Acciona Samsung Bouygues Joint Venture
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WestConnex M4-M5 Link Tunnels

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27 October 2020

Mr Carl Dumpleton
Team Leader
Motorways, Infrastructure Management

CoA E193 – Lodgment of Groundwater Modelling Report (CoA E192 / 193)

Dear Carl,

In accordance with Condition of Approval E193, please find attached a copy of the Groundwater Modelling Report for the Department's information - **Attachment 1.** This report has been built on knowledge obtained through compliance with CoA E192.

Extensive consultation has been undertaken in the development of this report including consulting with DPIE - Water and NRAR on both the preliminary and final modelling reports - **Attachment 2.**

Overall consultation with DPIE - Water & NRAR involved:

- Provision of the preliminary model, report & associated meeting on 19 August 2019
- Significant efforts by ASBJV to encourage the provision of comments on the preliminary model and report (August 2019 – March 2020)
- A letter from Alison Collaros dated 22 June 2020 providing comments on the preliminary model and report (These comments were supplied approx. 10 months after the provision of the preliminary model, report and presentation meeting)
- Provision of the final model and report & associated review meeting on 9 July 2020
- A final letter from Ms Alison Collaros dated 27 August providing no further comments or objections on the final CoA E193 model and report.

In addition to the consultation above and to ensure demonstrable compliance with the groundwater modelling requirements, ASBJV had the groundwater model reviewed by two independent subject matter experts (SMEs) – Dr Noel Merrick of SLR Consulting on the preliminary model and Brian Barnett from Jacobs on the final model and report.

A copy of Brian Barnett's review is incorporated as Appendix BE of the attached Report.

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Regards,

Grant Sainsbery
Environment & Sustainability Manager
Acciona, Samsung, Bouygues Joint Venture

Attachment 1 - Groundwater Modelling Report
Attachment 2 - Evidence of Consultation
Attachment 3 - CoA E193 Compliance Check list

WestConnex M4-M5 Link Tunnels

Hydrogeological Numerical Modelling Report

M4-M5 LINK TUNNELS

Client: M4-M5 Link Group | Project No.: 259954 | Date: 21/09/2020

Document No.: M4M5-JAJV-PRW-GEO-GW02-RPT-0006

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Document Control

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	Name	**Position**	**Date**	**Signed / Approved**
Originator(s)	Irena Krusic-Hrustanpasic	Principal Hydrogeologist	21/09/2020	Irena Krusic-Hrustanpasic
Review	Stephen Barrett	Principal Geotechnical	21/09/2020	
Approval	Mark Rees	JAJV Area Manager	21/09/2020	

Document Revisions

Rev No	Date	Issue / Description	Approved
A	28/02/2019	Issued for DCD external review	MR
B	20/12/2019	Issued for SDD external review	MR
C	19/06/2020	Issued for FD external review	MR
D	21/09/2020	Issued for FD2 external review	MR

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Glossary of Terms and Abbreviations

General

Abbreviation	Definition
2D / 3D	2/3 Dimensional
AIP	Aquifer Interference Policy
AS	Australian Standards
ASS	Acid-sulfate soil
BoM	Bureau of Meteorology
BS	British Standard
BSGS	Botany Sands Groundwater Source
CD	Concept Design
CDR	Cross Discipline Review
CH	Chainage
CHAIR	Construction Hazard Assessment Implications Review
cm	Centimetre
CRD	Cumulative rainfall deficit
D&C	Design and Construct Project
DCD	Developed Concept Design
DD	Design Drawings Certified for Construction
Deed	WestConnex M4-M5 Link Design and Construct Project Deed
DNAPL	Dense non-aqueous phase liquid
DPI	Department of Primary Industries
EIS	Environmental impact statement
EPA	(NSW) Environment Protection Authority
FDD	Final Design Documentation
GDE	Groundwater Dependent Ecosystems
GDR	Geotechnical Data Report
g/L	Grams per litre
GSSA	Groundwater and soil salinity assessment

Abbreviation	Definition
GWL	Groundwater level
HDR	Hydrogeological Design Report
IC	Independent Certifier
IFC	Issue for Construction Documentation
JAJV	Jacobs Aurecon Joint Venture
K_h	Horizontal hydraulic conductivity
K_v	Vertical hydraulic conductivity
Kg/ha/yr	Kilogram per hectare and year
Km	Kilometre
L	Litre
LNAPL	Light non-aqueous phase liquid
LOP	Level of protection
LPFZ	Luna Park Fault Zone
LSBJV	Lendlease Samsung Bouygues Joint Venture
Lugeon	Unit of permeability; 1 Lugeon is approximately equivalent to 1 x 10^{-7} m/s.
M	Mega (i.e. 10^{6})
M4East	The WestConnex Stage 1b: M4 East Project
m	Metre
MCoA	Minister's Conditions of Approval
MGA	Map grid Australia
m AHD	Metres above Australian height datum
m bgl	Metres below ground level
mg/L	Milligram per litre
mm	Millimetres
New M5	WestConnex Stage 2: New M5 Project
NEPM	National Environment Protection Measure
NSW	New South Wales
pH	Hydrogen potential (negative 10-base logarithm of the positive hydrogen ion concentration)

Abbreviation	Definition
ppm	Parts per million
PSD	Professional Services Deed
RFI	Request for Information
RL	Relative level
RMS	Roads and Maritime Services of New South Wales
SCBGS	Sydney Central Basin Groundwater Source
SDD	Substantial Detailed Design
SiD	Safety in Design
SMC	Sydney Motorways Corporation
SSI	State Significant Infrastructure
STN	Station
SWTC	Scope of Works Technical Criteria
TfNSW	Transport for New South Wales
TDS	Total dissolved solids
The Project	WestConnex Stage 3a: M4-M5 Link Project
µg	Micrograms
µS	Micro-Siemens
VWPs	Vibrating wire piezometers
WCX	WestConnex
WDA	WestConnex Delivery Authority
WFZ	Woolloomooloo Fault Zone
WHS	Workplace Health and Safety

1. INTRODUCTION

Jacobs Aurecon Joint Venture (JAJV) has engaged Golder Associates Pty Ltd (Golder) to provide hydrogeological numerical modelling services for the M4-M5 Link Project (WestConnex Stage 3a). The modelling work has been undertaken in three stages as the site investigation results for the detailed design investigation became available and the final design for the tunnels was developed.

The first stage of numerical modelling work was completed in January 2019 and results were presented in the Preliminary Modelling Report issued on 28 February 2019 (Report Reference No M4M5-JAJV-PRW-GEO-GW02-RPT-0006 Hydrogeological Numerical Modelling Report, Version A). The second stage was completed in December 2019 and results were presented in the SDD Modelling Report issued on 20 December 2019 (Report Reference No M4M5-JAJV-PRW-GEO-GW02-RPT-0006 Hydrogeological Numerical Modelling Report, Version B).

The FD model presented here builds on findings from the previous two modelling stages and considers all of the results from LSBJV detail design investigation that has now been completed.

Presented in this report is:

- a summary of the hydrogeological setting used to develop the FD model. Understanding of the hydrogeological setting has advanced considerably since the SDD report issue based on the results of the pumping test at Hawthorne Canal.
- the hydrogeological parameters adopted in the FD model for the main hydrostratigraphic units, including parameters for the high permeability zone at Hawthorne Canal originally identified by the packer testing in this area.
- the model development methodology and updated finite element mesh, which now includes both the New M5 and M4 East motorway tunnels.
- FD predictions of tunnel opening and long-term groundwater inflows into the permanent works and the associated groundwater drawdowns. These were developed using the proposed tunnel construction sequence and schedule for the excavations.
- FD prediction of the time when quasy steady-state conditions are likely to be established once the tunnels are in operation.
- FD predictions of salt water migration from Iron Cove, Iron Cove Creek and Hawthorne Canal.
- results of the solute fate and transport modelling completed in the St Peters area and at O'Dea Reserve and Algie Park.

This report should be read in conjunction with the following factual and interpretive reports, which summarise the groundwater data, geological setting and hydrogeological setting that formed the basis of the FD model:

- M4M5-JAJV-PRW-GEO-GT02-RPT-0005 Geotechnical Interpretive Report;
- M4M5-JAJV-PRW-GEO-GW01-RPT-0005 Hydrogeological and Ground Gas Factual Report; and
- M4M5-JAJV-PRW-GEO-GW02-RPT-0005 Hydrogeological Design Report.

2. PROJECT OVERVIEW

The Project comprising the WestConnex M4-M5 Link Main Tunnel motorway, which once completed, will link the WestConnex M4 East and the WestConnex New M5 motorways. A new northbound (M110) carriageway and southbound (M120) carriageway (the mainline carriageways) will be constructed from the eastern end of the M4 East Motorway in Haberfield to the northern end of the New M5 Motorway at St Peters.

New ramps will also be constructed at Haberfield and St Peters to connect the M4-M5 Link Motorway to Wattle Street and Parramatta Road (Wattle Street Entry Ramp, M170 and Exit Ramp, M160) and the St Peters Interchange (SPI Entry Ramp, M190 and Exit Ramp, M180). Provisions for a future connection from the M4-M5 Link Motorway to the future Rozelle Interchange (WestConnex Stage 3b) will also be provided.

The mainline carriageways will be constructed entirely underground as drained excavations below the existing groundwater table, with tunnel invert levels ranging from approximately RL –45 m AHD to RL –15 m AHD. The new ramps will also be constructed predominantly underground and below the existing groundwater table, except for short sections where the ramps emerge at the ground surface and connect with the existing traffic network.

3. OBJECTIVES AND SCOPE OF NUMERICAL MODELLING

The overall objective of the numerical modelling work was to develop a three-dimensional regional groundwater model to assess tunnel inflows, groundwater drawdowns due to project development and saline water migration in accordance with MCoA E193 and REMMs GW6 and GW7 requirements. The results of the model have used to evaluate potential risks associated with the predicted groundwater impacts and to inform detailed design of the Project including grouting, durability and water treatment requirements.

The FD model was developed in three stages with the two previous modelling stages presented in Versions A and B of this report. The specific scopes for each of these modelling phases are summarised below:

DCD Modelling: The focus of the DCD modelling work was to inform the initial project wide design development and obtain:

- initial estimates of long-term groundwater inflows into the individual Project elements during operation
- assess the potential long-term groundwater drawdowns
- guide the next stage of model development work by providing insight into the main issues and uncertainty related to the data availability/collection, hydraulic properties of key hydrostratigraphic units and modelling approach.

These insights were subsequently used to develop the SDD and FD models.

SDD Modelling: The focus of the SDD modelling work was to assess potential movements of contaminated groundwater in the St Peters area from the former Sydney Park, Camdenville and Alexandria landfills, induced by construction and operation of the proposed tunnels. Specifically, at St Peters, the SDD modelling provided an initial assessment of:

- potential for migration and long-term concentrations of key contaminants of interest, which may reach tunnel elements from former landfill sites
- the time it might take for the key contaminants to reach tunnel elements

Additional model simulations were also undertaken to assess potential contaminant migration from the former O'Dea Reserve and Algie Park landfills on the M110 and M120 mainline carriageway tunnels.

The SDD modelling results were also used to update the following project wide aspects of the SDD Hydrogeological Design Report:

- further assessment of potential ground treatment requirements for limiting groundwater inflows to the main tunnels, ramps and ventilation tunnels
- potential impacts on beneficial groundwater uses, surface water flows, groundwater dependent ecosystems
- the potential for salt water migration from Iron Cove, Hawthorne Canal and Iron Cove Creek
- potential groundwater inflow volumes, water quality, and contaminate concentrations for assessment and design of the in-tunnel water collection and treatment systems.

The results of the contaminant transport modelling are presented in Appendix BD of this report.

FD Modelling: The focus of the FD modelling work was to address all of the SWTC and MCofA requirements in relation to groundwater modelling and considers all of the results from the LSBJV detailed design investigation and support the final design of the Project. This included:

- adding the M4 East tunnels into the model
- completing a transient calibration based on the pumping test results at the Hawthorne Canal
- completing a steady state calibration based on water level data across the project corridor including data gathered from other projects
- incorporation of the proposed construction sequence
- finalising the inflow and drawdown assessment

- finalising the assessment of the potential for salt water migration from Iron Cove, Hawthorne Canal and Iron Cove Creek
- finalising the assessment of potential changes in the groundwater contribution to stream baseflow
- completing a detailed sensitivity analysis to assess the predictive ability of the model.

The FD model was developed in accordance with the National Water Commission's Australian Groundwater Modelling Guidelines (Barnett et al, 2012). It has been designed to meet Class 2 (with elements of Class 3) criteria set out in Table 2-1 of the Guidelines.

Throughout the DCD, SDD and FD modelling work, the numerical models and modelling outputs were periodically cross-checked and/or updated with observations from the LSBJV Detailed Design Site Investigation and the early tunnel excavations as the information became available. This included:

- Inclusion of the pumping test results and key groundwater insights from the test into the FD model.
- In tunnel geotechnical observations and encountered ground conditions checked against the project geological model.
- In tunnel inflow observations cross checked against inflow expectations at key locations such as Johnstons Creek and the Woolloomooloo Fault Zone.
- Ongoing groundwater level monitoring to check and re-calibrate initial groundwater conditions across the study area, together with any subsequently observed drawdowns from the early tunnel excavations as a guide to aquifer response.
- Revision of model grouting locations and extents, based on what was actually observed and carried out as the tunnel excavations were advanced.

4. GEOLOGICAL AND HYDROGEOLOGICAL SETTING

4.1 General

The geological and hydrogeological setting was developed from the Project geological, geotechnical and groundwater investigation data and interpretations as present in the following project wide reports;

- Geotechnical Data Report, M4M5-JAJV-PRW-GEO-GT01-RPT-0005
- Geotechnical Interpretive Report, M4M5-JAJV-PRW-GEO-GT02-RPT-0005
- Groundwater and In Ground Gas Factual Report, M4M5-JAJV-PRW-GEO-GW01-RPT-0005

4.2 Geological Units

The main stratigraphic units within the broader area of the Project alignment are summarised in Table 4-1 and a geological plan of the area is provided as Figure 4-1.

Table 4-1: Main Stratigraphic Units

Geological Period	Stratigraphic Unit	Description
Quaternary	Anthropogenic materials	Anthropocene age unconsolidated materials: Minor filling for housing and light commercial buildings. Significant controlled filling for the construction of large commercial/industrial units, infrastructure projects and reclaimed land (bay areas). Landfills developed within former shale quarries (brick pits), including Alexandria Landfill, Sydney Park, Camdenville Park, O'dea Reserve and Algie Park. The landfills have been backfilled with varying waste materials, including putrescible and industrial wastes, and were not lined at the time of landfilling.
	Botany Basin sand deposits (Qhd)	Holocene sands and clays. Occurs within and east of the St Peters interchange
	Undifferentiated alluvial/estuarine sediments (Qhs and Qha)	Holocene and Pleistocene age interbedded sandy and clayey sediments. Occur within valleys of the current watercourses including Hawthorne Canal, Iron Cove Creek, Whites Creek, Johnstons Creek, Alexandra Canal, and also in Botany Bay.
Jurassic	Volcanic intrusions	Dykes, sills and diatremes. Typically, less than 3 m wide and oriented between 005º - 035º and 090º - 120º
Triassic	Ashfield Shale (Rwa)	Forms a capping to bedrock within the central and southern areas of the Project alignment. Where the full profile is present the unit typically consists of four sub-group members: Mulgoa Laminite: Interlaminated siltstone and very-fine sandstone. Regentville Siltstone: Dark grey mudstone, shale and siltstone with occasional fine-grained sandstone laminae. Kellyville Laminite: Interlaminated siltstone and very-fine sandstone. Rouse Hill Siltstone: Dark-grey to black mudstone or shale.
	Mittagong Formation (Rm)	Interbedded fine-grained sandstone and siltstone.

Geological Period	Stratigraphic Unit	Description
	Hawkesbury Sandstone (Rh)	Medium to coarse-grained quartzose sandstone comprising cross-bedded or sheet sandstones, massive sandstone and shale/siltstone interbeds. Present beneath the entire Project area. Variably weathered with deeper weathering profile within incised watercourses.

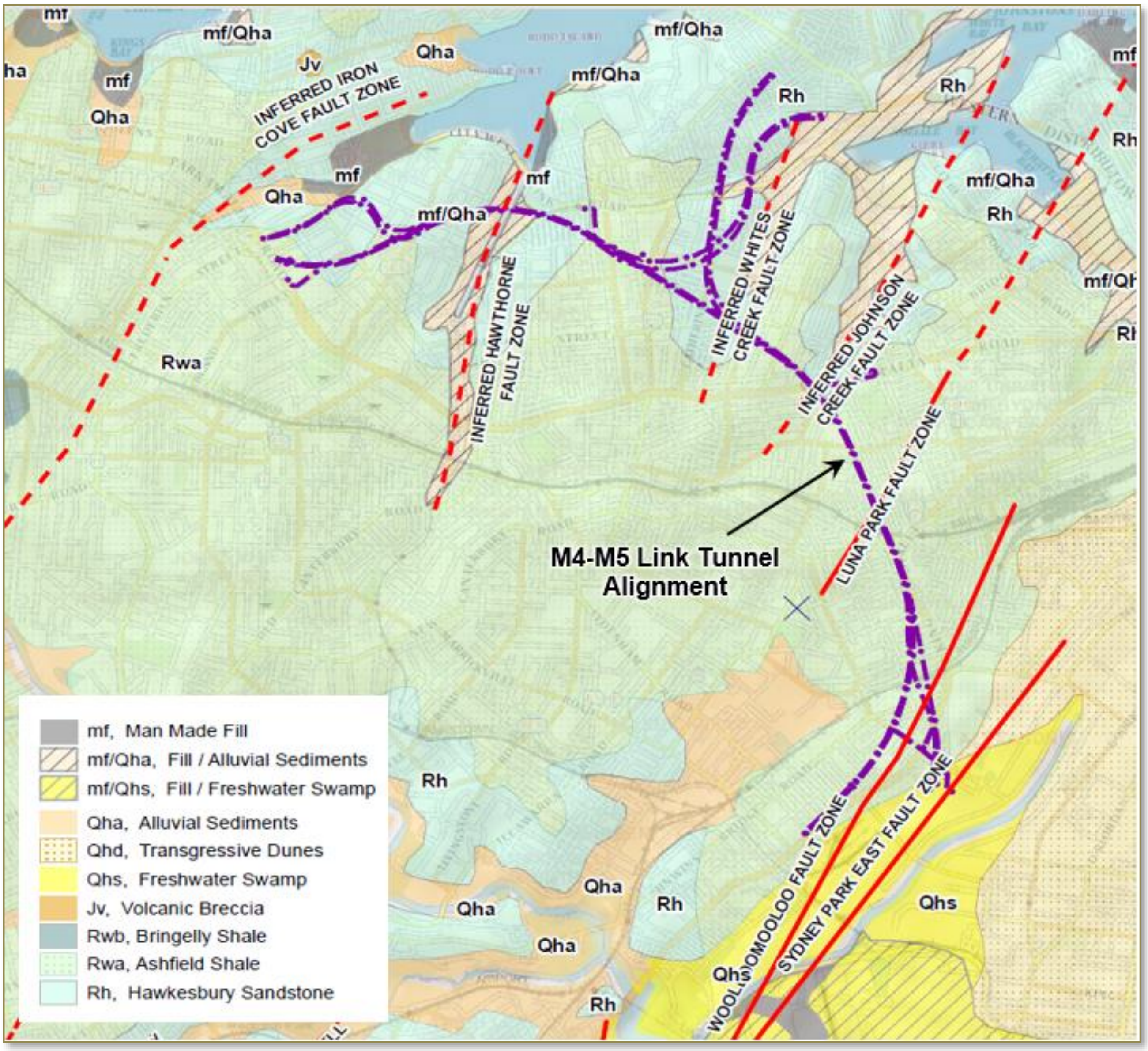


Figure 4-1: Geological Plan of Project Area, based on Geology of Sydney Maps 1:100,000 series (map sheet 91310)

4.3 Hydrostratigraphic Units

The stratigraphic units that will be encountered along the Project alignment were deposited/formed under different conditions, which resulted in significant variability of materials contained within some units (i.e., Quaternary sediments). Consequently, the hydrogeological characteristics of the units or parts of a unit, and their roles in the groundwater flow system are complex and variable. Tectonic activities including faulting, and bedding parting associated stress relief processes have also locally significantly affected the hydrogeological characteristics of geological units.

A summary of hydrogeological classification of main stratigraphic units and their roles in the groundwater flow system is provided in Table 4-2.

Table 4-2: Hydrogeological Classification of Stratigraphic Units and their Role in Groundwater Flow System

Stratigraphic Unit	Hydrogeological Classification of the Unit	Main Occurrence
Anthropogenic materials	Generally an aquifer, unconfined, porous media with highly variable hydraulic properties but most likely higher that hydraulic conductivity of surrounding rock mass. Anthropogenic fill within the landfills could form isolated unconfined aquifers. Leachate from contaminated fill could impacted the surrounding groundwater levels and quality.	5 former landfill sites within the Project Corridor.
Botany Sands (includes Qhs and Qhd)	Aquifer, unconfined, porous medium, horizontal hydraulic conductivity (Kh) moderately greater than vertical (Kv), high yielding where sands dominate.	To the east of the Project corridor (St. Peters area).
Undifferentiated alluvial sediments (Qha)	Aquifer, unconfined, porous medium, horizontal hydraulic conductivity (Kh) slightly to moderately greater than vertical (Kv). moderately yielding.	Within valleys of watercourses that cross the alignment.
Volcanic intrusion (dykes and sills)	Generally, a barrier to groundwater flow perpendicular to the feature. Could provide significant groundwater pathways along their contacts with adjacent host rock due to shearing.	Various locations. Likely occurrences based on current understanding of geological setting are variable across the whole Project domain but in particular in the vicinity of the St Peters and Wattle Street Interchanges.
Ashfield Shale	A low-yielding fractured rock aquifer of moderate to low significance throughout the Sydney region. Consists mostly of low permeability siltstone and laminate with groundwater flow occurring mostly via saturated fractures. Horizontal hydraulic conductivity (Kh) expected to be one to two orders of magnitude greater than vertical (Kv). Higher hydraulic conductivities are expected within the main faults/fracture zones.	Encountered in central and southern parts of the Project alignment.
Mittagong Formation	Hydraulic properties and role in the groundwater flow system similar to that of the Ashfield Shale.	Similar to that of the Ashfield Shale.
Hawkesbury Sandstone	Primary regional aquifer and an aquifer of moderate importance in the Sydney area. Fractured rock medium comprising several 'stacked' aquifers which form the aquifer system. Interbedded shale lenses can provide local or extensive confining layers, creating separate aquifers with different hydraulic properties. Moderate to low hydraulic conductivities typically highly enhance by faulting, fracturing and bedding parting. Horizontal hydraulic conductivity (Kh) expected to typically be about one order of magnitude greater than vertical (Kv), although the ratio may locally varied, particularly in areas of stress relief.	Encountered over entire area.

Hydrostratigraphic units are hydraulically continuous, scale independent and mappable units that can be defined based on their hydraulic properties. A hydrostratigraphic unit may include a geological formation, part of a formation or a group of formations. Some of the stratigraphic units that have similar hydraulic properties and role in the groundwater flow system, such as Mittagong Formation and Ashfield Shale, can therefore be considered as part of a single hydrostratigraphic unit.

On this basis the following hydrostratigraphic units were distinguished within the Project domain for purpose of the FD model development:

- Landfill Waste – includes anthropogenic fill within the major landfills.
- Botany Sands Aquifer – includes Botany Basin sands (Qhd) and undifferentiated estuarine sediments in the vicinity of the Alexandra Canal (Qhs), i.e., south eastern part of the broader Project area.
- Quaternary Alluvial Aquifer – includes Quaternary sediments within valleys of the current watercourses.
- Ashfield Shale Aquifer – includes residual soils, Ashfield Shale geological unit, Mittagong Formation and igneous intrusions (sills, dykes and diatremes).
- Hawkesbury Sandstone Aquifer - includes residual soils, Hawkesbury Sandstone geological unit and igneous intrusions (sills, dykes and diatremes).

4.4 Groundwater Conceptual Model

4.4.1 General

The groundwater conceptual model was developed based on the hydrogeological and geological data collected at monitoring wells and boreholes drilled during the LSBJV detailed design site investigation, the pre-contract award investigations and data that has been made available from other projects within the broader area of the Project corridor.

A number of aquifers exist within the broader area of the Project corridor. The relationship between the aquifers is suggested by the available data to be complex and variable at the regional and local scales. Groundwater levels and flows within aquifers are also affected by natural and man-made structures and processes.

The FD model was developed based on conceptualisation detailed in the Hydrogeological Design Report. Key aspects of this conceptualisation which provided the basis for the development and calibration of the numerical model are summarised below.

4.4.2 Groundwater Levels

Groundwater level data from a total of 144 monitoring wells was used to develop the FD model. The monitoring period for this group of wells spans across multiple years, with the majority of data collected between 2016 and 2020.

The groundwater level data set indicates the groundwater levels vary significantly across the Project corridor and between individual aquifers, with the observed levels ranging from about RL -12 m AHD in WCX-BH157 in the Hawkesbury Sandstone in a New M5 Motorway project well to about RL 33 m AHD in LSB-MT-BH1009a in the Ashfield Shale.

Typically, the higher groundwater levels were observed in areas of higher topography and in the residual soils adjacent to Sydney Park, while the lower groundwater levels were observed adjacent to surface water streams, canals and bays. Groundwater levels below sea level (approximately RL 0 m AHD) were observed in a number of monitoring wells located in the vicinity of the New M5 motorway, the recently excavated Sydney Metro tunnels and the Alexandria Landfill.

Average groundwater levels for each monitoring well were used to assess the correlation between ground surface elevation and the groundwater levels as shown in Figure 4-2. This has been used to assist with the development of initial groundwater level contours within areas where no monitoring data has been available and to assist with model calibration. This data set was also used to screen for suitable calibration targets and to identified potential outliers as further discussed in Section 6.

After data quality review and filtering, a total of 69 data points was utilised for the shallow groundwater system assessment. Groundwater levels from four monitoring wells were reported to be negative, i.e. below RL 0 m AHD (WCX-BH157, LDS-BH-3046A; LDS-BH-5007; LDS-BH-3045A) and, therefore, were excluded from assessment as not being representative of the natural initial groundwater system (shown as red dots in

Figure 4-2). The regression equation for the shallow groundwater system calculated based on the remainder of data points (a total of 65) was:

Water level = 0.7517(Ground Elevation) – 2.4924

A correlation coefficient (R) of 0.89 (R^2 of 0.79) suggests a strong positive correlation between ground elevation and water levels in the shallow aquifer system.

A total of 53 data points was utilised for the assessment of correlation between deep groundwater levels and ground surface elevation. The data set was filtered for negative groundwater levels and outliers with significantly higher or lower residual values. A total of five points were excluded from the assessment on this basis. This included wells LSB-MT-BH1012, MT_BH20, MT_BH21, WCX-BH109, LDS-BH-2008A. The regression equation for the deep groundwater system based on the remainder of data points (a total of 48) was:

Water level = 0.2901(Ground Elevation) – 0.7114

A correlation coefficient (R) of 0.84 (R^2 of 0.72) was calculated for the deep aquifer system suggesting a slightly less strong correlation than for the shallow groundwater system. The slope of the regression line for the deeper aquifer system is gentler than for the shallow aquifer, which suggests less of a relationship between ground surface elevation and groundwater levels.

The results of the regression analysis also indicate that for each aquifer system a vertical head gradient exists between the deep and shallow systems. This has been confirmed by groundwater level/pressure monitoring in paired wells/VWPs that were installed at several location throughout the Project corridor.

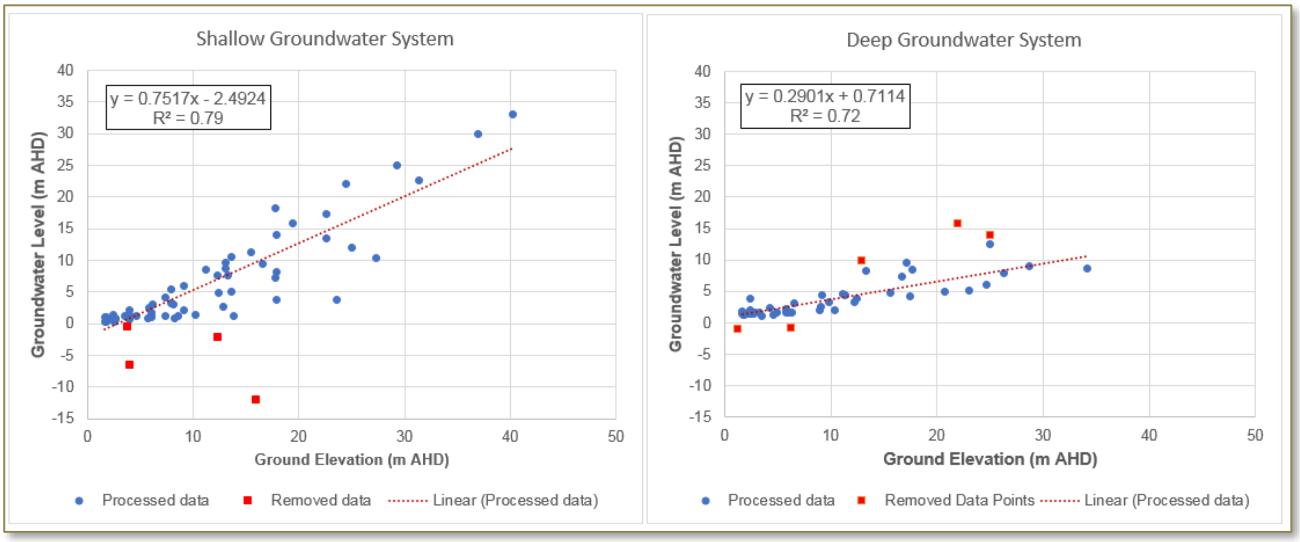


Figure 4-2: Correlation between Groundwater Levels and Ground Surface Elevation

The interpreted pre-tunnelling regional groundwater level contours considering all of the above data and factors are shown in Figure 4-3.

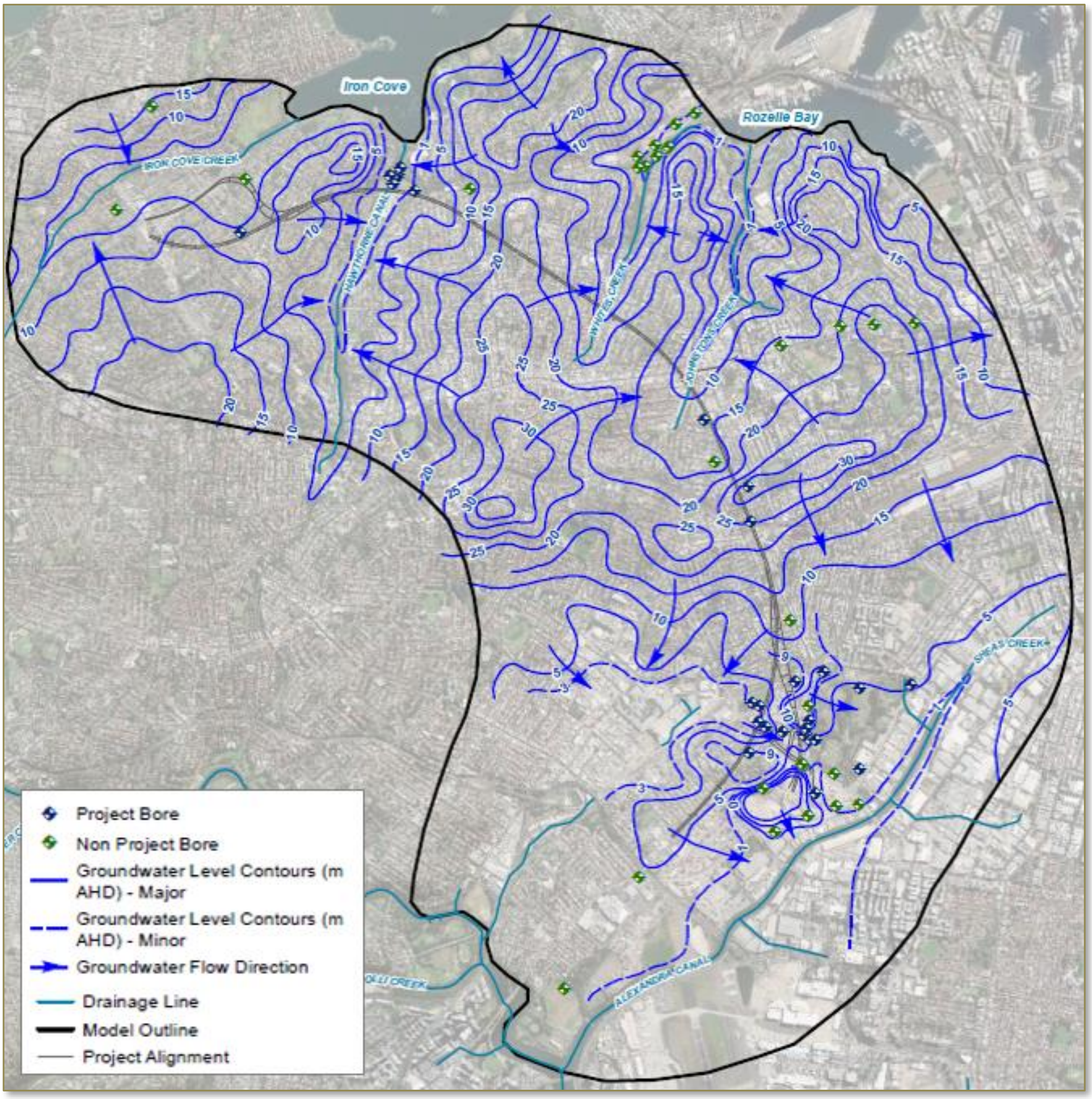


Figure 4-3: Inferred Groundwater Level Contours for the Model Domain.

4.4.3 Recharge

Recharge to groundwater occurs predominantly from rainfall infiltration and potentially from buried watermains, water transfer tunnels, stormwater drains and sewers, as well as from irrigation of parks and domestic gardens.

The rate of recharge is expected to vary greatly, from moderately high over open and grassed areas, to very low over built and paved areas. The nature of surface cover and hydraulic conductivity of the exposed geological unit would significantly affect the overall recharge rate.

Direct recharge to the Hawkesbury Sandstone from rainfall occurs in areas where the unit is extensively exposed and fractured. Where overlain by the Ashfield Shale, recharge to the Hawkesbury Sandstone is inferred to be significantly reduced due to the lower vertical conductivity of the Ashfield Shale and generally clayey weathering profile near surface. Recharge to the sandstone aquifer is further reduced within areas of topography relief and moderate slopes, where runoff rather than recharge is likely to dominate.

Direct recharge to the Ashfield Shale occurs in areas where Ashfield Shale is fractured and extensively exposed, although in areas where it has been weathered near surface, the rate of recharge is inferred to be reduced, resulting in increased surface runoff.

Based on the geology map of the Sydney area (Figure 4-1), the Ashfield Shales and residual soils derived from the shales are exposed over the majority of the broader Project area. The overall recharge rate from the rainfall within the numerical model domain, therefore, is expected to be generally low.

Higher recharge rates, however, are expected within the areas of exposed Botany Sands at the southwestern limit of the Project area.

To develop an understanding of the relationship between rainfall recharge and groundwater level response, long term groundwater level data available for seven monitoring wells was compared with the daily rainfall CRD (cumulative rainfall departure from average) for a period from 2016 to 2019. This included shallow wells SP-BH02, HB-BH03, HB-BH15 and MT-BH14, and deep wells HB-BH14, HB-BH12 and MT-BH02. Groundwater levels from all these wells, but MT-BH02, showed a correlation with rainfall. This ranged from fair to very good in the shallow groundwater wells, while it was fair in HB-BH12 and good in HB-BH14. The results of this analysis, including hydrographs for each of these wells, are presented in the Hydrogeological Design Report (Section 5.0).

Groundwater levels/pressures recorded in the deep alluvial monitoring wells (LSB-HC-PT-OW1a to LSB-HC-PT-OW5a) and shallow VWPs (LSB-HC-PT-OW1b to LSB-HC-PT-OW5b) prior to commencement of the Hawthorne Canal pumping test were used to assess potential recharge rates for the Hawthorne Canal alluvial aquifer. Two noticeable rainfall events occurred during this pre-pumping period, on 30 August 2019 (total of 40 mm) and between 17 September 2019 and 19 September 2019 (total of 102 mm over three days). Ranges of the groundwater responses are illustrated in Figure 4-4 (low end) and Figure 4-5 (high end) with groundwater levels shown by the blue line and the rainfall intensity by the red bars.

The rate of response in monitoring wells/VWPs varied considerably between events and at monitoring wells/VWPs. No clear responses to the August 2019 rainfall event were observed at any of the alluvial wells/VWPs. However responses to the September 2019 rainfall event were observed at LSB-HC-PT-OW3 , LSB-HC-PT-OW4 and LSB-HC-PT-OW5 (Figure 4-5), with the responses ranging from 0.11 m to 0.39 m in VPWs, and from 0.04 m to 0.06 m in alluvial wells. No clear responses were observed to the September 2019 event at monitoring locations LSB-HC-PT-OW1 (Figure 4-4) and LSB-HC-PT-OW2.

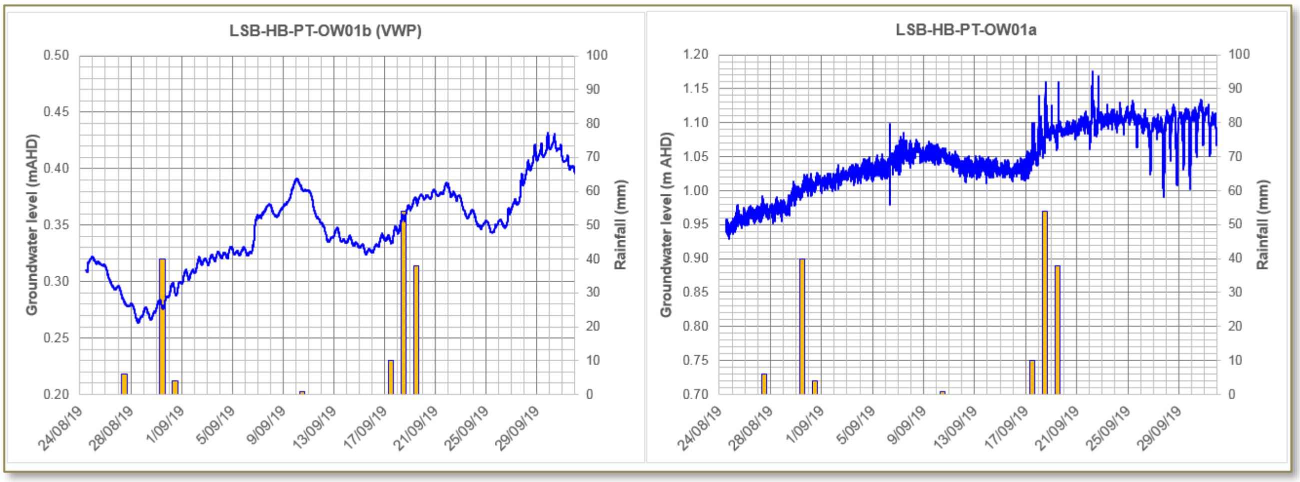


Figure 4-4: Groundwater Levels/Pressures responses to Rainfall as Monitoring Location LSB-HC-PT-OW01

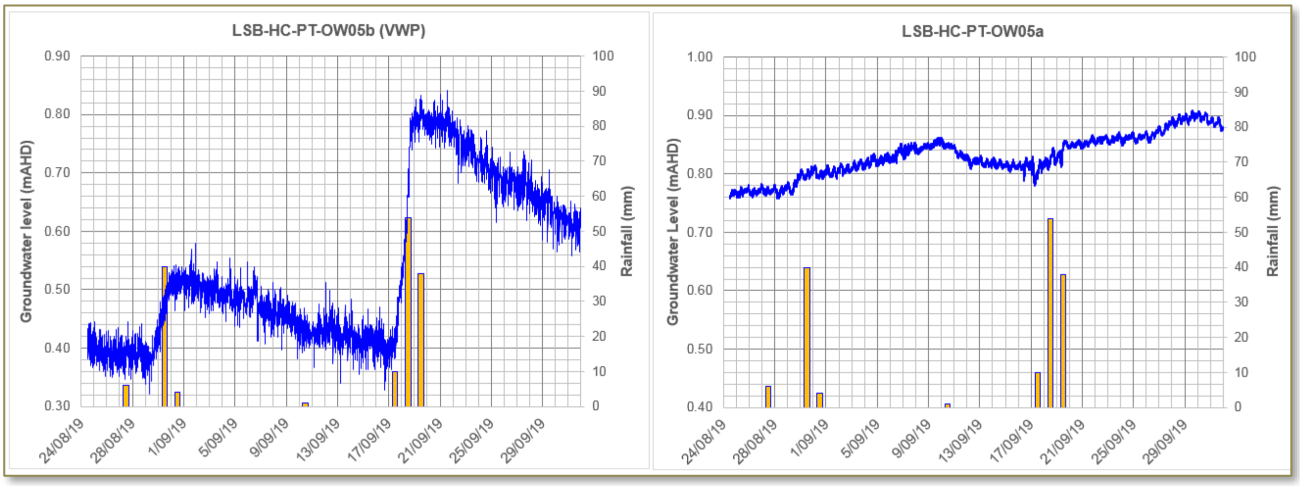


Figure 4-5: Groundwater Levels/Pressures responses to Rainfall as Monitoring Location LSB-HC-PT-OW05

The following was concluded from these observations in the Hawthorne Canal area:

- Groundwater response to rainfall varies significantly from location to location.
- No response to rainfall intensities of 40 mm or less were observed, suggesting recharge rates to be depended on rainfall intensity and duration.
- Responses of groundwater to rainfall within the deeper alluvium aquifer was about 20% or less of those observed at the water table. This suggests that the majority of the rainfall which infiltrated into the shallow aquifer zone was potentially lost through evapotraspiration, or was discharged into the canal, rather than penetrating into the deeper aquifer systems below.

Although no conclusive and quantifiable estimates of potential groundwater recharge rates could be derived due to high variability in groundwater responses with respect to location and rainfall intensity, the above assessment suggested that average total infiltration rates over larger areas are likely to be generally low.

The infiltration rates adopted at the end of FD model calibration are further discussed in Section 6.4.2.

4.4.4 Groundwater Discharge

Groundwater discharge occurs to surface water bodies (creeks, drains and bays), via localised groundwater extraction/dewatering and to the atmosphere by evapotranspiration.

Groundwater discharge to the surface water bodies may occur directly from the hydrostratigraphic units, (i.e. Quaternary aquifers connected directly with the surface water) or indirectly through other units (i.e. from the rock aquifers below to the overlying Quaternary sediments).

Groundwater discharge also occurs via abstraction from landfills, such as at the former Alexandria and Sydney Park landfills.

The Alexandria Landfill is an unlined landfill and waste material is therefore in direct contact with the underlying Ashfield Shale and adjacent Quaternary sediments. It is understood that leachate has been extracted from a landfill sump to prevent off-site leachate impacts. The sump extraction was designed to manage leachate levels within the sump at an elevation not greater than RL -16 m AHD (WestConnex Stage 2 M5 Design Hydrogeological Report[1] and HydroSimulation, 2017[2])

Groundwater extraction from the Quaternary sediments has also been undertaken since 2001 at the landfill to reduce leachate generation and the leachate extraction rate. It is understood this operation ceased after a barrier wall (cut-off wall) was installed as part of the New M5 motorway development. The cut-off wall has been

1 WestConnex Stage 2 M5, 2017: Design Package Report, Hydrogeological Design Report, FD, Document M5N-GOL-DRT-100-200-GT-1525-P

2 Hydrosolutions, 2017: Westconnex M4-M5 Link, Groundwater Modelling report for AECOM Pty Ltd

installed along the eastern boundary of the landfill site to significantly reduce groundwater inflow from the Quaternary sediments.

The former Sydney Park landfill is also unlined and consequently hydraulically connected to underlying Ashfield Shale aquifer. There is a localised leachate management system in place in the eastern part of the site which limits maximum groundwater levels within this section of the park.

Groundwater discharge to the existing drained tunnels within the project corridor will also locally impact on the groundwater system, by lowering the natural groundwater levels.

Evapotranspiration was also considered to be significant component of the water budget and source of water loss from the groundwater system.

4.4.5 Groundwater Flow System

Under natural conditions, groundwater levels are expected to reflect a subdued expression of topography, with groundwater flowing from topographical highs towards main rivers, creeks and bays. Under such circumstances no groundwater levels below the sea level would be expected to occur. The groundwater levels observed in a number of the available monitoring wells, however, indicate that the flow system within the project corridor has been highly modified with man-made structures and processes heavily influencing groundwater levels and flow directions.

The inferred pre-tunnelling regional groundwater flow directions are presented in Figure 4-3. As indicated in this figure, the Project Corridor lies broadly within the Parramatta River (i.e, Rozelle and Iron Cove Bays) and Cooks River groundwater catchments. Rozelle Bay and Iron Cove Bay to the north and north west and the Cooks River to the south and southeast, are interpreted to be the regional groundwater discharge points.

Locally groundwater flow is controlled by the local sub-catchments including:

- Iron Cove Creek, Hawthorne Canal and Johnston Creek sub-catchments of the Rozelle Bay and Iron Cove Bay catchment
- Alexandra Canal and the surface water drainage system in the broader Sydenham area sub-catchment of the Cooks River regional catchment.
- In addition to the natural flow control discharge points, the groundwater flow pattern at the local scale is also affected by man-made structure and processed such as:
  - leachate extraction at Alexandria Landfill
  - the drained mainline tunnels and ramps associated with the New M5 East and New M4 motorways
  - active excavation associated with the drained M4-M5 Link mainline tunnels and ramps, and
  - limited groundwater recharge from the infiltration of rainfall due to urban development.

Overall, downward vertical hydraulic gradients typically exist between the shallow and deep aquifer zones within the areas of aquifer recharge. Upwards vertical hydraulic gradients typically exist adjacent to the discharge areas with sub-artesian groundwater conditions inferred in the Hawkesbury Sandstone at and beneath Hawthorne Canal. Vertical head differences up to 15 m have been observed within the areas adjacent to and near existing drained underground structures.

Dykes, fault zones and zones of enhanced bedding parting due to valleys stress relief are expected to influence groundwater levels and flow directions by increasing or inhibiting groundwater flow, with the extent depending on the degree of weathering and the degree of fracturing associated with these structures. Examples of the types of effects which might be observed when encountering such features include:

- Groundwater flow being limited across dykes but enhanced parallel to them.
- Zones of enhanced bedding parting, resulting in increased groundwater inflows along them.
- Fault zones resulting in higher groundwater flow along them due to increased hydraulic conductivities along them when compared to the surrounding rock. Observations from the New M5 Motorway and current excavations for M4-M5 Link, however, suggest the hydraulic conductivities of the fault zones tend to be lower than those observed from the bedding partings and the inflows tend to be less persistent.

