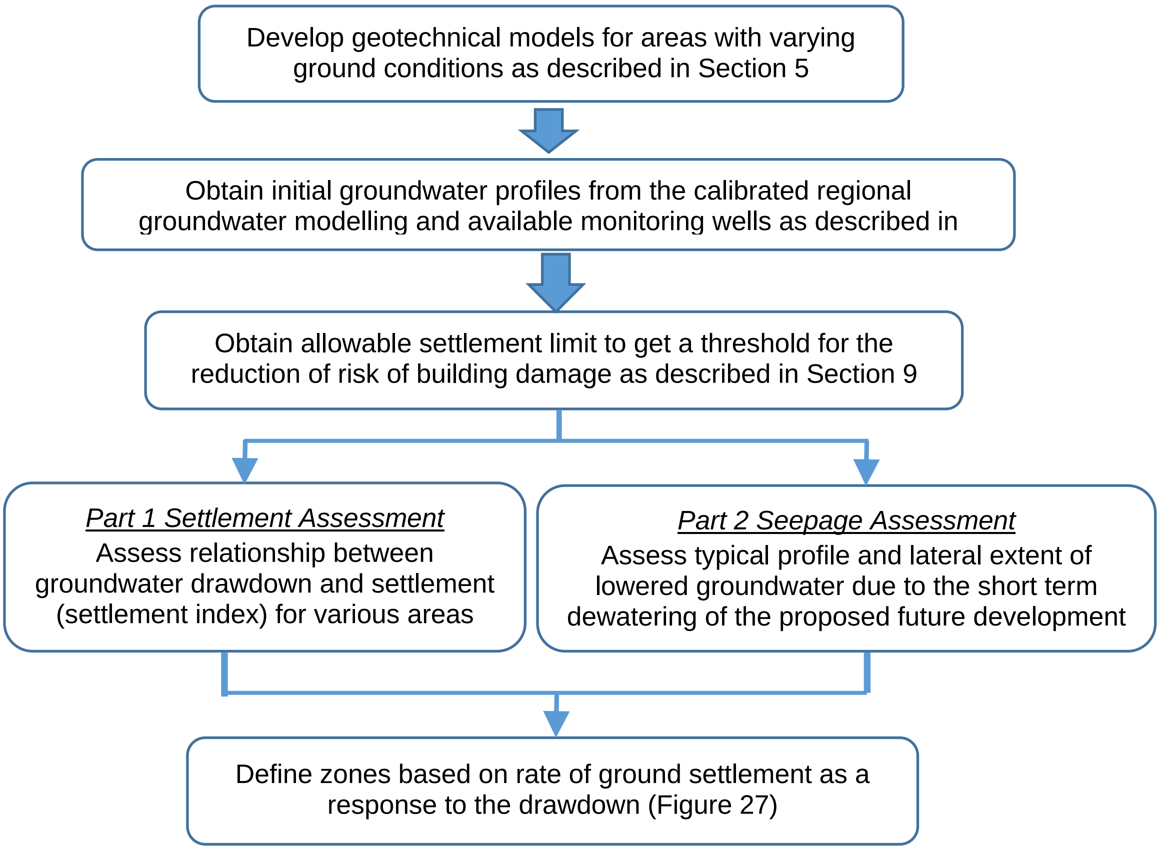

Figure 25 Flow chart showing general methodology for the settlement assessment

10.1.2 Settlement index analysis

Settlement analyses corresponding to predefined depths of groundwater drawdown were conducted for the majority of the data points outlined in Figure 1. For each data point, site specific geotechnical model was developed based on the available geotechnical investigation data. The results of all assessed settlement points were subsequently compiled to form a "Settlement Index Plot" in response to an assumed fixed groundwater drawdown depth. The drawdown depth of up to 5 m was considered because an uncontrolled dewatering of 2-level basement construction could potentially result in up to 5 m lowering of the original water table.

Based on this Settlement Index Plot together with the consideration of the spatial variability in ground conditions between the data points, a more generalised settlement map was developed, which shows degrees of susceptibility to dewatering-induced ground surface settlement for different sub-divided zones within the Double Bay study area. The settlement index provides a means to understand the response of ground settlement to various levels of drawdown for a given location, irrespective of any profile of groundwater drawdown caused by any particular development.

The dewatering induced settlement for each data points was analysed based on one-dimensional (1D) method where soil layers were modelled as follows:

- Elastic models with characteristic Young's moduli for granular materials
- Consolidation models with recompression and compression coefficients for fine grained soils

The compressibility properties adopted for the different soil / rock units are summarised in Table 12. These engineering parameters were derived on the basis of:

- Available information from past studies (e.g. GHD Longmac's Groundwater and Geotechnical Study in 2001, ref. R1)
- Review of in-situ testing results from available geotechnical investigation data
- Review of geotechnical laboratory testing results where available
- Use of empirical or semi-empirical correlations applicable for similar soil types
- Our experience on local geology, projects with similar soil types and challenges

Groundwater levels obtained from the regional groundwater modelling as well as from calibration against monitoring wells were adopted as initial groundwater levels in the settlement analyses. This assumption had to be made as the actual timing of future construction relative to the dry and wet seasons was not known at the time of our assessment.

10.1.3 Seepage analysis - relationship between dewatering and groundwater lowering

The influence of dewatering at the development sites on the levels of groundwater lowering in the adjacent areas depends on a number of factors including depth of dewatering, depth of the cut-off level in relation to an impermeable sub-soil layer, and the soil types generally encountered on site. Seepage analysis has been carried out to assess typical characteristics of groundwater lowering due to future construction dewatering. The analysis was conducted by using two-dimensional (2D) Finite Element (FE) method by means of a commercially available computer program Seep/W (Geostudio 2019).

The short-term seepage analysis was carried out on 5 geotechnical sections (Sections AA, BB, CC, DD and EE) as outlined in Figure 6.

The employed procedure for seepage analysis is described below:

- Each of the analysed models was extended between the fixed boundaries at which the influence of the dewatering could be considered to be negligible due to constant water recharge or discharge. For Sections AA and EE, the model started at the uphill side at the south and ended near the harbour at the north where the water discharged into the bay. These represented the two boundary conditions with fixed total head. The remaining geotechnical sections were extended between the eastern hillside and the western hillside.
- The initial conditions were modelled and compared against the regional groundwater modelling and available monitoring well data to calibrate the assumed total head at the boundary conditions as well as the foundation permeability.
- Only one development excavation was considered in each analysis. The selected future development was based on the excavation which would likely induce the highest magnitude of drawdown depth and furthest lateral impact covering greater extent beyond both sides of the excavation. Typically this was related to the deepest excavation with respect to the elevation and partial depth cut-off (refer Section 9.1 for theoretical background)
- The size and depth of basement excavation was modelled as per the details supplied by Council in the Brief. Where this information was not provided in the brief for a given location, a 2-basement excavation with dewatering and partial cut-off was assumed. As the detailed configuration of the adopted retention system and dewatering plan are not available at the time of this assessment, the following assumptions were incorporated:
  - The adopted retention system has been conservatively assumed to provide partial cut-off and extended up to a minimum depth below the excavation of equal to the excavation depth. This assumption was necessary as the seepage flow path depends on the groundwater cut-off condition (refer to Section 9.1). For a 2-basement excavation, the excavated depth was assumed to be typically about 6 m below existing ground surface. The depth of the retention system that also served as partial groundwater cut-off was assumed to be typically about 12 m below existing ground surface.
  - The groundwater was lowered to about 0.5 m deep below the base of excavation by using spear points installed along the perimeter of the retention system inside the excavation footprint.
- The model (Geotechnical Section AA) extending along the main groundwater flow direction was calibrated against known information of groundwater drawdown likely induced by the construction of 4-8 Patterson St. Based on this information (ref. R9 and R17), it is understood that the groundwater at 14 Forest Road was encountered at about 2 to 2.55 m at the time of investigation which roughly occurred during the construction of 4-8 Patterson St where the dewatering took place.

Additional seepage analysis was also conducted to assess the impact of full-depth cut-off for comparison purposes. This latter analysis was carried out for the assumed future development at 1 Cross Street which comprise 4-level basement as per the Brief. Although the basement excavation for 1 Cross St (ref. Figure 1 for location) development will be relatively deep compared to those of other developments, the site is located adjacent to the hillside in the northeast of the Double Bay study area. As such, we have assumed a full-depth cut-off system for this development (ref. Figure 35).