5. NUMERICAL GROUNDWATER MODEL DEVELOPMENT

5.1 General

The numerical modelling work was undertaken using the FEFLOW modelling code (Version 7.2). FEFLOW is a finite element modelling code developed by Wasy Institute in Germany (DHI-WASY). The code is capable of simulating saturated and unsaturated groundwater flow under complex boundary conditions.

The finite element formulation of the groundwater flow equation allows for very efficient discretization of the numerical grid for a large study area and complex geological setting. Its use of constrained boundary conditions also allows for greater flexibility in the simulation of underground infrastructure projects.

This code is extensively used in both the private and public sector and is widely recognised in the industry as one of the state-of-the-art codes for groundwater flow and contaminant transport modelling.

5.2 Model Set Up

The extent of the model domain is shown in Figure 5-1. The model covers an area of about 34.5 km^2 extending between MGA projection, Zone 56 grid lines 326,000 m and 333,800 m in the easterly direction and 6,243,600 m and 6,251,300 m in the northerly direction.

Figure 5-1: Model Domain

The FD Model finite element mesh within the model domain consisted of more than 683,500 triangular elements. The size of the elements varies over the model area, with the mesh density increasing towards the tunnels, geological structures, surface water bodies, landfills and the Alexandria Landfill cut-off wall. The size of the triangular elements in a horizontal projection varies as follows:

- about 2.5 m close to the Alexandria Landfill cut-off walls;
- 5 m to 15 m within the St Peters area;
- 10 m to 20 m close to the mainline tunnel alignment and other underground structures (ramps, interchanges); and
- about 120 m close to the boundaries of the model domain.

The finite element mesh for the FD Model is shown in Figure 5-2.

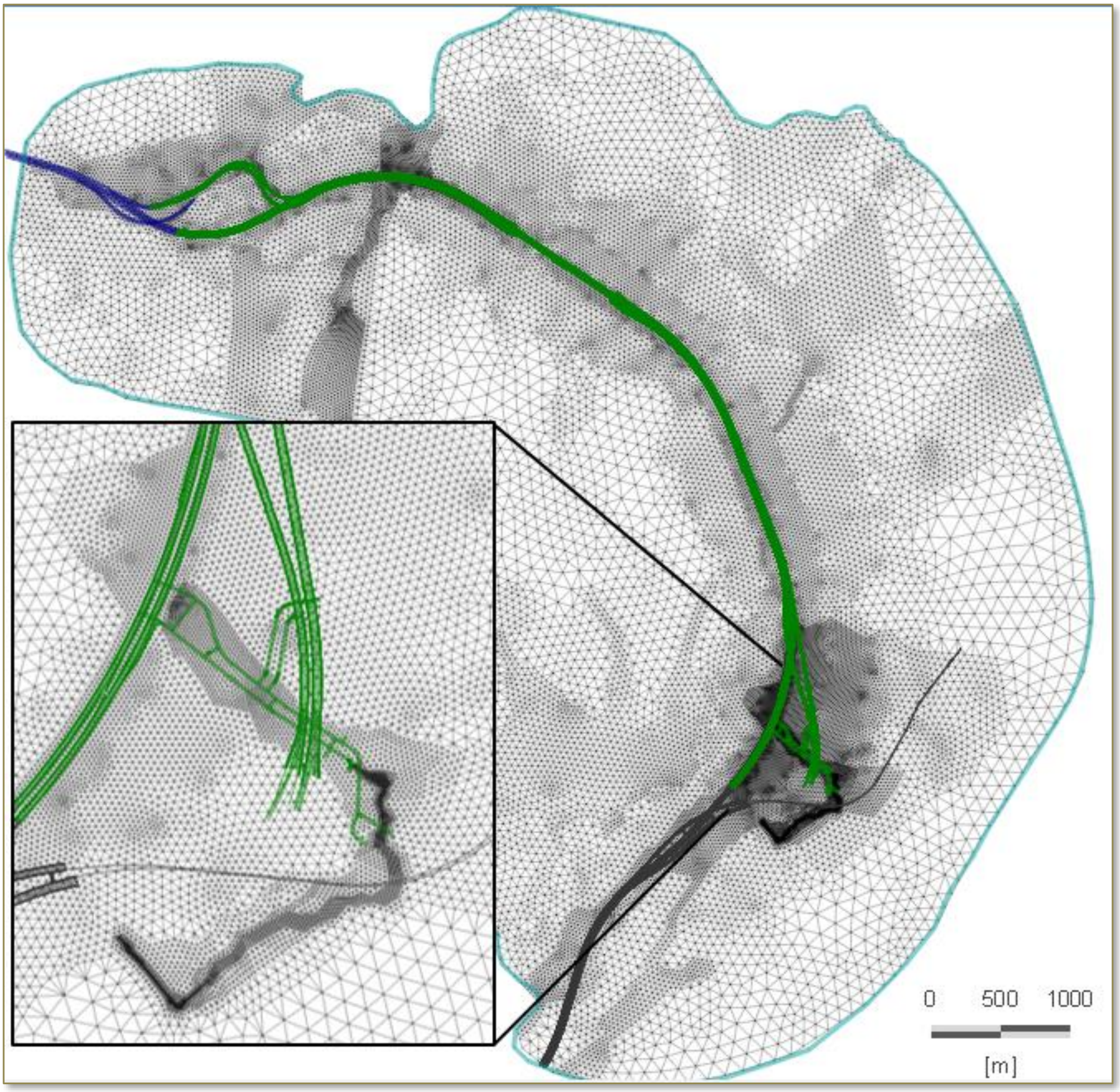


Figure 5-2: FD Model Finite Element Mesh (M4-M5 Link tunnels shown in green, New M5 tunnels shown in black)

The top of the model was set to the topographic elevation obtained from 1 m topographical contours within the study area. The base of the model was set at an elevation of RL -120 m AHD (about 75 m deeper than the deepest section of the mainline carriageway tunnels). It was judged that groundwater flow contribution from the deeper zones of the Hawkesbury Sandstone aquifer system below this elevation would not likely be significant.

The stratigraphy within the model was developed using outputs from a 3D geological model of the domain area produced using the Leapfrog geological modelling software package as described in the Geotechnical Interpretive Report (M4M5-JAJV-PRW-GEO-GT02-RPT-0005). The outlines of the geological units in the model at the ground surface (top of the model) were set to be consistent with the boundaries shown in the Leapfrog model. Fill, if present, was not included in the FD model except within the landfill areas.

The surface outlines of the geological units as included in the FD model are shown in Figure 5-3.

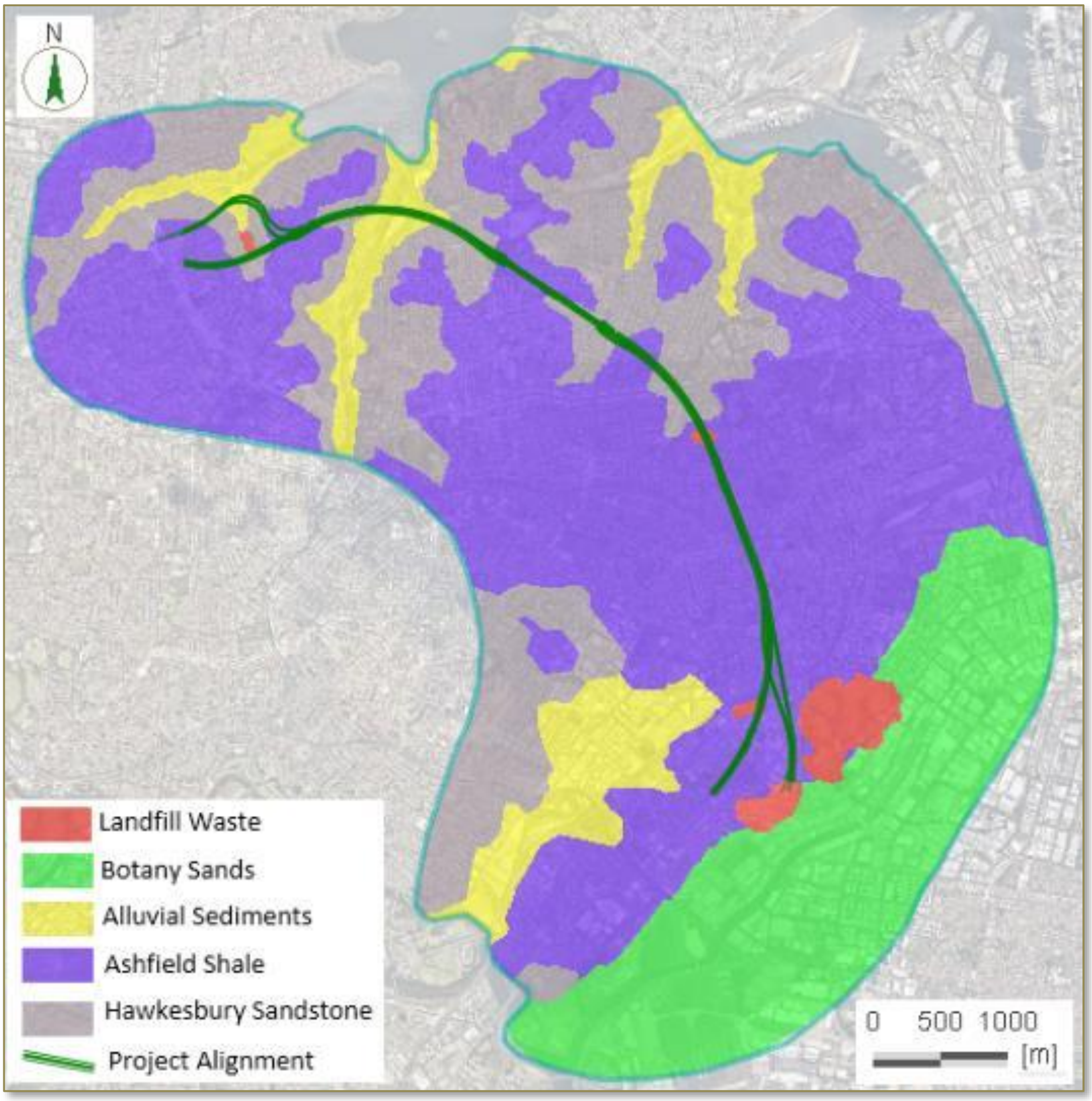


Figure 5-3: Surface Outlines of Model Hydrostratigraphic Units

The vertical distribution of geological units was also based on the interpreted geological unit boundaries in the Leapfrog model and the geological long and cross sections in the Geotechnical Interpretive Report.

From a hydrogeological perspective the Quaternary sediments (undifferentiated alluvium and Botany Sands), Ashfield Shale, Hawkesbury Sandstone and landfill waste were considered to be of primary significance for the groundwater impact assessment and therefore, elevation contours of the base of these units were used to set up the major model slices and layers. The Mittagong Formation was incorporated into the Ashfield Shale layer due to similarity in the hydraulic properties of the two units and the Mittagong Formation's small thickness relative to the thickness of the other major units.

Additional slices were also added between these major slices to:

- enable tunnel simulation within the model profile;
- to allow for better simulation of the vertical hydraulic head gradients and solute transport during dewatering across otherwise thick model layers; and
- to accommodate simulation of localised grouting around the tunnel sections in areas of higher inflow.

The refinement slices were distributed, either equally between the top and base of a particular hydrostratigraphic unit or based on top/invert levels of the feature represented by the refinement slice, such as the mainline tunnels or ramps. They were also adjusted to accommodate simulation of the New M4 and M5 tunnels.

In total, the FDD model was vertically divided into 12 layers, corresponding to 13 model slices. This allowed for definition of the base of each of the above main hydrostratigraphic units, provision for simulation of the vertical flow through each of the hydrostratigraphic units and for the placement of boundary conditions at the nodes representing the tunnels ramps. A justification for the model slices and their objectives is provided in Table 5-1.

Table 5-1: Model Slices and Objectives

Slice No	Description and Main Objective	Layer and Hydrostratigraphic Unit
1	Major slice: Model surface, elevation based on surface topography	Layers 1 to 2: Landfill Waste (Layer 1), Quaternary sediments and residual soils (Layers 1 and 2).
2	Major slice: Base of Landfill Waste	
3	Major slice: Base of Quaternary sediments (Botany Sands and Alluvial Sediments)	Layers 3 to 6: Ashfield Shale and Hawkesbury Sandstone where shales not present. Layer 3 Pleistocene alluvial sediments at Hawthorne Canal.
4	Refinement slice	
5	Refinement slice– based on the SPI shallow section invert levels where above the Hawkesbury Sandstone (up to CH1540 in M190 and CH1520 in M180)	
6	Refinement slice – based on the SPI invert level (from CH1540 to CH1700 for M190 and CH1520 to CH1680 for M180) and shallow sections of the PS21 (up to CH450 in MDS1 and CH150 in MDS2) where above the Hawkesbury Sandstone	
7	Major slice: Base of Ashfield Shale	Layers 7 to 12: Hawksbury Sandstone
8	Refinement slice – based on the PS21 invert levels where within the Hawkesbury Sandstone (MDS1 from CH450 to CH500 and MDS2 from CH150 to CH220) and SPI ramps from CH1700 to CH1820 at M190 and CH1680 to CH1720 at M180.	
9	Refinement slice -based on the Mainline Tunnel invert levels, deep section of PS21 and the SPI ramps invert levels where within the Hawkesbury Sandstone. (MDS1 from CH450 to connection with M120, MDS2 from CH220 to connection with M120, M190 from CH1820 and M180 from CH1720).	
10 to 12	Refinement slices – locally adjusted based on the M5 invert levels	
13	Major slice: Base of the model	

The vertical relationship between the layers in illustrated in Figure 5-4 at the Alexandria Landfill. The cross section was chosen because it intersects almost all of the major hydrostratigraphic units included in the model. A 3D view of the model layers distribution is shown in Figure 5-5.

Where an associated hydrostratigraphic unit was not present laterally, the layer was generally assigned a minimum thickness of 1 m in the model.

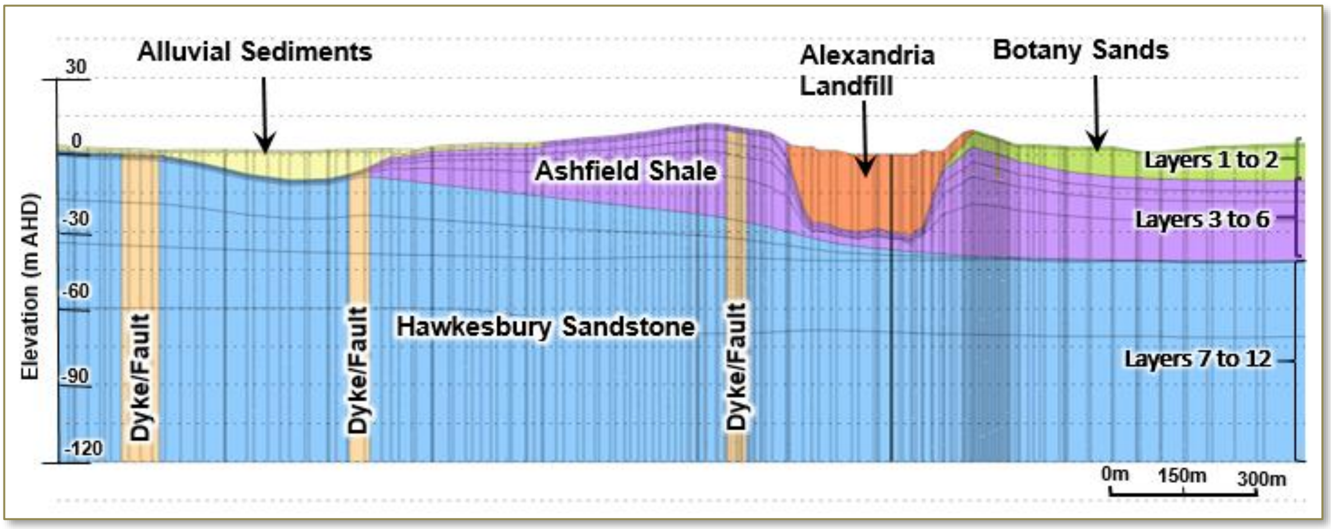


Figure 5-4: Vertical Distribution of the Model Layers

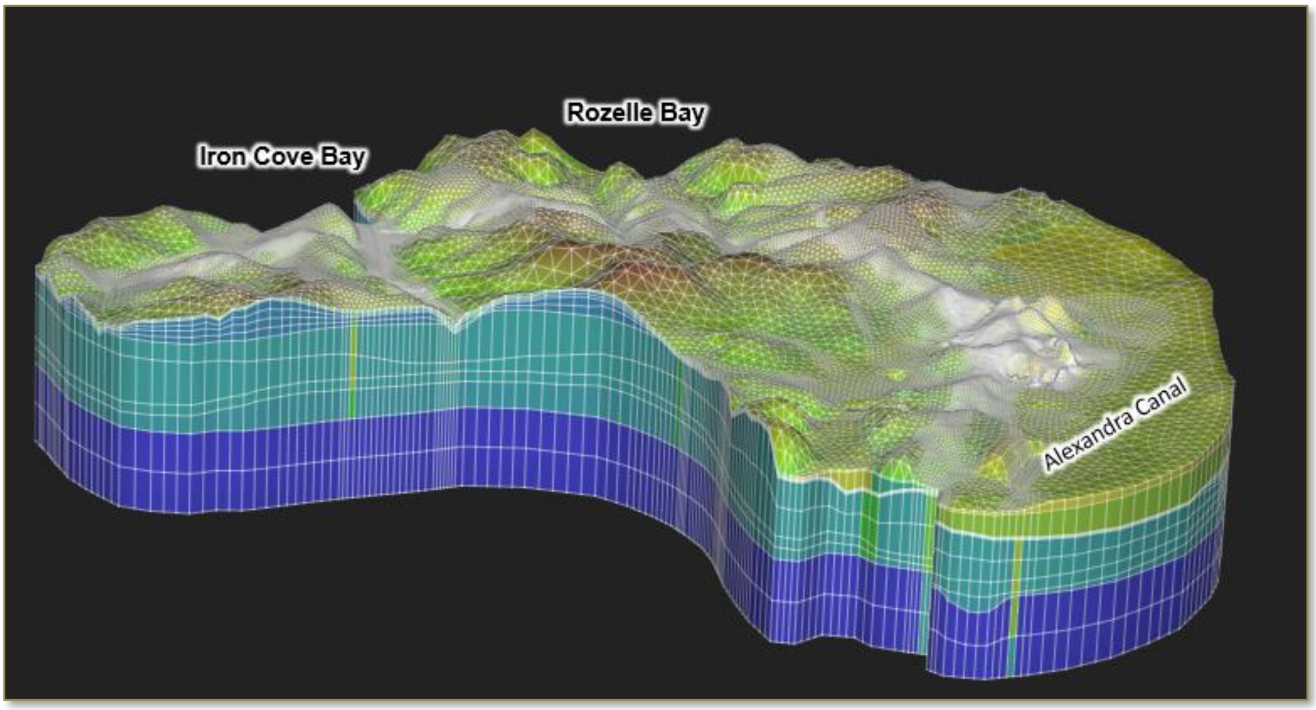


Figure 5-5: 3D View of Model Layers Distribution

5.3 Model Boundary Conditions

Three types of boundary condition were assigned to the model mesh: fixed head, head dependent flux and no-flow (zero flux). Positions of the boundary conditions in plan are shown in Figure 5-6. The boundaries were set in different layers, which corresponded to the conditions, elevations and type of features represented by the assigned boundaries.

Fixed head boundary conditions were applied to nodes along some of the natural features (rivers, drainage lines and large surface water bodies) to simulate interactions of these features with the groundwater system. These features included:

- Sydenham drainage lines
- Airport Lake in Alexandria
- Cooks River
- Sydney Harbour at Rozelle Bay and Iron Cove Bay

The boundary heads for the drainage lines were defined based on the 'ground' surface levels at these locations, assuming the surface water level to be at least 1.0 m below ground surface. Heads of RL 0 m AHD were assigned to the nodes along the Cooks River, Rozelle Bay and Iron Cove Bay, as well as along the tidal sections of the surface watercourses draining into these features.

Fixed head boundary conditions were also used to simulate leachate extraction at the Alexandria landfill. The fixed head boundary conditions were applied to the nodes at location of the leachate extraction sumps and leachate drains that connect into the sumps. The boundary conditions were constrained to allow only for groundwater extraction.

The head values initially set at the model nodes were adjusted during the calibration process to generally match the groundwater levels observed within this area.

Fixed head boundary conditions were also applied to nodes representing the drained mainline tunnels, ramps and ventilation tunnels for the M4-M5 Link and M4 East and New M5 motorways. All nodes were constrained to allow only for groundwater inflow into these structures. These boundary conditions assume free drainage into the tunnel drainage system during construction and operation and no resistance to the groundwater inflow.

Head dependent flux boundary conditions (Cauchy boundary condition). This type of boundary condition allows flow to occur into or out of the model domain, depending on the groundwater level calculated at a model node and the head defined as the boundary condition. The direction of the flow (out of or into the model) depends on the relative difference between these two levels, i.e., whether the calculated groundwater level is above or below the boundary condition head level. The water flux is calculated based on the head difference and the connectivity (referred to in the model as transfer) between the groundwater and the boundary feature.

A head dependent boundary condition was assigned for:

- Alexandra Canal
- Johnstons Creek
- Whites Creek
- Hawthorne Canal
- Iron Cove Creek
- Eastern model boundary nodes to allow for groundwater to leave or enter the model to/from distant groundwater features such as Botany Bay and Mill Stream to the south and the Black Water Bay palaeochannel to the northeast.

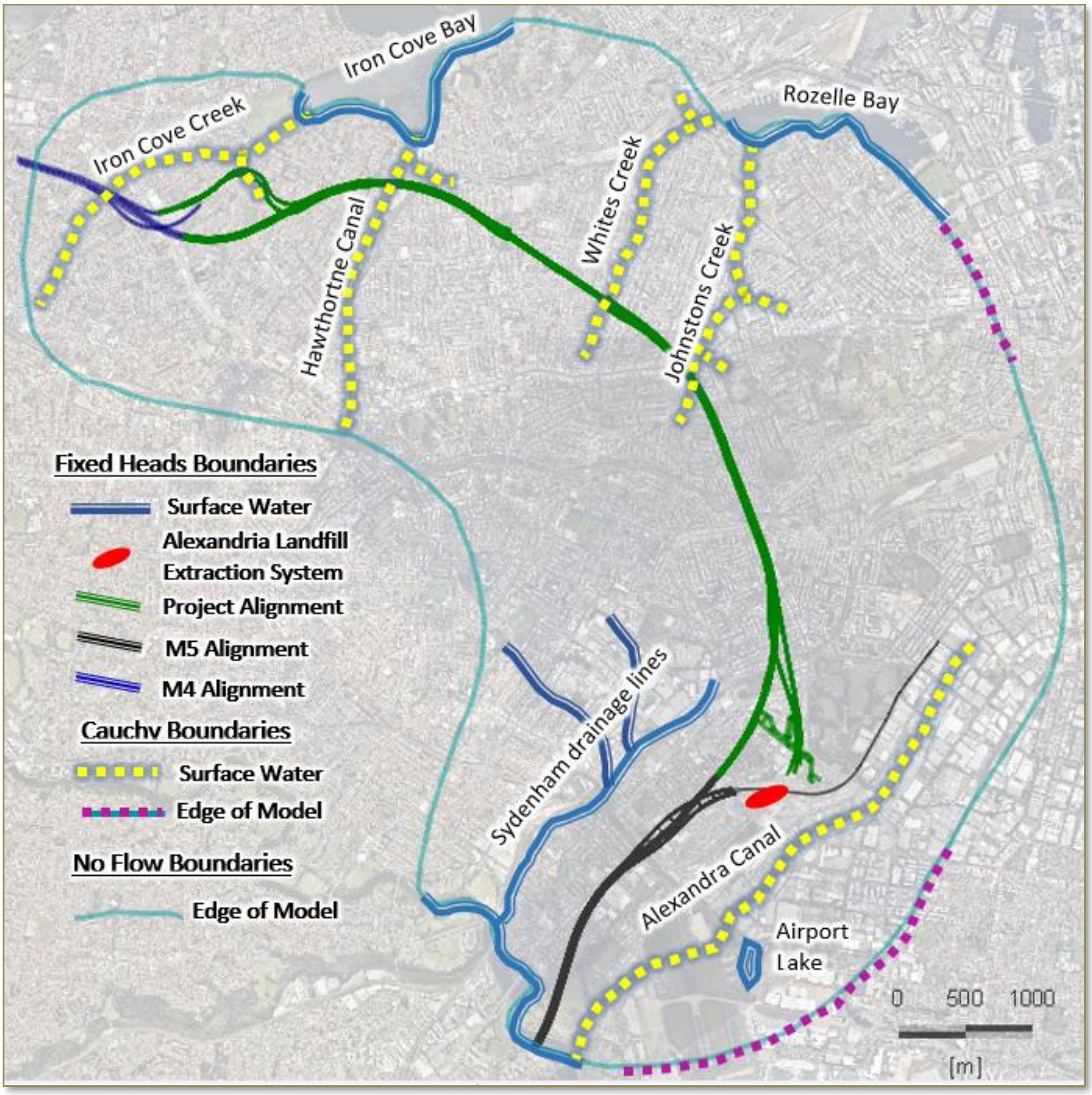


Figure 5-6: Model Boundary Conditions

The Cauchy boundary condition values for the creeks and canals were defined based on the water surface elevation being 1.0 m to 1.5 m below the topographic values provided or groundwater levels indicated in the monitoring bores close to the modelled feature. The nodes along the minor tributaries and upper reaches of the creeks were constrained to allow for groundwater discharge only.

No-flow boundary conditions were assigned along the western and south western boundaries of the model domain, as well as sections along the northern and eastern boundaries of the model domain (Figure 5-6). With exception of the south-western boundary, the no-flow boundary conditions were applied where no flow towards the mainline tunnels was expected due to the presence of a catchment divide or the flow being controlled by the M4 East or New M5 motorways.

To assess the potential effects of a no-flow boundary also being present along the southwestern model boundary, an additional sensitivity run was undertaken. The results of this sensitivity analysis are discussed in Section 9.

Additionally, a no flow boundary was set at the base of the model domain, as it was conceptualised that groundwater flow from deeper in the Hawkesbury Sandstone was unlikely to contribute significantly to the groundwater inflow into the tunnel.

Recharge to groundwater was applied to the ground surface in the top slice of the model. The recharge values were adjusted during the calibration process to match the groundwater levels and head gradients inferred across the study area. In general, higher recharge values were assigned for the higher permeable formations (Botany Sands, alluvium sediments and landfills with no engineered landfill cap) and open space/parks area. The recharge rates adopted for the FD Model at the end of the calibration process are summarised and discussed in Section 6.4.2.

6. MODEL CALIBRATION

6.1 Calibration Approach

Calibration of the model was undertaken in both transient and steady state modes as follows:

- Transient calibration using the results from the Hawthorne Canal pumping test was undertaken first. A local scale numerical model focused on Hawthorne Canal was prepared for this purpose. An initial steady state simulation was also undertaken prior to transient calibration to check that initial groundwater levels fit reasonably well to the groundwater levels observed in the monitoring wells prior to the pumping test. Changes in the groundwater heads during the pumping test, rather than groundwater elevations was the main target of the calibration and therefore no comprehensive steady state calibration was undertaken for this small-scale model.
- Using the results from the Hawthorn Canal model calibration, the larger scale FD model was then re-calibrated in a steady state mode using the groundwater level observations recorded in the project monitoring wells and other level data available from the monitoring wells within the regional model domain.

Both of these models were run as unconfined saturated flow models, with the top model slice being defined as the phreatic surface and all other model slices set as dependent slices. A PCG solver (preconditioned conjugate-gradient method) was used and the error tolerance was set at 5×10^{-3} m.

Calibration was performed by adjusting recharge rates, hydraulic parameters, head boundary conditions and connectivity of the groundwater system to the surface water bodies until an acceptable fit between the model predicted and observed monitoring well levels was achieved. Data recorded during the pumping test at the Hawthorne Canal, groundwater levels collected at the project monitoring wells between 2016 and 2020, and the available groundwater level data from the M4 East and New M5 Projects were all utilised for model calibration.

6.2 Pumping Test Calibration

6.2.1 Hawthorne Canal Model Setting

Borehole packer tests indicated a zone of increased permeability within the bedrock profile under the Hawthorne Canal palaeovalley. This zone was observed, typically, to occur between 30 m and 40 m below ground surface (RL -40 m AHD to RL -30 m AHD) and is inferred to corresponds to discrete horizontal bedding partings and fracture opening in response to valley stress relief. This zone is further referred herein as the “higher permeability rock zone”. The pumping test was designed to be undertaken within this zone to understand its potential extents and hydraulic properties. Additionally, the aim of the test was to further characterise hydraulic properties of the alluvial sediments, hydraulic connectivity between rock and alluvium units and connectivity between groundwater and surface water within the Hawthorne Canal area.

The local scale numerical model was therefore developed as follows:

- Adopting the numerical model reported in the SDD Modelling Report (Report Version B) as the starting point, but only retaining the model domain area to the west of Whites Creek and the Whites Creek Fault (i.e. the model domain to the east of the Whites Creek and Whites Creek Fault was cut out).
- The model layers were updated based on the 3D geological model documented in the FD Geotechnical Interpretive Report.
- The Hawthorne Canal alluvium was split into three layers to enable simulation of the vertical head gradients observed during the test:
  - a top layer with maximum thickness of 3 m represented combined fill and upper Holocene sediments;
  - a middle layer varying in thickness from 1 m to 10 m representing Holocene sediments; and
  - a bottom layer up to 5 m thick representing Pleistocene sediments.
- Top and bottom elevations of the Model layer 9 and layer 10 were updated to correspond to the top and base of the higher permeability rock zone inferred from the results of the hydraulic testing. The base of the

layer 10 (model slice 11) was set at about RL -40 m AHD within the test area and the top of layer 9 (model slice 9) at about RL -30 m AHD. The pumping well screen was contained within the layer 10.

- The model mesh was refined within the pumping test area to enable replication of the groundwater cone of depression that has been observed during the test. The size of the triangular elements in a horizontal projection was about 0.2 m at the pumping well location and approximately 4 m within the pumping test area.

6.2.2 Data Used and Initial Parameters

Groundwater levels prior to, during and post the pumping test groundwater extraction phases were monitored in the following wells:

- 11 pumping test groundwater wells (including the pumping well)
- 5 vibrating wire piezometers (VWP) installed in the alluvial sediments; and
- an additional 13 monitoring wells already installed as part of the detailed design site investigation.

Data from all of these wells was utilised for model calibration.

The main groundwater extraction phase for the pumping test lasted for 27 days. During this period an extraction rate of about 2.0 L/s was maintained for 23 days when it was increased to 2.5 L/s. While several pumping stoppages occurred during the main pumping test phase, their length were short and did not compromise the integrity of the overall pumping test.

To decrease numerical model run times, the model was simplified by not attempting to simulate the pumping stoppages, but instead making a slight adjustment to the simulated pumping rate. As all stoppages occurred during the pumping test period where the extraction rate was about 2.0 L/s, it was possible to calculate an equivalent constant pumping rate of 1.98 L/s, based on total water volume extracted during this period of the test.

In developing the pumping test calibration dataset, original datasets from the data loggers and VWPs (which contain tens of thousands of datapoints) were simplified to approximately 40 datapoints each (including the recovery period). When selecting these datapoints, parts of the original dataset affected by pumping stoppages were avoided. At the same time, it was ensured that all key characteristics of the pumping test response and recovery curves were captured. Where logger/VWP artefacts manifested as minor water level fluctuations in the original datasets, the datapoints were reviewed to check that the selected values were generally reflective of average water level at that point in time.

Interpretation of the pumping test results using analytical solutions suggested hydraulic conductivities of the Hawkesbury Sediments higher permeability rock zone to be within the low 10^{-6} m/s range and specific storage mid to high 10^{-6} m^{-1} range (assuming aquifer thickness of 30 m). Vertical hydraulic conductivity of the rock above the pumping test interval through which leakage from the overlying alluvial aquifer is occurring, was calculated to be between mid-10^{-8} m/s to mid-10^{-7} m/s. This was based on an inferred “leaky aquitard” thickness of 16 m. Although these values were derived from an analytical solution that consider a simplified, isotropic and porous media model, they provided initial values for model calibration purposes. Initial hydraulic conductivity values for the Hawthorne Canal alluvial aquifer were based on the results of the CPT testing, which suggest hydraulic conductivities for the Holocene sediments to be in a range from 10^{-9} m/s to 10^{-7} m/s and for the Pleistocene sediments from 10^{-7} m/s to 10^{-6} m/s.

Details of the pumping test, monitoring undertaken and results are reported in Appendix F of the Hydrogeological Design Report, while details of the CPT testing are reported in the Geotechnical Factual Report.

Groundwater responses to the August 2019 and September 2019 rainfall events were used to assess potential ranges of porosity within the alluvial sediments. As discussed in Section 4.4, these responses varied significantly between monitoring location and rainfall intensity. However, based on range of responses observed, an overall specific yield value of 0.2 or less, was considered to be a reasonable approximation of this parameter. For the purposes of the modelling 0.2 was therefore adopted as an initial representative value.

6.2.3 Results of Calibration and Statistics

Simulated transient groundwater drawdowns and temporal differences from observed groundwater drawdowns are shown in observed versus modelled hydrographs in Appendix BA.

The groundwater drawdowns in the deep Hawkesbury Sandstone rock zone (rock interval between RL -40 m AHD and RL -30 m AHD) at the end of pumping derived by the calibrated model are shown in Figure 6-1, with the spatial hydraulic conductivity distribution and extent of the higher permeable zone derived from the calibration process shown in Figure BC10 (Appendix BC).

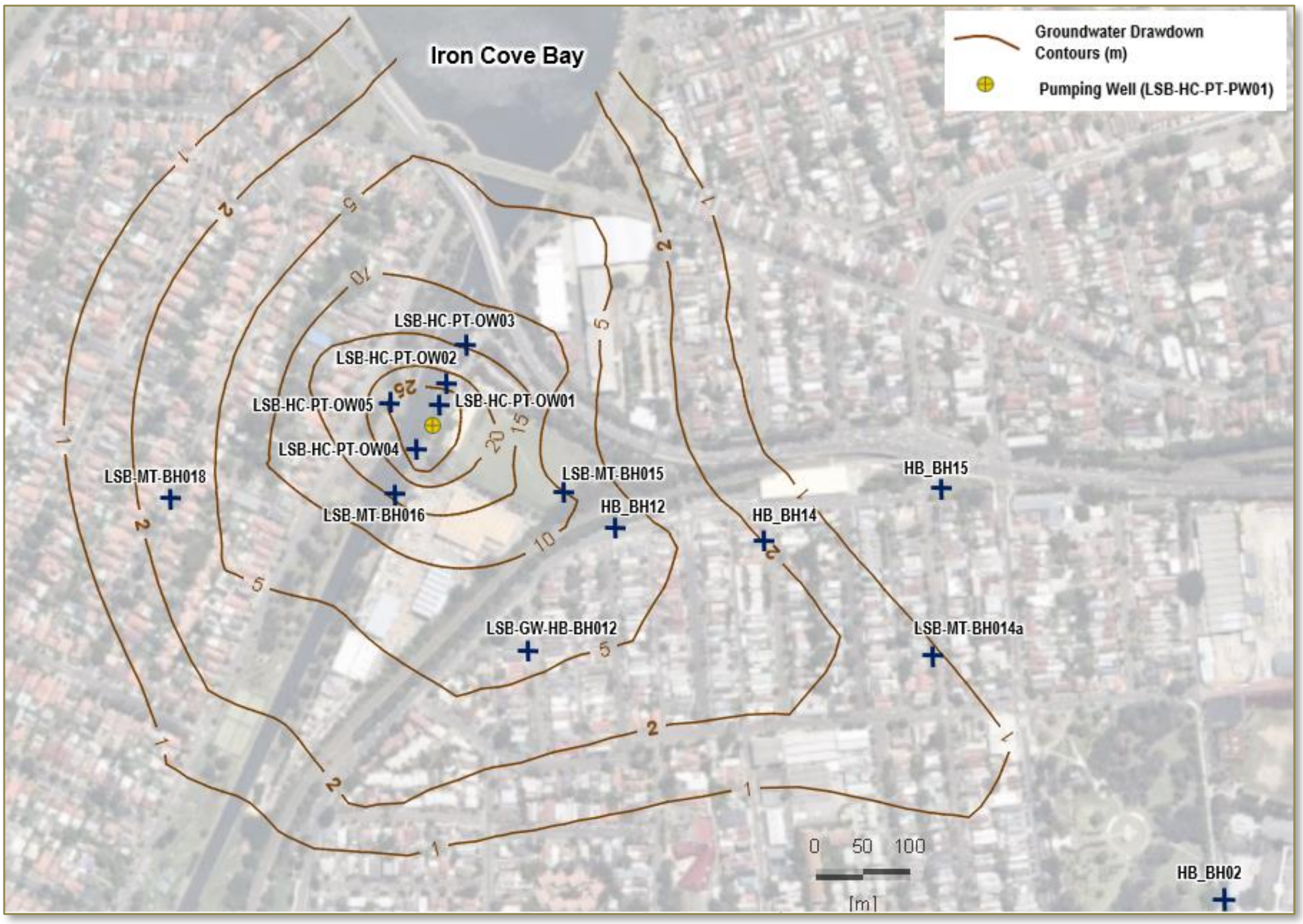


Figure 6-1: Calibrated Model Groundwater Drawdown in Hawkesbury Sandstone at end of Pumping Test

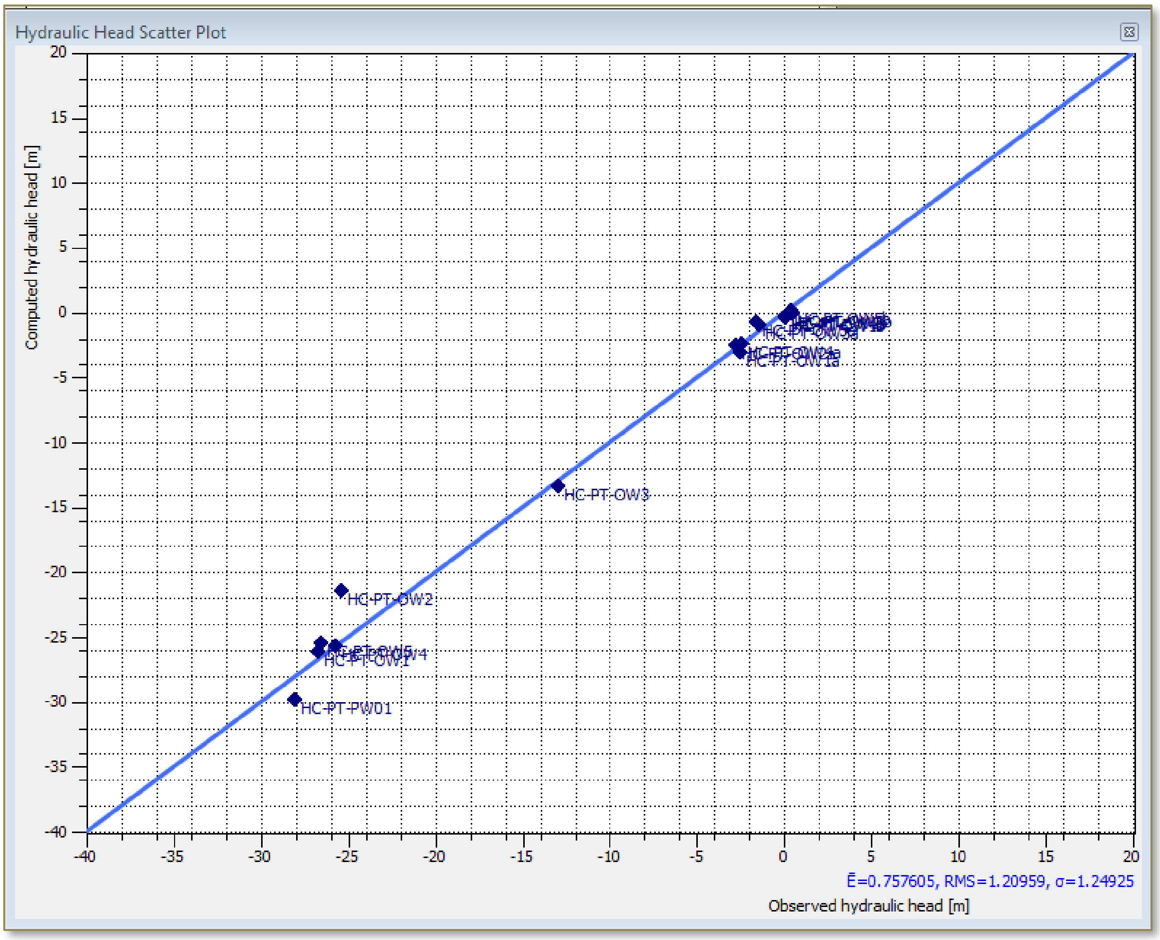
As shown in the hydrographs in Appendix BA and summarised in Table 6-1, the model achieved a very good fit with groundwater drawdowns observed in the deep Hawkesbury Sandstone observation wells (OW' series of the wells) located closer to the pumping well and, overall, a good fit with drawdowns observed in the alluvial wells. This is also demonstrated in Figure 6-2 by calibration statistics (scatter plot) at the end of the pumping test for these wells. A poorer fit, however, was achieved for the Hawkesbury Sandstone wells LSB-GW-HB-BH12, HB_BH14 and LSB-MT-BH014a.

The shape of the groundwater recovery curves generated by the model for deep Hawkesbury Sandstone wells also suggests a satisfactory calibration was achieved with respect to the rock storage parameters and seepage rates from the shallower Hawkesbury Sandstone aquifer zones.

Table 6-1: Observed versus Modelled Groundwater Drawdowns at the End of Hawthorne Canal Pumping Test

Well ID	Groundwater Drawdown at End of Pumping (m)		
	Observed	Modelled	Residual
LSB-GW-HB-BH03	0	0.03	-0.03
LSB-GW-HB-BH08d	18.9	13.42	5.48
LSB-GW-HB-BH12	15.5	6.89	8.61
HB_BH12	>6.4	6.34	
HB_BH14	7.6	0.55	7.05
HB_BH15	0.5	0.19	0.31
LSB-HC-PT-OW1*	28.2	27.02	1.18
LSB-HC-PT-OW1*a	3.7	3.47	0.23
LSB-HC-PT-OW1b*	0.4	0.70	-0.30
LSB-HC-PT-OW2*	26.8	22.28	4.52
LSB-HC-PT-OW2a*	3.9	2.94	0.96
LSB-HC-PT-OW2b*	0.2	0.52	-0.32
LSB-HC-PT-OW3*	14.3	14.11	0.19
LSB-HC-PT-OW3a*	2.1	1.22	0.88
LSB-HC-PT-OW3b*	0	0.31	-0.31
LSB-HC-PT-OW4*	27.1	26.64	0.46
LSB-HC-PT-OW4a*	3.4	2.76	0.64
LSB-HC-PT-OW4b*	0.8	0.61	0.19
LSB-HC-PT-OW5*	28	26.41	1.59
LSB-HC-PT-OW5a*	2.5	1.09	1.41
LSB-HC-PT-OW5b*	0	0.21	-0.21
LSB-HC-PT-PW01**	28.4	30.81	-2.41
MT_BH02	>1.20	0.36	
LSB-MT-BH1014a	5.4	1.08	4.32
LSB-MT-BH1015	8.6	9.88	-1.28
LSB-MT-BH1015a	0	0.03	-0.03
LSB-MT-BH1016	>9.65	16.43	
LSB-MT-BH1018	6.6	3.01	3.59
LSB-MT-BH1013a	2.8	0.61	2.19

Note: ** - Pumping well: * – Observation wells installed for purpose of pumping test monitoring, OW' denotes deep Hawkesbury Sandstone wells, OWa' denotes wells installed at the base of Hawthorne alluvium and OWb' denotes Vibrating Wire Piezometers installed close to water table within the alluvial sediments.



(Notes to figure: E = mean difference; RMS = root mean square error; σ = standard deviation)

Figure 6-2: Observed versus Modelled Groundwater Levels at the End of Pumping Test for Test Site Monitoring Wells (LSB-HC-PT-OW', LSB-HC-PT-OWa' and LSB-HC-PT-OWb' series and LSB-HC-PT-PW01)

The hydraulic properties adopted at the end the pumping test calibration process are listed in Table 6-4. Key findings derived from the calibration process were as follows.:

- The hydraulic conductivities of the higher permeability rock zone are generally, in the order of 2.0e-06 m/s (horizontal (Kh) equal to vertical (Kv)).
- The hydraulic conductivity of the rock below the higher permeability rock zone likely decreases with depth.
- A narrow easterly trending feature with hydraulic conductivity of 2.0e-05 m/s is present within the higher permeability rock zone, in addition to a localised area centred around the pumping well with a hydraulic conductivity of 8e-05 m/s. These hydraulic conductivity values are, generally, consistent with the higher end of Lugeon values reported for the Hawthorne Canal area.
- The Hawthorne Canal Fault has a significantly lower hydraulic conductivity than previously inferred. Hydraulic conductivities (Kh equal to Kv) of 1.5e-07 m/s and 1.0e-07 m/s were derived for this feature.
- A second higher permeability rock zone, which is a few metres thick, is inferred to be present higher in the rock profile beneath the Hawthorne Canal alluvium. This is also believed to be associated with a horizontal stress relief feature. A horizontal hydraulic conductivity of 1.0e-06 m/s and vertical hydraulic conductivity of 1.0e-08 m/s were derived for this zone.
- Holocene and Pleistocene alluvial sediments to be of relatively low hydraulic conductivities, 1.5e-07 m/s and 5.0e-07 m/s respectively (Kh equal to Kv). This is within the results of the CPT testing undertaken during the LSJV detail design investigation.
- Low connectivity with the Hawthorne Canal surface water with respect to surface water recharge to groundwater as discussed further in Section 9.

- Specific storage of the rock to be relatively low, within mid 10^{-6} m^{-1}, which is consistent with the pumping test analytical interpretation and published values.
- Specific storage of the alluvium sediments to be about one and a half orders of magnitude greater than the value derived for the rock.

Overall, the results of the Hawthorne Canal Model calibration suggest that the groundwater flow system is dominated by the inferred higher permeability zones associated with discrete horizontal bedding partings and fracturing, rather than subvertical features associated with inferred Hawthorne Canal Fault zone as initially postulated.

6.3 FD Model Steady State Calibration

6.3.1 Approach and Data Used

Following the successful calibration of the Hawthorne Canal model, the material properties adopted at the end of the calibration process were upscaled into the FD model, with the material properties within the Hawthorne Canal and surrounding area kept constant during the FD model calibration process.

Groundwater level data was available for a total of 144 monitoring wells within the FD model domain, which include data from:

- 80 monitoring wells installed during the LSBJV detailed design site investigation for the M4-M5 Link tunnels;
- 41 monitoring wells installed during the pre-contract award investigation; and
- 23 monitoring wells installed for the M4 East and New M5 projects.