Table 12 Summary of geotechnical properties for all foundation units

Unit/Material	Bulk Unit Weight (kN/m^3)	Compressibility parameters for fine-grained soil		Undrained Shear Strength s_u (kPa)	Young's Modulus of Elasticity E for sandy soils (MPa)	Young's Modulus of Elasticity E for sandy soils (MPa)	
		Compression Ratio CR (1)	Recompression Ratio RR (2)			Horizontal permeability k_h (m/day)	Vertical permeability k_v (m/day)
1 – Fill	18	N/A	N/A	N/A	10	5	0.5
2A – Very soft to soft Clay	16	0.1	0.014	12	N/A	4.3×10^{-5}	4.3×10^{-6}
2B – Firm Clay	17	0.1	0.014	30	N/A	2.1×10^{-5}	2.1×10^{-6}
2C – Stiff to hard Clay	19	0.1	0.014	85	N/A	8.6×10^{-6}	8.6×10^{-7}
3A – Very loose to loose Sand	17	N/A	N/A	N/A	5	2.5	0.25
3B – Loose to medium Dense Sand	18	N/A	N/A	N/A	10	1.5	0.15
3C – Dense to very dense Sand	20	N/A	N/A	N/A	30	1.0	0.1
3D – Mix of Sand and Clay (typically Clayey Sand)	18	N/A	N/A	N/A	10	0.2	0.02
4A – Very soft to soft Peat 4B – Very soft Sandy Peat	14	0.35	0.058	7	N/A	4.3×10^{-4}	4.3×10^{-5}
4C – Firm Peat	15	0.325	0.054	30	N/A	8.6×10^{-5}	8.6×10^{-6}
4D – Stiff to hard Peat	17	0.3	0.05	85	N/A	8.6×10^{-6}	1.3×10^{-7}
5A – Residual Soil (Clayey Sand)	19	N/A	N/A	N/A	50	1.0	0.1
5B – Extremely to highly weathered Sandstone	21	N/A	N/A	N/A	100	0.1	0.01
5C – Moderately weathered to Fresh Sandstone	23	N/A	N/A	N/A	250	0.02	0.002

Notes to Table 12:

(1) CR = $c_c/(1+e_0)$ where c_c is compression index and e_0 is the initial void ratio.

(2) RR = $cr/(1+e_0)$ where cr is recompression index and e_0 is the initial void ratio.

10.2 Settlement Index Plot and settlement zones

The analysed settlement index obtained for various drawdown depths was used to assess the sensitivity of ground settlement response to the groundwater drawdown due to construction dewatering. The contours of assessed settlement index in response to an assumed 1m depth of groundwater drawdown are presented as Figure 26. This assessed Settlement Index Plot shows similarity to the isopach map of upper peat layer thickness depicted in Figure 8 in terms of the locations of peat and the assessed settlement concentrations.

Note that there are inherent uncertainties associated with the settlement index plot owing to the following factors:

- Potential spatial variability in ground conditions between settlement points that could result in differential settlements beneath structures
- Uncertainty within locations where sufficient investigation data was not available.

Based on the Settlement Index Plot depicted in Figure 26 and the uncertainties outlined above, a more generalised settlement map was developed that delineates three settlement zones with different degrees of susceptibility to dewatering-induced ground surface settlement. The settlement zones and their descriptions are given in Table 13 below. The zones and their extent were superimposed in a plan with topographical contours as shown in Figure 27. Further, the variation of settlement with drawdown depths was plotted for various areas. These plots are shown as Figure 28 to Figure 31. The significance of these settlement plots are further discussed in Section 10.4.

Table 13 Description of various Settlement Zones

Zone Assignment	Description	Typical Settlement for given Drawdown Levels
A	Areas which are highly sensitive to drawdown due to the ground conditions. Consequently, higher settlement magnitude can likely occur and adversely impact adjacent properties.	• Settlement of more than 15 mm for 1 m drawdown depth • Differential settlement which can exceed 15 mm for drawdown up to 4 – 5 m
B	Areas which are moderately sensitive to drawdown due to the ground conditions. Although the assessed settlement was generally less than Zone A, this zone can continue settling with the increase in drawdown due to thicker soil profile or compressible layer located at deeper strata.	• Settlement of up to 15 mm for 1 m drawdown depth • Settlement can exceed 15 mm for excessive drawdown up to about 4 – 5 m
C	Areas which are less sensitive to drawdown due to ground conditions (e.g. shallow bedrock, lower original water table with respect to soil layers)	• Settlement of less than 5 mm for 1 m drawdown depth • Settlement is likely to be limited with the increase in drawdown depth due to shallow rock profile

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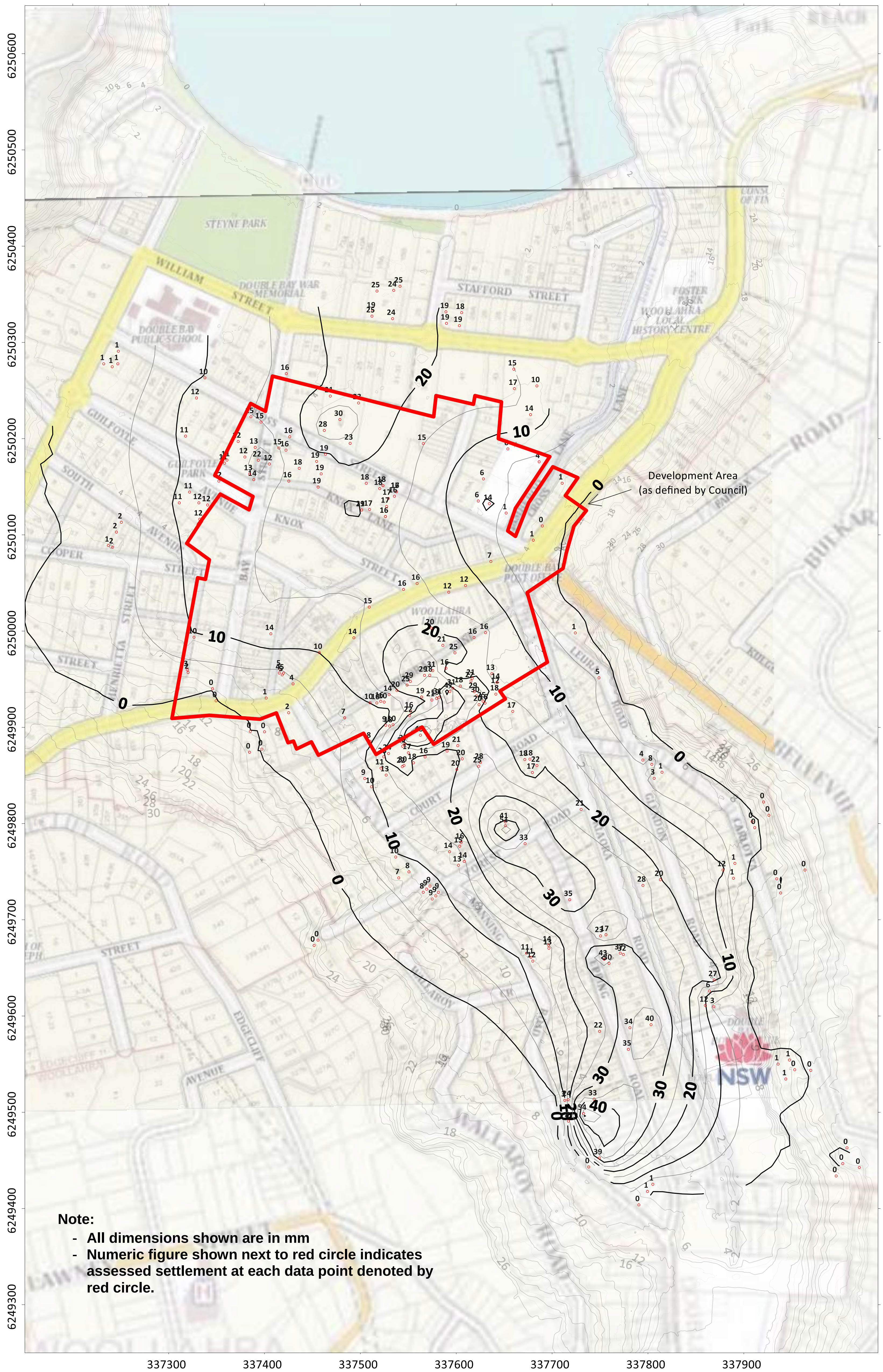


Figure 26 Contours of assessed settlement index for 1-m drawdown depth

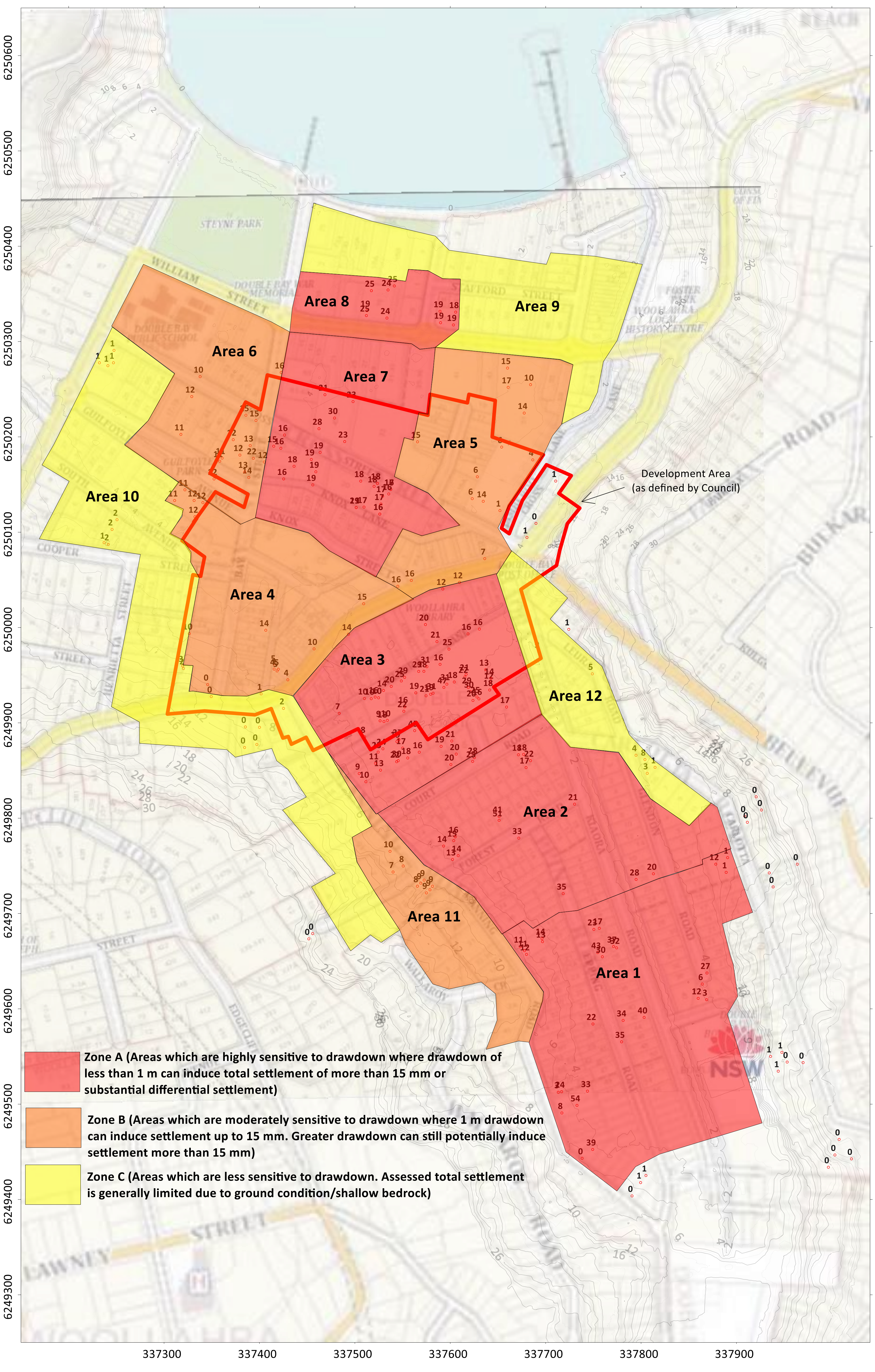


Figure 27 Settlement zones and their extent on a plan with topographical contours

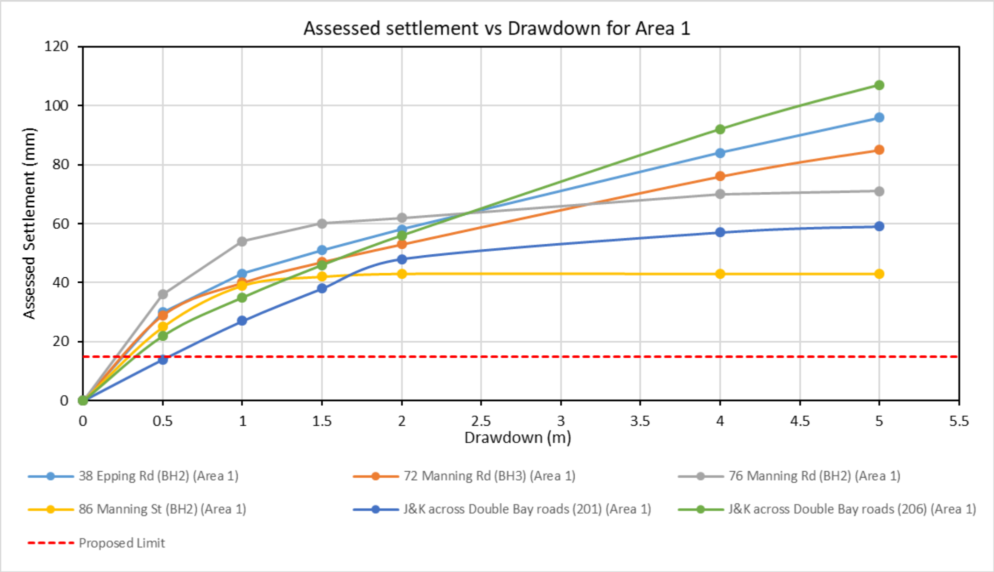


Figure 28 Settlement Index for various drawdown for Area 1 (Settlement Zone A)

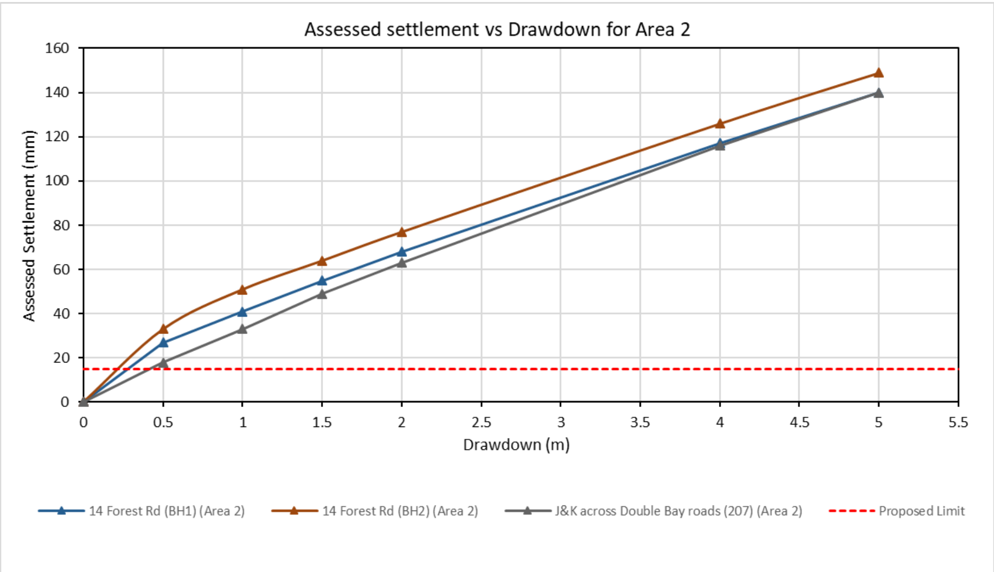


Figure 29 Settlement Index for various drawdown for Area 2 (Settlement Zone A)

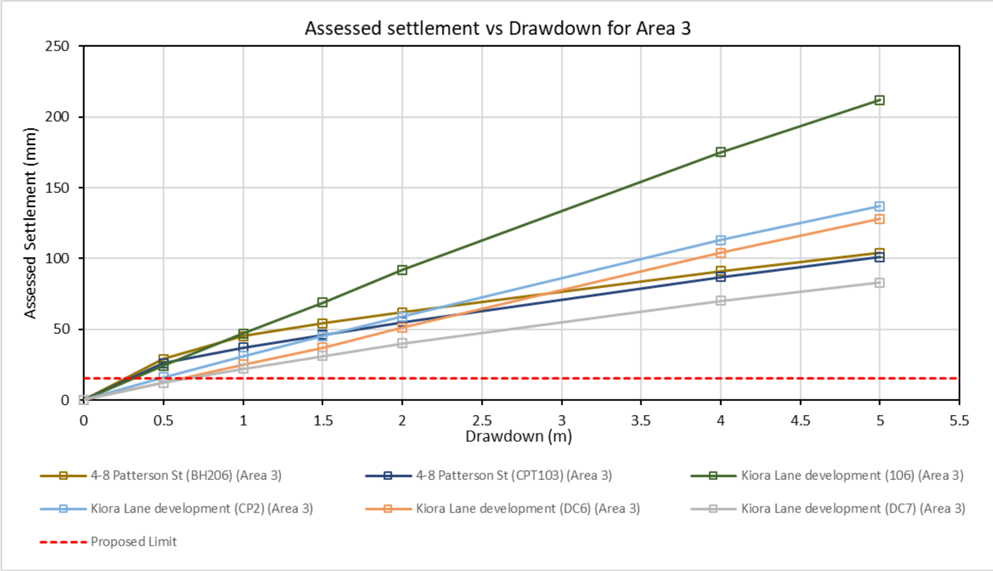


Figure 30 Settlement Index for various drawdown for Area 3 (Settlement Zone A)

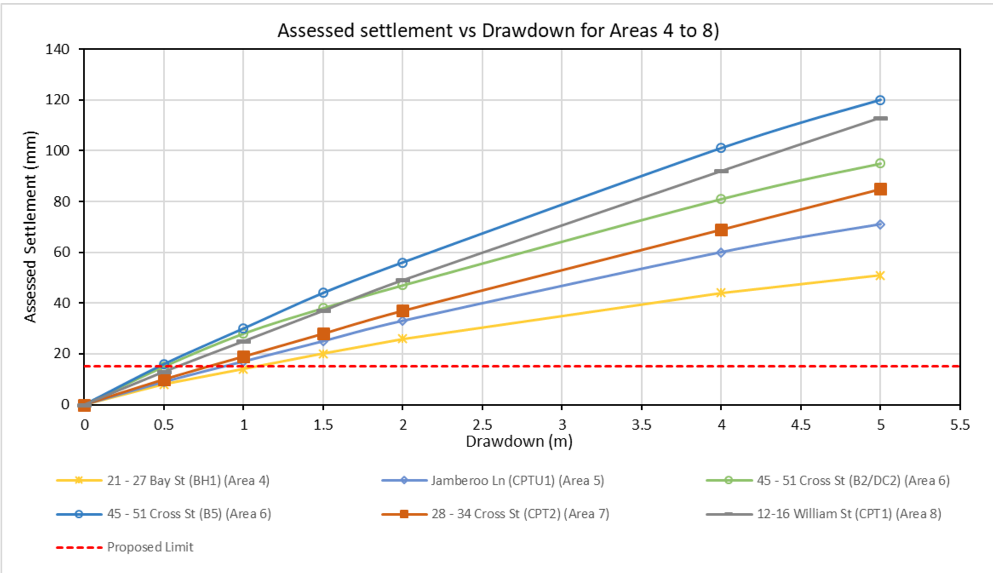


Figure 31 Settlement Index for various drawdown for Areas 4 to 8 (Settlement Zones A and B)

10.3 Seepage analysis

The groundwater levels related to the initial conditions for geotechnical section AA is indicated in Figure 7. For other geotechnical sections, the initial groundwater levels adopted in the seepage analysis are given in Appendix B (Figures B1, B3, B5 and B7 for Geotechnical Sections BB, CC, DD and EE, respectively).