A quality control review of the data was performed prior to use. As the model was to be calibrated for steady state conditions, the groundwater level dataset was checked to see if it had been influenced by any of the recent infrastructure construction within the project corridor. The groundwater hydrographs and correlation between the groundwater levels and ground surface elevations discussed in Section 4.4 were utilised for this assessment. As results of this review, groundwater level data from a total of 121 monitoring wells was deemed as suitable for the steady state model calibration and data from 23 monitoring wells was rejected. Calibration based on temporal changes, however, was not considered valuable due to the limited number and overall distribution of monitoring wells with four years of monitoring data.

Where more than one observation point was available for a monitoring point prior to dewatering activities, an average groundwater level was used otherwise a stable level estimated from the data trend was adopted. Although the timing of groundwater levels measurements varied between individual locations, the final calibration dataset was assessed to be representative of initial groundwater conditions and as such suitable for use as the basis of calibration of the FD model.

6.3.2 Calibration Statistics

Calibration statistics, based on the predicted verse observed groundwater levels at the 121 monitoring wells are summarised in Table 6-2 and shown graphically in Figure 6-3. The dashed lines on the calibration graph define the +/-5 m difference limits between the modelled and observed groundwater heads, i.e. points within dashed lines have absolute difference between modelled and observed heads less than 5 m.

The locations of the wells used for model calibration, the well ID's and the difference between observed and modelled groundwater levels at these wells are shown in Figure 6-4. These differences are also summarised in Table BB1 in Appendix BB.

Table 6-2: Steady State Model Calibration Statistics, FD Model

Number of Observation Points	121
Maximum Residual (m)	7.96
Minimum Residual (m)	-6.78
Residual Mean (m)	-0.36
Absolute Residual Mean (m)	1.59
Normalised Root Mean Squared (NRMS)	5.04%
Root Mean Squared (RMS) Error (m)	2.21
Correlation Factor (R^2) Observed vs Modelled Levels	0.89

At the completion of the steady state calibration process, a normalised root mean squared error (NRMS) of about 5 % and root mean squared (RMS) error of about 2.2 m were achieved.

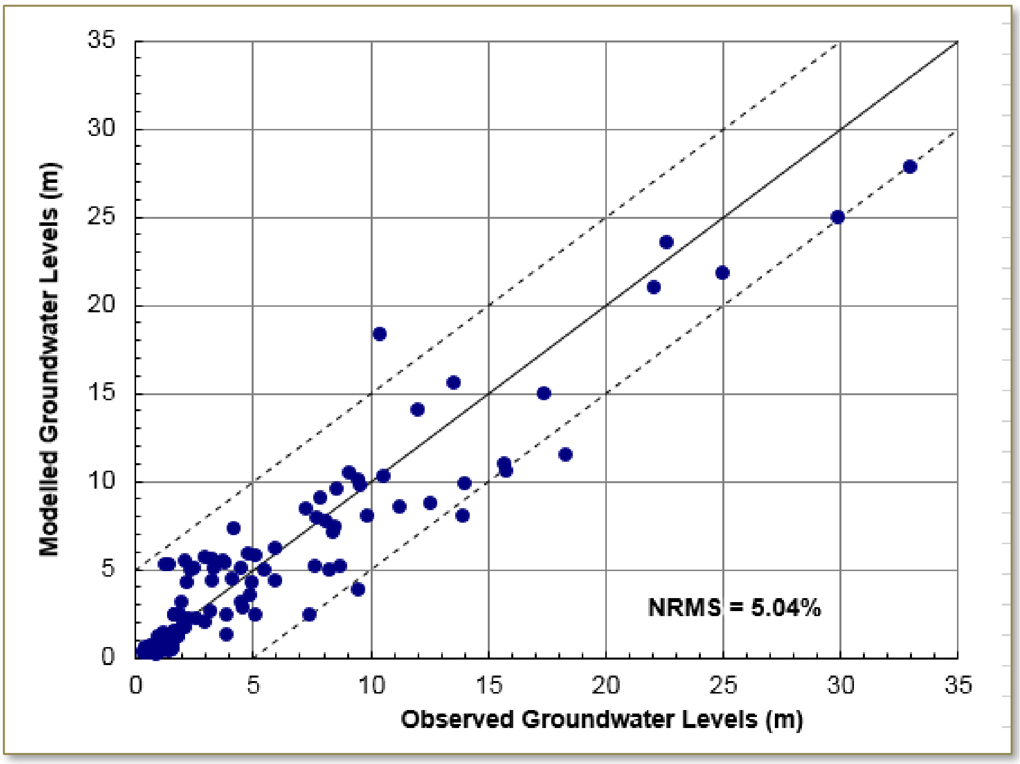


Figure 6-3: Steady State FD Model Calibration Graph

A water balance error at the end of the steady state calibration process of about 0.004% was achieved.

A summary of the steady state water balance is provided in Table 6-3.

Table 6-3: Water Balance Statistics, FD Model

Boundary Conditions	Flux (m^3/day)	
	Out of Model	Into Model
Fixed Head boundaries	483	5.2
Cauchy Boundaries	547	0.3
Well boundary	0	0
Aerial Recharge	0	1025
Total	1030.01	1029.97
IMBALANCE	0.041 (0.004%)	

Based on these calibration statistics and the model's ability to generally match observed groundwater levels, expected head gradients and groundwater drawdowns observed during the Hawthorne Canal pumping test, the FD Model was considered to have been calibrated to a satisfactory level for the purposes of assessing the potential groundwater inflows and drawdowns, and saline water migration in accordance with the MCoA E193 and REMMs GW6 and GW7 requirements.

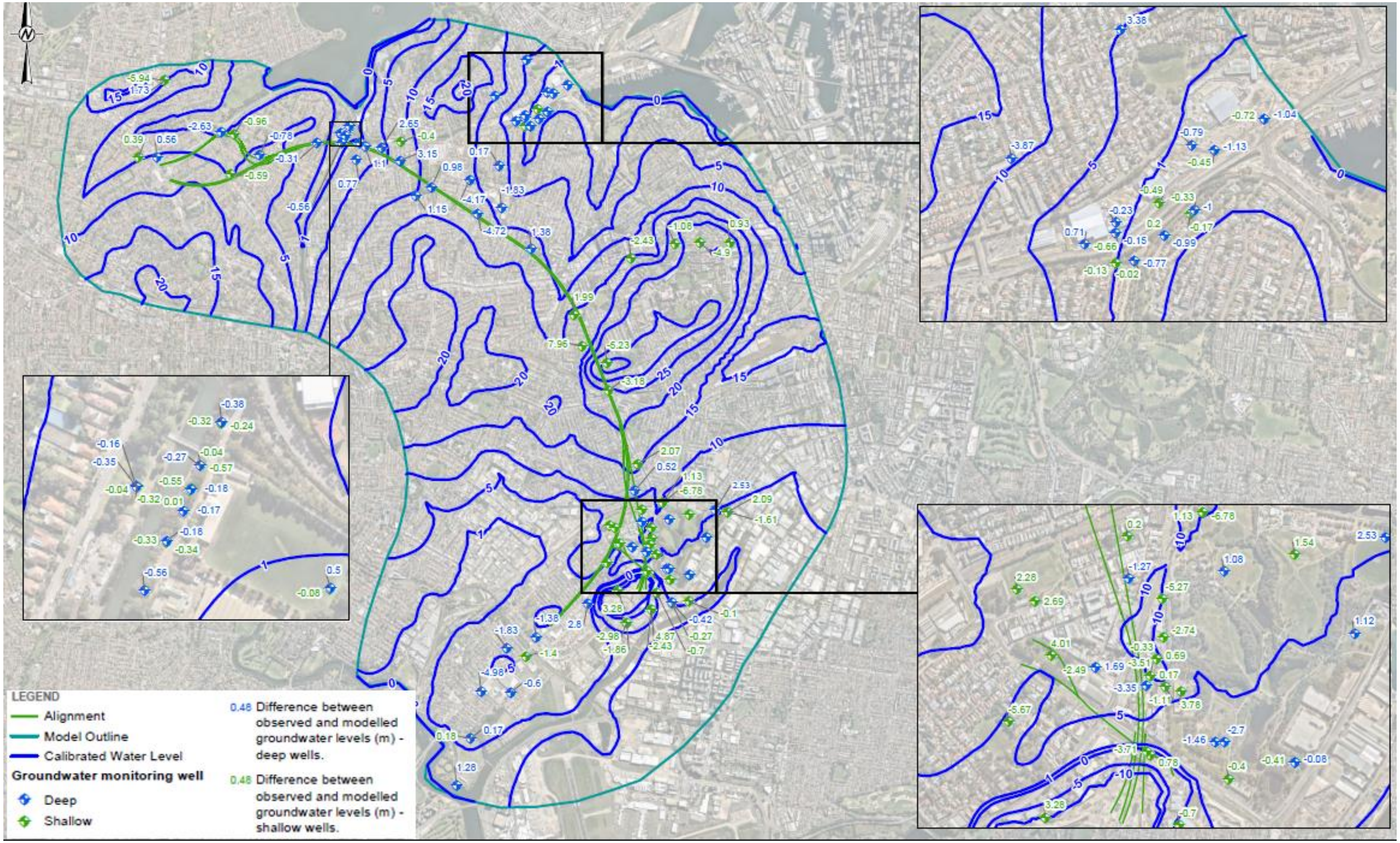


Figure 6-4: Calibrated FD Model Groundwater Levels at the Water Table and Residual Calibration Errors

6.4 Parameters Adopted

6.4.1 Hydraulic Parameters

Hydraulic conductivities of the hydrostratigraphic units adopted at the end of the FD Model calibration for each of the model layers are shown in Figures BC1 to BC13of Appendix BC. A summary of the hydraulic conductivity ranges adopted for individual hydrostratigraphic units is provided in Table 6-4 below.

Hydraulic conductivities in the x and y directions (Kx and Ky) were equal for all hydrostratigraphic units in the model. Vertical hydraulic conductivity (Kz) varied between individual units as listed in Table 6-4. In general, however, Kz was one order of magnitude lower than Kx or Ky for rock units, and half to one order of magnitude lower for the alluvial sediments. For the landfill waste, fault zones and residual soil units, hydraulic conductivities in each direction were generally kept equal.

Storage parameters were derived through the calibration of the Hawthorne Canal pumping test and then were upscaled to the FD model. The storage parameters adopted for the model hydrostratigraphic units which were not included in the transient calibration were also refined based on publish values and past project experience in these geological units.

The specific yield and effective porosity values adopted for the Hawkesbury Sandstone and Ashfield Shales (1 % to 3 %) were also comparable with the values used by Hydrosimulation for WestConnex M4-M5 Link Modelling (Hydrosimulation, 2017) and by GHD (INFO Doc – 441, GHD 2018)[3] for the SPI Interchange contaminant groundwater assessment. The specific storages adopted for the landfill waste and alluvial sediments are above 1.3e-05 m^{-1} as proposed by Raue[4] to be a plausible upper boundary of specific storage for unconsolidated, compressible sediments. Additionally, it should be noted that these sediments are largely unconfined and therefore the specific yield is the key parameter for groundwater flow rather than specific storage.

The storage values which were adopted in the model for all units are listed in Table 6-4. The sensitivity of the model results with respect to storage parameters was tested and the results of this assessment are discussed further in Section 9.

Table 6-4: Hydraulic Properties Adopted in the FD Model

Hydrostratigraphic Unit	Hydraulic Conductivity Range (m/s)		Specific Yield / Effective porosity	Specific Storage (1/m), Ss
	Kx = Ky	Kz		
Landfill Waste	5e-06 to 6.4e-05	5e-06 to 6.4e-05	0.20	5e-04
Alluvial Sediments and reclaim land (costal area)	1.5e-07 to 2e-05	1.5e-07 to 2e-05	0.15 and 0.2	1e-04
Botany Sands (also includes undifferentiated estuarine sediments)	3.8e-06 and 1e-05	2.5e-07 and 1e-06	0.15	1e-04

3 INFO DOC – 441: GHD, 2018: Project Giant – St Peters Interchange Information Document Review of Potential Technical Solutions to Landfill Related Contamination, prepared for NSW Treasury, 1st June 2018

4 Rau, G. C. at all: Quantifying compressible groundwater storage by combining cross-hole seismic surveys and head responses to atmospheric tides, Water Research Laboratory, NSW Australia

Hydrostratigraphic Unit	Hydraulic Conductivity Range (m/s) Kx = Ky	Kz	Specific Yield / Effective porosity	Specific Storage (1/m), Ss
Residual Soils – (including extremely weathered Ashfield Shale around Sydney Park, Ashfield Shale and Hawkesbury Sandstone soils)	2e-08 to 9e-07	2e-09 to 9e-07	0.12 and 0.15	1e-04 (clays) and 1e10-5
Ashfield Shale- bulk	5e-08 1e-08 (localised)	5e-09 1e-09 (localised)	0.03	5e-06
Woolloomooloo Fault – Ashfield Shale	2e-06	2e-06	0.04	6e-06
Luna Park Fault – Ashfield Shale	5e-07	5e-07	0.04	6e-06
Johnsons Creek and Whites Creek Faults – Ashfield Shale	2e-07	2e-07	0.04	6e-06
Haberfield Dyke – Ashfield Shale	5e-07	5e-07	0.04	6e-06
Hawkesbury Sandstone - bulk	1e-07 1e-08 and 8e-08 (deeper zones)	1e-08 1e-09 and 8e-09 (deeper zones)	0.01	5e-06
Hawkesbury Sandstone – high permeability rock zones (Hawthorne Canal)	1e-06 to 8e-05	1e-08 to 8e-05	0.02	4e-06
Woolloomooloo Fault – Hawkesbury Sandstone	1e-07 to 6e-06 (reducing with depth)	1e-07 to 6e-06 (reducing with depth)	0.01 and 0.04	6e-06
Luna Park Faults – Hawkesbury Sandstone	1e-07 and 2e-06 (lower at depth)	1e-07 and 2e-06 (lower at depth)	0.01 and 0.04	6e-06
Johnsons Creek and Whites Creek Faults – Hawthorne Sandstone	1e-08 to 1e-06 (reducing with depth)	1e-09 to 1e-06- (reducing with depth)	0.01 and 0.04	6e-06
Hawthorne Canal Fault – Hawkesbury Sandstone	1e-08 to 1.5e-07 (reducing with depth)	1e-09 to 1.5e-07 (reducing with depth)	0.01 and 0.04	6e-06
Haberfield Dyke – Hawkesbury Sandstone	1e-08 to 1.5e-06 (reducing with depth)	1e-09 to 5e-06 (reducing with depth)	0.01 and 0.04	6e-06

Note: Location of the Fault Zones and Dyke as adopted in the model are shown in Figures BC1 to BC13 (Appendix BC)

6.4.2 Transfer Rates and Recharge

Connectivity of groundwater and surface water in the model is controlled by transfer-in (surface water transfer into the model) and transfer-out (groundwater discharge to surface water out of the model). General, it is expected that transfer-out is greater than transfer-in, i.e., easier for groundwater to discharge into a surface water body than to surface water to infiltrate into groundwater. This is based on a concept that clean groundwater "flushes" the pore space in the layer beneath the surface water body, while in contrast, infiltrating surface water that is typically rich in suspended material tends to clog the pore space. Additionally, clogging of the pore space within the layer beneath the surface water body could also lead to development of unsaturated flow conditions below the water body, which in turns reduce hydraulic conductivity of the aquifer and connectivity of the water body with groundwater.

During the calibration process the initially assigned surface water to groundwater seepage transfer rate values were adjusted to assist with matching the observed groundwater levels particularly during the Hawthorne Canal pumping test. A degree of connectivity between the Alluvial aquifer and the canal had a strong influence on ability of the model to replicate the observed groundwater drawdowns in the alluvial wells/VWPs during pumping, as well as the initial (pre-pumping) groundwater levels observed in the alluvial and Hawkesbury Sandstone monitoring wells.

The transfer values were varied during the model calibration until a satisfactory fit to pre-pumping groundwater levels and drawdowns were achieved. Overall, the model calibration suggested a medium to low connectivity between the alluvium aquifer and the canal. A strong connection between the aquifer and canal resulted in initial groundwater levels in the alluvium aquifer being significantly lower than observed, as well as in significantly lower predicted groundwater drawdowns during the pumping. This is in line with the water pressures initially observed in the VWPs, which showed low responses to tidal changes in the canal, suggesting a subdued connection to this waterbody.

The transfer rate values adopted at the end of Hawthorne Canal model calibration were then upscaled to the other water bodies within the regional model. These values were further adjusted based on groundwater levels observed in the monitoring wells adjacent to the creeks and canals.

The finally adopted transfer rate (leakage coefficient) associated with the surface water features Cauchy boundary conditions are summarised in Table 6-5.

Table 6-5: Transfer Rates (Leakage Coefficient) Values Adopted in the FD Model

Surface Water Feature	Surface Water to Groundwater Seepage (1/day)	Groundwater Discharge into Surface Water Bodies (1/day)
Alexandra Canal	6.0E-05	3.0E-03
Hawthorne Canal and tributary – concrete lined section	5.0E-05	2.0E-02
Hawthorne Canal – unlined section	1.0E-04	3.0E-02
Iron Cove Creek – concrete lined section	6.0E-06	1.0E-02
Iron Cove Creek – unlined section	2.0E-04	2.0E-02
Iron Cove Creek tributary	2.0E-04	2.0E-05
Whites Creek over alluvium	5.0E-04	6.0E-02
Whites Creek over residual soils	2.0E-04	2.0E-02
Johnstons Creek over alluvium	5.0E-04	6.0E-02
Johnstons Creek over residual soils	2.0E-04	2.0E-02

Transfer rates for groundwater discharge/recharge at the eastern edge of the model domain were adjusted to calibrate groundwater levels within this part of the model, i.e., groundwater levels in the Botany Sands and undifferentiated alluvium aquifers. The values adopted at the end of calibration were 3.0 x 10^{-4} 1/day to 5.0 x 10^{-3} 1/day.

The initial recharge values used in the model were adjusted during the calibration process. Final recharge rates that were adopted at the end of calibration are summarised in Table 6-6 and distribution across the model domain is shown in Figure 6-5. These recharge rates reflect the net rates that reach the groundwater table and as such, indirectly account for any losses due to evapotranspiration processes. This should be taken into consideration when these values are compared with the typical recharge rates published for the greater Sydney area, as discussed in the Hydrogeological Design Report.

Overall, higher recharge rates were adopted for the parks, open space areas and Botany Sands, while lower recharge were adopted for the urbanised areas and for the Camdenville landfill where an engineered cap has been placed over the waste.

Table 6-6: Net Recharge Values Adopted in the FD Model

Hydrostratigraphic Unit	Net Recharge Rates Adopted	
	mm/year	% of annual rainfall
Landfills	4 to 22.0	0.4 to 2.4
Residual clays between Sydney Park Pits and around	5 to 44	0.5 to 4.8
Quaternary sediments (Alluvium Sediments and Botany Sands)	5.5 to 51	0.6 to 5.5
Ashfield Shale and Hawkesbury Sandstone, urban – general	4 to 7	0.4 to 0.8
Ashfield Shale and Hawkesbury Sandstone, urban – parks and open space areas	13 to 43	1.4 to 4.7

The groundwater table across the model domain is predominantly within the Ashfield Shale and the net recharge rate adopted for this model unit would significantly contribute to the overall model water balance and calibrated groundwater levels. An average net recharge rate of about 0.7% of the annual rainfall was adopted for this area after calibration of the model, with net recharge rates up to about 5% adopted for the parks and open space areas.

This is comparable to the 1% of rainfall adopted for the exposed Ashfield Shale and residual soil in the M4-M5 Link model developed by Hydrosimulation (Hydrosimulation, 2017) and the 0.5% to 1.5 % of rainfall adopted for the New M5 numerical model (WestConnex Stage 2 M5, 2017) within its northern extent.

It should be noted that both, the Hydrosimulation and the New M5 model values are representative of direct rainfall recharge, which excludes evapotranspiration losses, i.e., they would be higher than the net overall recharge rate applied in the FD Model where groundwater tables are above the maximum depths of evaporation adopted in these models.

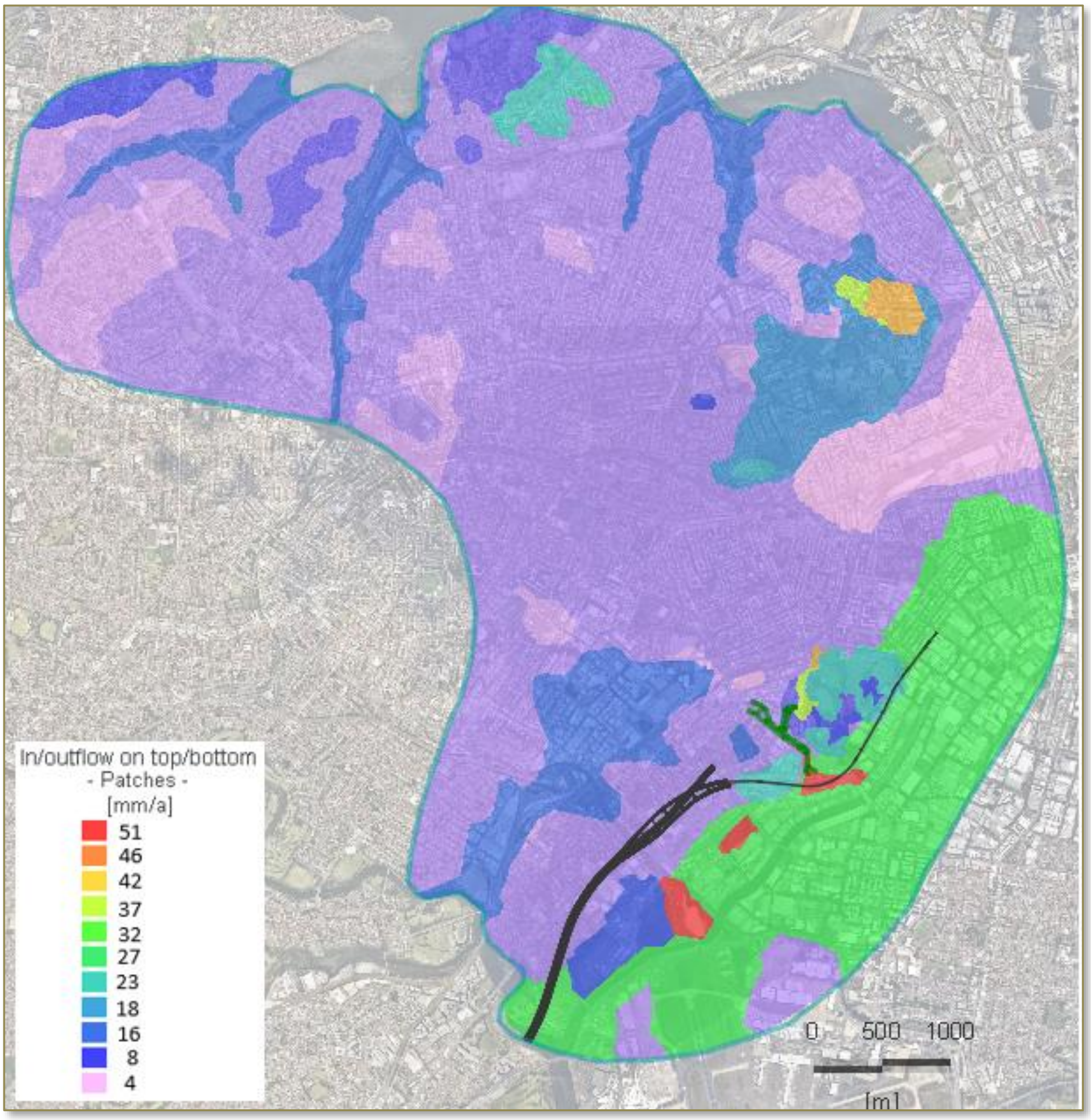


Figure 6-5: Recharge Distribution across Model Domain

7. PREDICTIVE SIMULATION SETTING

7.1 General

The Project construction and operation predictive simulation was undertaken in transient mode to assess:

- Potential groundwater inflows into the mainline tunnels, ventilation tunnels and ramps at tunnel opening and how they are likely to change during tunnel operation until steady state conditions are achieved
- timing for the near steady-state inflows to establish
- how the groundwater levels are likely to change until steady state conditions are achieved
- how groundwater drawdowns are likely to develop
- the potential for salt water migration using particle tracking.

In addition to features included in the model calibration (surface water features and extraction at Alexandria landfill,), the predictive scenario also included simulation of:

- the mainline carriageway tunnels (M110 and M120), Wattle Street Ramps (M160 and M170) and SPI Ramps (M180 and M190);
- Sump Rooms at Haberfield (SPR27010) and SPI (SPR27120);
- ventilation tunnel and associated cross passages (MDS1 CH 250 to CH 766, MDS2, MDS3, MDS4, MDS5, MDS6, and MDS 11);
- temporary access tunnels (MDS1 CH0 to CH250 and MDS9);
- M4 East and New M5 motorways; and
- hydraulic barrier (cut-off wall) that has been constructed at the eastern boundary of the Alexandria Landfill.

The representation of the cut-off wall in the model was based on the New M5 design drawings (Drawing Reference Nos. M5N-GOL-DWG-900-116-EV-0011 to 0017, provided in Info Doc 232). A hydraulic conductivity of 1 x 10^{-9} m/s was adopted for the model elements representing the cut-off wall.

7.2 Predictive Simulation Approach

In the numerical model, the tunnels were simulated as fully drained structures. Based on the previous DCD and SSD modelling results, higher permeability rock zones such as faults and dykes were identified as potential areas where high groundwater inflow may occur and could lead to SWTC inflow exceedances. Ground improvement in the form of fissure grouting was therefore assessed as being required in these areas to achieve the SWTC inflow limits within the model at or within three years of tunnel opening. The following areas consistent with the proposed surface and in-tunnel grouting programs were included in the FD Model:

- *Surface grouting*: The surface grouting of the rock was applied in the model within the Hathorne Canal area between about CH6470 and CH6680 in M110 and between about CH6470 and CH6690 in M120. The grouted block was modelled to extent from below the base of the alluvial sediments to about 15 m below the tunnel invert level. The width of the grouted block in the model is about 100 m to 110 m as per the drawings included in the JAJV Document No M4M5-JAJV-TUN-TGE-TG17-DRG-1012, dated 11/03/2020 (Figure 7-1). This block was modelled as having a bulk hydraulic conductivity of 0.5 Lu (5 x 10-8 m/s) upon completion of the grouting program.
- *In-tunnel grouting*: The in-tunnel grouting was applied in the model at selected locations where the tunnels or ramps intersect or are likely to intersect inferred higher permeability rock zones in the model. A 10 m wide grouted envelop was assumed to be grouted around the ramps and tunnels where fissure grouting was undertaken. Hydraulic conductivity of the grouted block was modelled as having a bulk hydraulic conductivity of 0.5 Lu (5 x 10^{-8} m/s) upon completion of the grouting program. The following in-tunnel grouting was simulated in the model:

- Haberfield Dyke: About a 40 m long section around each of the Wattle Street ramps (M160 and M 170) at intersection with the Dyke.
- High permeability rock zone within the Hawthorne Canal area: About a 890 m long section in the M110 tunnel and about a 290 m long section in the M120 tunnel. These sections of tunnel were selected as they passed through the inferred higher permeability zone in the model which was derived from the pumping test calibration process.
- Luna Park Fault: About a 60 m long section within around each of the twin mainline tunnels (M110 and M120) at the intersection with inferred fault zone.
- Woolloomooloo Fault: A grouted envelope around each of the PS21 Ventilation tunnel and Cross Passage within the inferred Woolloomooloo Fault zone. Total length of grouted section in the model is about 210 m.

No fissure grouting was assumed for other tunnel sections in the baseline predictive case.

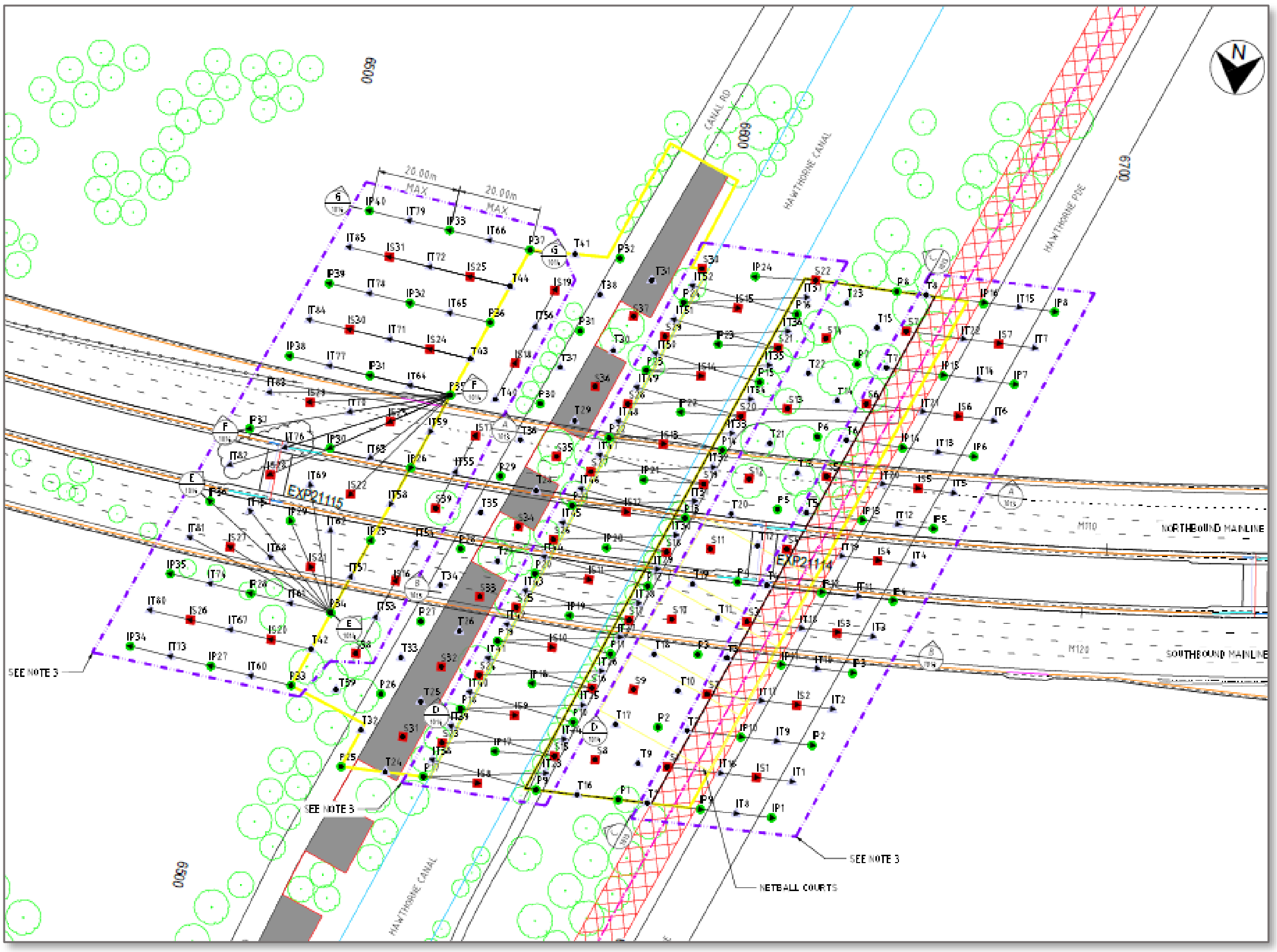


Figure 7-1: Pre Tunnel Grouting Primary, Secondary and Tertiary Hole Locations at Hawthorne Canal (taken from JAJV Document No M4M5-JAJV-TUN-TGE-TG18-DRG-1012, date 11/03/2020)

All material properties and boundary conditions remained the same as adopted at the end of FD model steady state calibration process. To encounter for impacts of the New M5 and M4 tunnels on the M4-M5 link pre-construction groundwater levels, an initial model simulation was undertaken in a transient mode that included both tunnels fully excavated. Groundwater levels obtained from this initial simulation were then used as a starting point for the M4-M5 Link Project simulation.

Excavation of all underground structures in the model followed the proposed construction sequence and excavation schedule. Excavation progression, however, was simplified with the excavations completed in a series of 15-day advances to reduce model simulation time to a practical time frame. The simulation was undertaken for 100 years after completion of the excavations.

It should be noted that the regional model was not re-calibrated to the groundwater inflows that have been observed during the current excavation period, which includes qualitative observation of groundwater inflows as part of the permit to tunnel and ground conditions mapping process. These inflow observations, however, were used to cross check model predicted excavation inflow rates and locations, i.e., where key water inflows occur and indicatively how long it takes for such flows to reduce. Quantitative measurement and recording of cumulative tunnel inflows that will be made once excavation is completed and the use of water for tunnelling excavation and support is ceased. This inflow information, together with the observed groundwater drawdown data from monitoring wells along the alignment, will then be used to re-calibrate the construction phase model as part of the next phase of regional groundwater numerical modelling work.

The following key milestone dates were adopted for the FD predictive simulation:

- Model Time 0 on 1 April 2019
- The start of excavation on 15 April 2019
- The end of excavations on 1 June 2021
- Opening of the Project on 1 March 2023
- End of simulation on 1 June 2121.

7.3 Model Formulation

The predictive model was run utilising Richards' equation for unsaturated/variable saturated media. This formulation allows for perched water development in the model, which was considered to be important for assessment of underdrainage of the alluvial aquifers.

The unsaturated/saturated option was used in order to achieve a more stable model and to improve the water budget for the predictive runs. Difference between unsaturated/saturated and phreatic model options is related only to treatment of the transient water table response in the model. Considering that the inflow into tunnels and development of the groundwater drawdown will be driven by the deeper groundwater system in the long term, these differences are not expected to significantly affect the model's inflow and drawdown predictions.

A standard iteration PCG solver (preconditioned conjugate-gradient method) was used and the error tolerance was set at 5×10^{-3} m as for the model calibration runs.

Unsaturated porosity and effective porosity for particle tracking was equal to specific yield adopted at the end of calibration process (Section 6.4.1).

8. MODEL FLOW PREDICTIONS

8.1 Groundwater Levels

The predicted long term groundwater level contours for the fractured rock aquifer at the mainline tunnel level, along with the inferred groundwater flow directions based on these levels are shown in Figure 8-1 for the model domain, in Figure 8-2 for the St Peters area and Figure 8-3 for the Hawthorn Canal area. Effects of the mainline carriageway tunnels drainage on the groundwater levels under the Hawthorn Canal alluvium are shown in a cross-section view in Figure 8-4 and in vicinity of the Sydney Park and Camdenville Park landfills in Figure 8-5. The initial water table prior to the start of excavation, as well as 50 years and 100 years after tunnel opening are also shown on Figure 8-4 and Figure 8-5, along with the groundwater contours and flow directions at 100 years after tunnel opening to indicate how the groundwater system is likely to change over time.

As shown in the figures, drainage to the mainline carriageway tunnels will be the dominant process, which will govern establishment of the long-term operation groundwater levels and regional flow patterns. While drainage of the ventilation tunnels, Wattle Street and St Peter ramp’s tunnels will also affect the long-term groundwater levels, the effect of this drainage will be localised and associated with the deeper sections of these tunnels and ramps. The model indicates that shallower sections of these ramps and ventilation tunnels are likely to be dry during the operational phase due to the dewatering caused by the deeper mainline tunnels. The exception to this would be during periods of sustained rainfall where localised, short term, transient inflows may develop. Additionally, transient groundwater inflows into shallower sections will likely be more immediate during operation than in the deeper mainline carriageway and ventilation tunnel sections due to the shorter flow path to the tunnel excavations.

The Haberfield and SPI sump rooms would continue to drain groundwater through the Operational Phase. This drainage, however, will affect a small localised area as shown in Figure 8-2 and Figure 8-3.

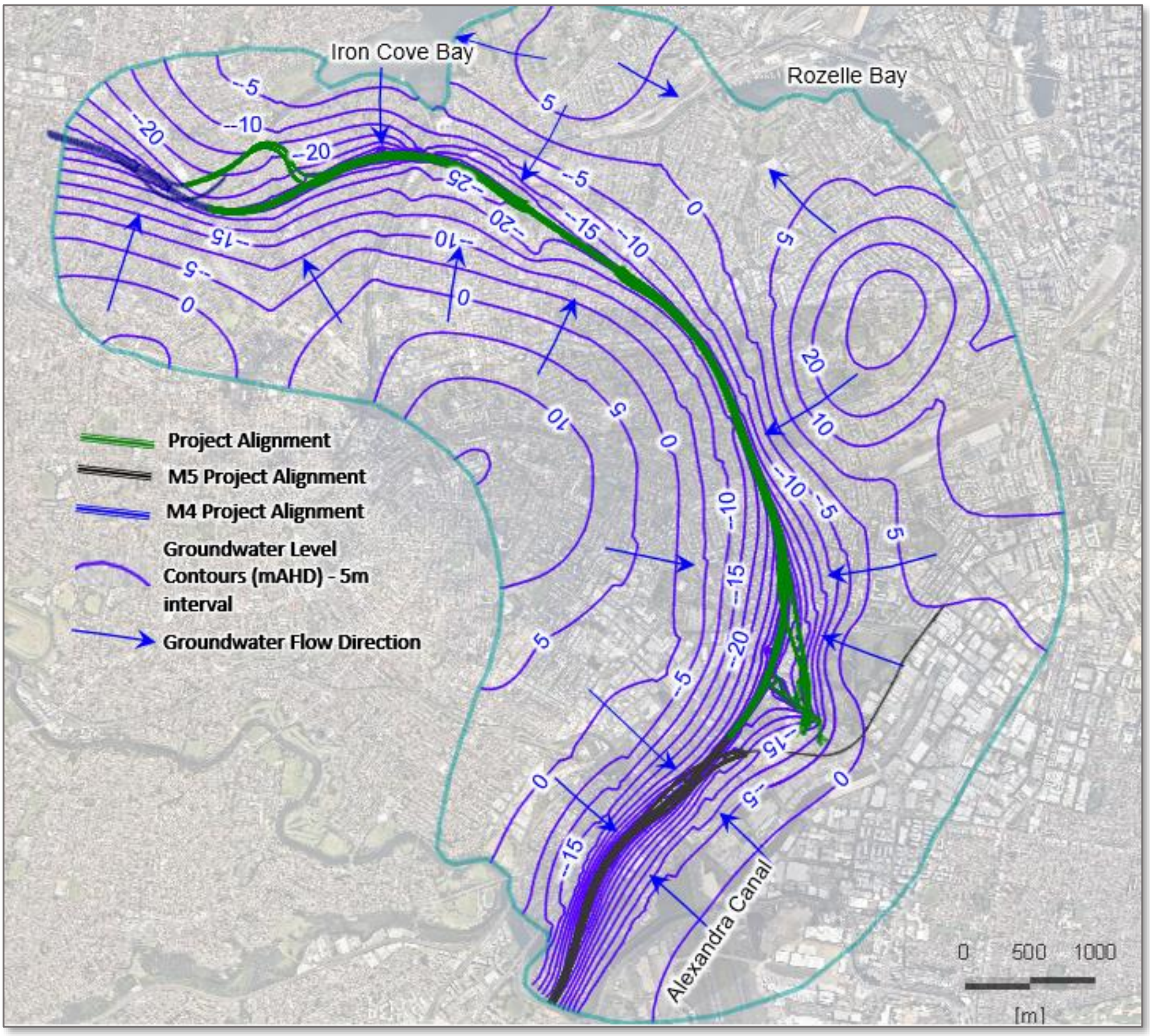


Figure 8-1: FD Model Predicted Groundwater Levels and Flow Directions within the Rock Aquifer at the Tunnel Level, Operational Phase (100 years after opening)

Drainage of the mainline carriageway tunnels is indicated to result in dewatering of Holocene alluvial sediments in vicinity of the alignment (Figure 8-4). A perched water, however, may develop within the grouted block above the mainline tunnels due to surface water leakage from the Hawthorn Canal as suggested by the modelling results.

Figure 8-5 shows that the long-term drainage of the mainline carriageway tunnels, SPI ramps and ventilation tunnels would result in the groundwater table to fall below the base of the Camdenville Park Landfill as infiltration through an engineered landfill cap is unlikely to be sufficient to maintain high groundwater levels under the landfill. Model also suggests that the leachate and groundwater levels within the Sydney Park would be less effected, which would lead to steep hydraulic head gradients to developed between the landfill waste and the tunnels at depth.

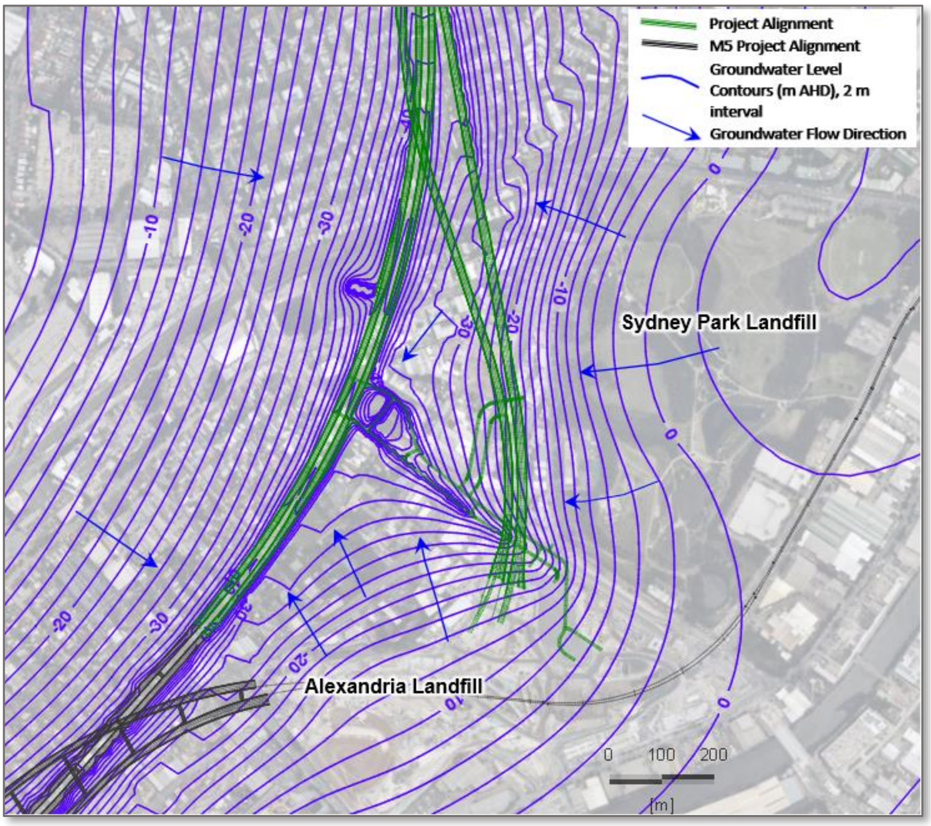


Figure 8-2: FD Model Predicted Groundwater Levels and Flow Direction within the Hawkesbury Rock Aquifer, the SPI Area, Operational Phase (100 years after opening)

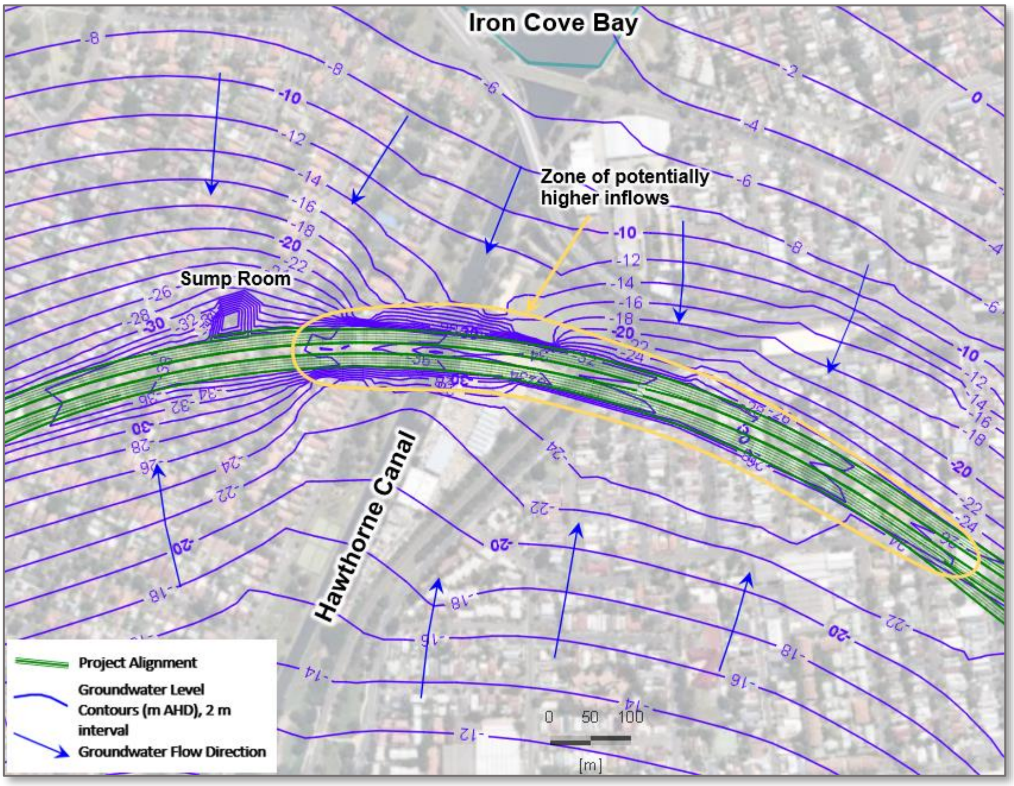


Figure 8-3: FD Model Predicted Groundwater Levels and Flow Direction within the Hawkesbury Rock Aquifer, the Hawthorn Canal Area, Operational Phase (100 years after opening)

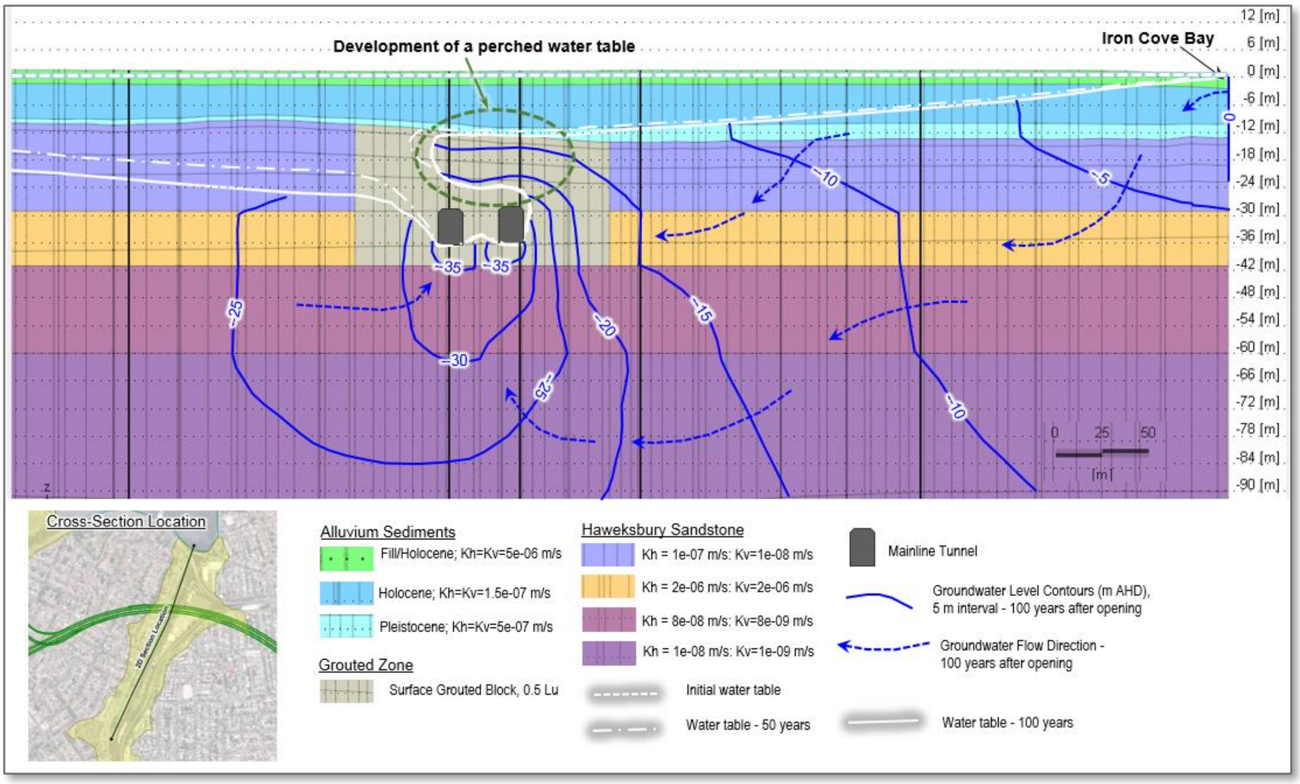


Figure 8-4: FD Model Predicted Effects of the Mainline Tunnels Dewatering on Groundwater Levels under the Hawthorn Canal Alluvium

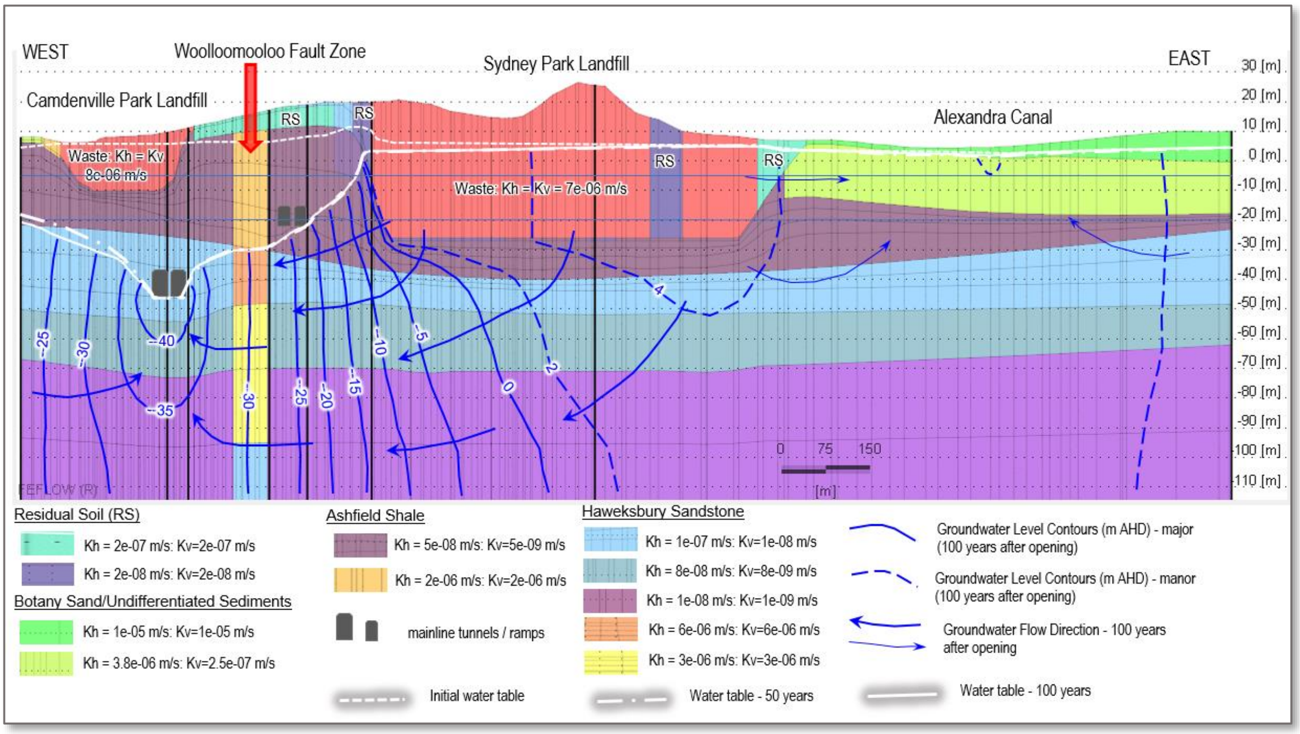


Figure 8-5: FD Model Predicted Effects of the Mainline Tunnels Dewatering on Groundwater Levels in Vicinity of Sydney Park and Camdenville Park Landfills

8.2 Groundwater Inflow

Overall estimates of the groundwater inflow into the mainline carriageway tunnels, Wattle Street ramps, SPI ramps. ventilation tunnels and sump rooms at opening, as well as Year 10, 20, 50 and 100 after excavation is completed are summarised in Table 8-1. Groundwater inflows estimated for individual tunnel design packages for the same time periods are provided in Table 8-2. Plans showing the tunnel design packages are included in Appendix BA of the main Hydrogeological Design Report (GW02 FD2). Time 0 in the model corresponds to 1 April 2019 (Section 7.2). These inflow rates are based on the hydraulic properties derived through the process of model calibration discussed in Section 6. Sensitivity of the predicted inflow rates to reasonable potential parameter/conceptual variations is discussed in Section 9.