Figure 32 shows the seepage analysis result for geotechnical section AA carried out for calibration purposes. This analysis incorporated the construction work at 4-8 Patterson St and the consequential lowering of the water table as observed within the property of 14 Forest Road. By adopting the configuration of retention system given in the respective reports (refs. R7 and

R8) as well as the dewatering depth of 0.5 m deep below the base of excavation, the resulting groundwater drawdown at 14 Forest Rd was assessed to be consistent with that observed during the investigations (refs. R9 and R17).

The change in water table due to the construction dewatering of the future development (7 – 17 Knox St) is presented as Figure 33 for geotechnical section AA. The resulting groundwater levels from this analysis are plotted in Figure 34 for comparison.

Similar assessment was also carried out for other geotechnical long sections by adopting the future development which caused the highest magnitude of drawdown. The results of these assessments are presented as Figures B2, B4, B6 and B8 in Appendix B.

Figure 35 presents the groundwater profile induced by the construction of 1 Cross St where dewatering and full depth cut-off were assumed. The result indicates that the assessed groundwater profile due to the full-depth cut-off underwent only minor change from the initial groundwater level for geotechnical section BB (Figure B1 of Appendix B) despite the application of dewatering within the excavation.

It should be noted that all figures referenced above were plotted with either 2 times or 4 times vertical scale exaggeration to fit the report page. As a result, any inclined straight line can appear to be have a steeper slope than what the actual slope is.

10.4 Discussions

10.4.1 Settlement due to short-term dewatering

The results of our seepage analysis pertinent to the impact of short term construction dewatering can be described as follows:

- The shape of lowered groundwater profile as a result of construction dewatering appeared to be relatively flat (refer to Figure 32 and Figure 33). This observation can usually be expected in soil layers with relatively high permeability such as sandy soils.
- Due to the shape of the lowered groundwater profile, the impact of construction dewatering in sandy soil layers can be expected to extend a fair distance beyond the excavation footprint (refer to Figure 33 and Figures B2, B4 and B6 in Appendix B). Based on Figure 34, this lateral impact can extend up to 800 m away from the excavation near the recharge point at the sandstone hillside. In a 3-dimensional context, this impact can cover substantial areas located within the above distance or areas between the excavation and hillside, whichever is the least, beyond all four sides of the excavation.
- Since groundwater reinjection/recharge was not considered in our seepage analysis, the depth of groundwater drawdown in the immediate vicinity of the excavation footprint was similar to that within the excavation where the dewatering took place. It is inferred that a drawdown of up to 4 – 5 m can potentially occur in the nearby area if appropriate control measures are not put in place.

The groundwater drawdown will induce settlement as described in Section 9. As inferred by the seepage analysis result (Figure 33) for geotechnical section AA, the settlement-related impact of such drawdown could extend over a distance of up to about 800 m which is close to the uphill boundaries.

The magnitude of assessed settlement will depend on the original and lowered groundwater levels as well as the ground conditions. The settlement index analysis has been conducted by considering an increasing drawdown depth of up to a maximum of 5 m below the original groundwater levels as shown in Figure 28 to Figure 31. These figures indicates the following points:

- The general trend of the analysis results indicates that the greater the groundwater drawdown depth, the greater surface settlement will be experienced in the different sub-divided areas. For example, the total settlement at Kiaora Lane in Area 3 can be up to as high as 210 mm for a total drawdown depth in excess of 5 m as shown in Figure 30.
- The shape of Settlement Index Plot gives an indication regarding the ground conditions. This can be discussed as follows:
  - It is interesting to highlight that the assessed total settlement experienced at certain areas in Area 1 towards Double Bay South (e.g. 76 and 86 Manning St) appear to plateau beyond 2 m depth of groundwater drawdown (see Figure 28). The main reason to this assessed behaviour is that the upper peat layers within this area are generally occurred at shallow depth and are within or above the existing groundwater fluctuation range. The further drop of water table due to dewatering will not incur additional loading to these shallow peat layers.
  - Settlement index assessed for data points located in the areas where bedrock is relatively deep indicate continuing increase in settlement with the increase in the groundwater drawdown. This increase is expected to continue further until the lowered groundwater level reaches the layer that is least susceptible (i.e. bedrock) to the drawdown induced settlement.
- Some variability in the assessed total settlements could be observed among the data points located within the same area. These spatial variability become more pronounced with the increase in total settlements which consequently can increase the risk of the occurrence of differential settlement. It can be recalled from Section 9.2.1 that certain value of differential settlement would be sufficient to increase the risks of building damage.
- For the different subdivided areas, the allowable drawdown depths associated with proposed settlement limit of 15 mm were assessed to vary between 0.2 m and 1.2 m. A corollary of this finding is that a 0.2 m depth of dewatering can be considered as a relatively safe limit to control building damage.
- As discussed above, the impact of dewatering the groundwater and water table drawdown could extend up to a considerable distance away from the dewatering location due to a relatively flat shape of the lowered groundwater profile. By considering this along with the sensitivity of ground settlement response in some areas to the drawdown, it is practical that the safe limit of dewatering of 0.2 m is applied throughout the Double Bay area.

From constructability viewpoint, it can be necessary to dewater sufficiently to enable the dry excavation during construction. If the above drawdown limits cannot be achieved, other controls are also available to reduce groundwater drawdown in the adjacent areas to within the acceptable limit. These include the following:

- Systematic groundwater reinjection/recharge during excavation dewatering
- Sufficient cut-off depth to limit groundwater drawdown outside of the excavations
- Elimination of the need for dewatering by providing a sealing layer on the excavation base which needs to be adequately designed to resist uplift pressure

Alternative measures can be considered on a case-by-case basis to allow for a review of the drawdown limit. These measures should include the undertaking of sufficient additional geotechnical investigations and subsequent analysis to demonstrate that settlement impacts of surrounding building are within acceptable limit.

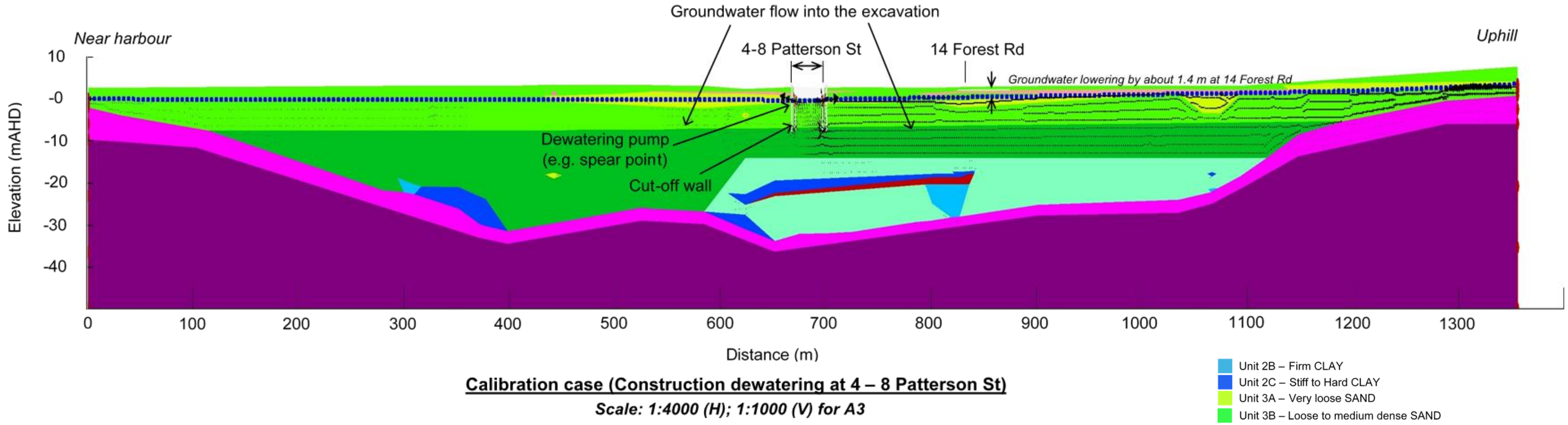
It is noted that the water table will likely stabilise to a level that is near the original groundwater levels (see Section 8.4.2) following a certain period after the dewatering is terminated. However, the settled ground and other environmental features impacted by the groundwater drawdown will not likely return to the original conditions.

10.4.2 Settlement due to long term drawdown

As described in Section 8.4.2, assessment of long term impact of the completed developments on the regional groundwater condition has been addressed by the regional groundwater modelling. This impact is expected to be mainly related to the cut-off system left in place which can affect the long-term groundwater flow. For this assessment, the impact of full depth cut-off was assumed to provide a critical scenario due to more blockage to groundwater flow. Our assessment indicates that the drawdown values due to the presence of full depth cut-off (ref. Figure 22) and partial depth cut-off (ref. Figure 23) considering all future developments (Figure 1) are about 0.3 m and 0.2 m, respectively.

The maximum drawdown induced by the presence of full depth cut-off (i.e. all future basement structures extending to bedrock) was assessed to be more than the proposed safe limit of groundwater drawdown of 0.2 m (per Section 10.4.1). Therefore, the permanent groundwater cut-off provided by full-depth basement structures without any mitigation measures should be avoided as part of future developments.

It can be inferred from our assessment that the groundwater drawdown of no more than 0.2 m can be achieved by limiting the permanent groundwater cut-off to a maximum of 12 m deep below the ground surface (i.e. partial depth cut-off). Alternatively, some forms of drainage measures could be adopted for full depth cut-off walls to control the long term impact of the completed developments on adjacent existing structures.



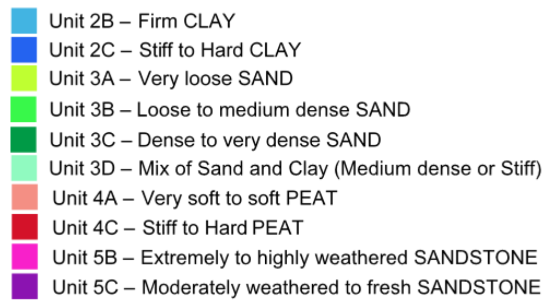


Figure 32 Seepage Analysis showing groundwater drawdown due to the construction dewatering of 4-8 Patterson Rd (Geotechnical Section AA)

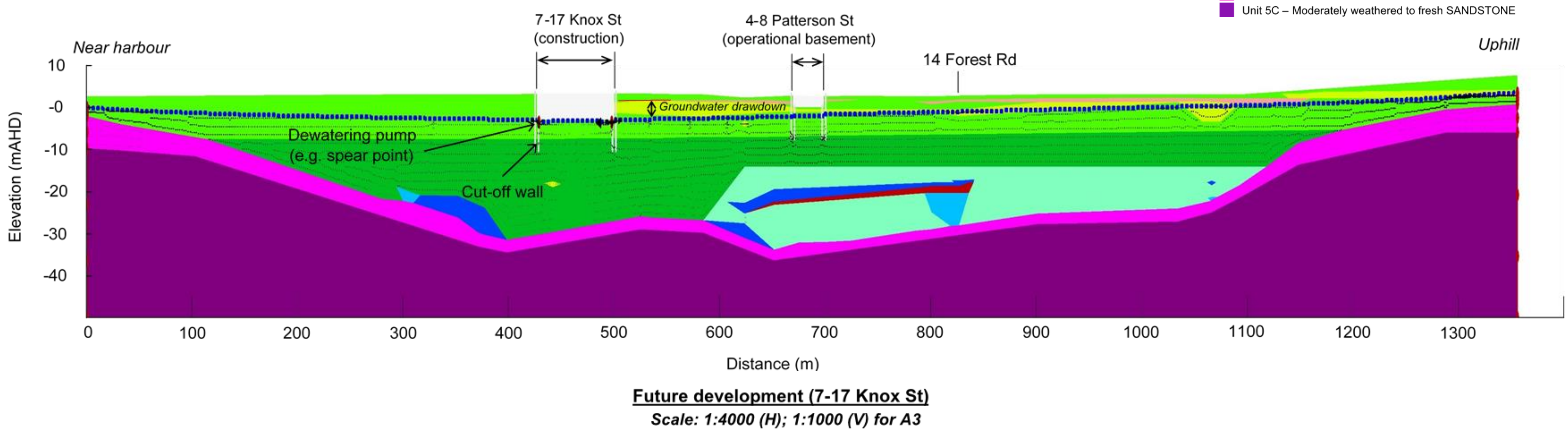


Figure 33 Seepage Analysis showing groundwater drawdown due to the future construction dewatering at 7 – 17 Knox St (Geotechnical Section AA)

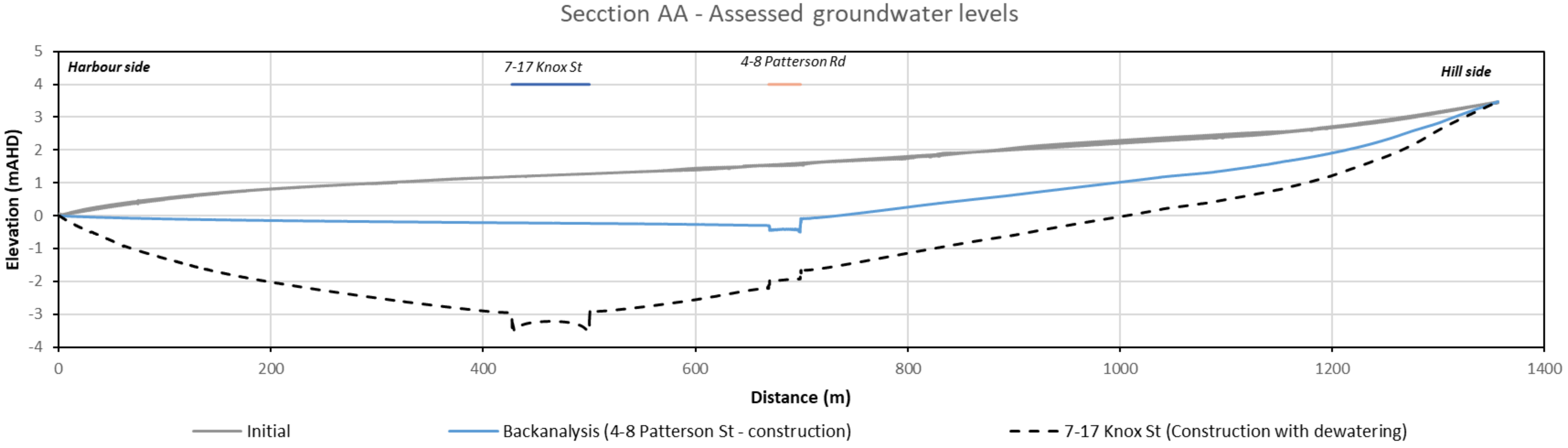


Figure 34 Seepage Analysis showing groundwater drawdown due to the construction dewatering for Geotechnical Section AA

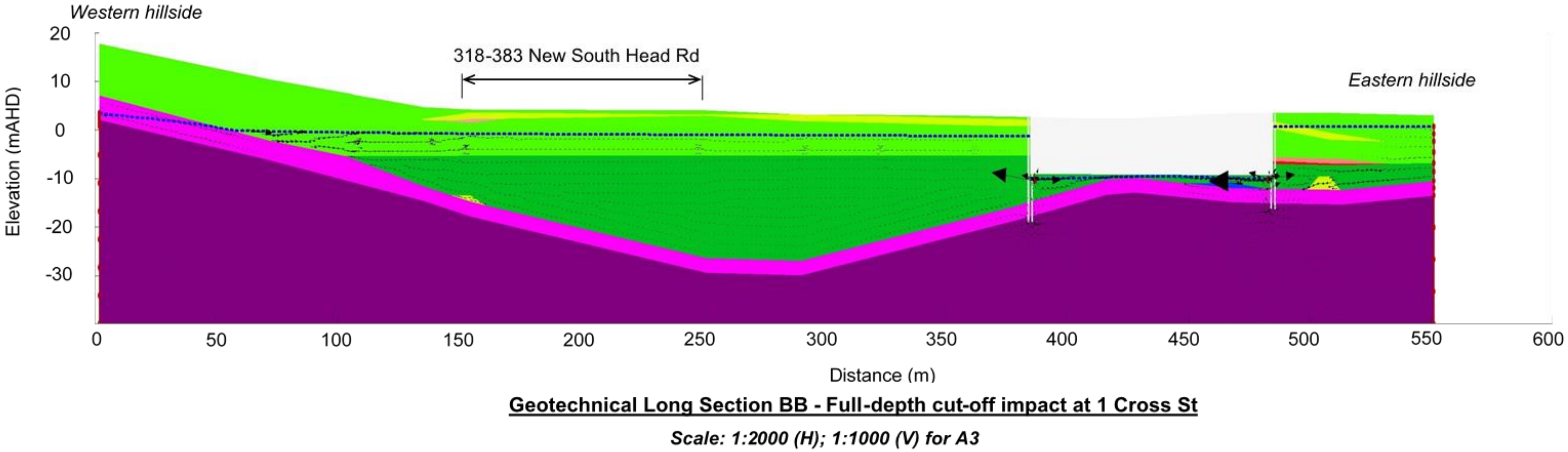


Figure 35 Seepage Analysis showing groundwater drawdown due to the construction dewatering of 1 Cross St with full-depth cut-off (Geotechnical Section BB)

11. Summary

The normally consolidated sediments within the valley underlying the Double Bay area form a highly productive water table aquifer (Alluvium), which is underlain by the less permeable fractured Bedrock aquifer. The Alluvium, comprising sand with minor silts, clay and peat, has high hydraulic conductivity and is readily replenished by rainfall-derived recharge, resulting in fresh groundwater with salinity of typically less than 400 mg/L. The water table fluctuates in response to seasonal variations in rainfall, with up to 1 m of variation observed in monitoring bores constructed within the Alluvium. In the area between Court Road and Epping Road, the water table has been observed to reach close to ground surface following wet periods. Groundwater within the Alluvium flows to the north, towards the coastal boundary which acts as a major point of discharge of groundwater. Groundwater also flows locally from topographically elevated areas on the valley edges to low-lying areas in the valley centre. The seasonal water table variations are less pronounced closer to the coastal boundary where the water table is constrained at mean sea level. The search of the Australian Groundwater Explorer identified 28 water supply bores and one irrigation bore within the Double Bay area, which are potentially utilising the shallow groundwater resource.