A graph showing combined groundwater inflows into the mainline carriageway tunnels (M110 and M120) over the full model simulation time is included in Figure 8-6. Groundwater inflows between time 0 and time 2.2 years are construction inflows. It should be noted that short term burst inflows may be experienced in the tunnels when they first encounter a water bearing feature, but in the majority of cases where these features are not hydraulically connected to a significant water source, these inflows are expected to dissipate within a few days of excavation.

The total long-term steady state groundwater inflow into the mainline carriageway tunnels M110 and M120 based on the baseline predictive simulation are in the range of about 2.5 L/s to 2.7 L/s (220 m^3/day to 235 m^3/day) for each carriageway. The total tunnel opening inflow rates are higher with the groundwater inflows predicted to be about 8.6 L/s (745 m^3/day) in M110 and 8.0 L/s (690 m^3/day) in M120.

Based on the model results, it is suggested that after an initial sharp decrease in the groundwater inflow rate, the rate of decrease will slow between years 10 and 20 after excavation completed and will continue to gradually decline up to about 85 to 90 years when the near steady state inflows will establish (Figure 8-6). A similar time frame for inflow stabilisation is also indicated for the SPI Ramps (M180 and M190) and PS21, while slowing of the groundwater inflow rates into the Wattle Street Ramps is indicated to occur around 5 years after excavation completed. The near steady state inflows into the SPI Ramps is predicted to occur about 75 to 80 years after excavation is completed and for the Wattle Street Ramps and PS21 60 to 70 years.

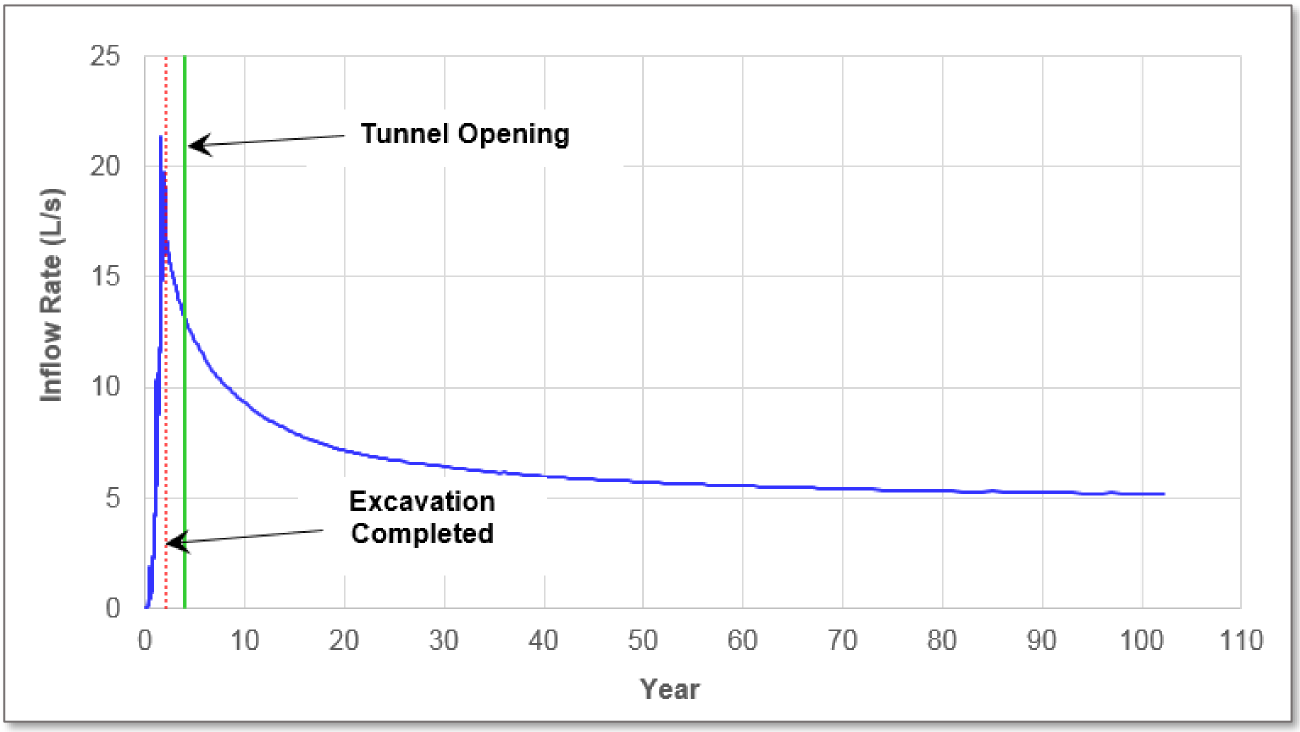


Figure 8-6: Model Predicted Combined Groundwater Inflow into Mainline Carriageway Tunnels (M110 and M120)

Table 8-1: Summary of FD Model Predicted Groundwater Inflows

Project Element (*length of element in the model*)	Groundwater Inflow									
	L/s	m^3/day	L/s	m^3/day	L/s	m^3/day	L/s	m^3/day	L/s	m^3/day
Years after excavation completed	**Opening (2 years)**		**10**		**20**		**50**		**100**	
Mainline Carriageway										
Southbound Mainline Carriageway (M120) *(7,480m)*	6.3	548	4.1	358	3.4	296	2.9	251	2.7	231
Northbound Mainline Carriageway (M110) *(7,450m)*	6.9	599	4.4	377	3.5	299	2.8	240	2.5	215
St Peter Interchange										
Exit Ramp (M190) *(1,020m)*	1.2	108	0.8	71	0.6	50	0.4	38	0.4	33
Entry Ramp (M180) *(1,160m)*	0.4	36	0.2	16	0.08	7	0.04	3	0.03	2
Wattle Street Ramp										
Entry Ramp (M170) *(565 m)*	0.3	24	0.2	16	0.2	14	0.1	12	0.1	12
Entry Ramp (M160) *(560 m)*	0.10	9	0.06	5	0.05	4	0.04	3	0.03	3
Ventilation Tunnels and Cross Passages										
Package PS21 *(1,350 m)*	1.0	89	0.8	66	0.6	52	0.5	47	0.5	46
Package PS08 *(110 m)*	0.04	3	0.03	2	0.02	2	0.02	2	0.02	2
Sump Rooms										
Haberfield end	0.07	6	0.06	5	0.06	5	0.06	5	0.06	5
SPI end	0.10	8	0.09	8	0.07	6	0.05	4	0.05	4

Table 8-2: FD Model Predicted Groundwater Inflows into Individual Design packages, Mainline Carriageway Tunnels

Package ID	Mainline Carriageway	Length of Drained Section (m)	Groundwater Inflow L/s	m³/day	L/s	m³/day	L/s	m³/day	L/s	m³/day	L/s	m₃/day
Years after excavation completed			Opening (2 years)		10		20		50		100	
PS01_WST	M110	490	0.28	24	0.19	16	0.17	14	0.15	13	0.14	12
	M120	484	0.24	21	0.17	15	0.16	14	0.15	13	0.15	13
PS01_ROE	M110	690	0.48	41	0.30	26	0.25	21	0.20	17	0.18	15
	M120	695	0.45	39	0.27	24	0.23	20	0.19	17	0.18	15
PS02	M110	957	1.12	96	0.64	55	0.54	47	0.41	36	0.34	29
	M120	960	1.06	92	0.63	54	0.55	47	0.44	38	0.37	32
PS03	M110	736	1.02	88	0.60	52	0.50	43	0.44	38	0.45	39
	M120	746	0.91	79	0.61	53	0.55	48	0.53	45	0.53	46
PS09	M110	710	0.74	64	0.52	45	0.35	31	0.28	24	0.26	22
	M120	723	0.81	70	0.53	46	0.43	37	0.39	33	0.37	32
PS10	M110	580	0.47	40	0.38	33	0.25	22	0.19	17	0.17	15
	M120	580	0.42	36	0.31	27	0.22	19	0.18	16	0.17	15
PS14	M110	229	0.26	23	0.16	14	0.14	12	0.11	9	0.09	8
	M120	231	0.24	21	0.15	13	0.13	11	0.11	10	0.10	9
PS16	M110	868	0.69	59	0.47	41	0.41	36	0.35	31	0.31	27
	M120	868	0.55	48	0.37	32	0.33	29	0.30	26	0.27	23
PS19	M110	713	0.44	38	0.20	18	0.17	15	0.14	12	0.13	11

Package ID	Mainline Carriageway	Length of Drained Section (m)	Groundwater Inflow									
			L/s	m³/day	L/s	m³/day	L/s	m³/day	L/s	m³/day	L/s	m₃/day
Years after excavation completed			Opening (2 years)		10		20		50		100	
	M120	657	0.40	35	0.21	18	0.17	15	0.15	13	0.14	12
PS20 (Rozelle section)	M110	604	0.64	56	0.31	26	0.26	22	0.22	19	0.21	18
	M120	662	0.38	33	0.20	17	0.16	14	0.14	12	0.14	12
PS20 (SPI section)	M110	853	0.80	69	0.59	51	0.42	37	0.28	24	0.21	18
	M120	864	0.88	76	0.68	59	0.49	42	0.33	28	0.25	22

The SWTC groundwater inflow requirement for drained tunnels is 1 L/s/km. Based on the total length of the St. Peters Interchange and Wattle Street ramp tunnels, and ventilation/exhaust tunnels, the FD model predicted inflow rates (Table 8-1) are consistent with the SWTC criteria.

Groundwater inflow rates (L/s)[5] and the total groundwater inflows per kilometre of the tunnel (L/s/km) estimated by the model at tunnel opening, 3 years after opening and long term steady state inflows are shown in Figure 8-7, Figure 8-8 and Figure 8-9 for the northbound mainline tunnel (M110) and in Figure 8-10, Figure 8-11 and Figure 8-12 for the southbound mainline tunnel (M120).

The model indicates that two sections of tunnel may exceed inflow criteria at the time of opening. These inflows are related to zones of increased rock permeability associated with the Luna Park Fault and Hawthorne Canal high permeability zone. The modelling results, however, indicate that the groundwater inflows will reduce to below 1 L/s/km across the alignment about 3 years after opening and that once steady state conditions are reached the inflow rates will be significantly less than 1 L/s/km criteria.

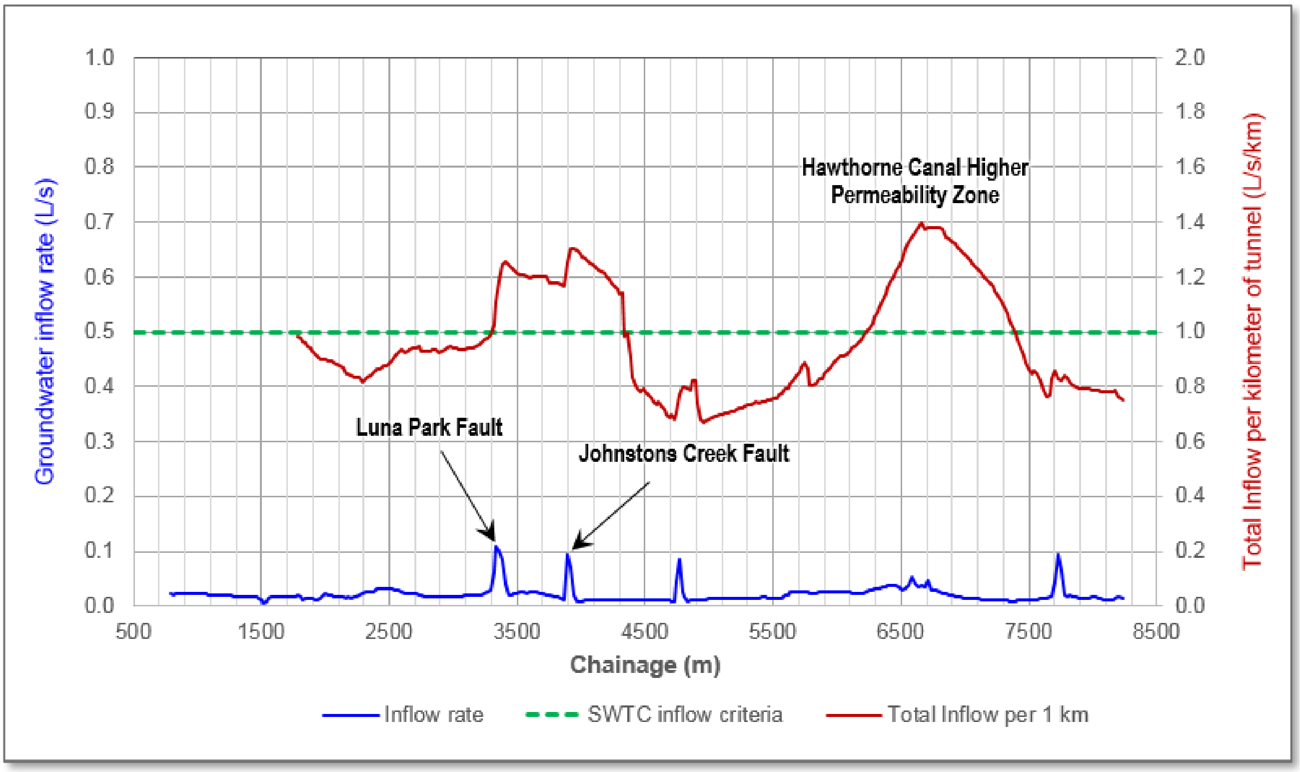


Figure 8-7: FD Model Predicted Inflow Rates into Northbound Main Carriageway (M110) and Total Inflows per Kilometre of Tunnel at Tunnel Opening (March 2023) Calculated from Connection with New M5 Tunnel

[5] Flow rate values are reported at model finite element nodes and are spaced at an approximate distance of 20 m along the tunnel alignment.

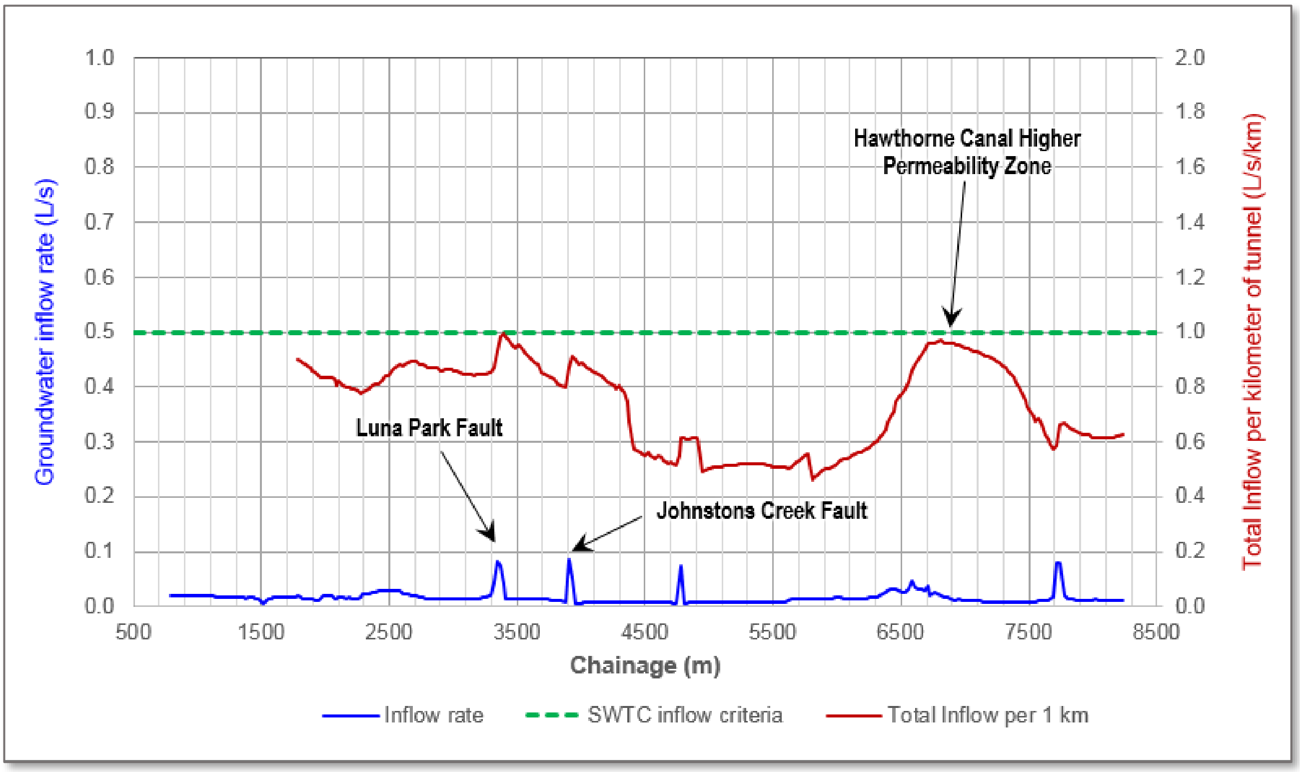


Figure 8-8: FD Model Predicted Inflow Rates into Northbound Main Carriageway (M110) and Total Inflows per Kilometre of Tunnel 3 years after Tunnel Opening Calculated from Connection with New M5 Tunnel

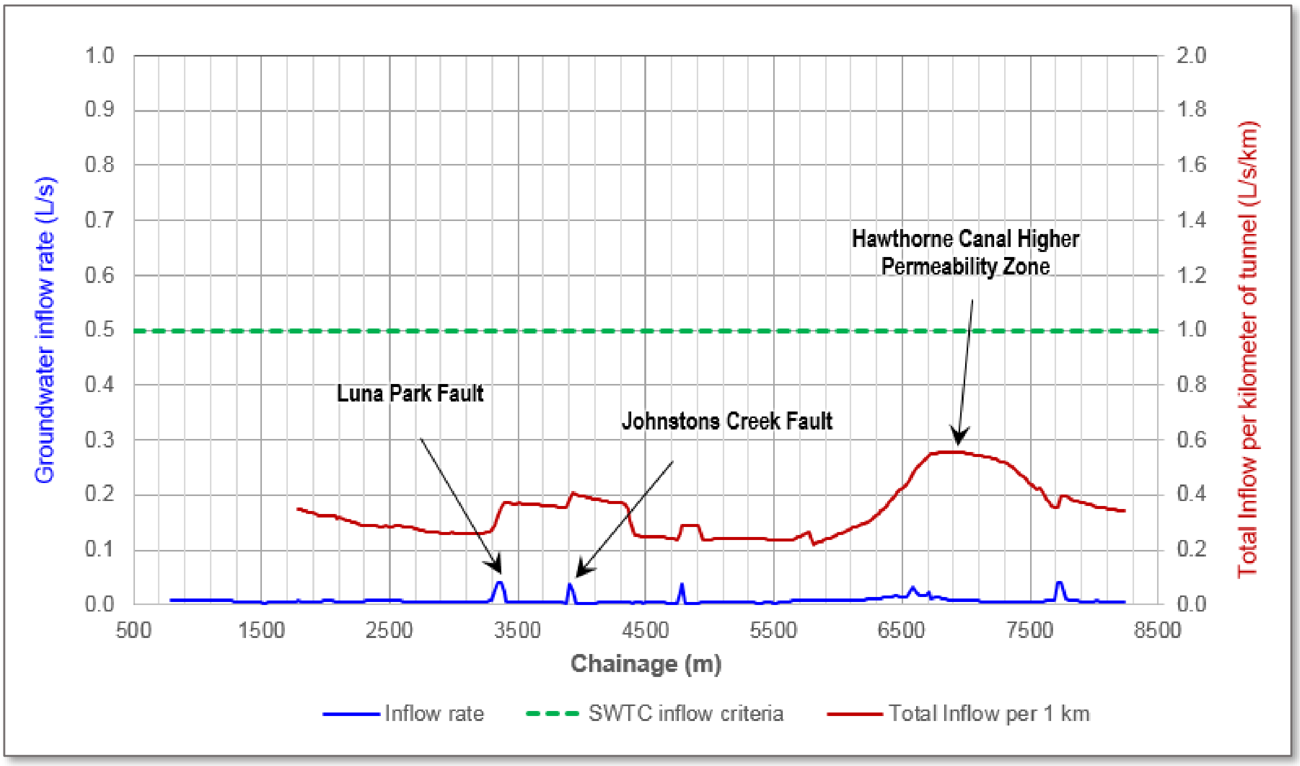


Figure 8-9: FD Model Predicted Steady State Inflow Rates into Northbound Main Carriageway (M110) and Total Inflows per Kilometre of Tunnel Calculated from Connection with New M5 Tunnel

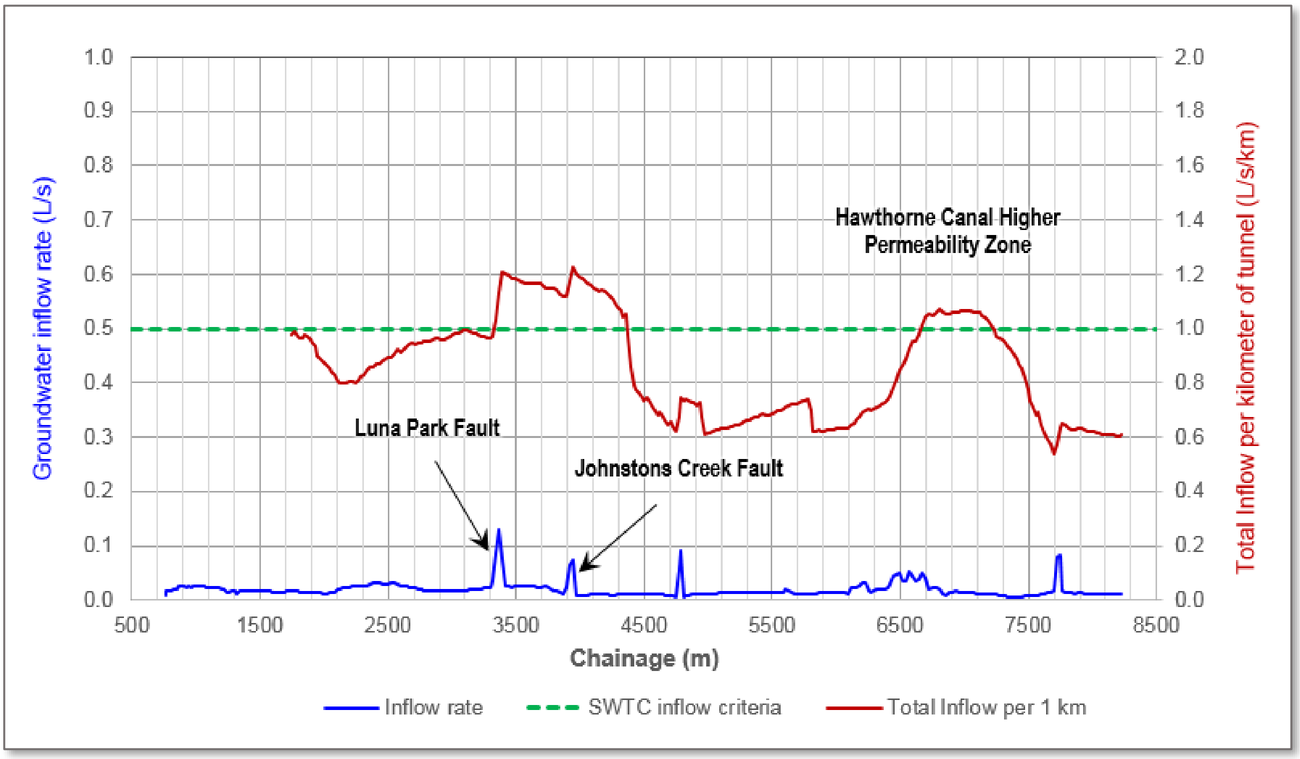


Figure 8-10: Model Predicted Inflow Rates into Southbound Main Carriageway (M120) and Total Inflows per Kilometre of Tunnel at Tunnel Opening Calculated from Connection with New M5 Tunnel

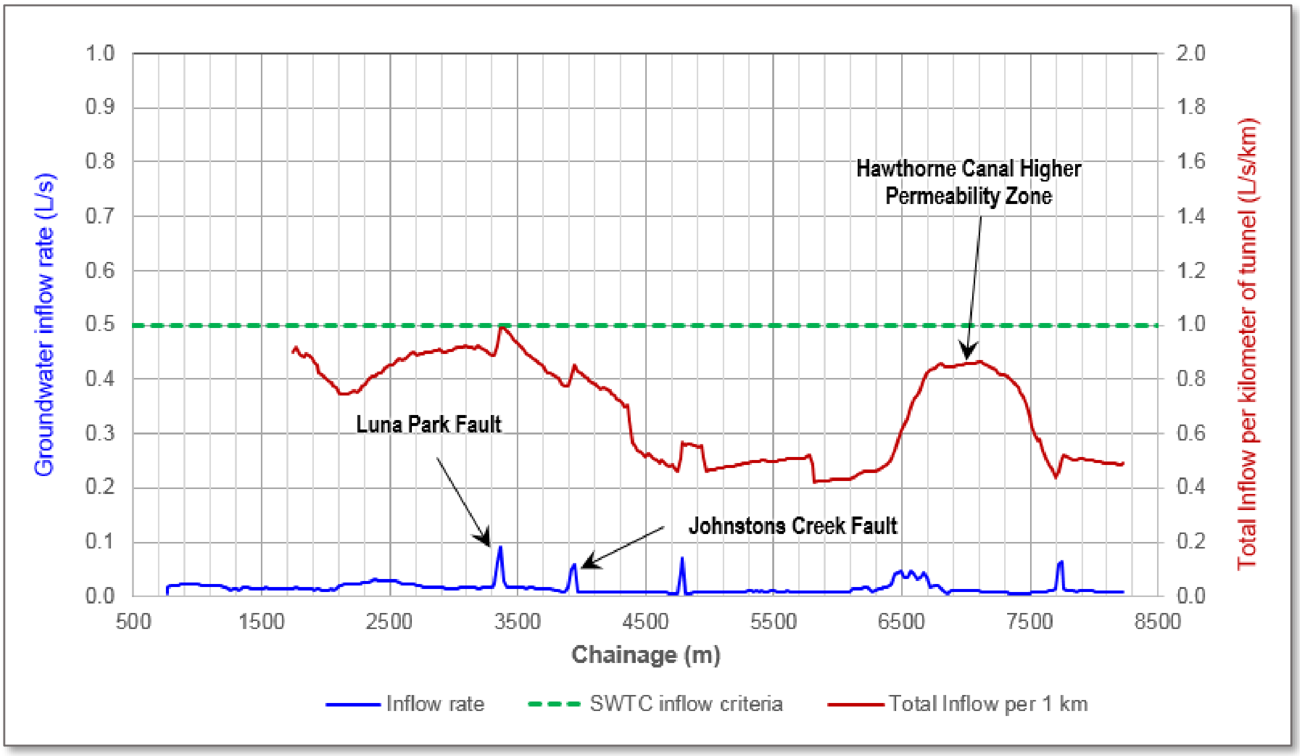


Figure 8-11: FD Model Predicted Inflow Rates into Southbound Main Carriageway (M120) and Total Inflows per Kilometre of Tunnel 3 years after Tunnel Opening Calculated from Connection with New M5 Tunnel

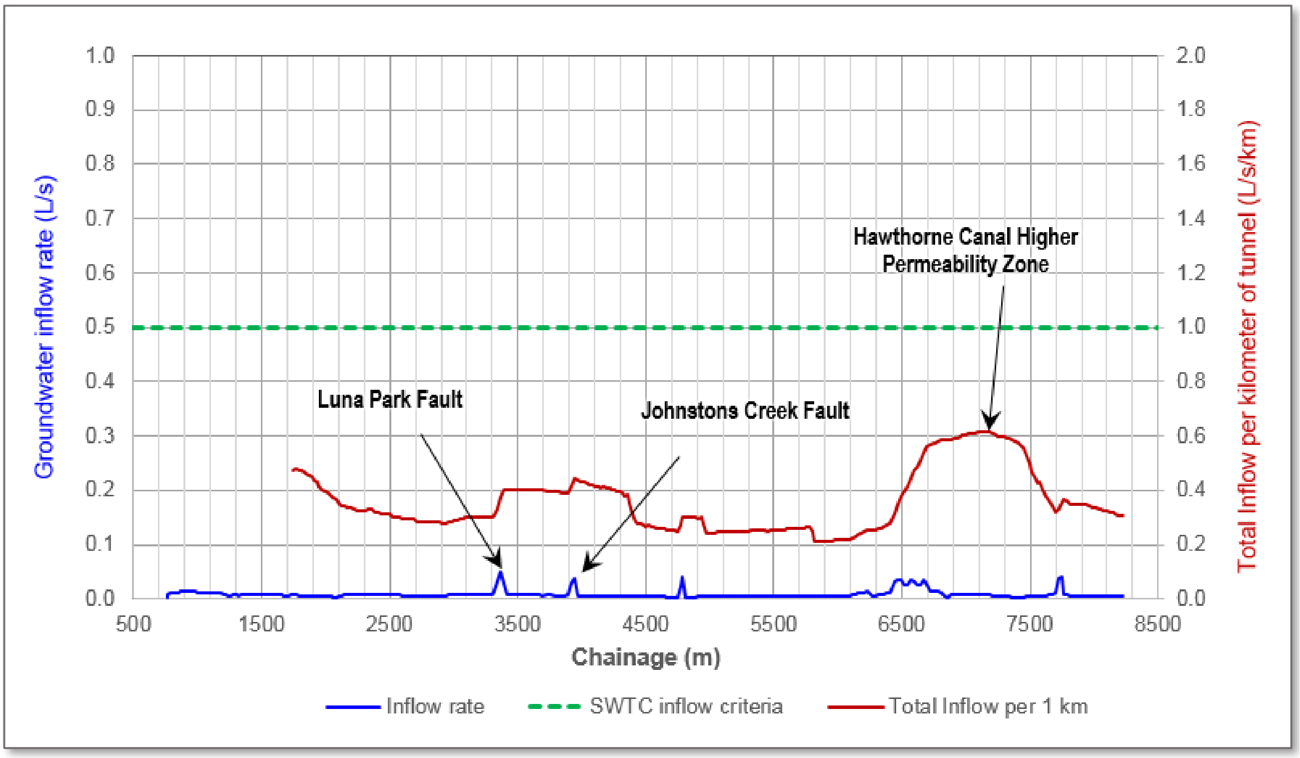


Figure 8-12: FD Model Predicted Steady State Inflow Rates into Southbound Main Carriageway (M120) and Total Inflows per Kilometre of Tunnel Calculated from Connection with New M5 Tunnel

8.3 Groundwater Drawdowns

The predicted long-term groundwater drawdown contours for the Hawkesbury Sandstone and Ashfield Shale fractured rock aquifers due to inflows into the tunnels are shown in Figure 8-13. The predicted long-term groundwater drawdown contours within the SPI area are shown in Figure 8-14 and at the base of alluvial sediments within Iron Cove Creek and Hawthorne Canal in Figure 8-15.

Groundwater drawdowns within the fractured rock aquifer for the fully drained tunnel conditions are indicated to range up to about 55 m adjacent to the deepest section of the mainline carriageway tunnels (West of the Sydney Park, PS09 and PS10, and about 58 m at the Sump Room SPR27120 (Figure 8-14). The maximum groundwater drawdowns at the mainline tunnel level are controlled by the mainline and ventilation tunnels invert levels and are indicated to range from 35 m to 55 m (i.e., maximum tunnel invert depth from 35 m to 55 m below the predicted initial water table). The effects of deep Sump Rooms will be localised around these structures. Laterally the groundwater drawdowns are suggested to extent more than 2 km away from the tunnel alignment. The model results also suggest that where highly permeable structures are present such as faults zones, they would locally extend the lateral extent of the drawdowns away from the tunnel alignment.

Overall, groundwater drawdowns resulting from drainage of the deeper mainline tunnels and deeper sections of the ventilation tunnels, overshadow the localised drawdowns developed around the shallower sections of the ramp tunnels. This is to be expected for fully drained tunnels.

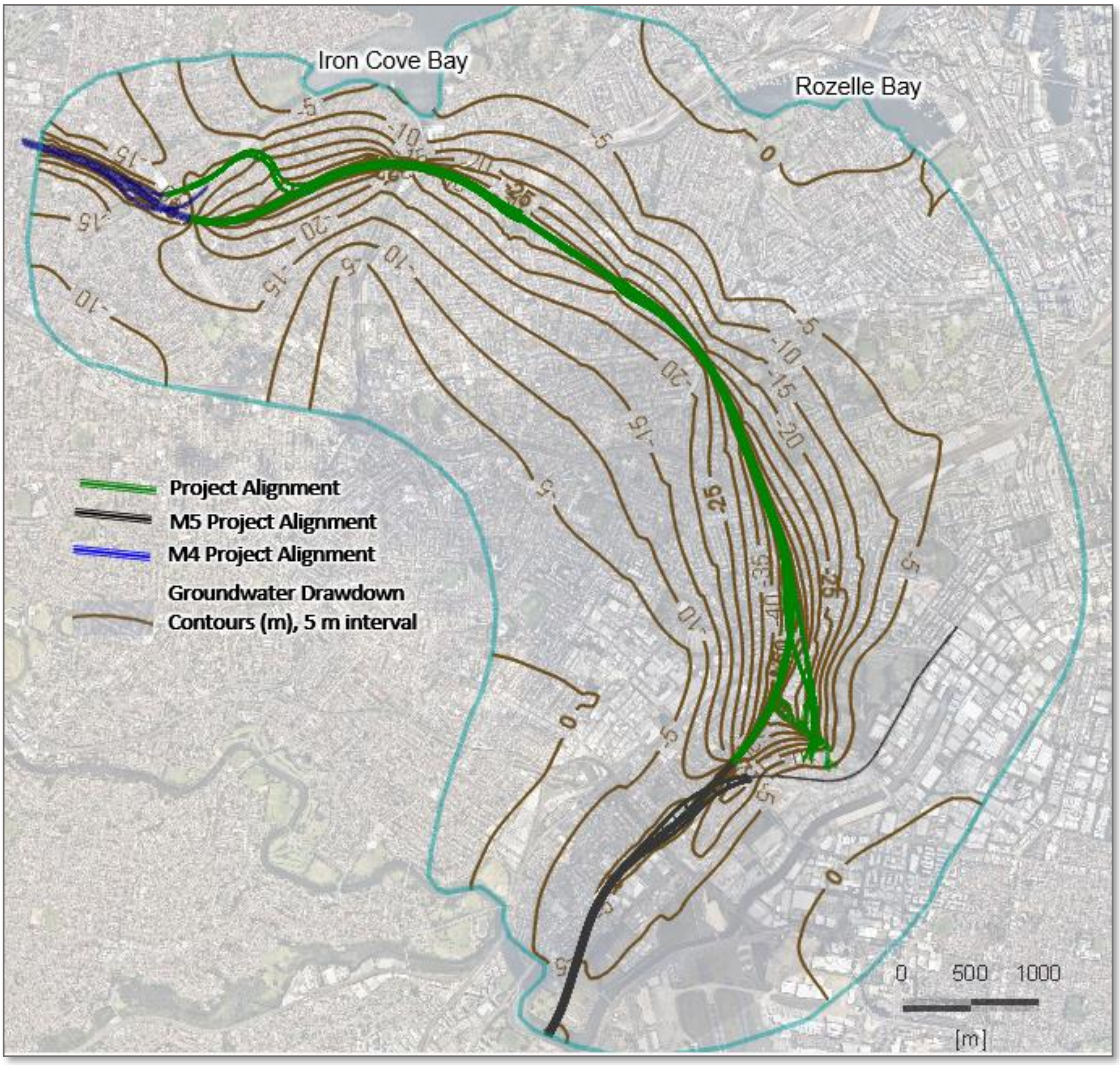


Figure 8-13: FD Model Predicted Groundwater Drawdowns at the Mainline Tunnel Level, Operational Phase (100 years after opening)

Groundwater drawdowns at the base of the Alluvial sediments associated with the Hawthorne Canal and Iron Cove Creek 100 years after M4-M5 Link opening are shown in Figure 8-15. These drawdown ranges suggest that the sediments would be under-drained over the majority of their extent, with the exception of relatively short distances from the bays and the upper sections of these palaeochannels. However as illustrated in Figure 8-4, the modelling also predicts that there is a potential for a perched water table to locally develop in the sediments due to surface water recharge from the canal.

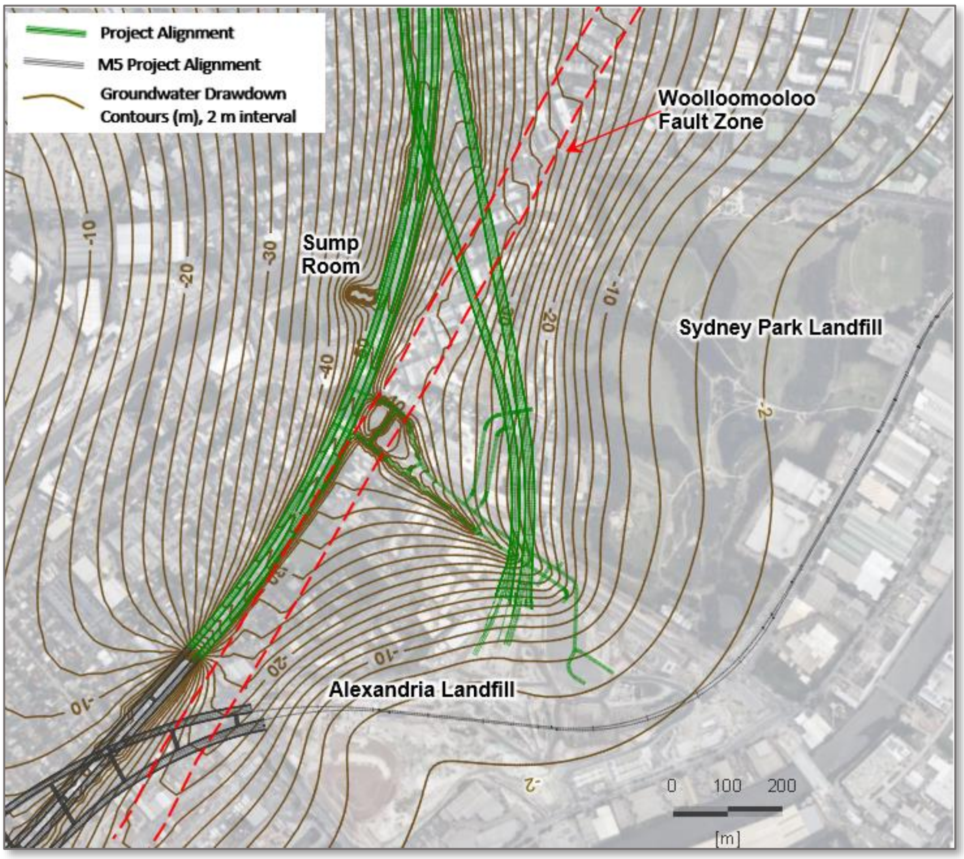


Figure 8-14: FD Model Predicted Groundwater Drawdowns at the Mainline Tunnel Level within the SPI Area, Operational Phase

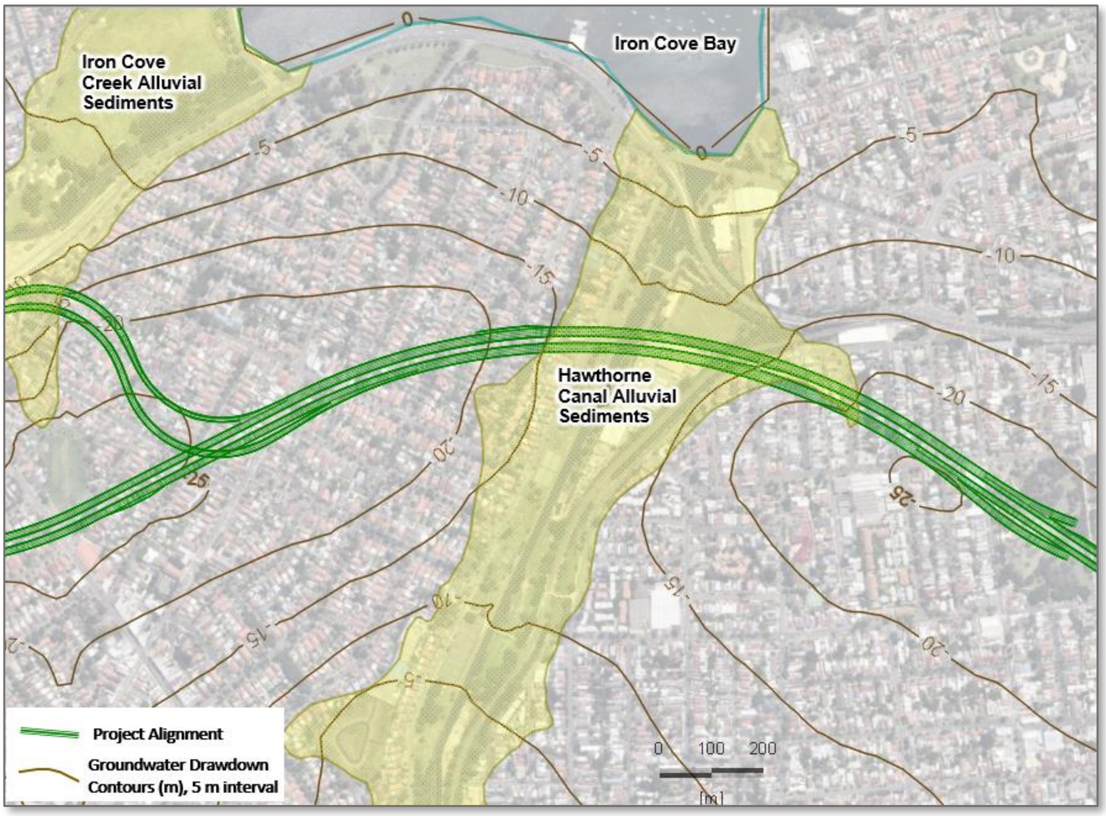


Figure 8-15:FD Model Predicted Groundwater Drawdowns at the Base of Holocene Sediments, Operational Phase

8.4 Salt Water Migration

The FD model indicates salt water migration from Iron Cove, Iron Cove Creek and Hawthorne Canal would reach the northern portion of the mainline tunnels (approximately between CH6450 and CH7650) over their design life as illustrated in Figure 8-16. The salt water would travel faster along the Hawthorne Canal higher permeable zone and may reach the tunnel sections within this zone (approximately between CH6600 and CH6800) much earlier than other tunnels sections. The modelling results also suggest that water reaching the M120 carriageway approximately between CH6550 and CH6600, and between CH7100 and CH7300 is likely to be a mixture of water originating from Iron Cove and surface water from Hawthorne Canal and Iron Cove Creek respectively.

Saline water migrating from Iron Cove is indicted to reach the Wattle Street Exit Ramp M170 approximately between CH850 and CH1050 also during its design life as illustrated Figure 8-17.

The modelling also predicts that no saline water inflow is likely to occur into Exit Ramp M160.

Figure 8-16: Predicted Salt Water Particle Pathways for Mainline Carriageway Tunnels at 100 Years after Excavation, Operational Phase

Figure 8-17: Predicted Salt Water Particle Pathways for Wattle Street Ramps at 100 Years after Excavation, Operational Phase

The model also indicates that salt water migration from Rozelle Bay towards the Project is unlikely to occur due to M4-M5 Link Project development. Simulation of potential salt water migration due to the proposed Rozelle Interchange Project development was not considered as it is outside the M4-M5 Link Project scope.

8.5 Surface Water Features Water Fluxes Changes

A number of natural water features affecting the groundwater flow system were included into the model. Changes in the water exchange rates (recharge/discharge) to and from these features due to the Project development are summarised in Table 8-3.

While it is not possible to quantify the proportion of stream flow that groundwater contributes to due to the lack of gauging data, it is likely that the majority of the stream flow in each feature is derived from stormwater runoff or in the case of Hawthorne Canal and Iron Cove Creek from tidal inflows from Iron Cove. The overall changes in stream flows are, therefore, likely to be small.

Table 8-3: Water Fluxes To and Out of Main Surface Water Features

Surface Water Feature	Pre-Tunnel Construction				Long Term Operational			
	Groundwater discharge		Seepage to Groundwater		Groundwater discharge		Seepage to Groundwater	
	L/s	m³/day	L/s	m³/day	L/s	m³/day	L/s	m³/day
Iron Cove Creek	0.40	35	0.10	9	0.00	0	0.08	7
Hawthorne Canal	0.76	66	0.00	0	0.12	10	0.37	32
Whites Creek	0.36	31	0.00	0	0.19	16	0.02	2
Johnstons Creek	0.65	56	0.00	0	0.21	18	0.03	3
Alexandra Canal	4.50	389	0.02	2	2.45	212	0.10	9
Airport Lake	0.38	33	0.38	33	0.34	29	0.00	0
Sydenham drainage lines	0.80	69	0.01	1	0.35	30	0.34	29
Cooks River	0.04	3	0.00	0	0.02	2	1.07	92
Iron Cove Bay	0.30	26	0.00	0	0.11	10	0.45	39
Rozelle Bay	0.33	29	0.00	0	0.31	27	0.00	0

9. MODEL UNCERTAINITY AND SENSITIVITY ANALYSIS

It is important to note that all groundwater numerical models are an approximation of a natural groundwater flow system and geological setting. All models, therefore, include a degree of uncertainty associated with their predictive ability due to the assumptions and simplifications made during their conceptualisation.

The FD model has been calibrated to transient conditions using the results of the Hawthorne Canal pumping test. Although these stresses are relatively short with respect to the duration of tunnel's construction period and operational life, the transient calibration using the results of the Hawthorne Canal pumping test, combined with the subsequent regional scale steady state calibration based on the long-term monitoring data, has significantly improved the model's predictive ability.

The current uncertainty in the model's predictions was addressed using sensitivity analysis. The sensitivity analysis was undertaken in accordance with the Australian Groundwater Modelling Guidelines (Barnett et al., 2012) to gain an understanding of the effect of reasonable parameter/conceptual variations upon model predictions.

The following model predictions were used as the basis for the sensitivity analysis:

- Combined transient groundwater inflows to the mainline carriageway tunnels M110 and M120.
- The 20 m groundwater drawdown contour at the end of the predictive simulation (i.e., 100 years post-construction). The 20 m contour was selected as a representative indicator of drawdown extent.
- Transient drawdown in observation well LSB-HC-PT-OW04. This well was selected as a representative indicator of transient drawdown progression within the Hawthorn Canal area, as it is a deep well which is screened in close proximity to the invert of the mainline tunnels in this area.

When conducting the sensitivity analysis, a simplified approach was taken whereby all of the tunnels were excavated simultaneously. This was done to improve the traceability of the suite of sensitivity analysis simulations, by using a less computationally intensive simulation approach as a substitute.

Comparisons between the instantaneous excavation approach and the progressive excavation approach for each of the above three model predictions are presented in Figure 9-1, Figure 9-2 and Figure 9-3. Differences between these two approaches were found to be generally minor. The most notable prediction discrepancy is in the early inflows into M110 and M120 mainline tunnels (Figure 9-1). Ongoing long-term operational inflows, however, match well for both model runs.

The results of this comparative analysis indicate that the instantaneous excavation approach is a suitable substitute for the purpose of analysing the sensitivity of the model predictions to key model parameter and conceptualisation changes.

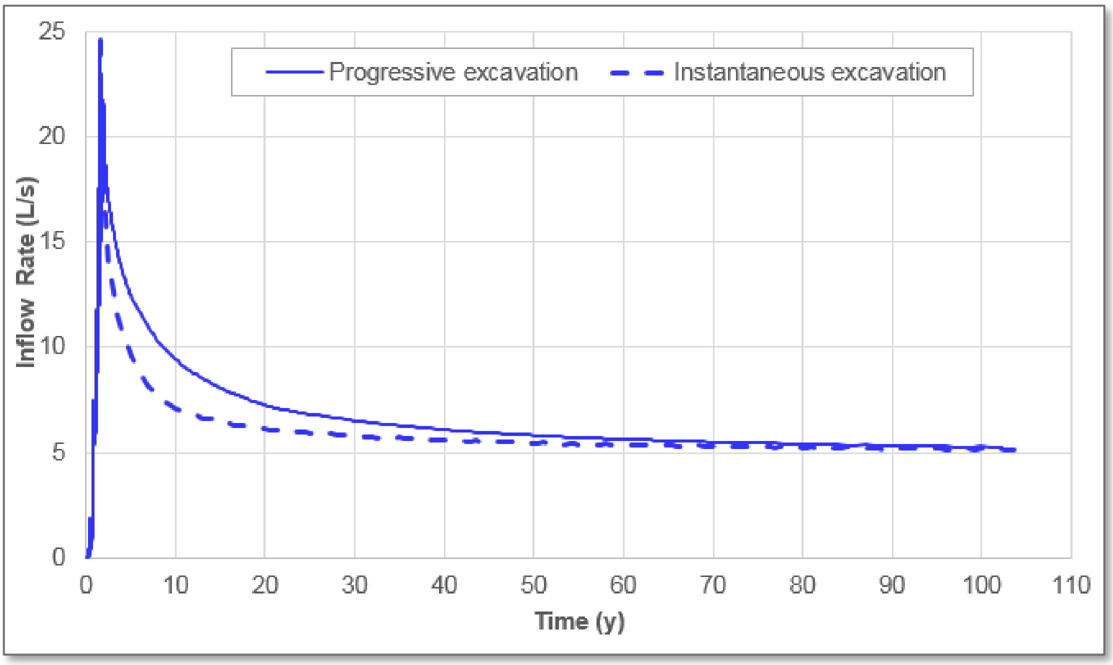


Figure 9-1: Combined Groundwater Inflow into Mainline Carriageway Tunnels (M110 and M120) for Progressive and Instantaneous Excavation, Sensitivity 0 Simulations

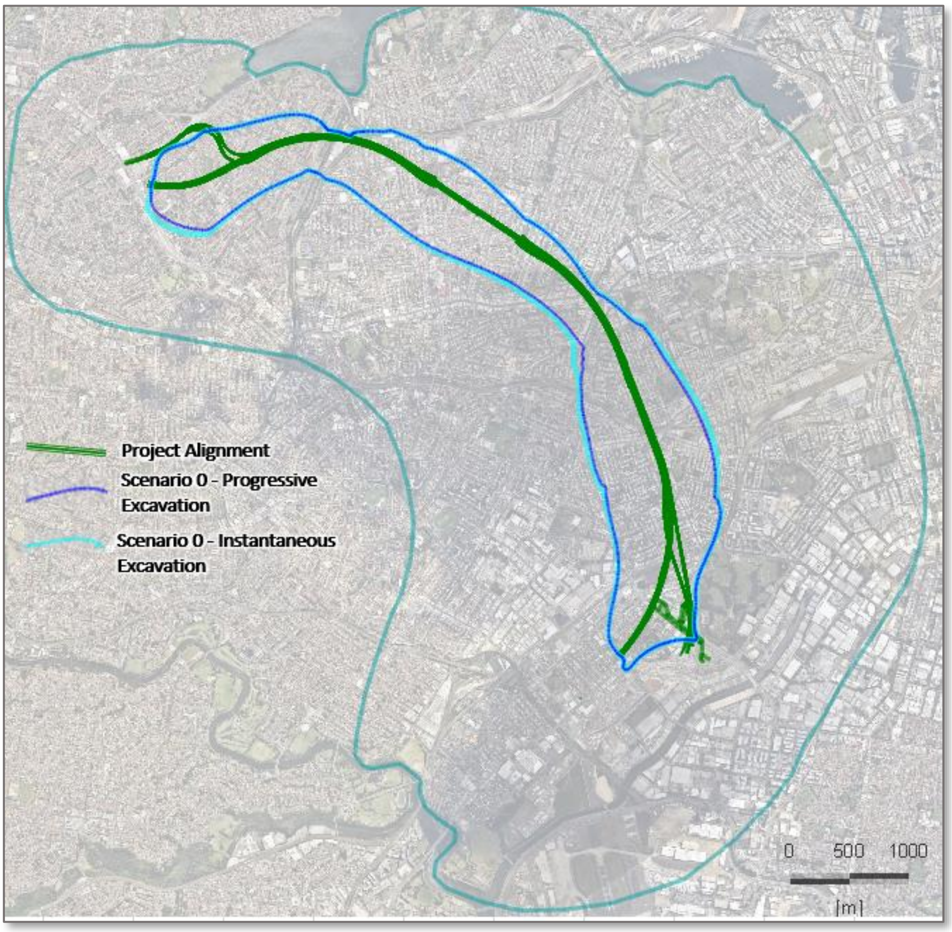


Figure 9-2: 20 m Groundwater Drawdown Contours for Progressive and Instantaneous Excavation Sensitivity 0 Simulations, Long Term Operational Phase

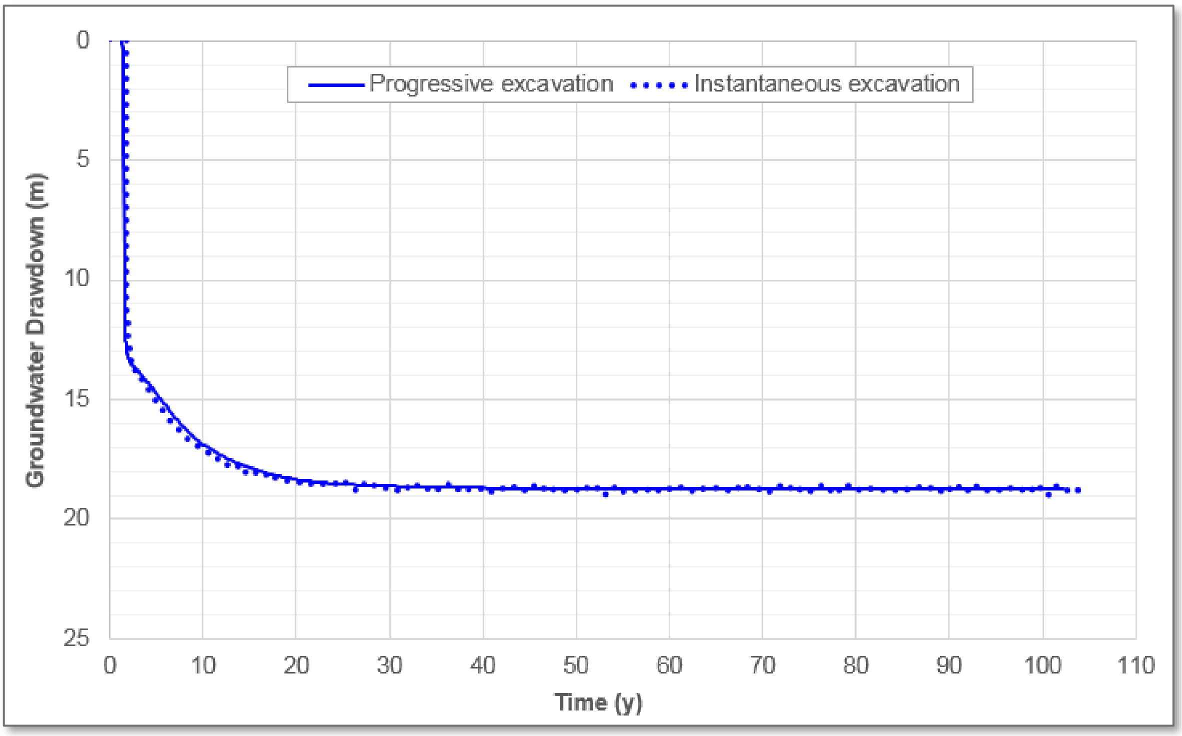


Figure 9-3: Model Predicted Groundwater Drawdowns at LSB-HC-PTOW04 Well for Progressive and Instantaneous Excavation, Sensitivity 0 Simulations.

On the basis of the comparative analysis results, the FD model was then tested for sensitivity of results to following key parameters and boundary conditions:

- the bulk hydraulic conductivity of the Hawkesbury Sandstone (i.e., the rock mass between discrete high permeability features) including horizontal and vertical conductivities;
- conductivity of the Woolloomoollo and Luna Park fault zones, and other higher permeability rock zones;
- porosity of the Hawkesbury Sandstone;
- connectivity of the surface water features with the underlying aquifers; and
- the boundary conditions along the southwestern edges of the model domain.

A total of 10 different scenarios were simulated, referred to further in the report as Sensitivity 1 to Sensitivity 10, which were then compared with the reference scenario (Scenario 0). Table 9-1 provides a summary of main model changes for each sensitivity simulation and overall changes in combined groundwater inflows into mainline carriageway tunnels (M110 and M120) and long-term operational 20 m contour drawdown extents.

Combined groundwater inflows into the mainline carriageway tunnels (M110 and M120) over the full model simulation time for each of sensitivity analysis simulation are shown in Figure 9-4. The extents of the 20 m drawdown contour at 100 years after tunnel excavation is shown in Figure 9-5 and predicted groundwater drawdowns in LSB-HC-PT-OW04 are shown in Figure 9-6.

Table 9-1: Summary of Model Sensitivity Simulation and Results

Sensitivity Analysis Simulation ID	Changes Relative to Sensitivity Simulation 0		
	Main model change(s)	**Increase / decrease in long-term inflow**	**20 m Drawdown Extents**
1	Vertical hydraulic conductivity of Hawkesbury Sandstone bulk rock (i.e., excluding faults) throughout the model reduced by an order of magnitude.	about 10% decrease	Up to about 170 m smaller where faults and higher permeability zones are not present (dark blue line).
2	Model nodes along southwestern model domain boundary (no-flow conditions in Sensitivity 0) converted to fixed head boundary conditions. Assigned fixed heads values were equal to steady-state head values at each of the model nodes.	less than 2% increase	Generally, within tens of metres of Sensitivity 0. Up to about 150 m smaller at the western end of the M4-M4 Link alignment (red line).
3	Horizontal and vertical hydraulic conductivity of Woolloomooloo and Lunar Park faults increased by an order of magnitude.	about 10% increase	Up to 1,300 m greater along Woolloomooloo Fault and 300 m greater along Lunar Park Fault (yellow line).
4	Vertical hydraulic conductivity of Hawkesbury Sandstone (bulk rock) throughout the model increased to be equal to horizontal hydraulic conductivity.	5% to 10% increase	Up to about 150 m greater where faults and higher permeability zones are not present (royal blue line).
5	Faults removed from model (i.e., horizontal and vertical hydraulic conductivities of faults decreased such that they are equal to background bulk values).	about 10% decrease	Varies between approximately 50 m greater or smaller throughout most of domain. Up to about 300 m lesser west of Haberfield Dyke (brown line).
6	Porosity in Hawkesbury Sandstone throughout model (including faults) increased to 0.1.	20% to 30% increase	Up to about 200 m smaller where faults and higher permeability zones are not present (dark yellow line).
7	Influence of streams removed from the simulation (i.e., in- and out-transfer rates set to 0)	less than 5% decrease	Up to about 500 m greater where faults and higher permeability zones are not present (pink line).
8	Influence of streams significantly enhanced (in- and out-transfer rates set to 10000)	20% to 30% increase	100 m to 300 m smaller in vicinity of streams/canals. Otherwise generally within a few tens of metres of Sensitivity 0 (green line).
9	Horizontal and vertical hydraulic conductivity of Hawkesbury Sandstone bulk rock (i.e., excluding faults) in deeper layers reduced. They were set to following values: Layers 7-9 – K_h = 3e-08 m/s$^-$, K_v = 3e-09 m/s[1] Layer 10 – K_h = 1e-08 m/s, K_v = 1e-09 m/s[1] Layers 11-12 – K_h = 5e-09 m/s, K_v = 5e-10 m/s	30% to 35% decrease	Up to about 200 m smaller where faults and higher permeability zones are not present (purple line.)

Sensitivity Analysis Simulation ID	Changes Relative to Sensitivity Simulation 0		
	Main model change(s)	Increase / decrease in long-term inflow	20 m Drawdown Extents
10	Porosity in Hawkesbury Sandstone throughout model (including faults) reduced to 0.001	2% to 3% decrease	Generally, within tens of metres of Scenario 0, and up to ~50 m greater where faults and higher permeability zones are not present. Up to ~200 m greater west of western dyke (Beige line.)

Two sensitivity analysis simulations (Sensitivity 6 and Sensitivity 8) resulted in a notable increase in groundwater inflows into mainline carriageway tunnels, about 1 L/s to 2 L/s greater than the Sensitivity 0, which represents approximately a 20% to 30% increase. As detailed in Table 9-1, these simulations involve pervasive high porosity of the rock and high surface water-groundwater connectivity, respectively. Both cases are considered unlikely to occur but have been assessed to understand the potential sensitivity of the model to such changes.

In the case of the Sensitivity 6 simulation, a bulk porosity as high as 0.1 may be possible in localised areas but is unlikely to be pervasive throughout a large area of the fractured rock aquifer. In the case of the Sensitivity 8 simulation, higher transfer rates governing stream-groundwater interaction (i.e., no resistance to flow interchange between groundwater and surface water) are also considered unrealistic based on the observed groundwater level responses to the pumping test at Hawthorne Canal and groundwater levels observed in the monitoring wells located adjacent to main surface water features within the model domain.

As shown in Figure 9-4, the lowest groundwater inflow rates into mainline carriageway tunnels are predicted by the Sensitivity 9 simulation, which includes between half an order to an order of magnitude reduction in the bulk horizontal and vertical hydraulic conductivities of the Hawkesbury Sandstone. A reduction of approximately 2 L/s predicted by this simulation, corresponds to about a 35% decrease relative to the Sensitivity 0 simulation.

The Sensitivity 9 simulation represents a potential conceptualisation in which groundwater flow is more dominated by fracture zones and faults, with bulk rock hydraulic conductivities substantially lower than inferred. The Sensitivity 1 simulation, which included reduction in the vertical hydraulic conductivity only, predicted a decrease in the groundwater inflow by about 10% only. Based on the hydraulic testing undertaken (packer tests) as part of the detailed design investigation, hydraulic conductivities of the rock in these ranges are possible locally, but not for the entire model domain.

The inflow predictions for all other sensitivity analysis simulations produced an envelope of inflow variability of approximately +/-10% relative to the Scenario 0 predictions.

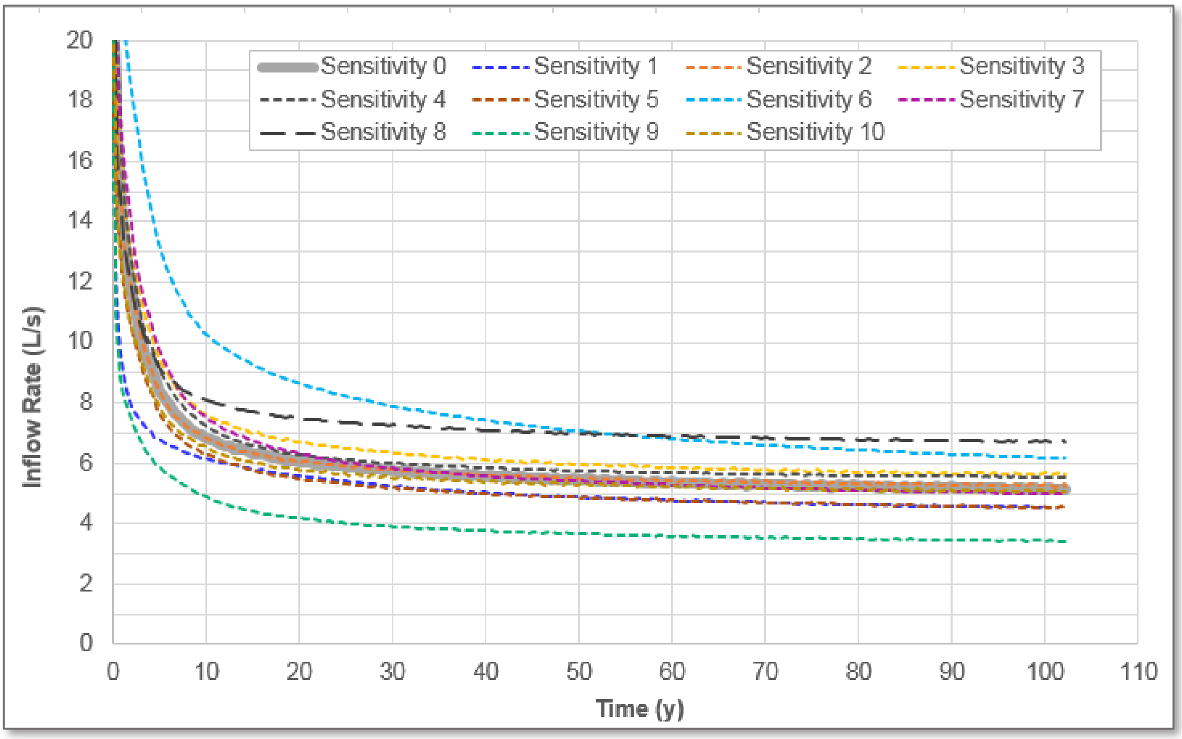


Figure 9-4: Model Predicted Combined Groundwater Inflow into Mainline Carriageway Tunnels (M110 and M120) for Sensitivity 0 to Sensitivity 10 simulations.

Increasing the hydraulic conductivity of the Woolloomooloo and Lunar Park faults (Sensitivity 3) resulted in increased groundwater drawdown propagation along these faults (Figure 9-5), while no observable drawdown changes were predicted at LSB-HC-PT-OW04 (Figure 9-6). The lack of change in well LSB-HC-PT-OW04 was not surprising given the well's distance from both faults.

Reduction in the connection between the surface water and groundwater as simulated by Sensitivity 7, results in a significant increase in drawdown extent along most of the project corridor and significant drawdown at LSB-HC-PT-OW04. On other hand Sensitivity 8 simulation, that assumes enhanced connectivity between the surface water and groundwater predicts the lowest groundwater drawdowns in the vicinity of the streams/canals and at LSB-HC-PT-OW04.

Simulations 7 and 8 demonstrate that the connectivity between surface and groundwater is a significant model assumption and if not characterised correctly will impact upon the model's predictive ability. The assumptions adopted for both simulations, however, are considered unrealistic based on the observed groundwater level responses to the pumping test at Hawthorne Canal and groundwater levels observed in the monitoring wells located adjacent to main surface water features within the model domain.

The smallest predicted drawdown extent through most of the model domain is predicted by the Sensitivity 9 simulation, which assumes significantly reduced horizontal and vertical hydraulic conductivities of the deeper Hawkesbury Sandstone zones (Figure 9-5). This simulation also results in the second smallest drawdown at LSB-HC-PT-OW04. It also predicted the lowest inflows into mainline carriageway tunnels as discussed above.

All other sensitivity analysis simulations predicted smaller differences in groundwater drawdown response and at LSB-HC-PTOW04.

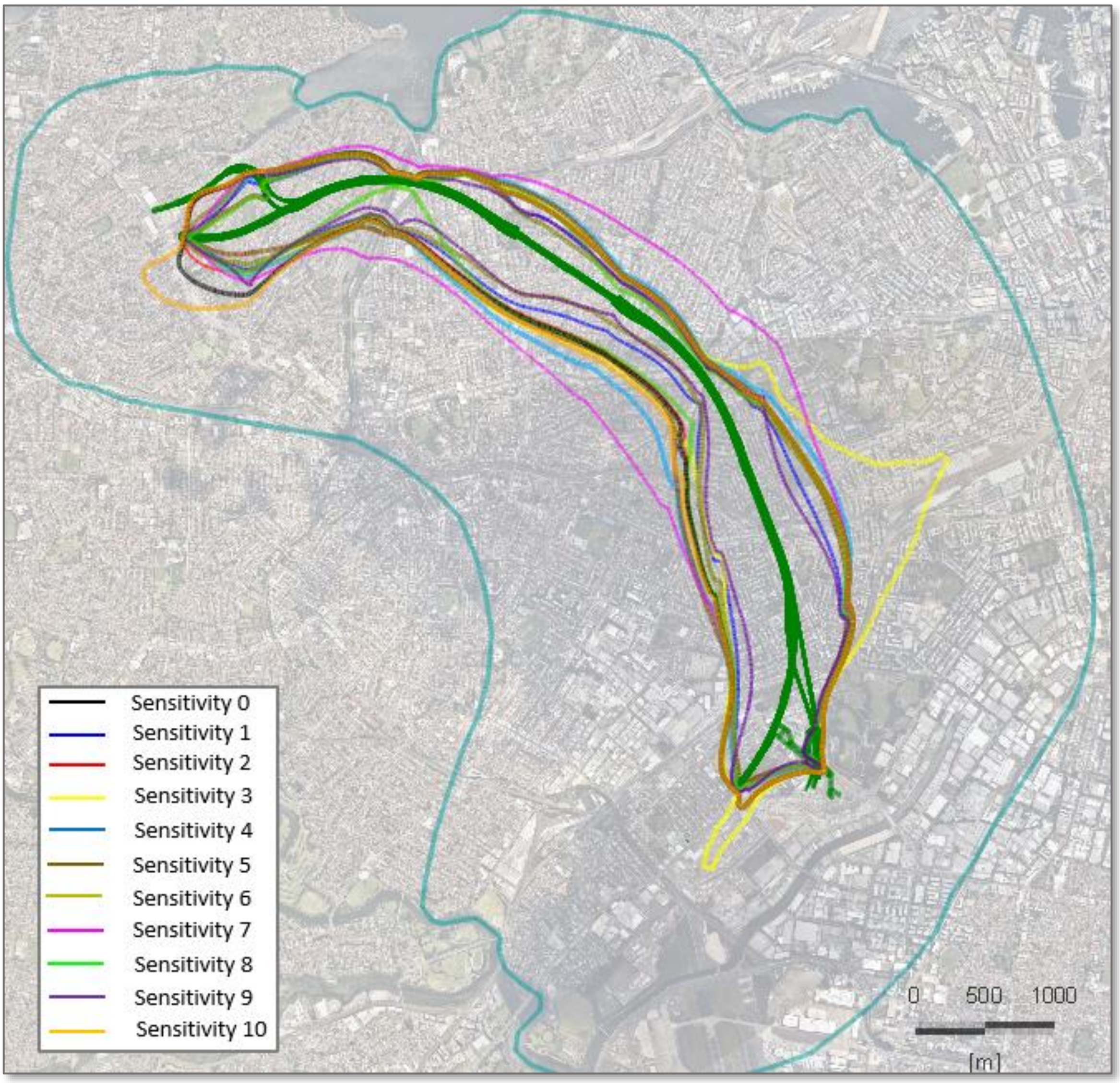


Figure 9-5: Model Predicted Extent of 20 m Groundwater Drawdown Contour for Sensitivity 0 to Sensitivity 10 Simulations.

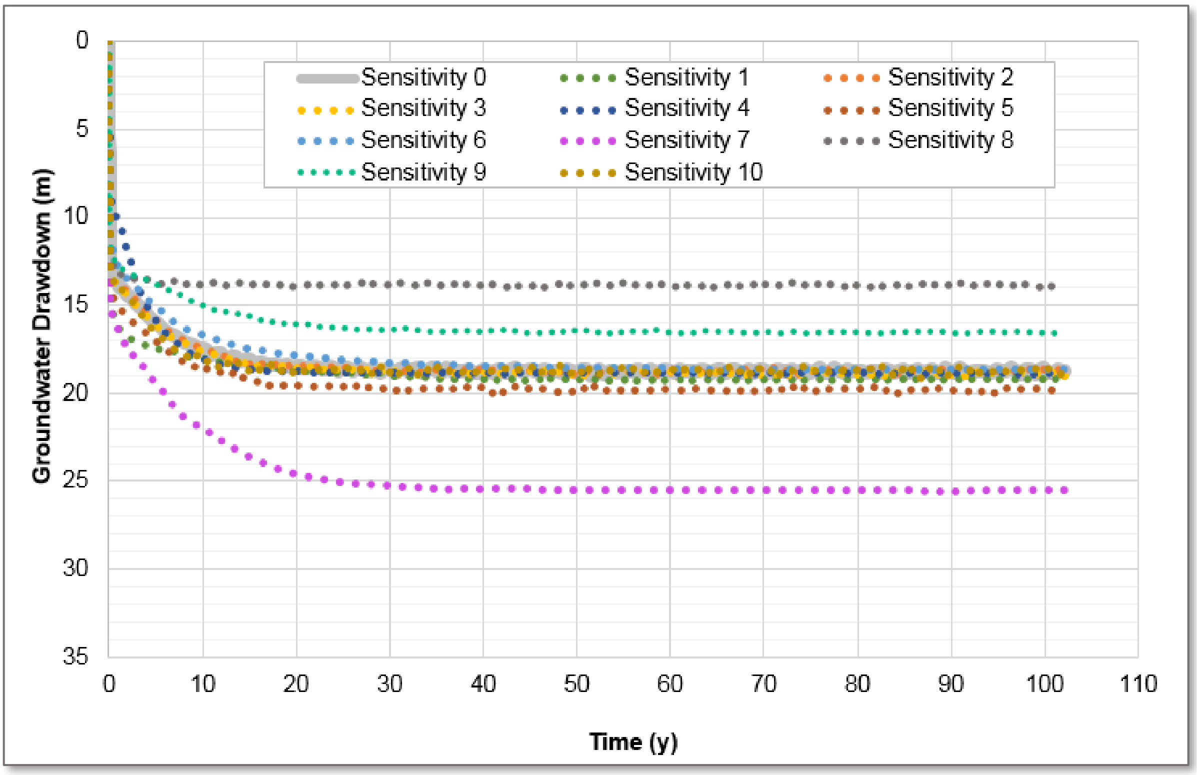


Figure 9-6: Model Predicted Groundwater Drawdowns at LSB-HC-PT-OW04 Sensitivity 0 to Sensitivity 10 Simulations.

In addition to understanding the effect of the sensitivity simulations on inflow and drawdown predictions, the sensitivity analysis also explored the effect of varying the specific storage assumptions on the model's ability to match the pumping test results at Hawthorne Canal. The specific storages were varied during the transient calibration process, with smaller specific storages found to provide a better match to the groundwater recovery curves in particular, but overall the effects on the predicted drawdown extents were indistinguishable. Additionally, observations of groundwater inflows for the excavations completed up to May 2020 are also suggesting both the Ashfield Shale and Hawkesbury Sandstone aquifers have low storage properties, as the groundwater inflows when encountered decrease relatively quickly and do not persist, while the groundwater drawdowns also develop and propagate away from the excavations rapidly.

In summary the results of sensitivity analysis indicate that the most realistic parameter variations that may have a significant effect on the model's predictive ability is the bulk hydraulic conductivity of the Hawkesbury Sandstone. All other parameter variations are inferred not to be realistic or unlikely to results in significant effects.

It should be noted however, that should the ground conditions encountered during excavation differ from those assumed in the geological model, the inflows and drawdowns predicted by the model could vary from those presented in this report. It is therefore important that the actual groundwater response to tunnel excavation be monitored during construction.

10. SUMMARY OF CONTAMINANT TRANSPORT MODELLING

Contaminant transport modelling was undertaken to assess the potential effects of the tunnel construction and operation on the movement of contaminated groundwater from the former landfills within the project corridor. These landfills were developed within former shale quarries (brick pits). For the purposes of the contaminant transport assessment, ammonia was selected as a representative species for the contaminant transport modelling and was used as a proxy for other solutes in the leachate. Results of the contaminant transport modelling were then used to inform the durability assessment for tunnel structures, the in-tunnel gas risk assessment and for the design of the tunnel water treatment plant.

A summary of the contaminant transport modelling, including the assumptions adopted, scenarios simulated, and the modelling results are detailed in Appendix BD. The findings of the modelling in relation to durability assessment, the in-tunnel gas risk assessment and the groundwater quality inputs for the design of the water treatment plant are documented and discussed in Contamination Assessment Report (M4M5-JAJV-PRW-GEO-CN01-RPT-0005).

11. MODEL CLASSIFICATION AND LIMITATIONS

The FD model presented in this report was developed and calibrated based on the extensive data collected during the detailed design site investigation. The model also builds on the knowledge obtained through other infrastructure projects within the Project area and monitoring data that spans over a multi-year period.

Considering the detail of the geological model the FD model has been based on, the quality of the calibration statistics and water balance and the model's ability to generally replicate the regional groundwater flow system, the FD Model is considered to have a Class 2 Confidence Level based on Table 2-1 of the Australian Groundwater Modelling Guidelines. While the model contains elements, which are associated with a Class 3 Confidence Level, it has been classified as Class 2 because the transient calibration stresses are relatively short when compared to the stresses which will be imposed on the groundwater system by construction and operation of the project. From a practical perspective however, obtaining such a transient data set to calibrate the model against is only possible well into the construction period.

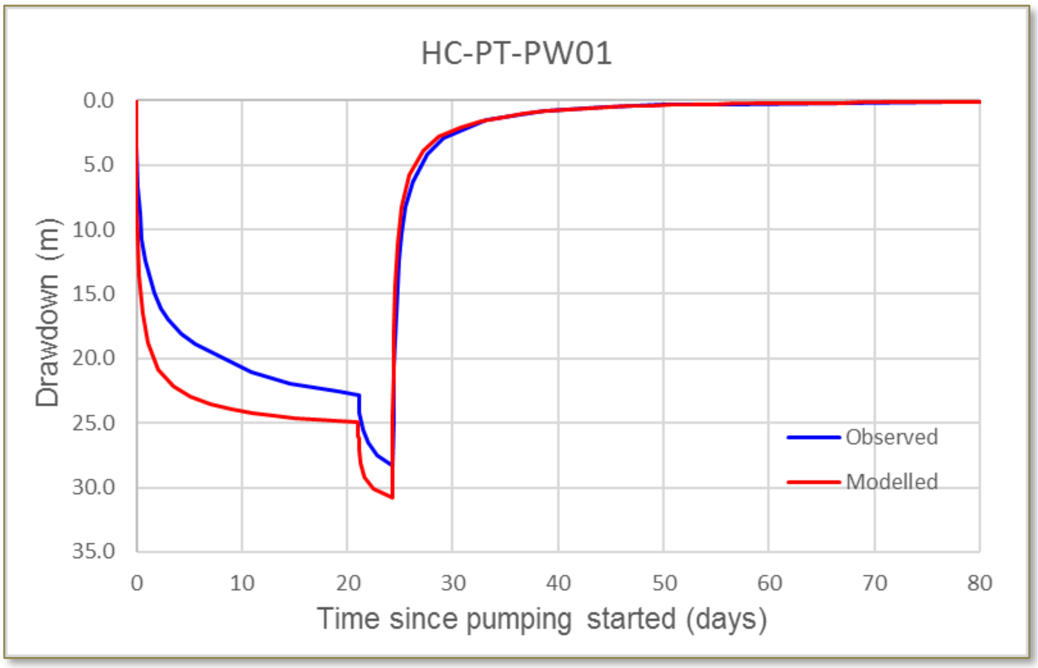
Based on the quality of the modelling work completed and the confidence level obtained, the model and the modelling results are considered to be compliant with the modelling requirements in MCoA E193 and REMMs GW6 and GW7.

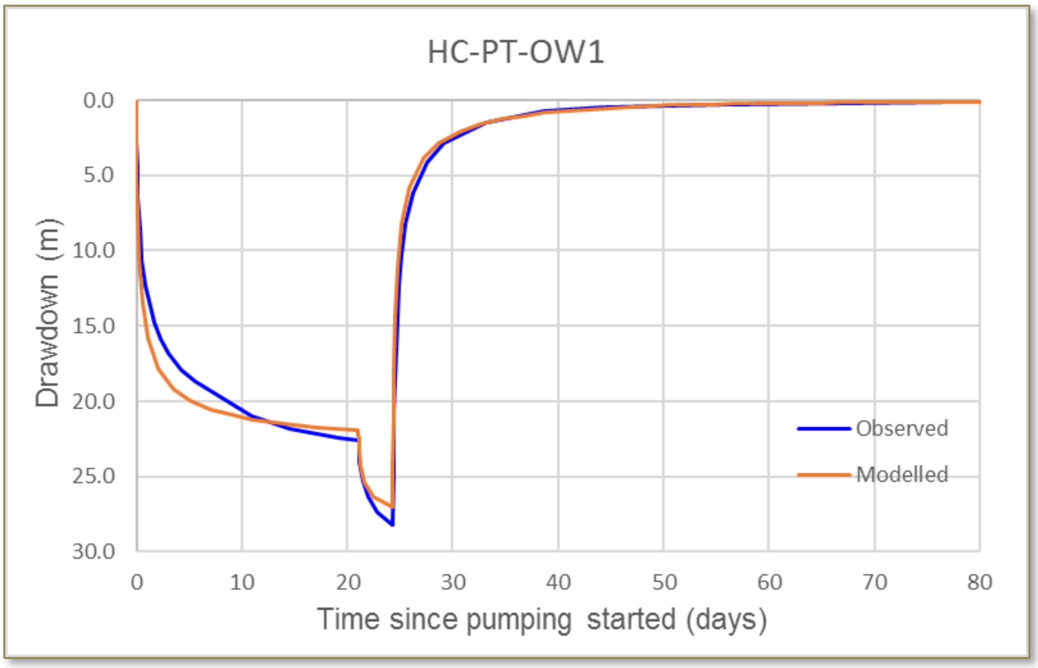
12. MODELLING PEER REVIEW

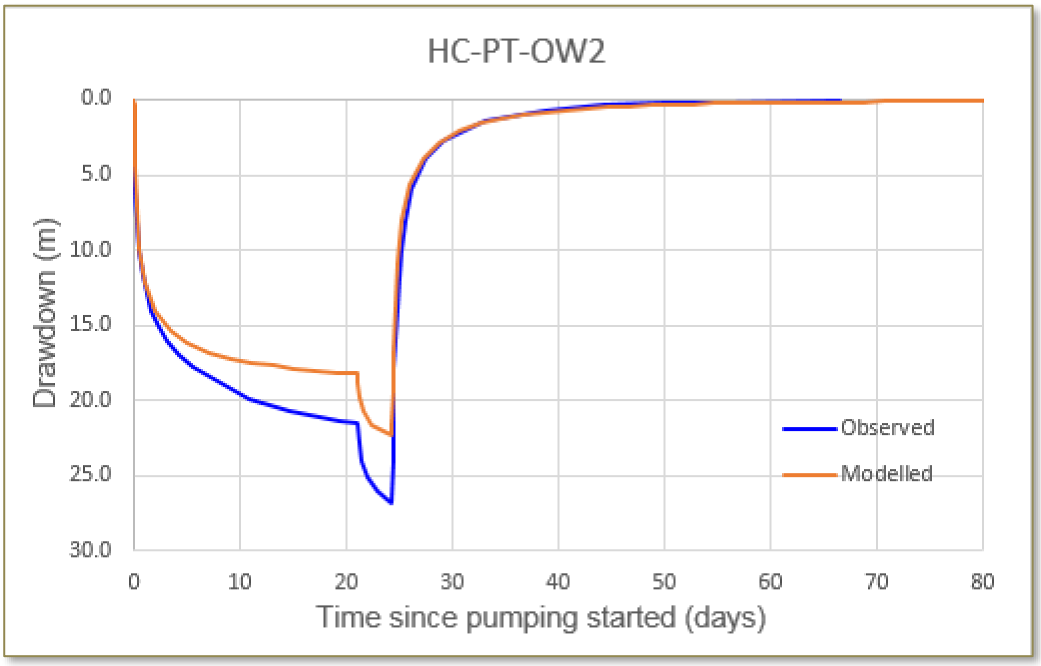
A peer review of the FD model calibration, inflow and drawdown predictions was completed by Brian Barnett of Jacobs. The review record is presented in Appendix BE.