Due to the shallow water table in the Double Bay area, there is high potential for future developments to interact with groundwater. The nature of interaction may be short term, during construction when the water table is lowered to enable dry excavations, or long term when the basements are constructed below the water table and alter the natural flow regime. To assist with the quantification of potential impacts and risks, a regional groundwater model has been developed and calibrated to available groundwater level data, using hydrogeological parameters that are considered realistic based on prior investigations and conditions observed to date.

The modelling provides an indication of areas within Double Bay that are naturally susceptible to shallow water table following wet periods, when the water table reaches close to ground surface. The depth to groundwater map, and associated seasonal range, provides useful screening tools for identifying future developments that have high potential/risk of groundwater interference. The modelling of cumulative impacts associated with multiple subterranean structures (basements) has shown that mounding and drawdown of the water table could occur over the long term albeit this is generally estimated to be less than 0.3 m conservatively assuming full depth cut-off (basements extending to the Bedrock), with mounding of <0.2 m in areas of shallow water table. On the other hand, the assessed drawdown due to cumulative impacts associated with multiple basements with partial depth cut-off is 0.2 m.

Our seepage analysis indicates that short term construction dewatering has a potential to lower the water table in the vicinity of the excavation to almost the same level as that in the excavation. Although the magnitude of this lowering reduced with the increase of distance away from the excavation, this lowering can occur over a long distance due to relatively high permeability of sand layers. This potentially covers a substantial majority of the Double Bay study area where residential and commercial buildings are located.

By referring to the Settlement Index Plots, such excessive dewatering if uncontrolled can potentially result in substantial amount of drawdown which can induce a total settlement in excess of 210 mm. Relatively high magnitude of total settlement and spatial variability in ground conditions are expected to increase the differential settlement. It should be noted that some structures, particularly old buildings and buried pipes, are sensitive to differential settlement. Consequently, an allowable settlement limit of 15 mm has been proposed for the purpose of this study based on the relevant Australian Standard AS2870-2011 and widely referred literature on the topic of building damage (Burland *et al.*, 2002). The corresponding dewatering drawdown to cause 15 mm settlement varies between 0.2 m and 1.2 m for areas grouped as Settlement Zone A (red) and Settlement Zone B (orange) respectively.

Imposing a drawdown limit to an acceptable value of 0.2 m is expected to assist in limiting the settlement and differential settlement to values related to ‘aesthetic’ damage category. The risk of settlement impact to the structures is still present if the assessed groundwater drawdown due to uncontrolled dewatering exceeds 0.2 m in some areas. The developed settlement zones can be used to highlight various areas and their sensitivity of settlement response to various drawdown depths.

For practical and constructability purposes, a drawdown which is greater than the acceptable limit may be required to allow for a dry condition in a multi-level basement construction. In this case, additional control measures should be put in place such as the reinjection of groundwater, controlled provision of full depth cut-off system or base seal capable of resisting uplift pressure. Alternatively, a review of this limit can be considered on a case-by-case basis by undertaking additional site investigations and impact assessment for the affected structures.

GHD understands the appropriate limits and control measures mentioned above will need to be documented in a Development Control Plan. It is expected that the outcome of this Geotechnical and Hydrogeological Study can be used as inputs to the formulation of this plan.

Appendices

Appendix A – List of supplied information

Set of information	Reference ID	Property associated with the Geotechnical Investigation Report [1]	Issued by
Information Package 1 from Council	R14	1 Transvaal Avenue	JK Geotechnics
	R15	10 Leura Road	JK Geotechnics
	R16	12 Carlotta Road	JK Geotechnics
	R17	14 Forest Road	JK Geotechnics
	R18	14 Pinehill Avenue	JK Geotechnics
	R19	15 Cooper Street	JK Geotechnics
	R20	17 Carlotta Road	JK Geotechnics
	R21	17 Ocean Avenue	JK Geotechnics
	R22	18-20 Carlotta Road	JK Geotechnics
	R23	19 Court Road	JK Geotechnics
	R24	20 Epping Road	JK Geotechnics
	R25	20 Glendon Road	JK Geotechnics
	R26	26 Epping Road	JK Geotechnics
	R27	31 Epping Road	JK Geotechnics
	R28	324 New South Head Road	JK Geotechnics
	R29	38 Epping Road	JK Geotechnics
	R30	38 Ocean Avenue	JK Geotechnics
	R31	45 Carlotta Road	JK Geotechnics
	R32	450 New South head Road	JK Geotechnics
	R33	47 Carlotta Road	JK Geotechnics
	R34	5 Carlotta Road	JK Geotechnics
	R35	55 Carlotta Road	JK Geotechnics
	R36	6 Transvaal Avenue	JK Geotechnics
	R37	72 Manning Road	Longmac
	R38	76 Manning Road	JK Geotechnics
	R39	8 Court Road	JK Geotechnics
	R40	382 New South Head Road	JK Geotechnics

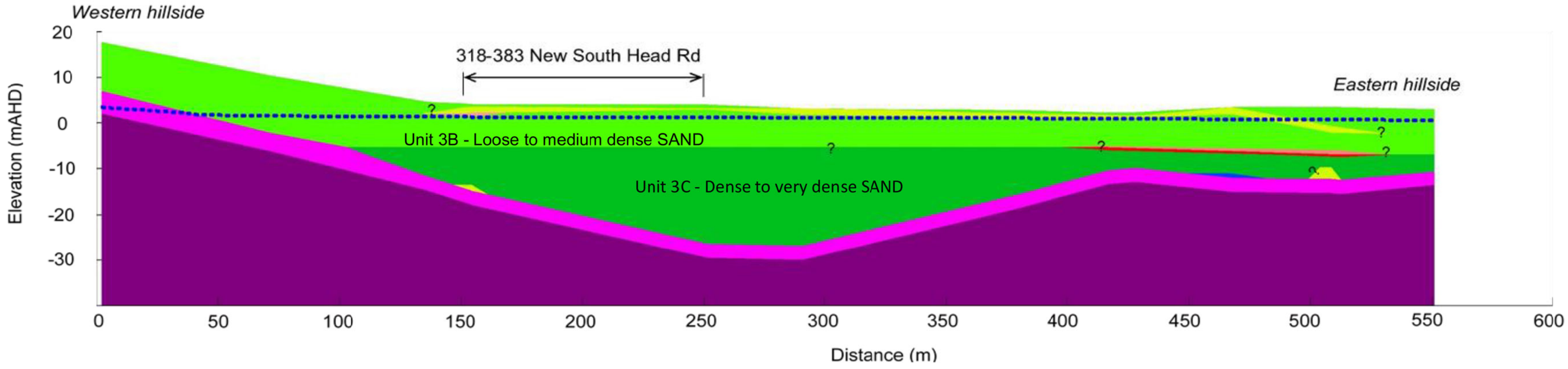
Set of information	Reference ID	Property associated with the Geotechnical Investigation Report [1]	Issued by
	R41	42 Glendon Road	JK Geotechnics
	R42	Pole investigation along New South Head Road	JK Geotechnics
	R43	Compliance with dewatering plan, 59 William Street, Double Bay, Ref: 17512W4 Let2.	JK Geotechnics, 2007
	R44	Groundwater Monitoring at 1 – 9 marathon Mews Double Bay NSW, Ref: 23626WH2Let.	JK Geotechnics, 2011
Information Package 2 from Council	R45	4 - 6 Forest Road	Douglas Partners
	R46	9 Clarence Place	
	R47	4-12 Guilfoyle Ave	JK Geotechnics
	R48	69 Bay St	
	R49	14 - 16 Court Rd	Douglas Partners
	R50	15 Cooper St	Douglas Partners
	R51	11 - 15 Guilfoyle Ave	JK Geotechnics
	R52	23 Manning Rd	JK Geotechnics
	R53	25 Manning Rd	JK Geotechnics
	R54	59 William St	JK Geotechnics
	R55	98 Manning Rd	Douglas Partners
	R56	12-16 William St	JK Geotechnics
	R57	4-8 Patterson St	JK Geotechnics
	R58	351 - 353 New South Head Rd	Martens
	R59	1 Court Rd	Grant Alexander
	R60	40 Manning Rd	JK Geotechnics
	R61	22 William St	JK Geotechnics
	R62	86 Manning St	GeoEnviro
	R63	16 Manning Rd	JK Geotechnics
	R64	61 - 63 Bay St	N/A
	R65	45 - 51 Cross St	Ground Test
	R66	36 - 48 Bay St	URS
	R67	19 - 27 Cross St	