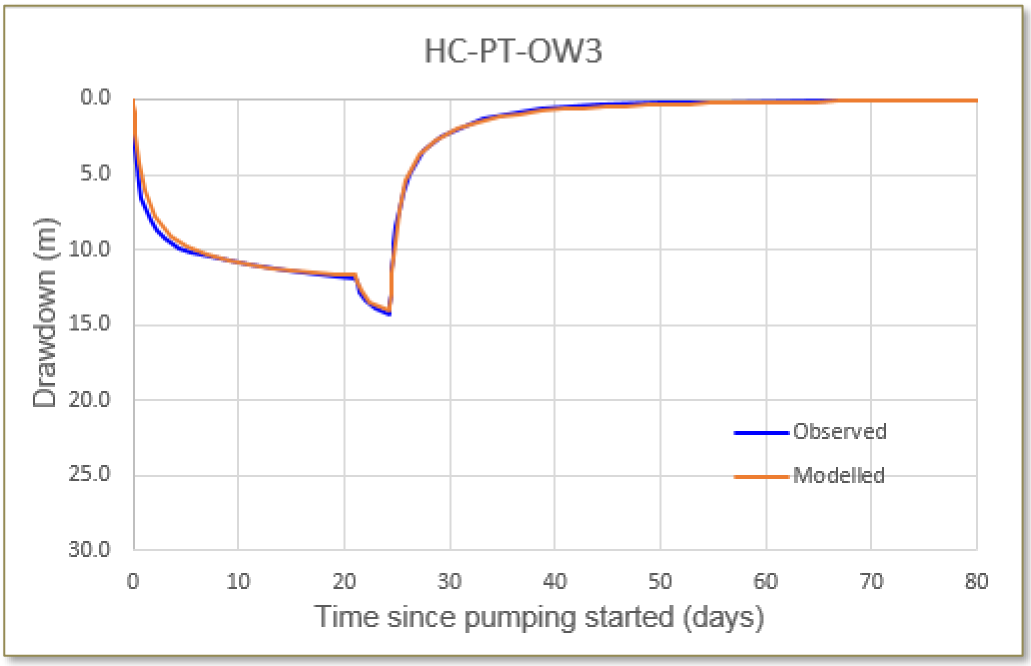
Appendix BA

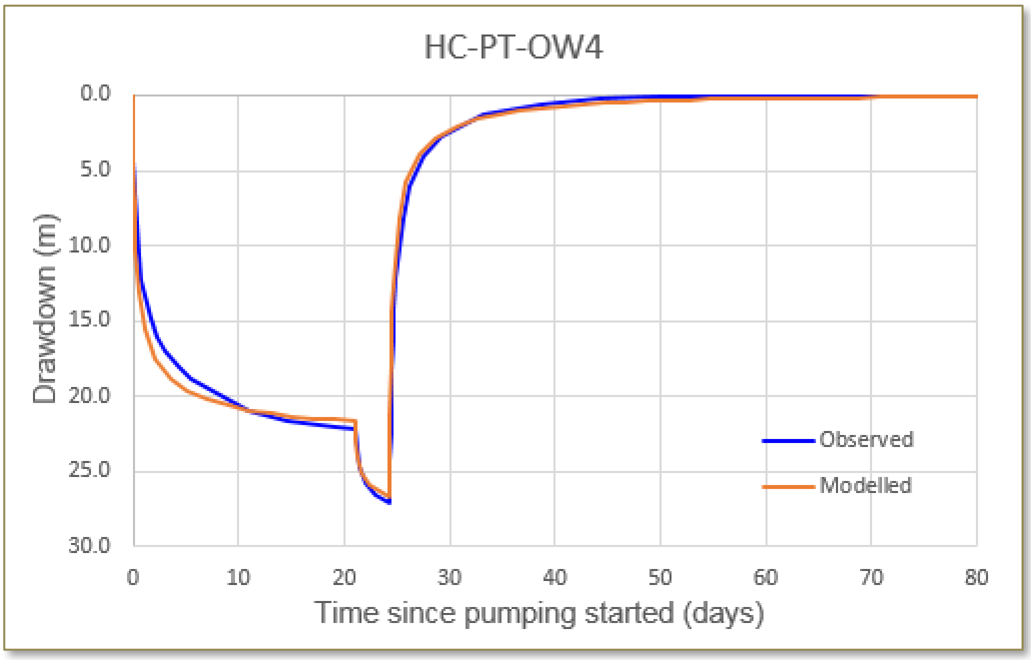
Simulated versus Observed Groundwater Drawdowns, Transient Model Calibration

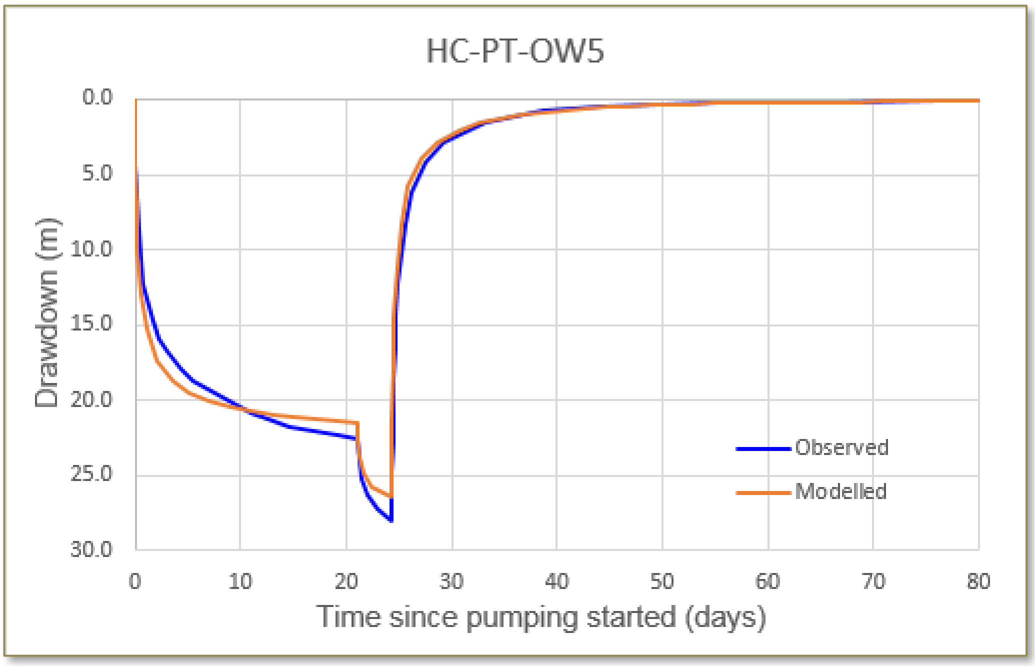

HC-PT-PW01
0.0
5.0
10.0
15.0
20.0
25.0
30.0
35.0
Drawdown (m)
0
10
20
30
40
50
60
70
80
Time since pumping started (days)
Observed
Modelled

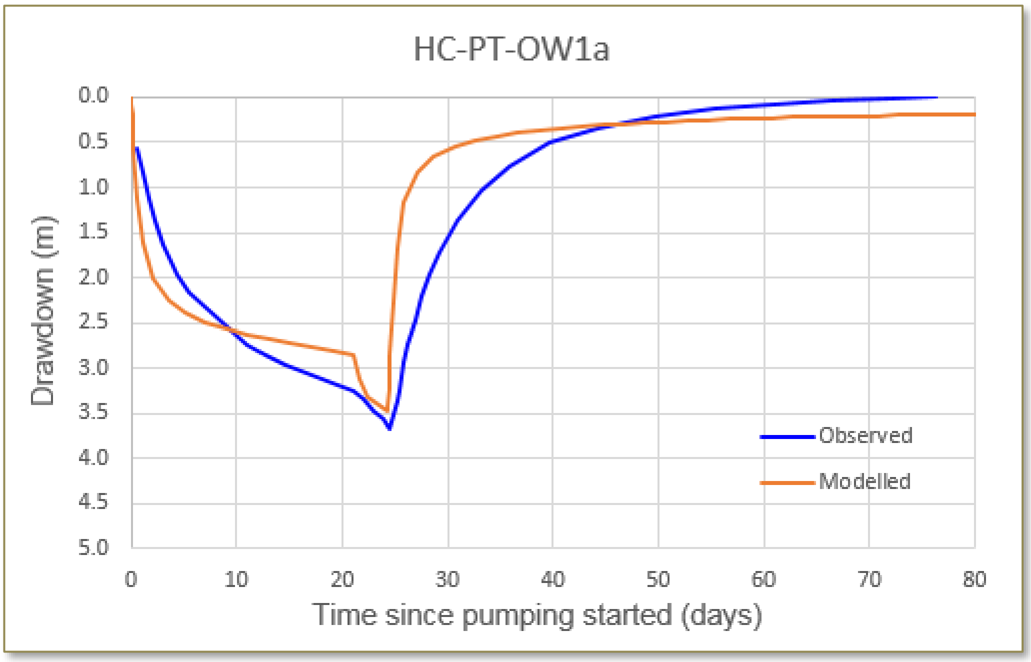

HC-PT-OW1
0.0
5.0
10.0
15.0
20.0
25.0
30.0
Drawdown (m)
0
10
20
30
40
50
60
70
80
Time since pumping started (days)
Observed
Modelled

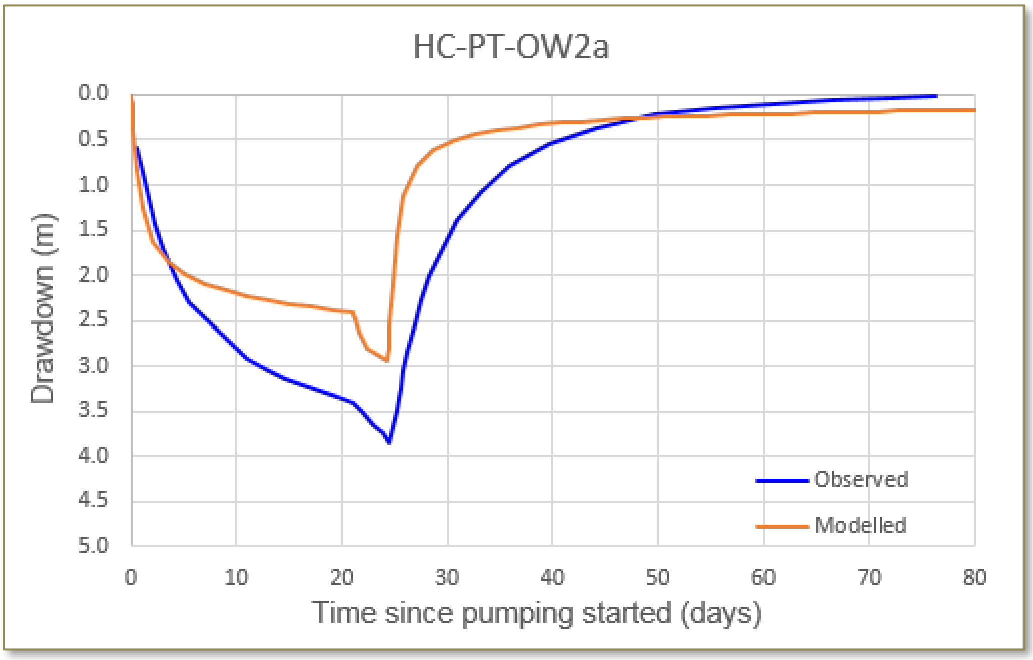

HC-PT-OW2
Drawdown (m)
0.0
5.0
10.0
15.0
20.0
25.0
30.0
0
10
20
30
40
50
60
70
80
Time since pumping started (days)
Observed
Modelled

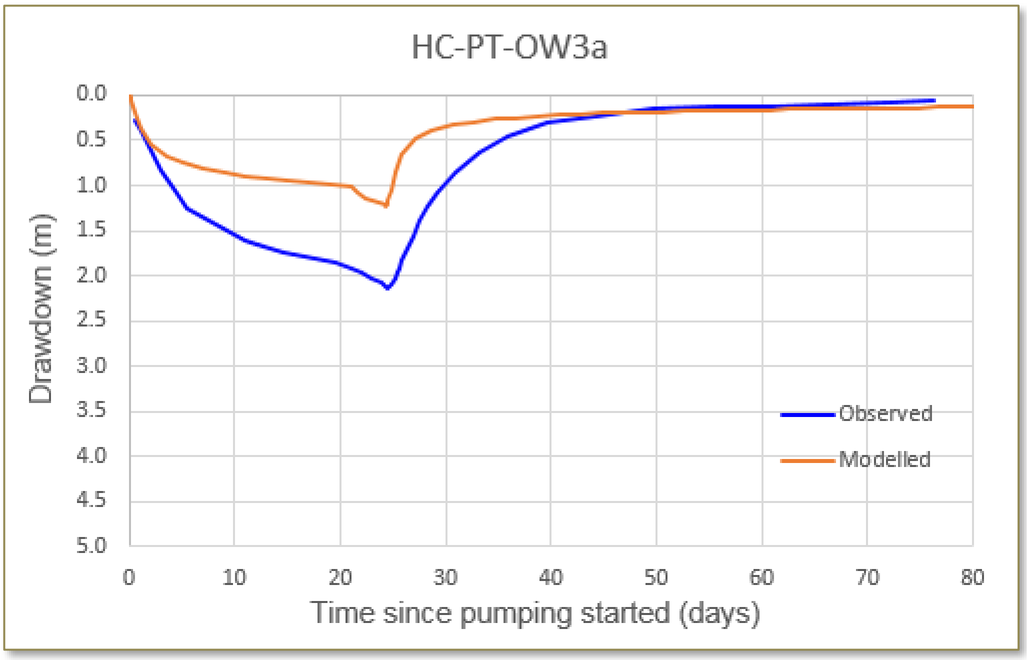

HC-PT-OW3
Drawdown (m)
0.0
5.0
10.0
15.0
20.0
25.0
30.0
0
10
20
30
40
50
60
70
80
Time since pumping started (days)
Observed
Modelled

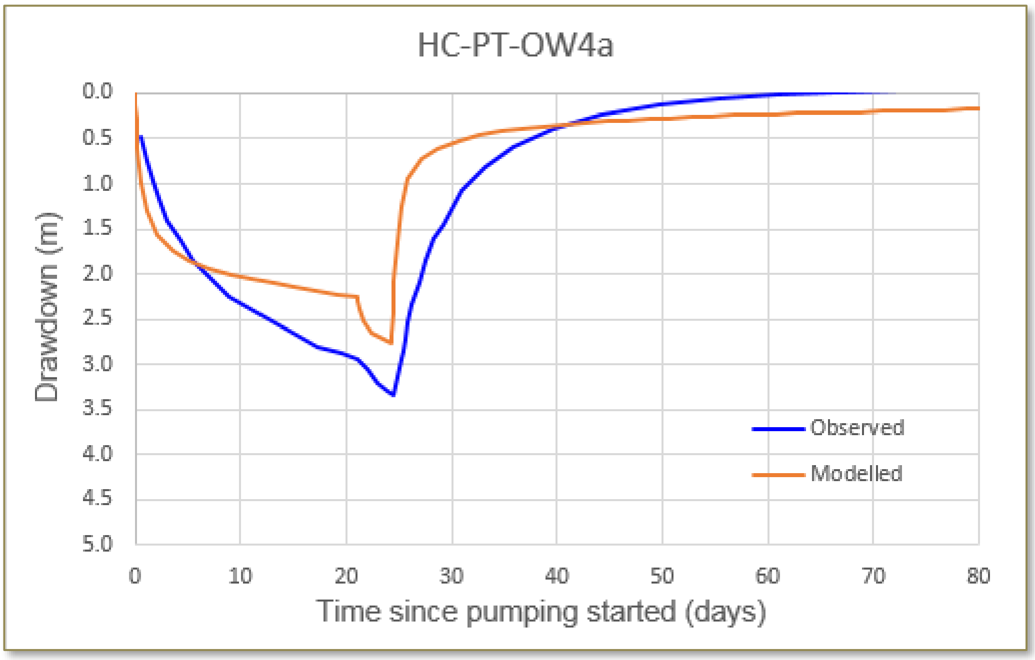

HC-PT-OW4
0.0
5.0
10.0
15.0
20.0
25.0
30.0
Drawdown (m)
0
10
20
30
40
50
60
70
80
Time since pumping started (days)
Observed
Modelled

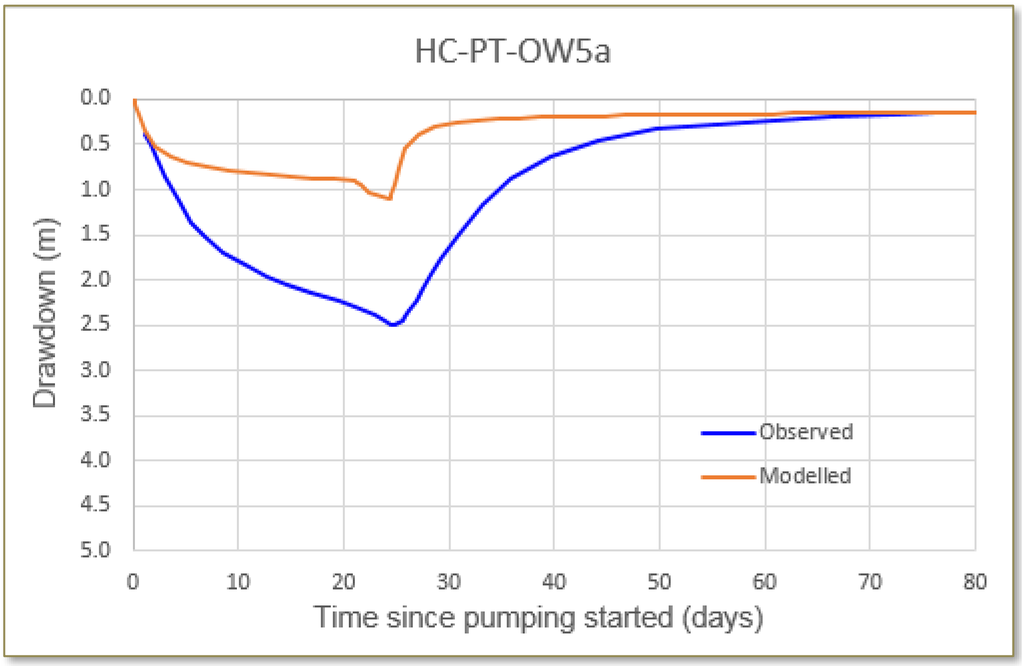

HC-PT-OW5
0.0
5.0
10.0
15.0
20.0
25.0
30.0
Drawdown (m)
0
10
20
30
40
50
60
70
80
Time since pumping started (days)
Observed
Modelled

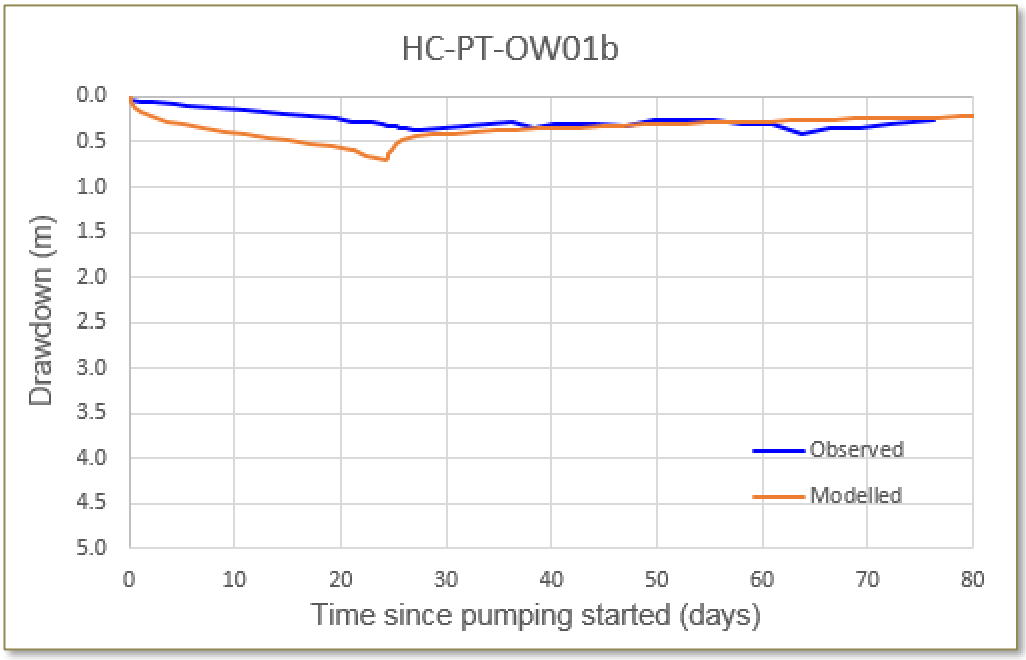

HC-PT-OW1a
Drawdown (m)
0.0
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0
4.5
5.0
0
10
20
30
40
50
60
70
80
Time since pumping started (days)
Observed
Modelled

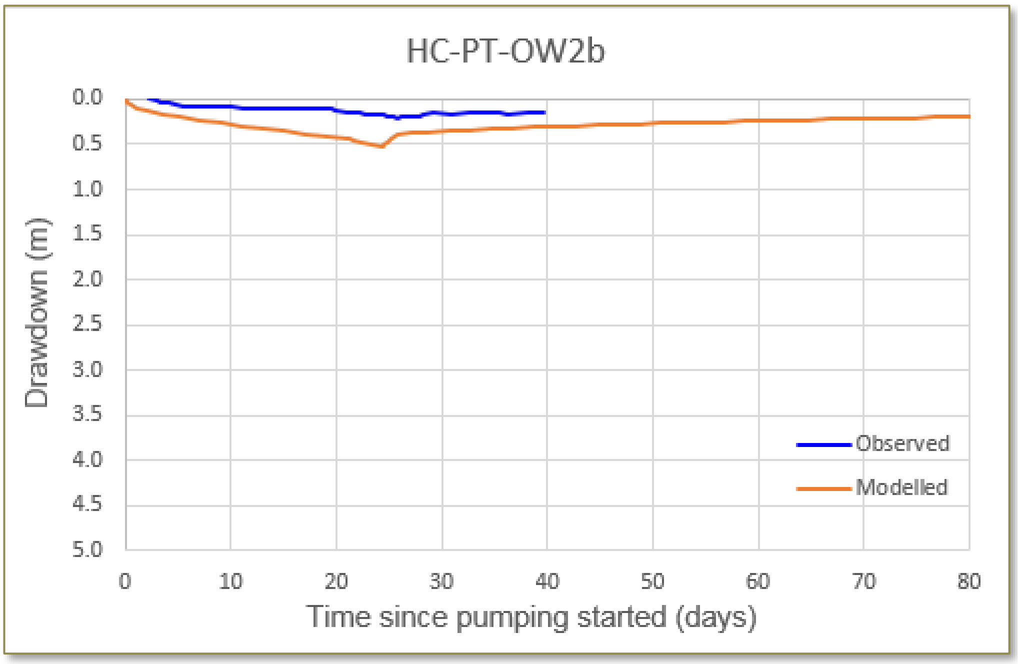

HC-PT-OW2a
Drawdown (m)
0.0
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0
4.5
5.0
0
10
20
30
40
50
60
70
80
Time since pumping started (days)
Observed
Modelled


HC-PT-OW3a
Drawdown (m)
0.0
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0
4.5
5.0
0
10
20
30
40
50
60
70
80
Time since pumping started (days)
Observed
Modelled

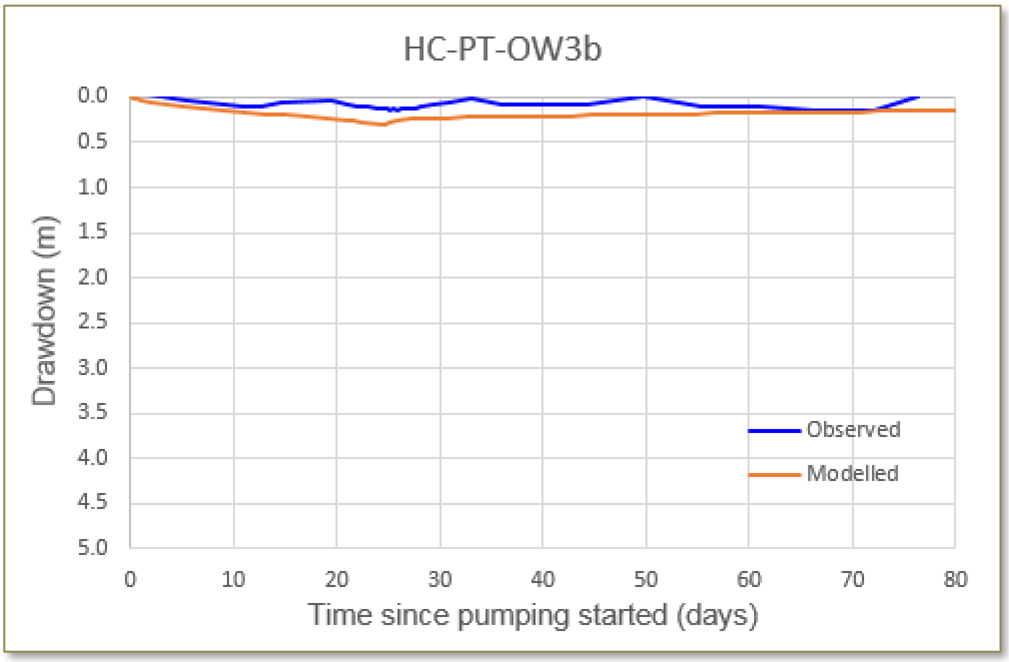

HC-PT-OW4a
Drawdown (m)
0.0
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0
4.5
5.0
0
10
20
30
40
50
60
70
80
Time since pumping started (days)
Observed
Modelled


HC-PT-OW5a
Drawdown (m)
0.0
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0
4.5
5.0
0
10
20
30
40
50
60
70
80
Time since pumping started (days)
Observed
Modelled

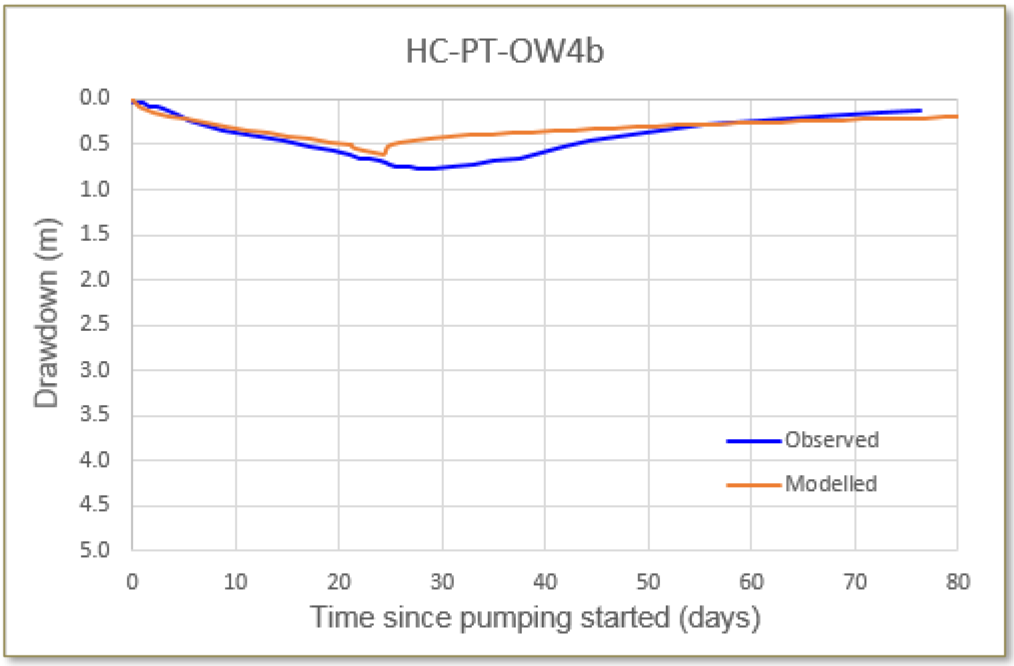

HC-PT-OW01b
Drawdown (m)
0.0
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0
4.5
5.0
0
10
20
30
40
50
60
70
80
Time since pumping started (days)
Observed
Modelled

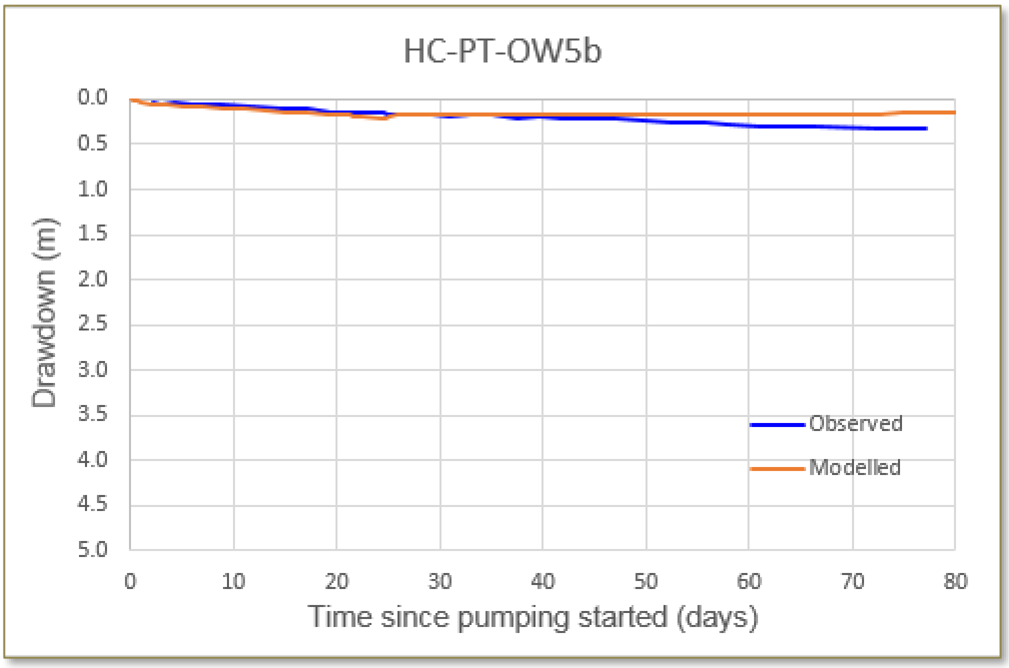


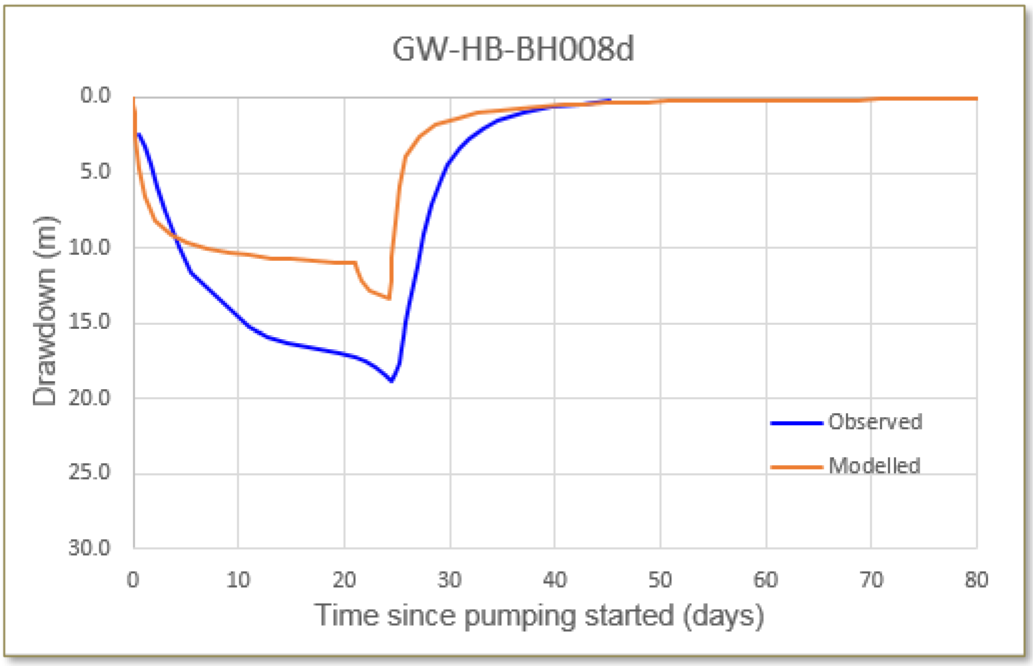
HC-PT-OW2b
Drawdown (m)
0.0
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0
4.5
5.0
0
10
20
30
40
50
60
70
80
Time since pumping started (days)
Observed
Modelled

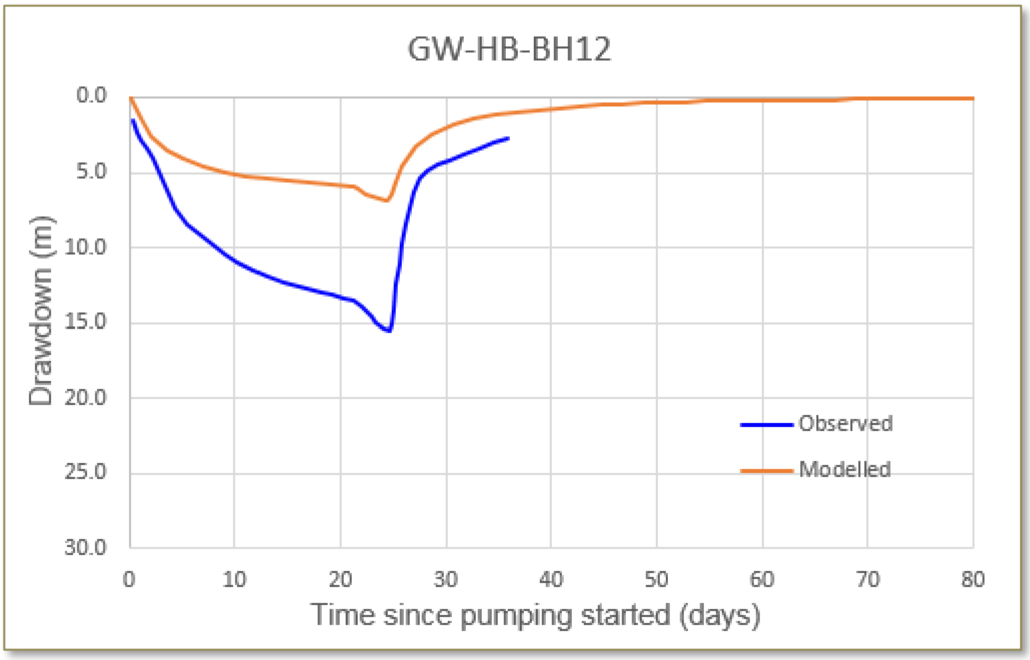


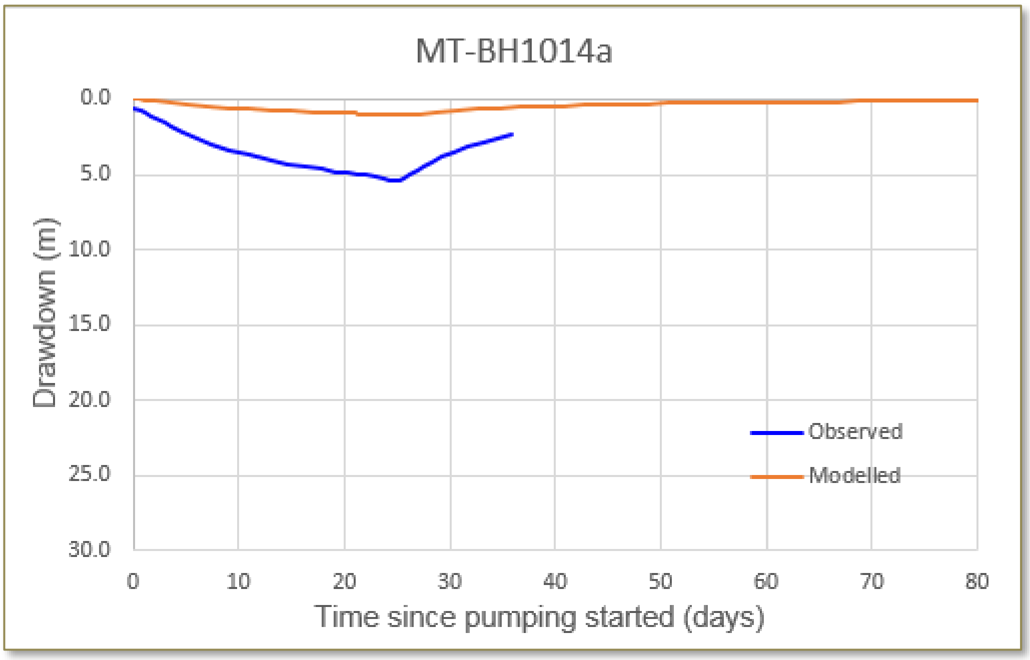
HC-PT-OW3b
Drawdown (m)
0.0
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0
4.5
5.0
0
10
20
30
40
50
60
70
80
Time since pumping started (days)
Observed
Modelled

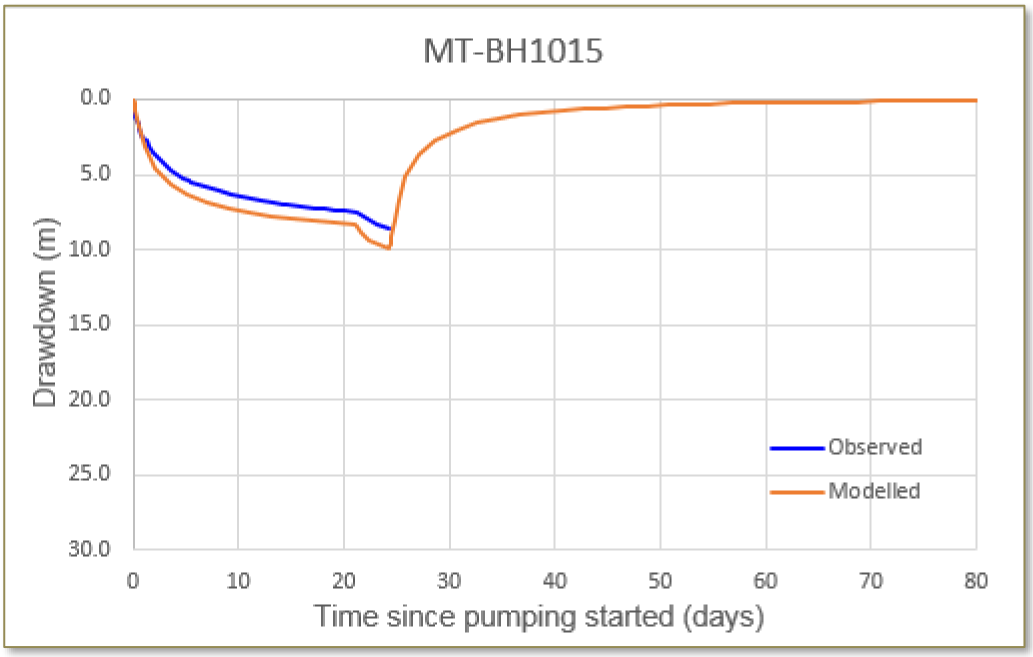

HC-PT-OW4b
Drawdown (m)
0.0
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0
4.5
5.0
0
10
20
30
40
50
60
70
80
Time since pumping started (days)
Observed
Modelled

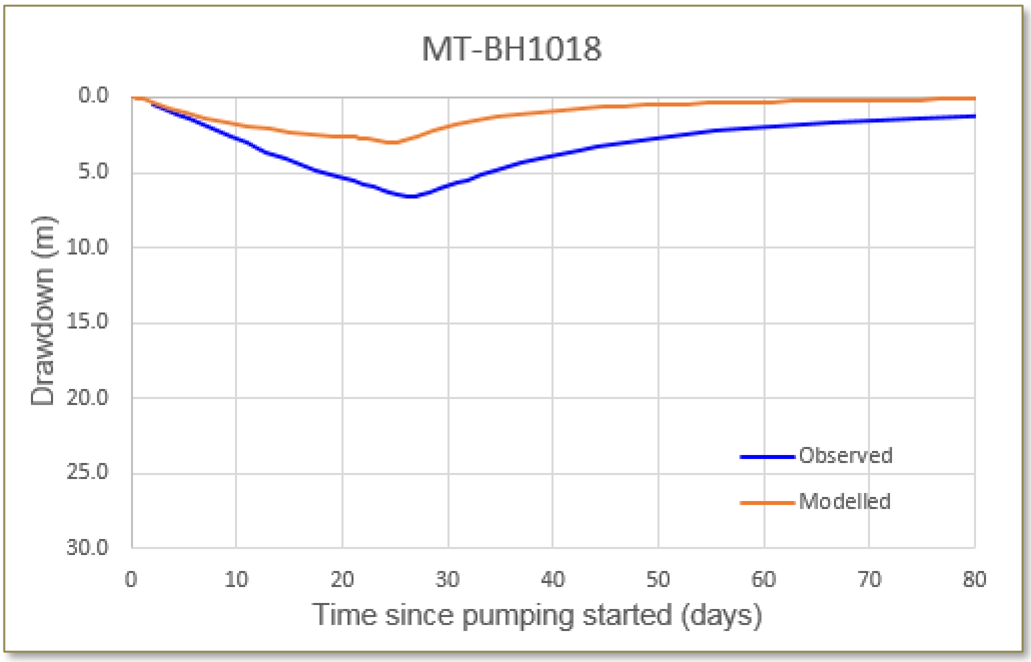

HC-PT-OW5b
Drawdown (m)
0.0
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0
4.5
5.0
0
10
20
30
40
50
60
70
80
Time since pumping started (days)
Observed
Modelled


GW-HB-BH008d
Drawdown (m)
0.0
5.0
10.0
15.0
20.0
25.0
30.0
0
10
20
30
40
50
60
70
80
Time since pumping started (days)
Observed
Modelled


GW-HB-BH12
Drawdown (m)
0.0
5.0
10.0
15.0
20.0
25.0
30.0
0
10
20
30
40
50
60
70
80
Time since pumping started (days)
Observed
Modelled


MT-BH1014a
0.0
5.0
10.0
15.0
20.0
25.0
30.0
Drawdown (m)
0
10
20
30
40
50
60
70
80
Time since pumping started (days)
Observed
Modelled


MT-BH1015
0.0
5.0
10.0
15.0
20.0
25.0
30.0
Drawdown (m)
0
10
20
30
40
50
60
70
80
Time since pumping started (days)
Observed
Modelled


MT-BH1018
Drawdown (m)
0.0
5.0
10.0
15.0
20.0
25.0
30.0
0
10
20
30
40
50
60
70
80
Time since pumping started (days)
Observed
Modelled

Appendix BB

Observed versus Modelled Groundwater Levels, Steady State FD Model Calibration

Table BB:1 Observed versus Modelled Groundwater Levels, Steady State FD Model Calibration

Well ID	Observed groundwater levels (m AHD)	Modelled groundwater levels (m)	Difference between observed and modelled levels (m)
Deep Wells			
HB_BH12	2.00	3.10	1.10
HB_BH14	2.35	5.00	2.65
LDS-BH-2008A	-1.00	0.28	1.28
LDS-BH-2011A	1.00	1.17	0.17
LDS-BH-2015	7.40	2.42	-4.98
LDS-BH-2019	4.50	3.12	-1.38
LDS-BH-3047A	1.20	0.78	-0.42
LSB-GW-HB-BH03	5.00	4.22	-0.78
LSB-GW-HB-BH08d	1.26	0.91	-0.35
LSB-GW-HB-BH12	1.65	2.42	0.77
LSB-HB-BH1006	3.90	1.27	-2.63
LSB-HC-PT-OW1	1.41	1.23	-0.18
LSB-HC-PT-OW2	1.41	1.14	-0.27
LSB-HC-PT-OW3	1.34	0.96	-0.38
LSB-HC-PT-OW4	1.43	1.25	-0.18
LSB-HC-PT-OW5	1.38	1.22	-0.16
LSB-HC-PT-PW01	1.41	1.24	-0.17
LSB-MT-BH1012	15.75	11.03	-4.72
LSB-MT-BH1013a	7.90	9.05	1.15
LSB-MT-BH1014a	4.20	7.35	3.15
LSB-MT-BH1015	1.90	2.40	0.50
LSB-MT-BH1016	1.80	1.24	-0.56
LSB-MT-BH1018	2.00	1.69	-0.31
LSB-SP-BH09	2.25	2.17	-0.08
LSB-SP-BH14	3.90	2.44	-1.46
LSB-SP-BH15	5.15	2.45	-2.70
LSB-SP-BH16	4.85	5.93	1.08
LSB-SP-BH17	3.25	4.37	1.12
LSB-SP-BH19	2.50	5.03	2.53
M4E-BH302	4.50	5.06	0.56
MT_BH02	8.60	9.58	0.98
MT_BH07	6.00	6.17	0.17
MT_BH11	9.10	10.48	1.38
MT_BH19	9.50	10.02	0.52
MT_BH20	9.90	8.07	-1.83
MT_BH21	14.00	9.83	-4.17

Well ID	Observed groundwater levels (m AHD)	Modelled groundwater levels (m)	Difference between observed and modelled levels (m)
RZ_BH01d	1.64	1.49	-0.15
RZ_BH15	1.70	2.41	0.71
RZ_BH16d	1.67	1.44	-0.23
RZ_BH19	1.53	0.74	-0.79
RZ_BH44d	1.59	0.46	-1.13
RZ_BH47d	1.45	0.41	-1.04
RZ_BH60	12.59	8.72	-3.87
RZ_BH64	2.10	5.48	3.38
SP_BH01_old	8.40	7.13	-1.27
SP_BH04_old	3.35	5.04	1.69
SP_BH06_old	8.27	4.92	-3.35
TC_BH01d	1.60	0.83	-0.77
TC_BH07d	1.60	0.61	-0.99
TC_BH09d	1.50	0.50	-1.00
WCX-BH103	4.60	2.77	-1.83
WCX-BH109	-0.70	2.10	2.80
WCX-BH153	3.20	2.60	-0.60
BH3103_141d	3.77	5.50	1.73
Shallow Wells			
BH3103_141s	13.97	8.03	-5.94
CM_BH01	17.40	14.97	-2.43
CM_BH04	22.10	21.02	-1.08
CM_BH06	29.90	25.00	-4.90
CM_BH10	22.60	23.53	0.93
HB_BH03	3.00	2.04	-0.96
HB_BH15	8.10	7.70	-0.40
LDS-BH-2011B	1.20	1.38	0.18
LDS-BH-2018	4.90	3.50	-1.40
LDS-BH-30456	1.15	-1.83	-2.98
LDS-BH-3045A	-0.50	-2.36	-1.86
LDS-BH-3046	1.50	-0.93	-2.43
LDS-BH-3046A	-6.50	-1.63	4.87
LDS-BH-3047	1.25	0.98	-0.27
LDS-BH-5007	-2.03	-1.25	0.78
LDS-BH-5022	1.20	1.10	-0.10
LSB-AP-BH01	5.50	4.91	-0.59
LSB-CP-BH01	3.25	5.53	2.28
LSB-CP-BH02A	3.00	5.69	2.69

Well ID	Observed groundwater levels (m AHD)	Modelled groundwater levels (m)	Difference between observed and modelled levels (m)
LSB-HC-PT-OW04a	0.86	0.53	-0.33
LSB-HC-PT-OW1a	1.07	0.52	-0.55
LSB-HC-PT-OW1b	0.36	0.37	0.01
LSB-HC-PT-OW2a	1.04	0.47	-0.57
LSB-HC-PT-OW2b	0.36	0.32	-0.04
LSB-HC-PT-OW3a	0.70	0.38	-0.32
LSB-HC-PT-OW3b	0.56	0.32	-0.24
LSB-HC-PT-OW4b	0.75	0.41	-0.34
LSB-HC-PT-OW5a	0.85	0.53	-0.32
LSB-HC-PT-OW5b	0.54	0.50	-0.04
LSB-MT-BH1002	9.50	3.83	-5.67
LSB-MT-BH1008a	25.00	21.82	-3.18
LSB-MT-BH1009a	33.00	27.77	-5.23
LSB-MT-BH1015a	1.40	1.32	-0.08
LSB-OR-BH01	13.55	15.54	1.99
LSB-SP-BH04b	8.50	7.39	-1.11
LSB-SP-BH05a	11.30	8.56	-2.74
LSB-SP-BH06a	7.70	7.90	0.20
LSB-SP-BH07	7.30	8.43	1.13
LSB-SP-BH07a	18.30	11.52	-6.78
LSB-SP-BH08	2.20	4.29	2.09
LSB-SP-BH08a	6.00	4.39	-1.61
LSB-SP-BH09a	2.60	2.19	-0.41
LSB-SP-BH10	8.70	5.19	-3.51
LSB-SP-BH10a	9.60	9.77	0.17
LSB-SP-BH11	5.10	5.79	0.69
LSB-SP-BH11a	10.60	10.27	-0.33
LSB-SP-BH12a	7.65	5.16	-2.49
LSB-SP-BH18	3.80	5.34	1.54
LSB-SPI-BH1001	0.90	0.20	-0.70
LSB-SPI-VT-BH1004a	1.45	5.23	3.78
LSB-SPI-VT-BH1006	1.30	5.31	4.01
M4E-BH301	4.10	4.49	0.39
MT_BH14	10.40	18.36	7.96
MT_BH18	12.00	14.07	2.07
RZ_BH01s	1.80	1.14	-0.66
RZ_BH44s	0.90	0.45	-0.45
RZ_BH47s	1.10	0.38	-0.72

Well ID	Observed groundwater levels (m AHD)	Modelled groundwater levels (m)	Difference between observed and modelled levels (m)
RZ_BH49s	1.20	0.71	-0.49
SP_BH02_old	15.80	10.53	-5.27
SP_BH09_old	2.75	-0.96	-3.71
TC_BH01s	0.70	0.68	-0.02
TC_BH06s	1.10	0.97	-0.13
TC_BH07s	0.40	0.60	0.20
TC_BH08s	0.60	0.43	-0.17
TC_BH09s	0.80	0.47	-0.33
WCX-BH122	2.14	1.74	-0.40
WCX-BH157	-12.00	-8.72	3.28