Set of information	Reference ID	Property associated with the Geotechnical Investigation Report [1]	Issued by
	R68	20-26 Cross St	Douglas Partners
	R69	16-18 Cross St - Groundwater assessment for proposed mixed use development (ref. *Douglas Partners Pty Ltd, 2016b*)	Douglas Partners
	R70	21 - 27 Bay St	JK Geotechnics
	R71	28 - 34 Cross St	Douglas Partners
	R72	434 - 440 New South Head Rd	Douglas Partners
	R73	2 - 10 Bay St	Douglas Partners
	R74	55 Bay St	Geotechnique
	R75	49 - 53 Bay St	Douglas Partners

Note:

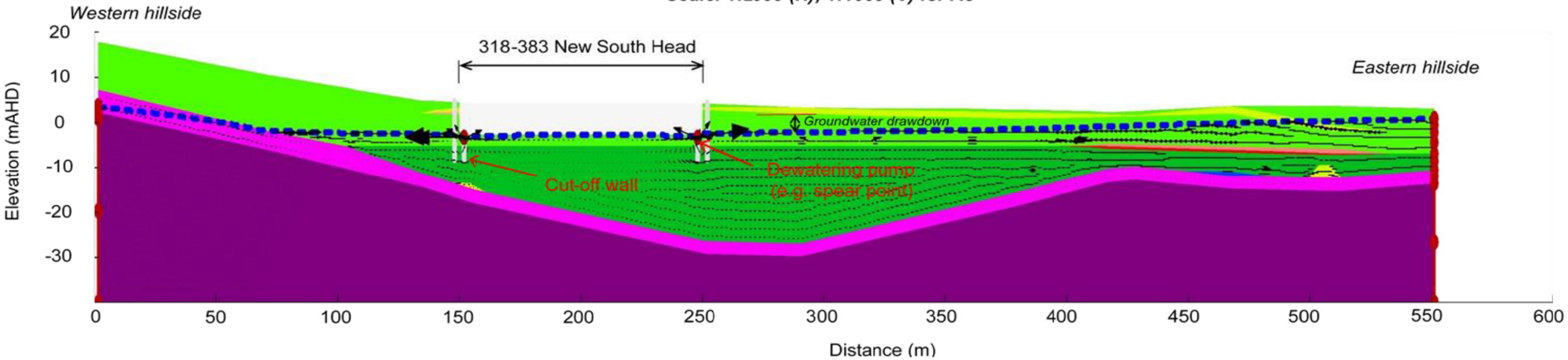
1. Unless otherwise noted, reports listed in the table are associated with the Geotechnical Investigation Report.

Appendix B – Geotechnical Long Sections and Initial Groundwater Levels



B1. Geotechnical Long Section BB (initial condition)

Scale: 1:2000 (H); 1:1000 (V) for A3



B2. Future development (7-17 Knox St) – Geotechnical Long Section BB

Scale: 1:2000 (H); 1:1000 (V) for A3

- Unit 2B – Firm CLAY
- Unit 2C – Stiff to Hard CLAY
- Unit 3A – Very loose SAND
- Unit 3B – Loose to medium dense SAND
- Unit 3C – Dense to very dense SAND
- Unit 3D – Mix of Sand and Clay (Medium dense or Stiff)
- Unit 4A – Very soft to soft PEAT
- Unit 4B – Firm PEAT
- Unit 4C – Stiff to hard PEAT
- Unit 5B – Extremely to highly weathered SANDSTONE
- Unit 5C – Moderately weathered to fresh SANDSTONE

GHD GEOTECHNICS

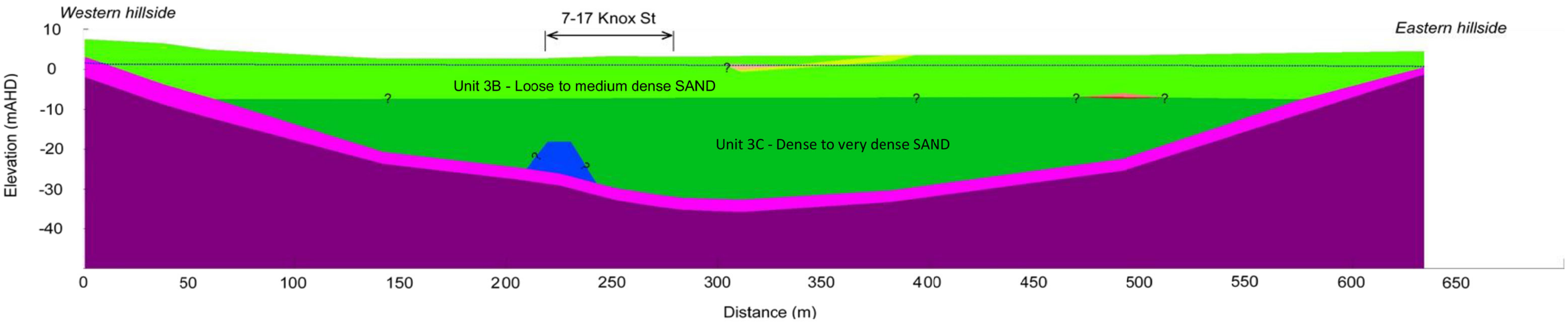
Woollahra Municipal Council
Double Bay - Hydrogeological and Geotechnical Study
Initial and lowered groundwater profile for geotechnical section BB

scale | as shown date | May 2020

job no | 21-12512436
file ref | 21-12512436-REP-1

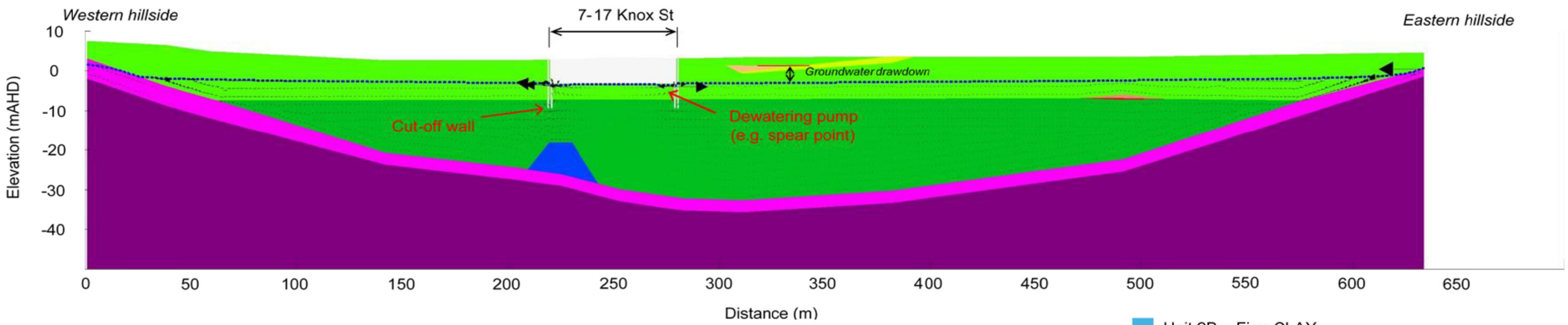
Figures B1 and B2

Level 2 29 Christie Street St Leonards NSW 2065 Australia T 61 2 9462 4700 F 61 2 9462 4710 E atnmail@ghd.com.au



B3. Geotechnical Long Section CC (initial condition)

Scale: 1:2000 (H); 1:1000 (V) for A3



B4. Future development (7-17 Knox St) – Geotechnical Long Section CC

Scale: 1:2000 (H); 1:1000 (V) for A3

- Unit 2B – Firm CLAY
- Unit 2C – Stiff to Hard CLAY
- Unit 3A – Very loose SAND
- Unit 3B – Loose to medium dense SAND
- Unit 3C – Dense to very dense SAND
- Unit 3D – Mix of Sand and Clay (Medium dense or Stiff)
- Unit 4A – Very soft to soft PEAT
- Unit 4B – Firm PEAT
- Unit 4C – Stiff to hard PEAT
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- Unit 5C – Moderately weathered to fresh SANDSTONE

GHD GEOTECHNICS

Woollahra Municipal Council
Double Bay - Hydrogeological and Geotechnical Study
Initial and lowered groundwater profile for geotechnical section CC

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file ref	21-12512436-REP-1
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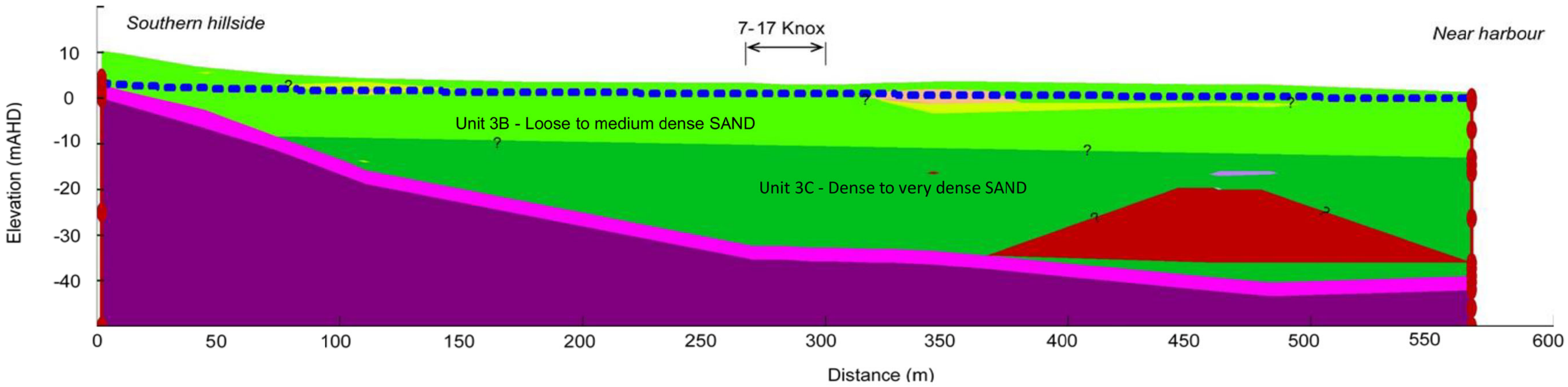
date	May 2020

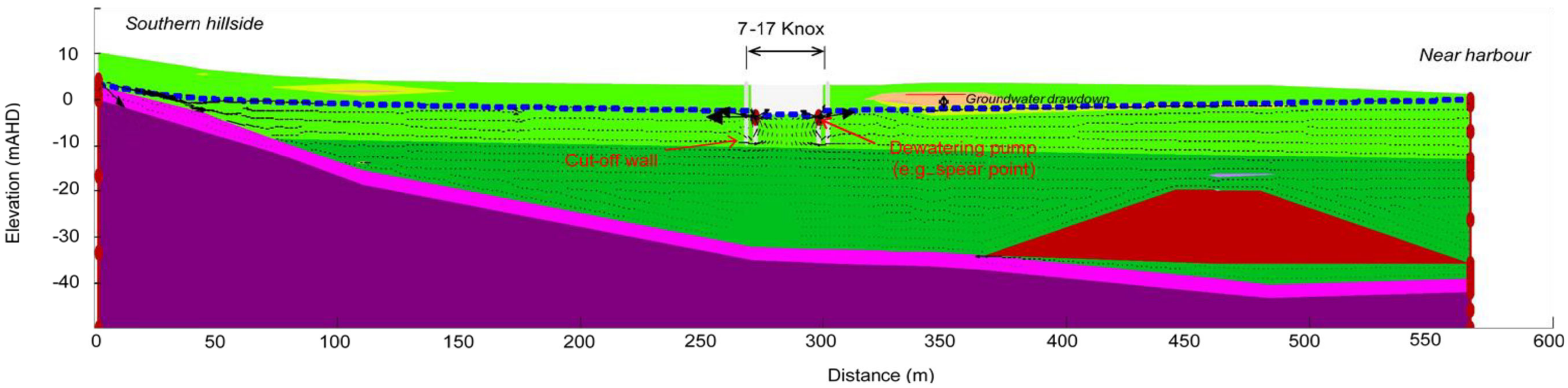
Figures B3 and B4

Level 2 29 Christie Street St Leonards NSW 2065 Australia T 61 2 9462 4700 F 61 2 9462 4710 E atnmail@ghd.com.au



B5. Geotechnical Long Section DD (initial condition)

Scale: 1:2000 (H); 1:1000 (V) for A3



B6. Future development (7-17 Knox St) – Geotechnical Long Section DD

Scale: 1:2000 (H); 1:1000 (V) for A3

- Unit 2B – Firm CLAY
- Unit 2C – Stiff to Hard CLAY
- Unit 3A – Very loose SAND
- Unit 3B – Loose to medium dense SAND
- Unit 3C – Dense to very dense SAND
- Unit 3D – Mix of Sand and Clay (Medium dense or Stiff)
- Unit 4A – Very soft to soft PEAT
- Unit 4C – Stiff to Hard PEAT
- Unit 5B – Extremely to highly weathered SANDSTONE
- Unit 5C – Moderately weathered to fresh SANDSTONE

GHD GEOTECHNICS

Woollahra Municipal Council
Double Bay - Hydrogeological and Geotechnical Study
Initial and lowered groundwater profile for geotechnical section DD

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Figures B5 and B6

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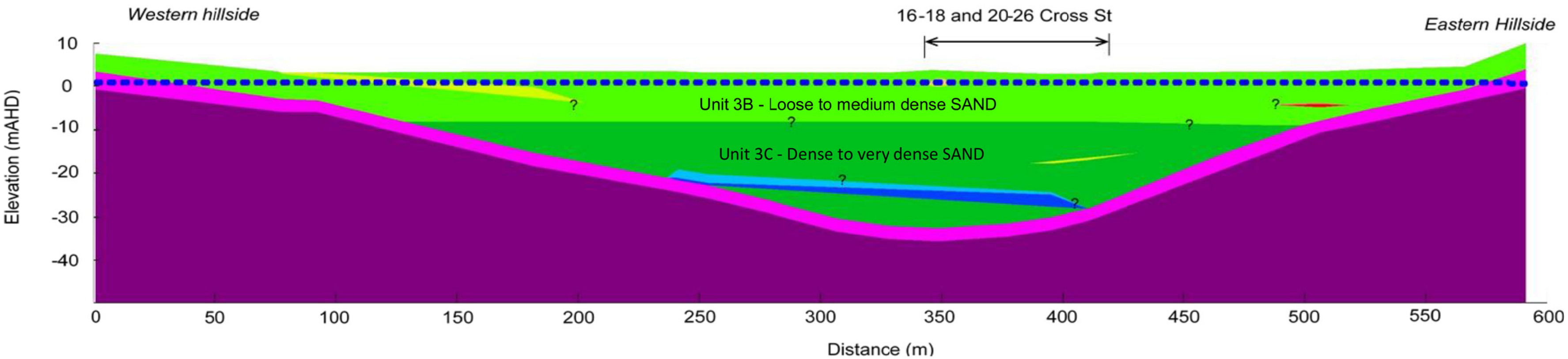
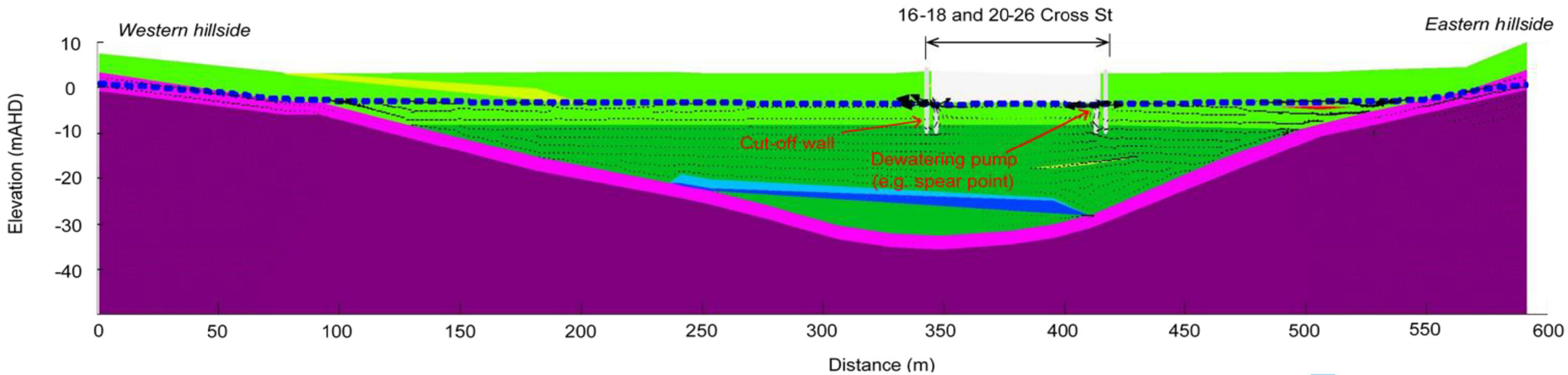


B7. Geotechnical Long Section EE (initial condition)

Scale: 1:2000 (H); 1:1000 (V) for A3



B8. Future development (16-18 and 20-26 Cross St) – Geotechnical Long Section EE

Scale: 1:2000 (H); 1:1000 (V) for A3

- Unit 2B – Firm CLAY
- Unit 2C – Stiff to Hard CLAY
- Unit 3A – Very loose SAND
- Unit 3B – Loose to medium dense SAND
- Unit 3C – Dense to very dense SAND
- Unit 3D – Mix of Sand and Clay (Medium dense or Stiff)
- Unit 4A – Very soft to soft PEAT
- Unit 4B – Firm PEAT
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- Unit 5B – Extremely to highly weathered SANDSTONE
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GHD GEOTECHNICS

Woollahra Municipal Council
Double Bay - Hydrogeological and Geotechnical Study
Initial and lowered groundwater profile for geotechnical section EE

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file ref | 21-12512436-REP-1

Figures B7 and B8

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