Appendix BC

Hydraulic Conductivity Distribution within Model Layers

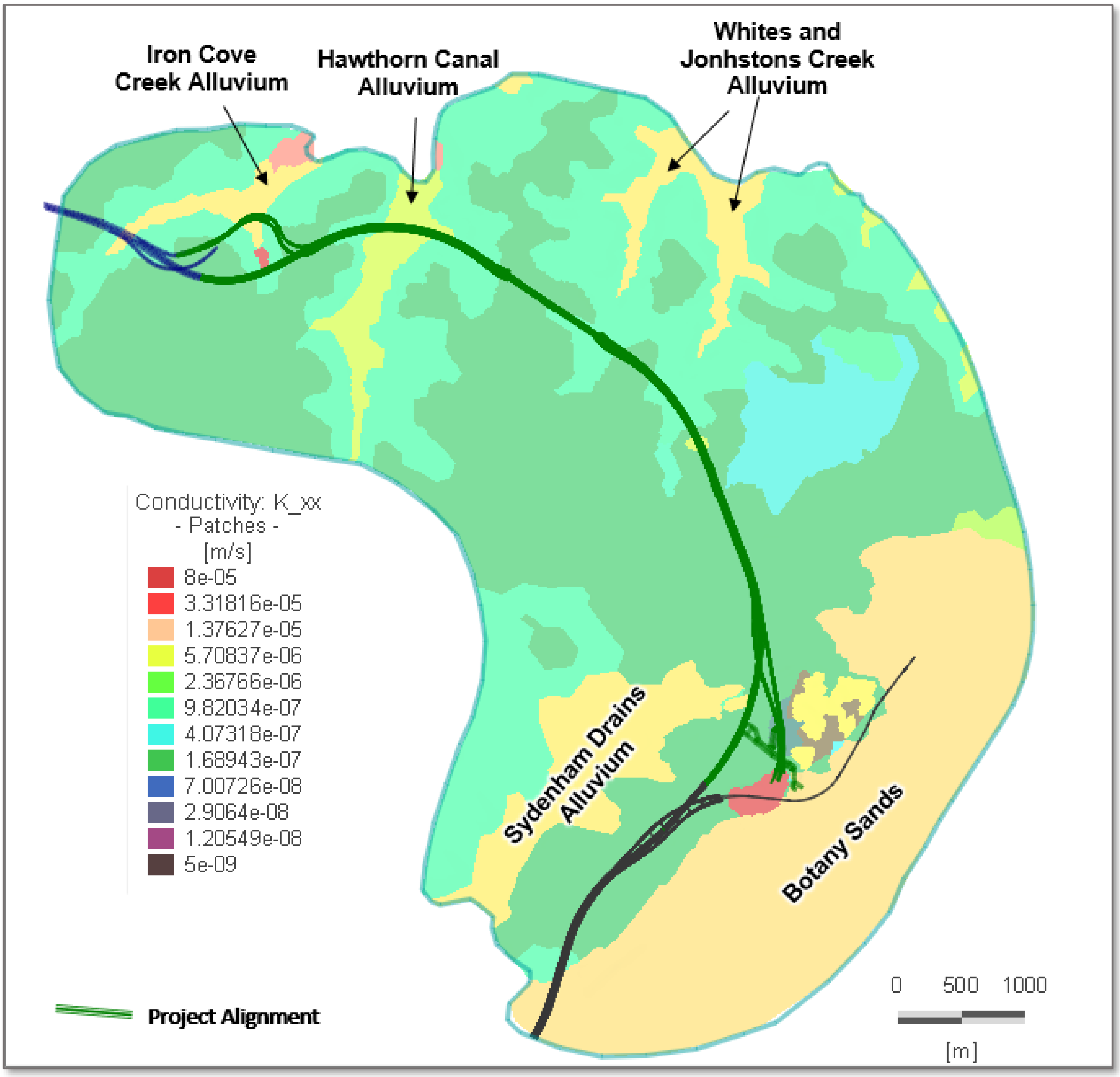


Figure BC1: Horizontal Hydraulic Conductivity Distribution within Layer 1

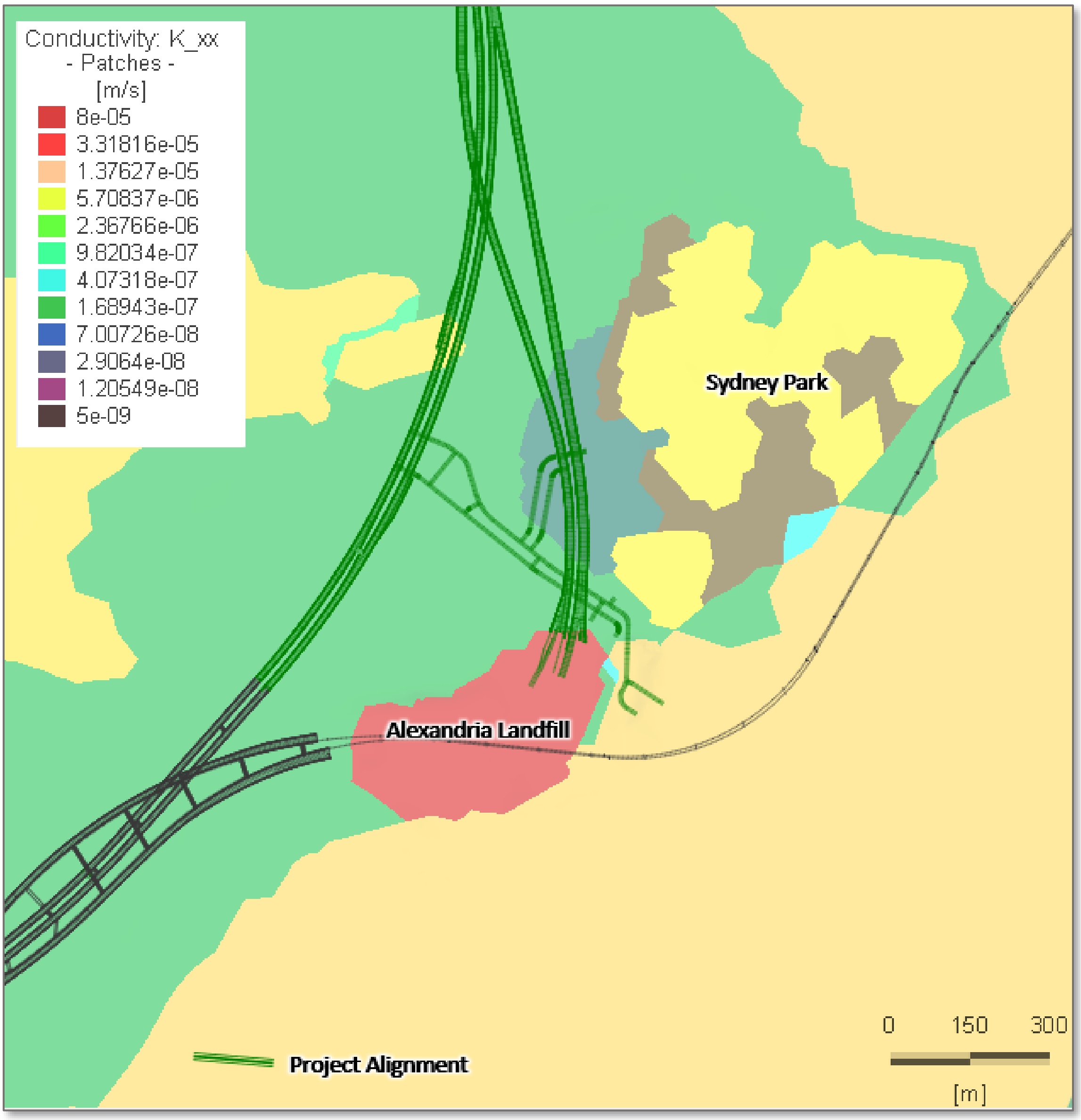


Figure BC2: Horizontal Hydraulic Conductivity Distribution within Layer 1, SPI Area Zoom-in

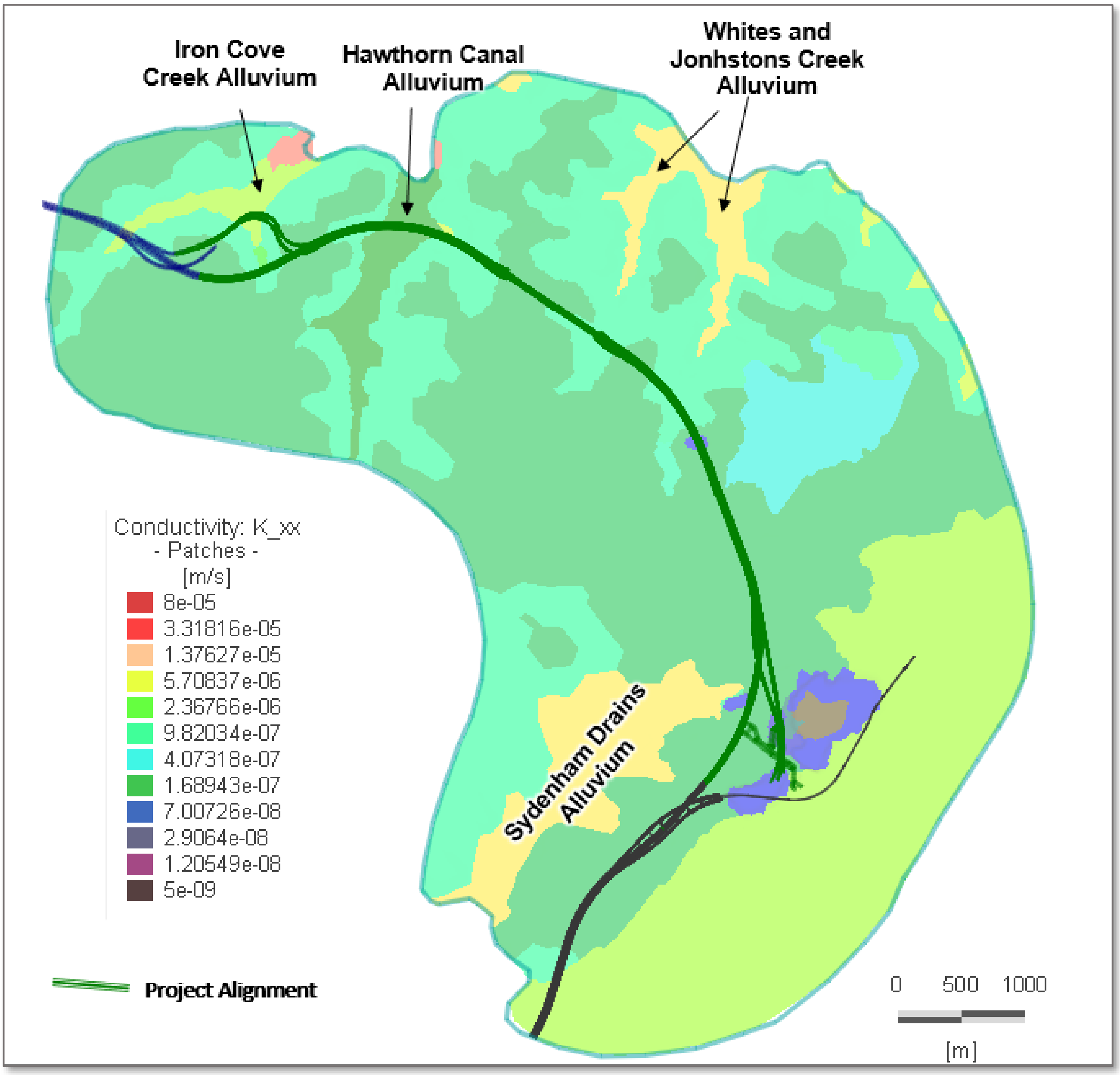


Figure BC3: Horizontal Hydraulic Conductivity Distribution within Layer 2

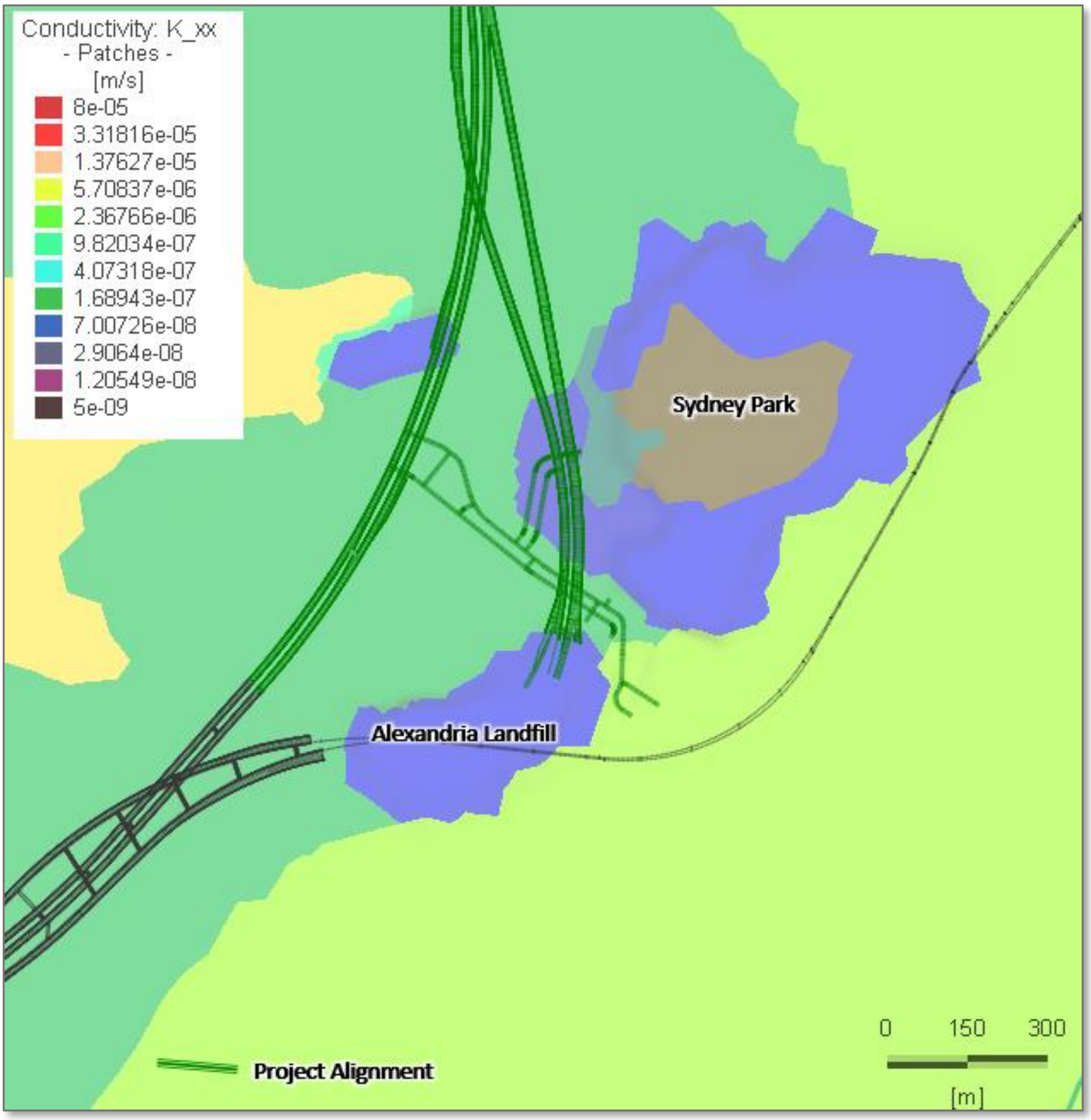


Figure BC4: Horizontal Hydraulic Conductivity Distribution within Layer 2, SPI Area Zoom-in

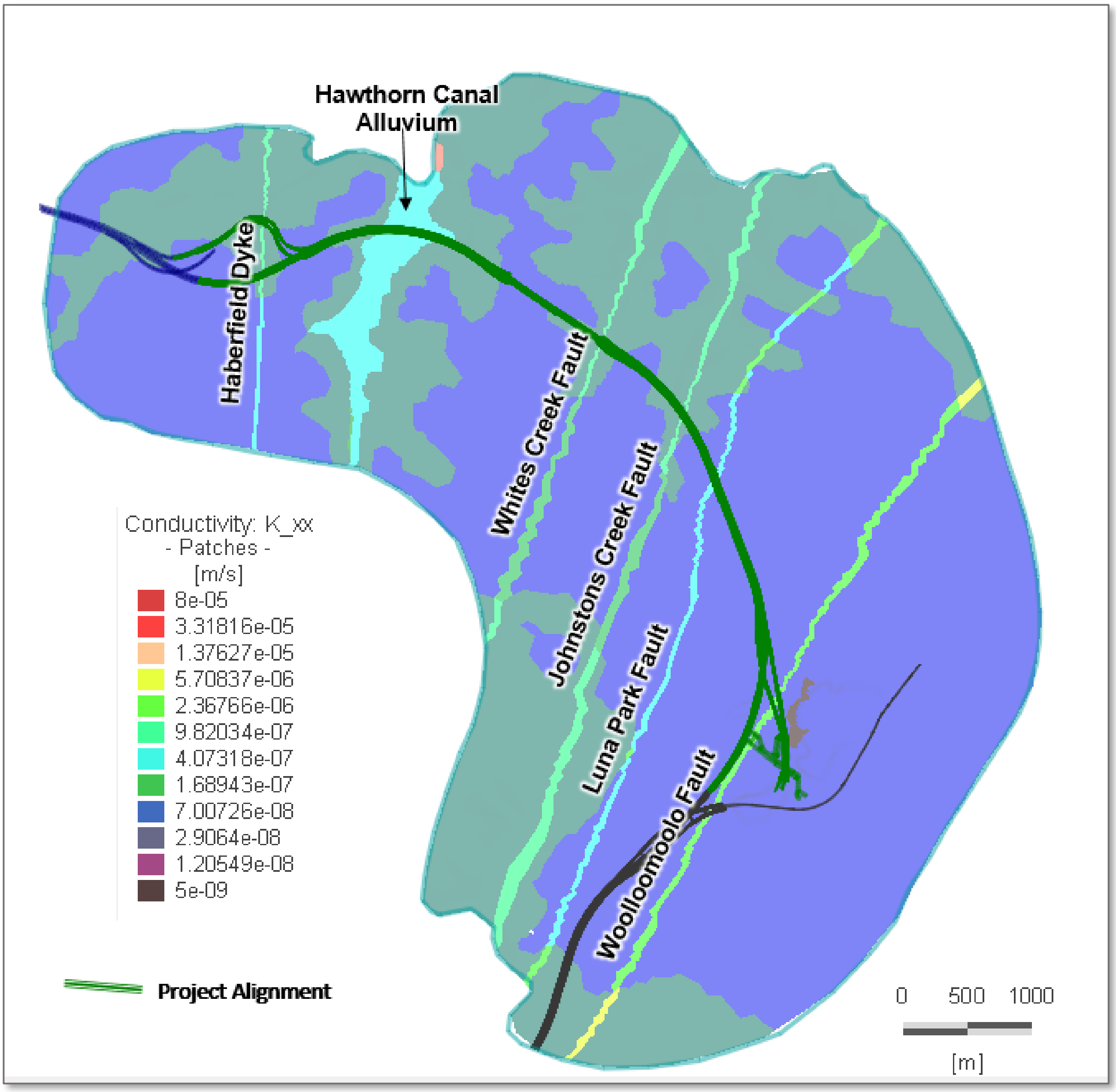


Figure BC5: Horizontal Hydraulic Conductivity Distribution within Layer 3

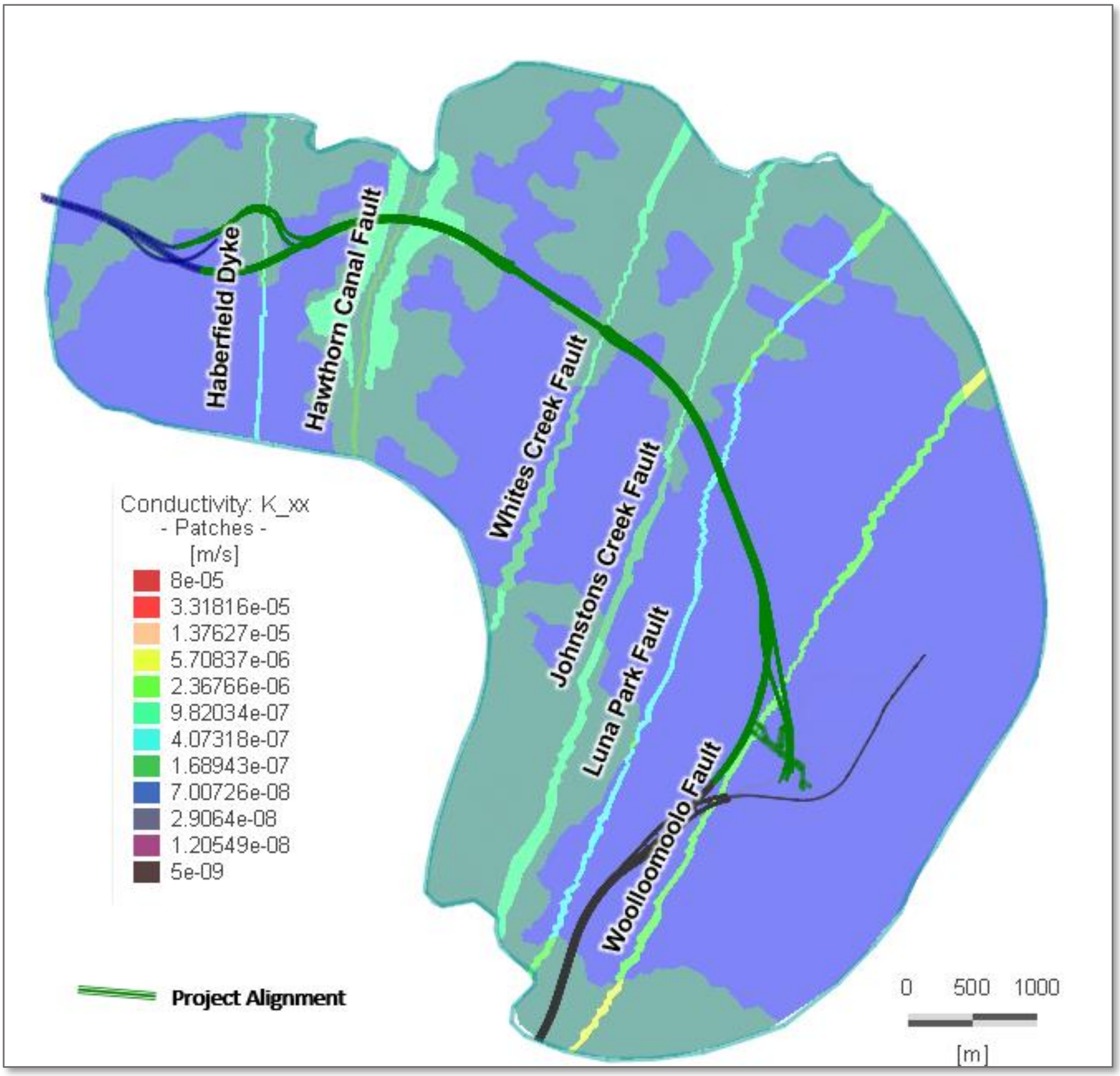


Figure BC6: Horizontal Hydraulic Conductivity Distribution within Layer 4

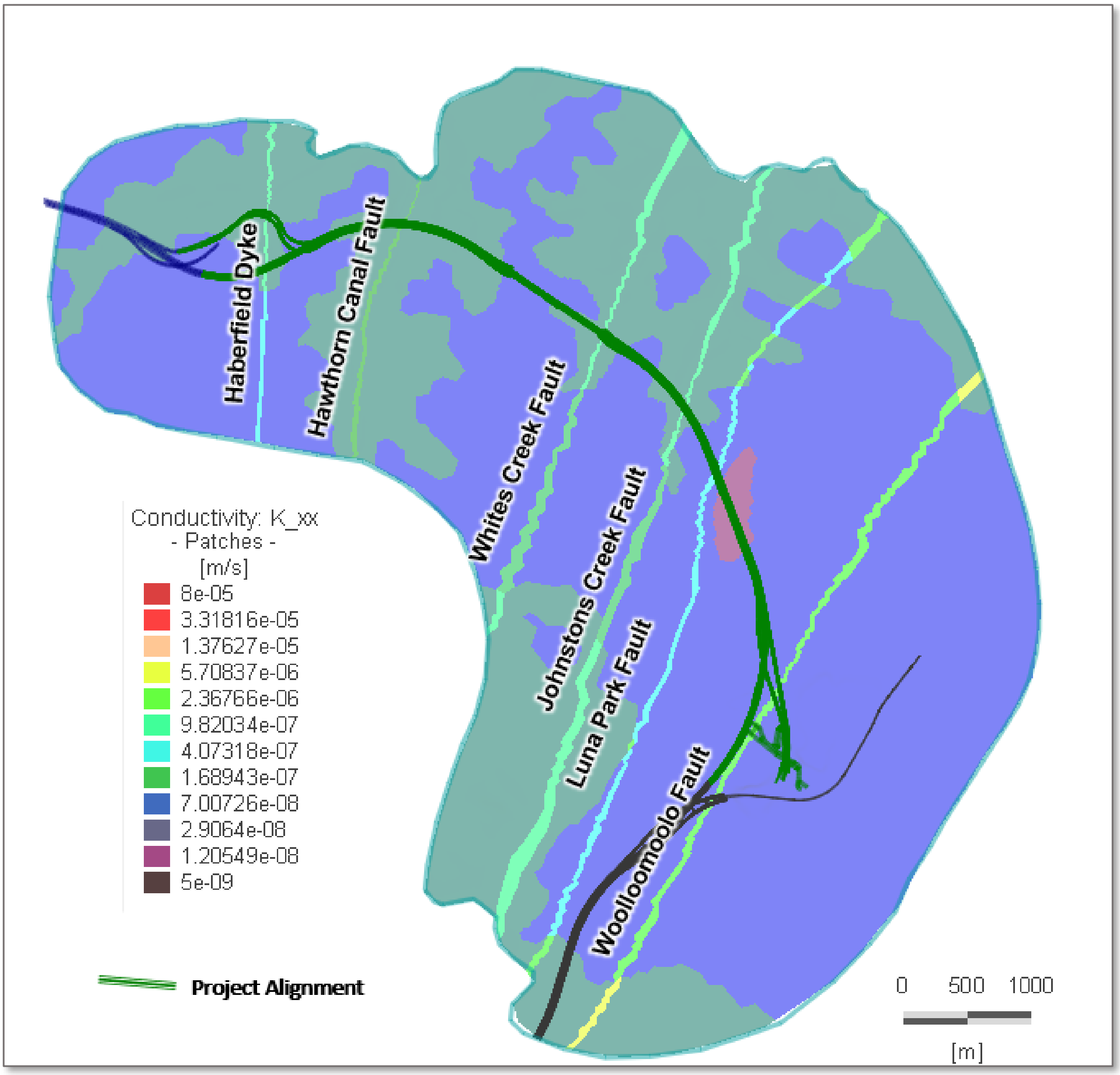


Figure BC7: Horizontal Hydraulic Conductivity Distribution within Layer 5

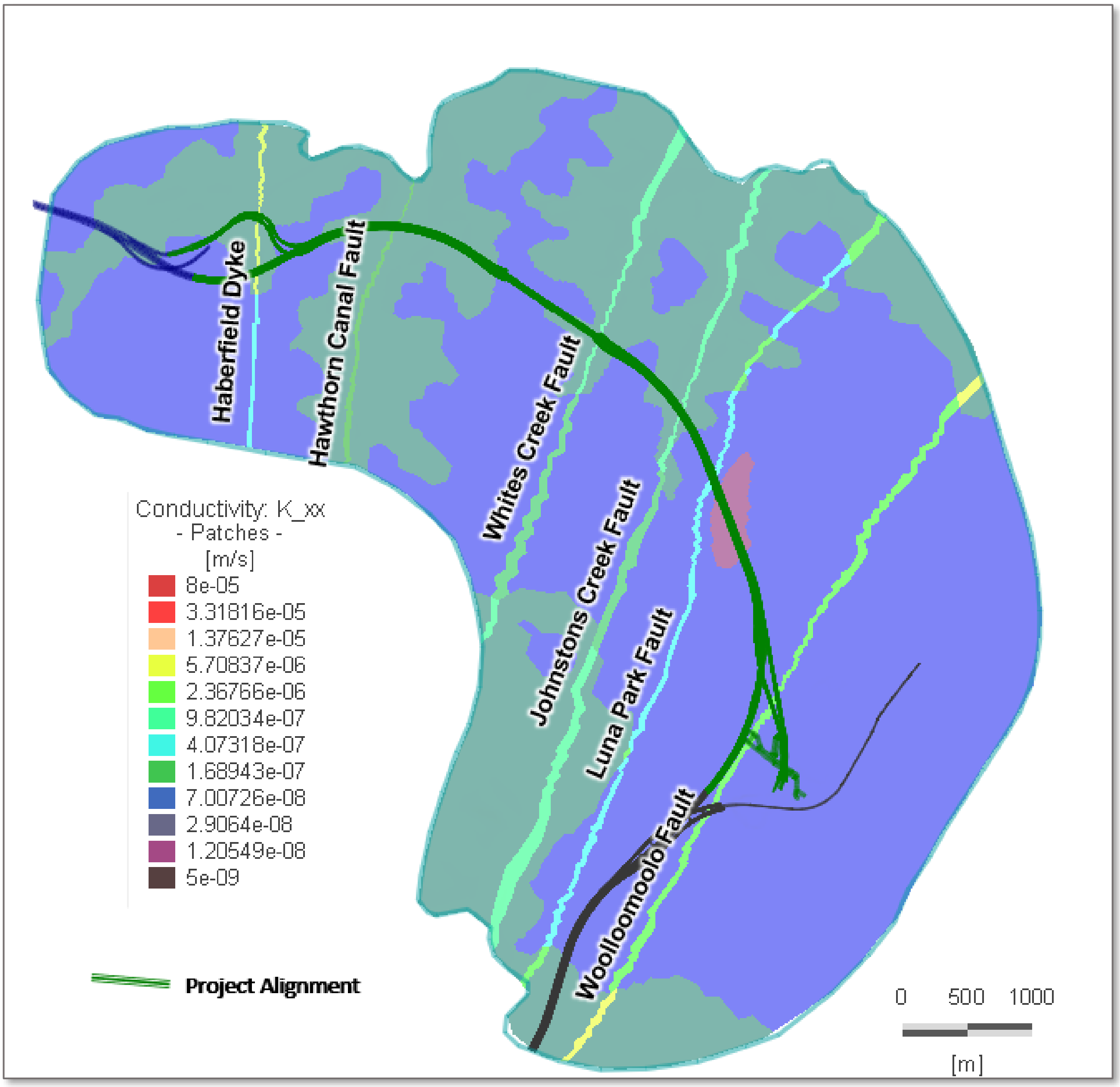


Figure BC8: Horizontal Hydraulic Conductivity Distribution within Layer 6

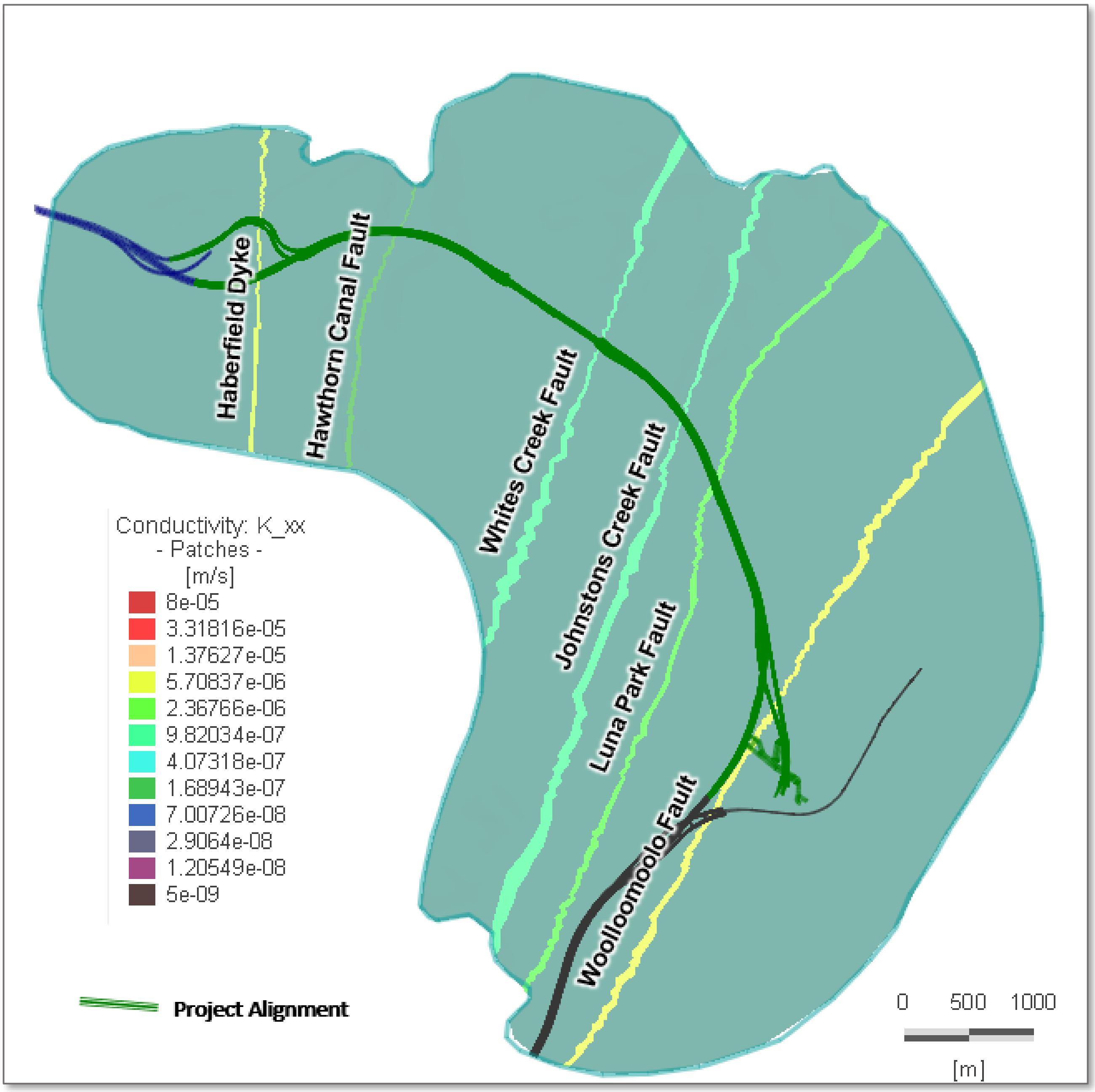


Figure BC9: Horizontal Hydraulic Conductivity Distribution within Layer 7

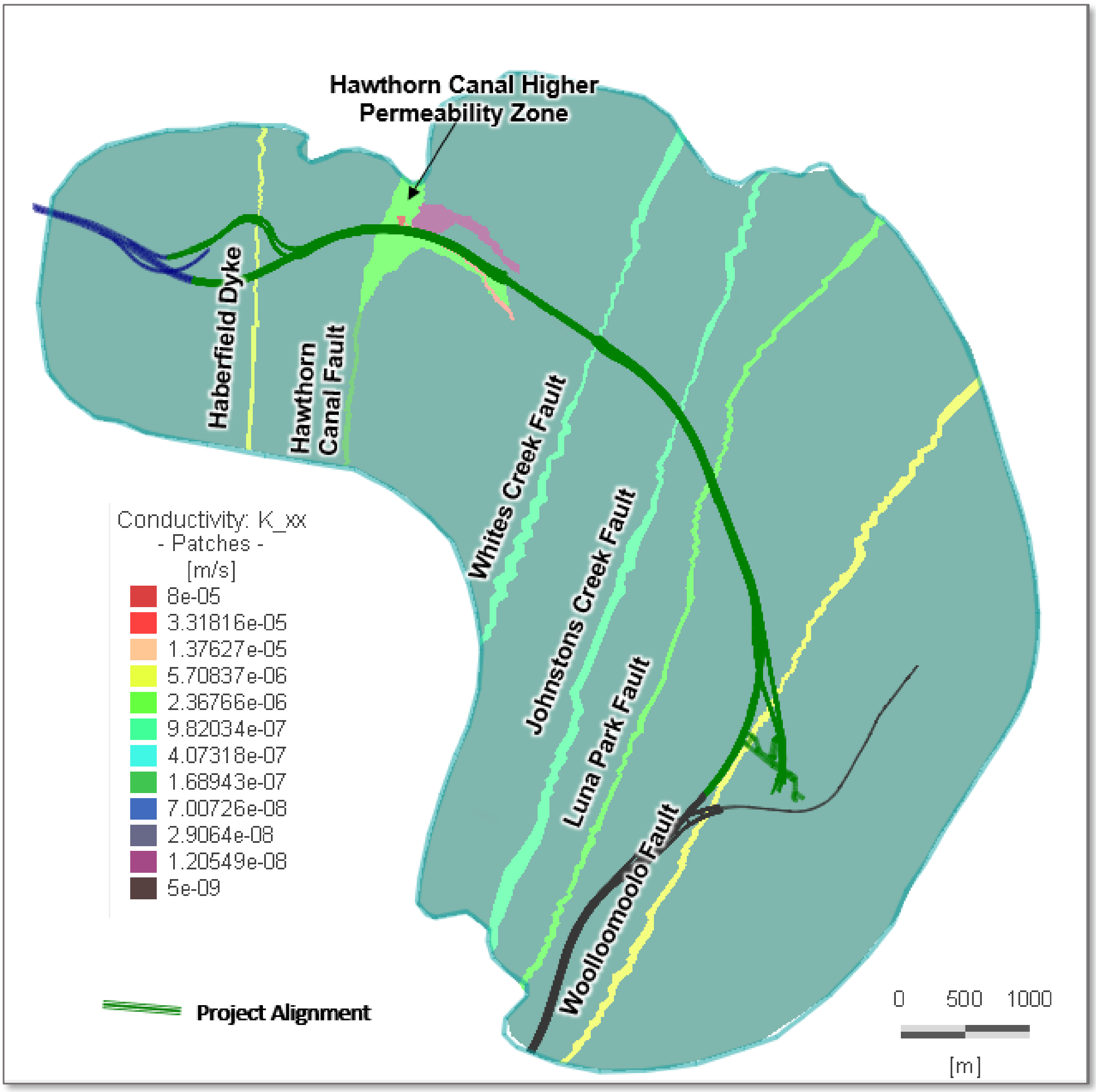


Figure BC10: Horizontal Hydraulic Conductivity Distribution within Layers 8 and 9

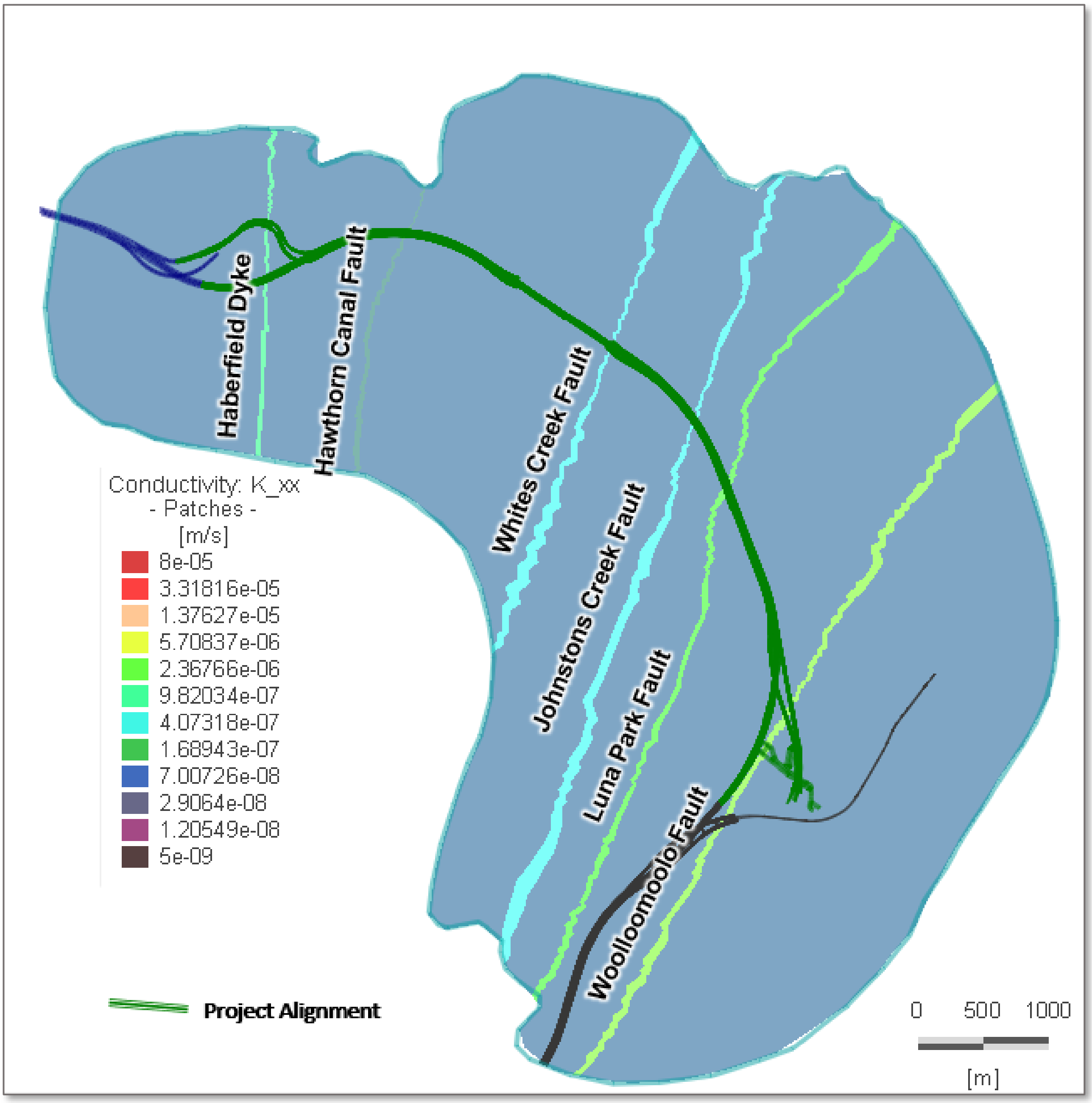


Figure BC11: Horizontal Hydraulic Conductivity Distribution within Layers 10

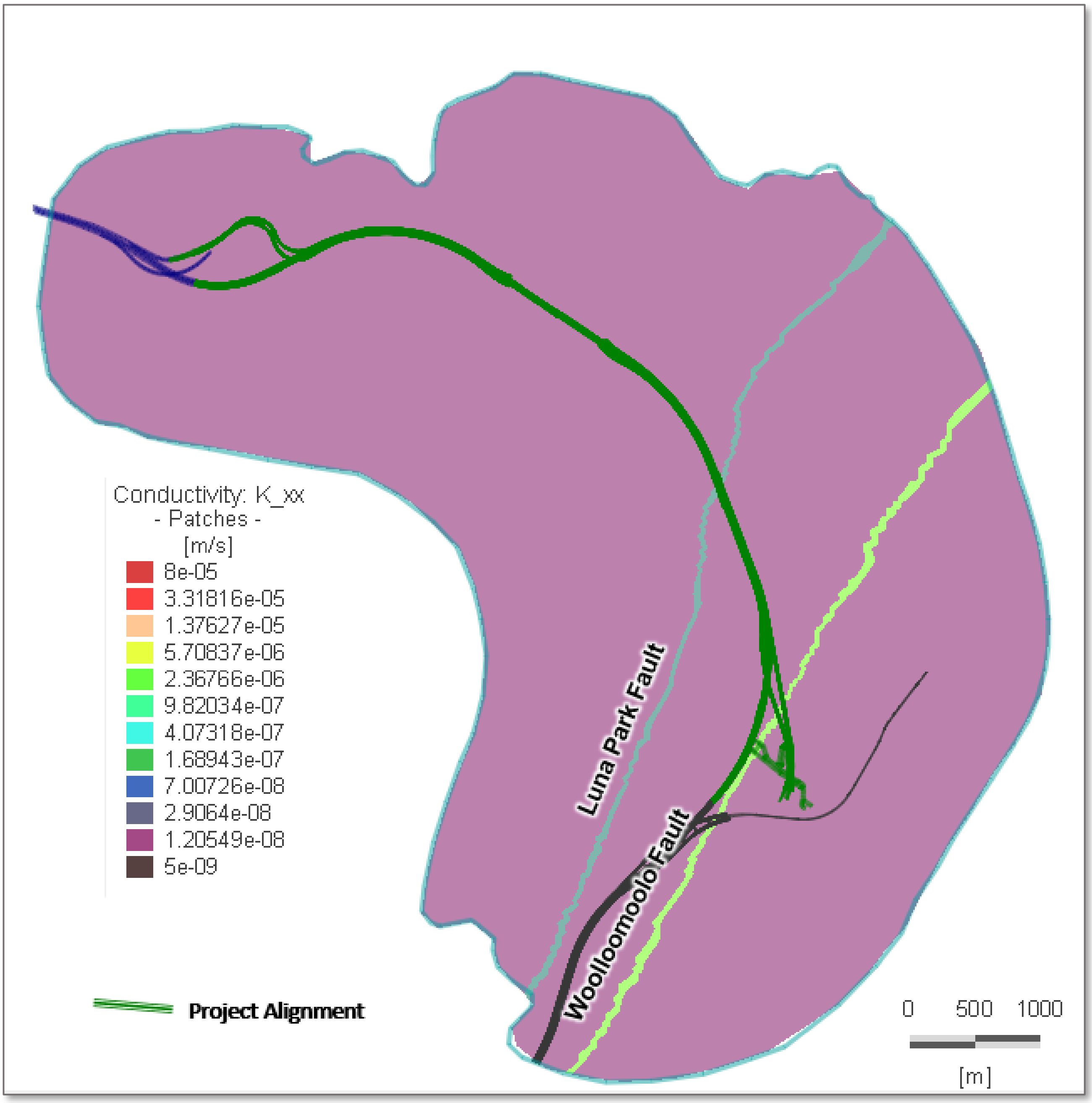


Figure BC12: Horizontal Hydraulic Conductivity Distribution within Layer 11

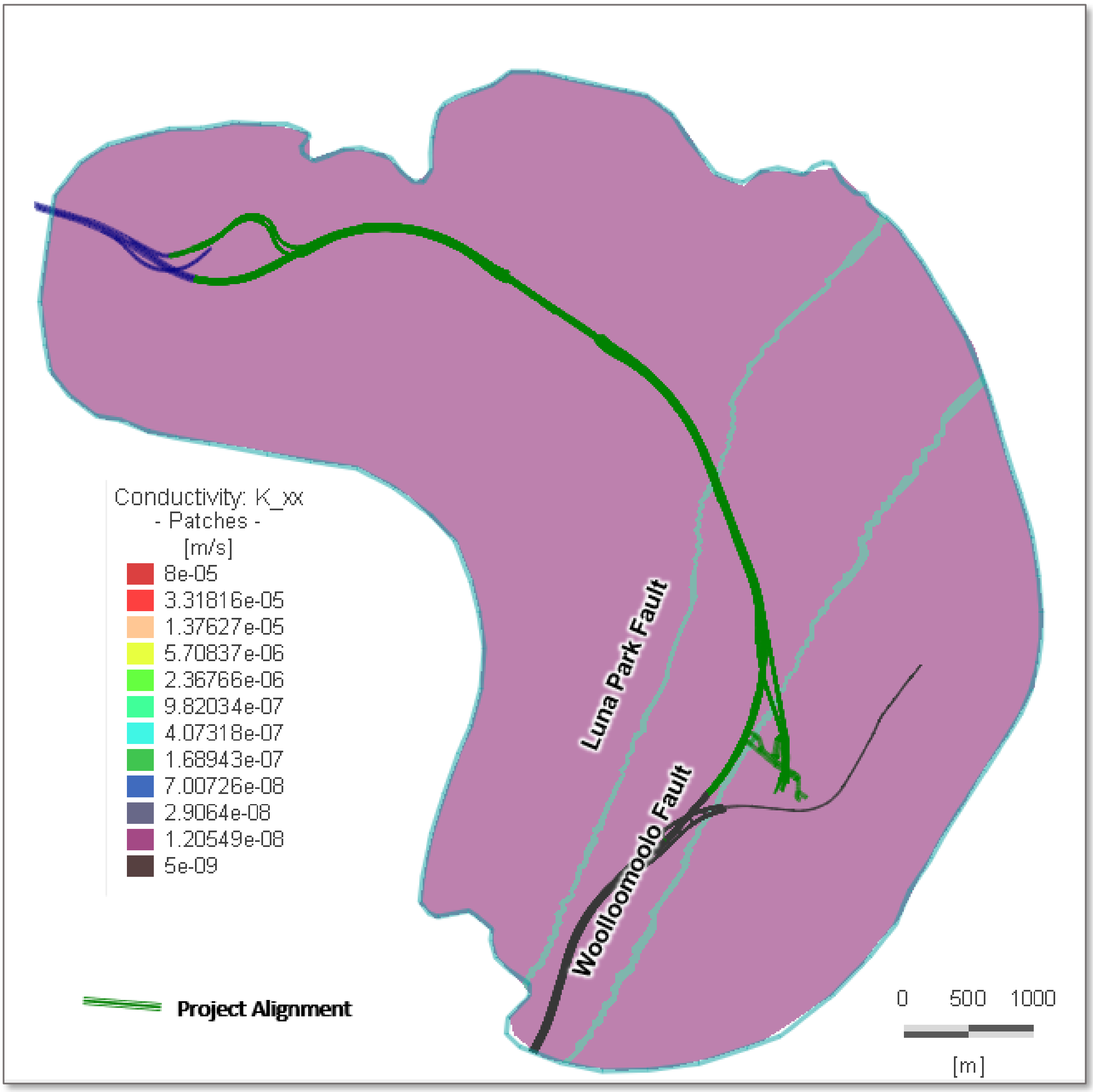


Figure BC13: Horizontal Hydraulic Conductivity Distribution within Layer 12

Appendix BD

Contaminant Transport Modelling

CONTAMINANT TRANSPORT MODELLING

Contaminant Transport Approach and Model Inputs

Contaminated Sources

A number of high-risk contaminant sources have been identified in the vicinity of the Project alignment, including the following former landfills that were developed within former shale quarries (brick pits):

- Alexandria Landfill
- Sydney Park
- Camdenville Park
- O'dea Reserve
- Algie Park.

These landfills have been backfilled with varying waste materials, including putrescible and industrial wastes, and were not lined at the time of landfilling. Available data indicate that the landfill leachate has impacted surrounding groundwater quality and construction and operation of the Project is likely to draw the landfill leachate towards the underground structures.

Contaminant transport modelling was undertaken to assess the potential effects of Project construction and operation on the movements of the landfill leachate. This information was also used to inform the durability assessment for tunnel structures, the tunnel gas risk assessment and for design of the water treatment plant.

Ammonia Transport Modelling

Several key contaminants of interest were identified in the landfill leachate and are discussed in the Contamination Assessment Report (M4M5-JAJV-PRW-GEO-CN01-RPT-0005). Groundwater drawdowns associated with the M4-M5 Link project will induced leachate to migrate into groundwater resulting in the groundwater quality being largely consistent with that of leachate.

Ammonia was selected as a representative species for the contaminant transport modelling and was used as a proxy for other solutes, i.e., to predict the range of contaminants expected to come out of the landfills and in contact with the project tunnels.

Given the unknown history of waste disposal as well as the age of the landfills (i.e. degradation of wastes / leaching of contaminants has been occurring for decades) longevity of the ammonia source concentrations is difficult to estimate without long term monitoring data. The following two scenarios were, therefore, considered in the model:

- Constant source: The use of constant source terms assumes that contaminants will continue to be generated at the current source concentrations from the waste via either degradation or leaching processes for the 100 year design life of the M4-M5 tunnels.
- Finite source: The use of finite source term assumes that further generation of contaminants from the waste either via degradation or leaching processes does not occur and that only contaminants within solution (current source concentrations within leachate within the landfills) were available to migrate.

It is considered that the modelling of the finite and constant source terms would provide a range that likely bounds the potential contaminant concentrations which may reach the individual tunnel elements over the design life of the structures.

The transport of ammonia in groundwater was modelled to occur through advection, dispersion and diffusion only (i.e. no sorption or degradation occurs in ammonia transport) as reactive transport would be generally different for each of the individual species. Although some attenuation of ammonia may occur along the flow path, the rate of attenuation is likely to be low based on the aquifer properties and the following conditions expected within the aquifers:

- Residual Ashfield Shale soils - Higher sorption through the residual clay horizons where the flow velocity is expected to be low, the ion exchange capacity of the soil is relatively high (but reduce over the time as soil become saturated by ammonia), and oxygen availability is likely to be high. However, these horizons are generally limited to shallower depths and the flow model indicated that the soils would be under-drained by the tunnels.

- Fractured Ashfield Shale rock – Benefits of sorption are expected to be considerably lower than through residual soils. The effects of sorption would be reduced by increased groundwater flow through the fractured zones and increased flow velocities induced by groundwater head gradients and low effective porosity of the fractured zones. The reduced contact surface area also means saturation will occur faster than it would in soils.
- Hawkesbury Sandstone – Benefits of sorption are expected to be nil due to limited organic carbon and cation exchange capacity. Flow velocity through the fractures would be relatively high which would further reduce attenuation.
- As ammonia contaminated groundwater moves from the landfill towards the tunnels it is expected that the lack of oxygen at depth within the groundwater system will restrict nitrification of ammonia and therefore, degradation is likely to be limited.

In the model the source of high ammonia is contained within Layer 1 (landfill pits). As drainage of tunnel commences, ammonia contained within the landfill moves laterally and vertically through the model layers towards the model nodes representing seepage points along the tunnels (nodes with the fixed head boundary conditions assigned at the tunnel invert levels). The model then calculates groundwater seepage rate, mass flux and concentrations of ammonia at each model node at the tunnels.

The modelling approach adopted was considered to be appropriate in the context of the design life of the tunnels and to be consistent with reviewed published literature relating to the likely behaviours of key contaminants associated with landfills including ammonia (Buss et al, 2004[6] and Kjeldsen et al, 2010[7]).

Scenarios Simulated and Model Inputs

Modelling Scenarios

A number of modelling scenarios were simulated to enable assessment of ammonia flow-paths from individual landfills (Pathway Scenarios) and ammonia concentrations likely to reach the individual tunnel structures due to the combined effect of all of the landfills (i.e. sources) within the St Peters area (Base Scenario). Two Base Scenario model simulations were undertaken adopting the finite and constant ammonia source conditions. The Pathway Scenarios were only simulated for the constant ammonia source condition.

The main objective of the Pathway Scenarios was to explore which contaminant sources would likely impact each tunnel design package. These scenarios also provided an insight into percentages of the source contaminant concentrations likely to reach the tunnel during the tunnel operational phase. The Pathway Scenarios included simulation of individual landfills only (i.e. a separate model was run for each of the landfills). Results from these simulations enabled estimation of percentages of source concentration that will arrive at the model nodes representing the individual tunnel structures from each landfill.

The objective of the Base Scenario was to assess the combine effect of all landfills identified within the Project alignment.

Several scenarios were also simulated to explore the sensitivity of the contaminate transport prediction to effective porosity, dispersity values, in tunnel grouting and variation in the geological setting. These scenarios are discussed in Attachment 1.

A summary of the contaminant transport modelling scenarios simulated is provided in Table BD1 below.

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6 S. R. Buss, A. W. Herbert, P. Morgan, et al. " A review of ammonium attenuation in soil and groundwater" Quarterly Journal of Engineering Geology and Hydrogeology 2004; v. 37; p. 347-359

7 Peter Kjeldsen , Morton A. Barlaz , Alix P. Rooker , Anders Baun , Anna Ledin & Thomas H. Christensen "Present and Long-Term Composition of MSW Landfill Leachate: A Review" Critical Reviews in Environmental Science and Technology, 32:4, 297-336, published 3 June 2010

Table BD:1 Summary of Modelling Scenarios

Modelling Scenario	Description	Objective
Preliminary Flow Model Scenarios with Conventional Contaminant Transport Parameters		
Landfill Pathway Scenario (constant ammonia source)	Individual landfill simulated in separate model runs. (a total of eight individual simulations). Nominal concentrations of 100 mg/L assigned to each landfill.	Assessment of ammonia flow-paths from individual landfills and percentages of the source concentrations likely to reach the tunnels.
Combined Landfill Base Scenario (finite and constant ammonia source)	All landfills were simulated. Ammonia (as N) concentrations based on the maximum ammonia concentration observed in the Alexandria Landfill and the most recent concentrations observed in other landfills (i.e. source). Woolloomooloo Fault grouted to 0.5 Lu at intersection with the PS21 and Hawthorne Canal Fault grouted to 1 Lu at intersection with the mainline tunnels.	Assessment of the combine effect of all high-risk landfills identified within the Project corridor assuming the ammonia (as N) concentrations adopted for each source will persist over 100 year simulation.
Sensitivity Analysis Scenarios (constant ammonia source only) – Included in Appendix BD-A		
No Grouting at Woolloomooloo Fault Zone (No Grouting Option Scenario)	No grouting of PS21 at the Woolloomooloo Fault intersection. The remainder of the model the same as in Base Scenario	To assess the potential for reduce extents of ammonia impacted tunnel by not grouting the WFZ and exceeding inflow criteria within PS21.
Impact of Dyke at SPI (Dyke Scenario)	A low permeability dyke postulated north of the ventilation tunnels (a barrier to flow) with the remainder of the model the same as in Base Scenario	Assess impacts of changes to the assumed geological setting on ammonia distribution and groundwater inflows.
Higher Dispersivity Scenario.	Base Scenario with longitudinal dispersivity increased from 10 m to 50 m and a transverse dispersivity from 1 m to 5 m.	Assess sensitivity of the model results to changes in adopted dispersivity values.
Low Effective Porosity Scenario	Base Scenario with effective porosity of the rock aquifer reduced to one half and one fifth of original values.	Assess sensitivity of the model results to changes in adopted effective porosity and consequently higher flow velocities.

Contaminant Transport Parameters and Source Concentrations

Given the size of Sydney Park and proximity to the tunnel, Sydney Park was broken into sub-source areas to reflect the varying ammonia concentrations observed in wells sampled within the park. The sub-areas shown below within Sydney Park were based the spatial extents of the former brick pits, established from historic aerial photographs. The sub-source areas included:

- South West Pit (SW P1)
- Central East Pit 1 and Pit 2 (CE P1 and CE P2)
- Central West Pit 1 to Pit 3 (CW P1 to CW P3)
- North Side Pit 1 and Pit 2 (NS P1 and NS P2).

Alexandria Landfill, Camdenvile Park, Algie Park and O'Dea Reserve were all modelled as single source areas.

For the Base Scenario the maximum historical ammonia (as nitrogen) concentrations reported were adopted for Alexandria Landfill and Camdenville Park. Sydney Park ammonia (as nitrogen) concentrations were adopted based on recent investigations completed by LSBJV (LSB-SP-BH14 to LSB-SP-BH19). A nominal concentration of 100 mg/L was adopted for the Algie Park and O'Dea Reserve based on available data and the age of these landfills.

The concentrations assumed for each of the source area were as follows:

- Alexandria Landfill – 400 mg/L
- Camdenville Park – 40 mg/L
- Algie Park – 100 m/L
- O'Dea Reserve – 100 mg/L
- Sydney Park SW P1 – 800 mg/L
- Sydney Park CE P1 and CE P2 – 1.4 mg/L
- Sydney Park CW P3 – 700 mg/L
- Sydney Park CW P3 – 1,200 mg/L
- Sydney Park NS P1 and NS P2 – 300 mg/L

Outlines of the Sydney Park pits and ammonia (as nitrogen) source concentrations are shown in Figure BD:1.

Figure BD:1 Ammonia as Nitrogen Concentration Distribution Adopted for Sydney Park Pits in the Base Scenario

The source of contamination is contained within the Layer 1 of the model. The elevation of the base of Layer 1 within Sydney Park is about RL -26 m AHD, which is about 16 m to 18 m above the mainline tunnel invert and invert of the deeper (western) section of the ventilation tunnels. The invert level of the main line tunnels and deeper of the ventilation tunnels corresponds to the top of model Layer 9.

Based on ammonia transport assumptions, the contaminant transport parameters adopted in the model were:

- Longitudinal dispersivity of 20 m[8]
- Transverse dispersivity of 2 m
- Molecular diffusion of 1 x 10^{-9} m^2/s.

The effective porosity values used for the contaminant transport assessment were assumed to be equal to the specific yield values used for the fluid flow simulation, as follows:

- Landfill waste – 0.2
- Alluvial sediments and Botany Sands 0.15
- Clayey Soils around Sydney Park – extremely weathered Ashfield Shale – 0.12
- Reworked and mixed Botany Sands and clays within the eastern Sydney Park (around former brick pits) – 0.18
- Residual Soil – Hawkesbury Sandstone; Hawkesbury Sandstone – 0.01
- Ashfield Shale - 0.05
- Faults and Dykes – 0.04

Fixed concentrations boundary conditions were used to simulate ammonia concentrations in the landfill leachate for the constant ammonia source scenario.

For the finite ammonia source scenario, mass concentrations were assigned to the nodes within the source zone at the model time of 0 day and then dilution of the concentrations was allowed to occur (i.e., depletion of the source contaminant mass) within the modelled landfills due to rainfall infiltration and groundwater flow through Sydney Park.

Pathways Scenarios Results

Table BD:2 lists the landfill sites and identifies design packages likely to be impacted by each of the sources based on the Pathways Scenarios modelling. The design packages within the St Peters area are shown in Figure BD:2. For the Pathways Scenarios flow conditions, contaminant transport modelling suggests that groundwater flow and seepage into the tunnels will be governed by the deeper sections of the mainline and ventilation tunnels, resulting in the contamination originating from Sydney Park passing under the shallower (southern) sections of the St Peters ramps. The results of the modelling also suggest that contaminants from the Sydney Park Central East and North Side pits are unlikely to reach the tunnels over the 100 year period as simulated by the Pathway Scenarios.

Table BD:2 Contaminant Sources and Design Packages Likely to be Impacted

Package ID		Alexandria Landfill	Sydney Park				Camdenville Park	O'Dea Reserve	Algie Park
			South West (SW)	Central East (CE)	Central West (CW)	North Side (NS)			
SPI	M190 M180				yes				
	PS08		yes						

[8] Adopted based on Longitudinal dispersivity versus scale of observation graph included in C. Zeng, and G. D. Bennett: Applied Contaminant Transport Modeling: Theory and Practice, 1995 (after Gelhar at al, 1992: A Critical review of data on field-scale dispersion in aquifers. Water Resours. Res. Vol 28, no 7, pp 1955-1974)

Package ID		Alexandria Landfill	Sydney Park South West (SW)	Sydney Park Central East (CE)	Sydney Park Central West (CW)	Sydney Park North Side (NS)	Camdenville Park	O'Dea Reserve	Algie Park
Vent. Tunnels	PS21	yes	yes		yes				
PS09	M120	yes			yes				
	M110						yes		
PS10	M120				yes				
	M110						yes		
PS02	M120							yes	
	M110							yes	
PS16	M120								yes
	M110								yes

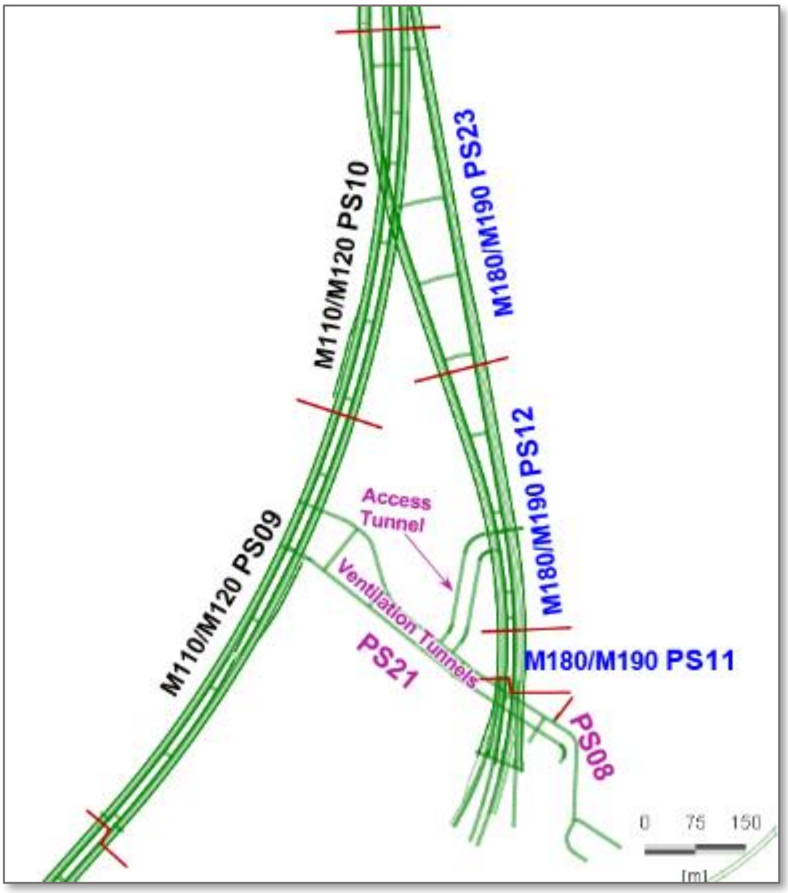


Figure BD:2 Design Packages within the St Peters Area

St Peters Area Base Scenario Results

Finite Ammonia Source

This modelling scenario assumes that no further generation of ammonia will occur at Sydney Park, Camdenville Park and Alexandria Landfill after tunnel construction, which is equal to time 0 days in the model.

The ammonia (as nitrogen) concentration distribution within the rock aquifer 50 years after tunnel construction is shown in Figure BD:3 and 100 years after tunnel construction in Figure BD:4. The contours shown in these figures represent ammonia (as nitrogen) concentrations in groundwater seepage, which are predicted to be collected by the mainline tunnels, ventilation tunnels and SPI ramps drainage system in the St Peters area.

As shown in Figure BD:3 and Figure BD:4 greatest impacts on the mainline and ventilation tunnels will be from contaminants within the Sydney Park Central West and South West pits leachate. This is to be expected as the highest ammonia concentrations were reported at these pits. The finite source results suggest that the length of tunnels and ramps sections within the St Peters area that could be exposed to ammonia (as nitrogen) greater than 250 mg/L would vary over the time. Overall, the extent of the 250 mg/L ammonia (as nitrogen) concentration impact is predicted to increase up to 40 years to 60 years after construction and then start to decline due to source mass/concentration depletion.

With respect to the SPI Ramps, the length of ammonia effected sections will depend on the extent of seepage into the tunnel. The model indicates seepage extents will reduced over time due to mainline tunnel under drainage. After first arrival, contaminant concentrations are estimated to increase before declining as the contaminant source depletes. The model predicts that the maximum length of the SPI Ramp exposed to ammonia (as nitrogen) concentrations at or above 250 mg/L will be around 45 years after construction.

Results for 50 years and 100 years after construction suggest the following tunnel section lengths could be exposed to ammonia (as nitrogen) concentrations greater than 250 mg/L assuming a finite ammonia source:

- 50 years – about 1,050 m in length of the ventilation tunnel including cross passages, 220 m in length of the M120 tunnel and 150 m of the SPI Ramp M190
- 100 years – about 800 m in length of the ventilation tunnel including cross passages, 190 m in length of the M120 tunnel and 100 m of the SPI Ramp M190.

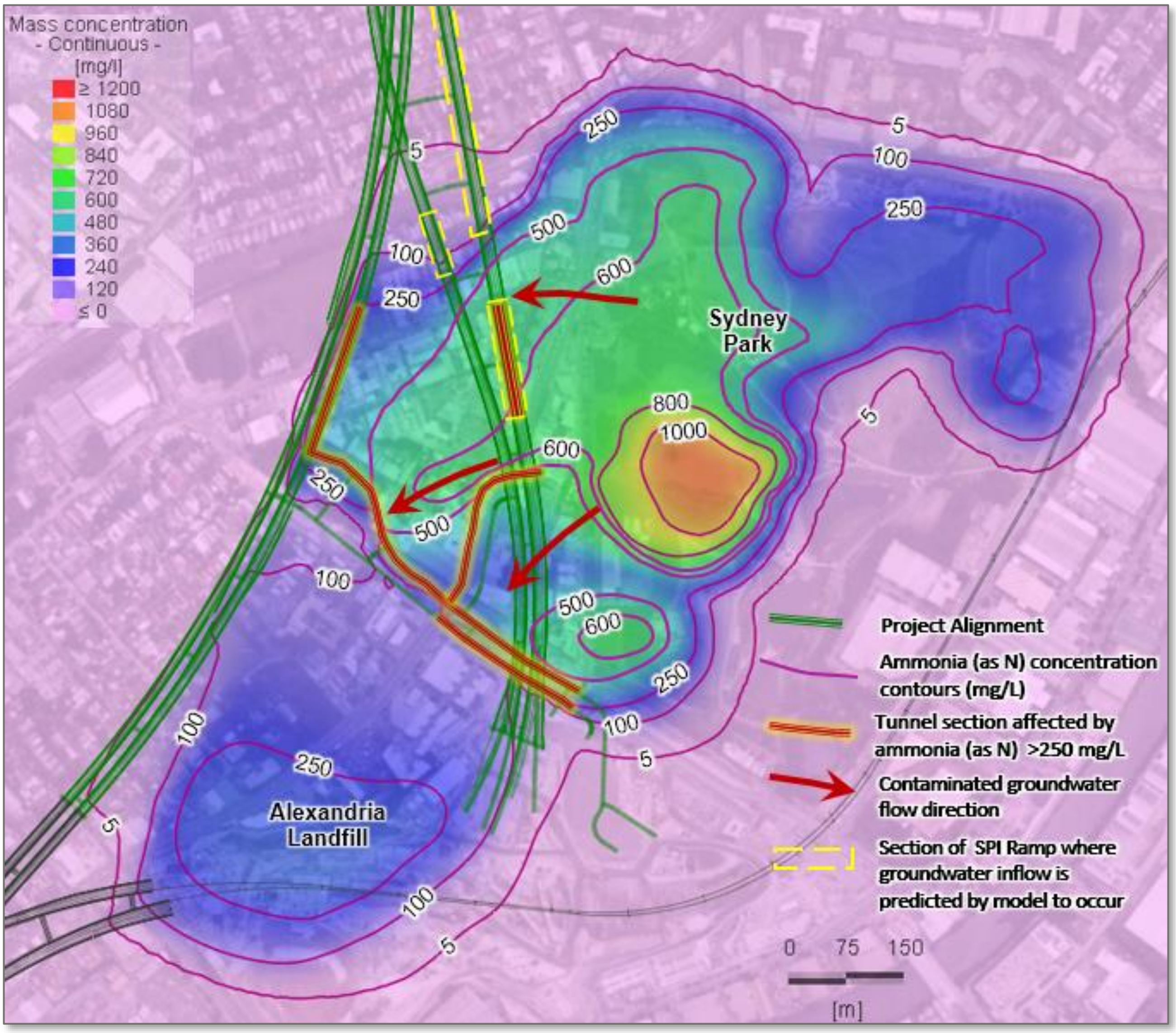


Figure BD:3 Ammonia (as Nitrogen) Concentration Distribution within the Rock Aquifer at the Mainline and Ventilation Tunnels Level 50 Years after Project Construction, Base Scenario – Finite Ammonia Source

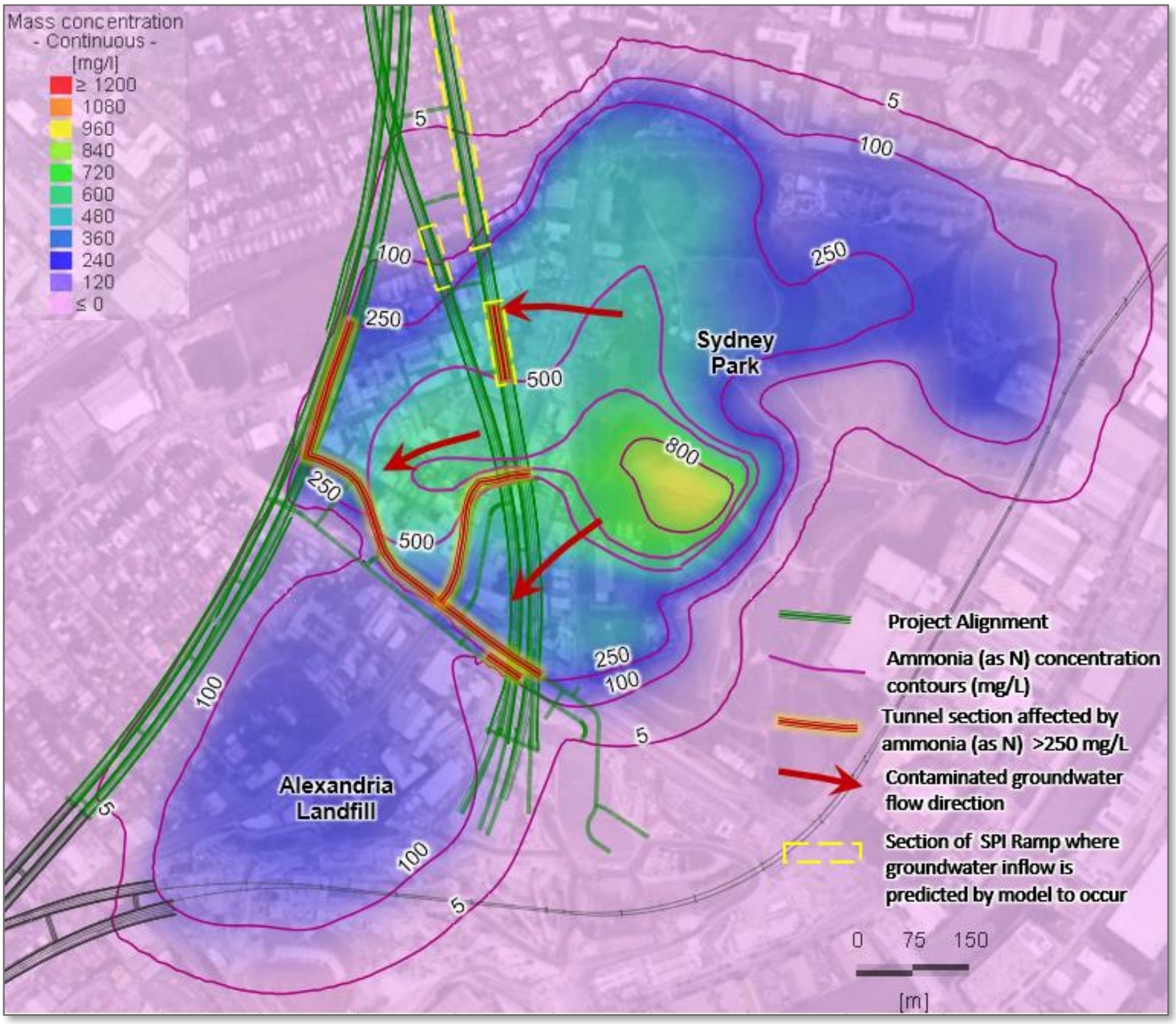


Figure BD:4 Ammonia (as Nitrogen) Concentration Distribution within the Rock Aquifer at the Mainline and Ventilation Tunnels Level 100 Years after Project Construction, Base Scenario – Finite Ammonia Source

Contamination arrival time and time to maximum concentration are both dependent on the distance between the relevant source and the tunnel. An overall summary of times to ammonia arrival and maximum concentration for the finite ammonia source is provided in Table BD:3.

Further discussion related to the solute breakthrough patterns including comparison between finite and constant source breakthrough patterns is included in Section *Summary of Contaminant Transport Modelling*.

Table BD:3 Concentrations Breakthrough, Finite Source Base Scenario Model

Package ID	Approximate NH3 Arriving Time	Time to NH3 greater than 250 mg/L	Time to Maximum NH3
Mainline Tunnels (PS09 and PS10)	3 to 10 years	24 to 27 years	45 to 65 years
Ventilation Tunnels (PS21)	1 to 9 years	4 to 30 years	25 to 80 years
SPI Ramps	3 to 5 years	16 to 33 years	40 to 55 years

Contamination originating from Camdenville Park, which is located close to the mainline will emerge in the mainline tunnels faster than the contamination originating from more distant sources, i.e. Alexandria Landfill and Sydney Park. The timing depends on vertical distances between the tunnels and the base of the landfills as well as the hydraulic properties of the hydrostratigraphic units separating the tunnels from the landfill, including presence of vertically orientated defects within the rock. The modelling results suggest that the time for contamination originated from the Camdenville Park landfill is likely to be in a range of about 2 years after tunnel construction.

In addition to distances from the sources, the concentration breakthrough pattern is dependent on the aquifer properties and, in particular, the effective porosity, dispersion and reactive transport assumptions in the model. A sensitivity analysis related to effective porosity, higher dispersivity and geological setting was undertaken to provide further insight into sensitivity of the model's prediction to the input values. These scenarios are discussed in Section *Sensitivity Analysis* and Attachment 1.

Additionally, the concentrations breakthrough pattern will also depend on the construction sequences, which will be taken into consideration in the FD modelling work.

Constant Ammonia Source

This modelling scenario assumes that maximum ammonia concentrations will persist within all sources over the 100 year design life of the Project.

The ammonia (as nitrogen) concentration distribution within the rock aquifer 50 years after tunnel construction is shown in Figure BD:5 and 100 years after tunnel construction in Figure BD:6. The contours shown in these figures represent ammonia (as nitrogen) concentrations in groundwater seepage, which are predicted to be collected by the mainline tunnels, ventilation tunnels and SPI ramps drainage system in the St Peters area, assuming maximum ammonia concentrations would persist within the Sydney Park pits, Camdenville Park and Alexandria landfills over design life of the project.

As shown in Figure BD:5 and Figure BD:6, the greatest impacts on the mainline and ventilation tunnels will be from contaminants within the Sydney Park Central West and South West pits leachate. This is to be expected as the highest ammonia concentrations were reported at these pits. The finite source results suggest that the length of tunnels and ramps sections within the St Peters area that could be exposed to ammonia (as nitrogen) greater than 250 mg/L would vary over the time. Overall, the extent of the 250 mg/L ammonia (as nitrogen) concentration impact increases up to around 50 years to 65 years and then remains relatively constant.

Results for time periods of 50 years and 100 years after construction suggest the following tunnel section lengths could be exposed to the ammonia (as nitrogen) concentrations greater than 250 mg/L assuming finite ammonia source:

- 50 years – about 1,080 m in length of the ventilation tunnel including cross passages, 250 m in length of the M120 tunnel and 150 m of the SPI Ramp M190
- 100 years – about 1,100 m in length of the ventilation tunnel including cross passages, 270 m in length of the M120 tunnel and 100 m of the SPI Ramp M190.

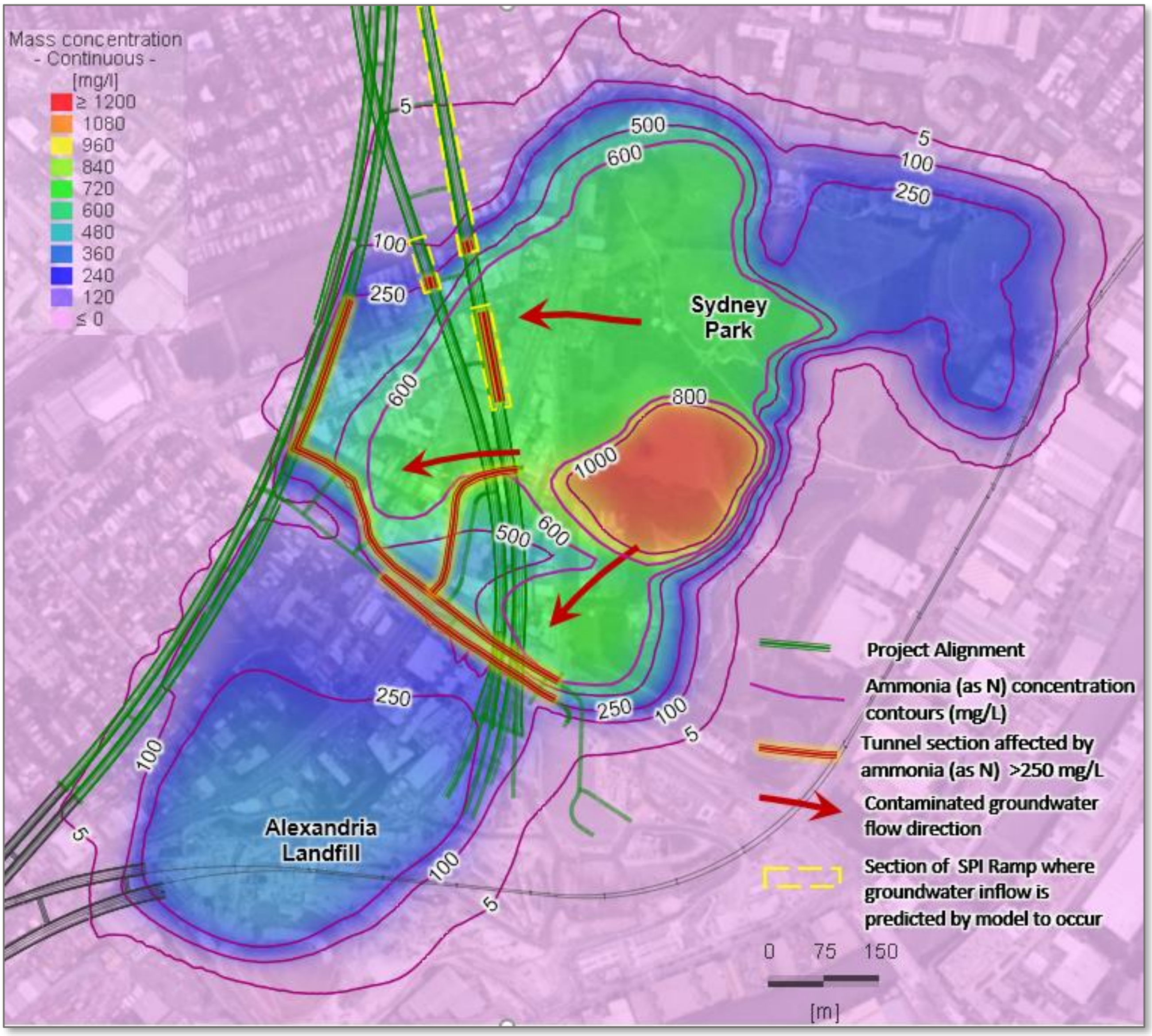


Figure BD:5 Ammonia (as Nitrogen) Concentration Distribution within the Rock Aquifer at the Mainline and Ventilation Tunnels Level 50 Years after Project Construction, Base Scenario – Constant Ammonia Source

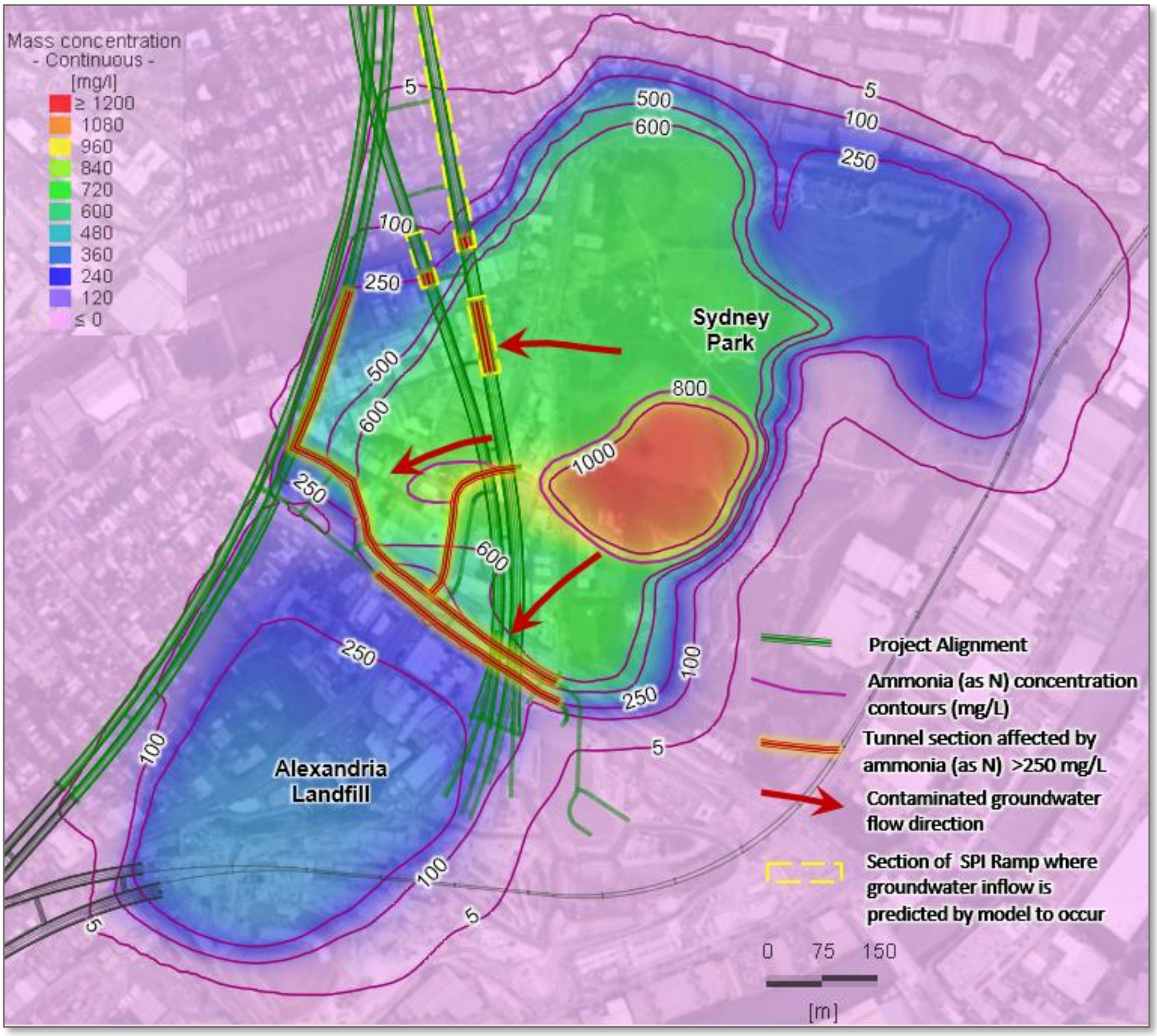


Figure BD:6 Ammonia (as Nitrogen) Concentration Distribution within the Rock Aquifer at the Mainline and Ventilation Tunnels Level 50 Years after Project Construction, Base Scenario – Constant Ammonia Source

As for the finite source scenario, contamination originating from the landfills will emerge in the tunnels at various times depending on the distances from the relevant sources, which will also affect the timing of when the inflow concentrations would approach the long-term (maximum) concentration levels. An overall summary of ammonia arrival time and maximum concentration approaching time for the constant ammonia source scenario is provided in Table BD:4.

Further discussion related to the solute breakthrough patterns including comparison between finite and constant source breakthrough patterns is included in Section *Summary of Contaminant Transport Modelling*.

Table BD:4 Concentrations Breakthrough, Constant Source base Scenario Model

Package ID	Approximate NH3 Arriving Time	Time to NH3 greater than 250 mg/L	Time to Maximum NH3
Mainline Tunnels (PS09 and PS10)	2 to 10 years	20 to 35 years	65 to 70 years
Ventilation Tunnels (PS21)	1 to 8 years	3 to 25 years	15 to 90 years

Package ID	Approximate NH3 Arriving Time	Time to NH3 greater than 250 mg/L	Time to Maximum NH3
SPI Ramps	2 to 5 years	10 to 30 years	55 to 70 years

Algie Park and O'Dea Reserve Landfills

These landfills will not interact with the St Peters Area.

The Algie Park and O'Dea Reserve landfills were only simulated assuming a constant ammonia source. The results of the simulations (discussed below) indicated impacts on the quality of groundwater seeping into the tunnel are likely to be low. No simulation finite source simulations were therefore undertaken for these landfills, as the impacts for these scenarios would be even less than from a constant source.

The predicted peak ammonia (as nitrogen) concentrations within the rock aquifer at the tunnel invert level are shown in Figure BD:7 for the Algie Park Landfill and in Figure BD:8 for the O'Dea Reserve Landfill. As shown in these figures, the peak concentration distribution is expected to be localised and limited to the tunnel sections immediately under the landfills.

Modelling results suggest that the arrival time for contaminants from each landfill is likely to be less than a year after tunnel construction. Concentrations are likely to approach maximum level within a relatively short time period after arrival (about one to five years). The modelling results also show the Algie Park and O'Dea Reserve landfills will not interact with the St Peters Area.

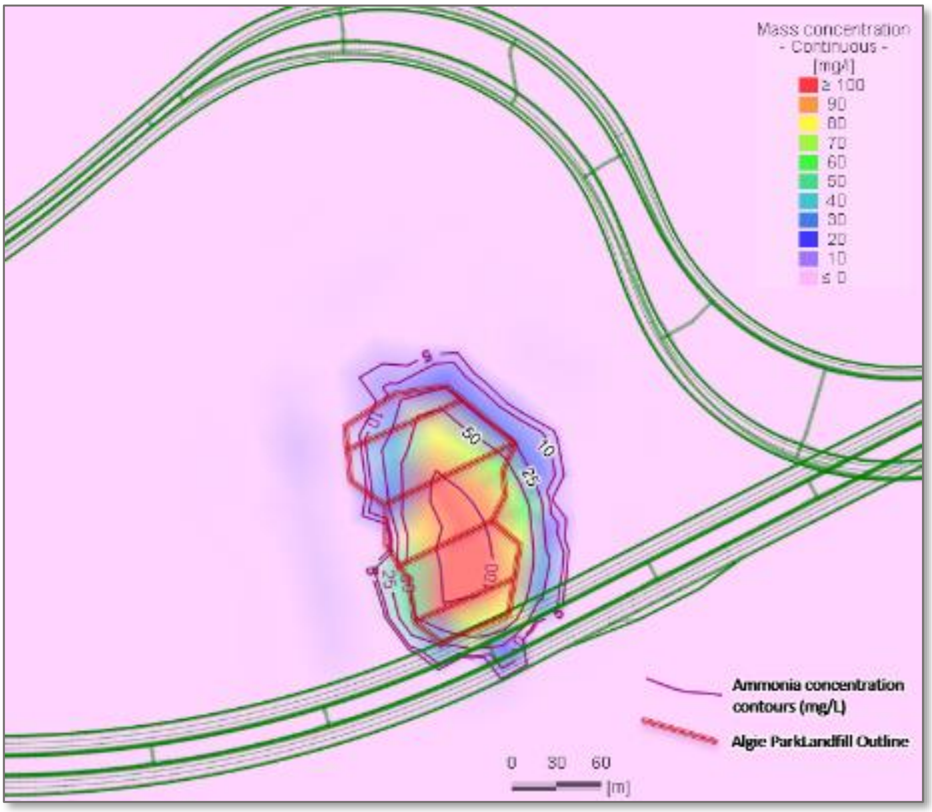


Figure BD:7 Peak Ammonia (as Nitrogen) Concentration Distribution Originated from the Algie Park Landfill, Assuming Source Concentrations of 100 mg/L, Base Scenario, 10 Years after Tunnel Construction

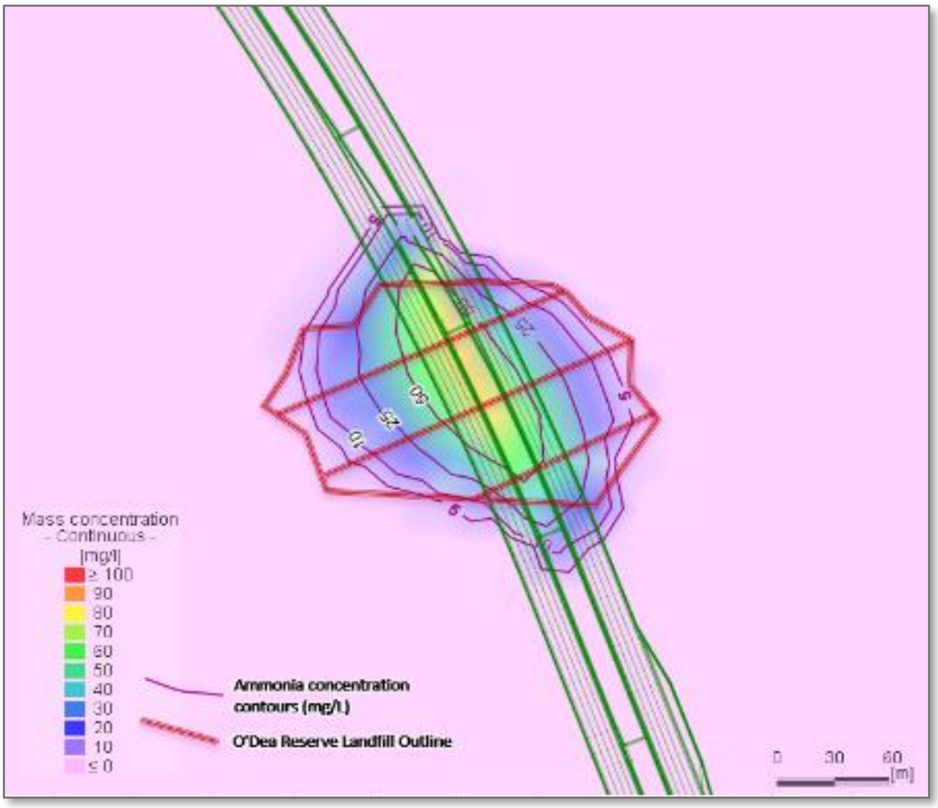


Figure BD:8 Peak Ammonia (as Nitrogen) Concentration Distribution Originated from the O'Dea Reserve Landfill, Assuming Source Concentrations of 100 mg/L, Base Scenario, 6 Years after Tunnel Construction

Sensitivity Analysis

The following scenarios were simulated as a part of the sensitivity analysis:

- No grouting of the PS21 Ventilation Tunnels within the Woolloomooloo Fault Zone – No Grouting Sensitivity Scenario
- Impact of a postulated low permeability dyke east west through the St Peters Area on ammonia distribution and groundwater flow system – Dyke Sensitivity Scenario.
- Impact of a higher dispersivity on the ammonia distribution – Higher Dispersivity Sensitivity Scenario.
- Impact of a low effective porosity of the rock aquifers on the ammonia distribution – Low Effective Porosity Sensitivity Scenario.

All these scenarios were simulated for a constant ammonia source conditions to explore relative changes of these potential scenarios' impacts on the ammonia distribution and groundwater flow pattern rather than to assess absolute ammonia concertation values that would reach the tunnels. The sensitivity runs indicate that the extent of tunnel impacted by ammonia concentrations could potentially be reduced by accepting an exceedance of inflow criteria, but this benefit could be negated should a low permeability dyke be encountered running east-west to the north of the ventilation tunnels

Results of these scenarios are presented in Attachment 1.

Summary of Contaminant Transport Modelling

Results from the finite and constant source Base Scenarios are considered to be sufficient to provide the range of potential ammonia concentration distributions expected over the design life of the tunnels.

The modelling suggests that distribution of the ammonia concentrations and arrival times shortly after tunnel construction is similar for both scenarios. As post-construction time increases, a difference between ammonia concentrations becomes more prominent as conceptualised in Figure BD:9. These differences are smaller closer to the source but increase with the distance from the source. In general, arrival of ammonia (as nitrogen) concentration greater than 250 mg/L at the tunnels for the constant source scenario is about 1 year to 5 years earlier than for the finite ammonia source scenario.

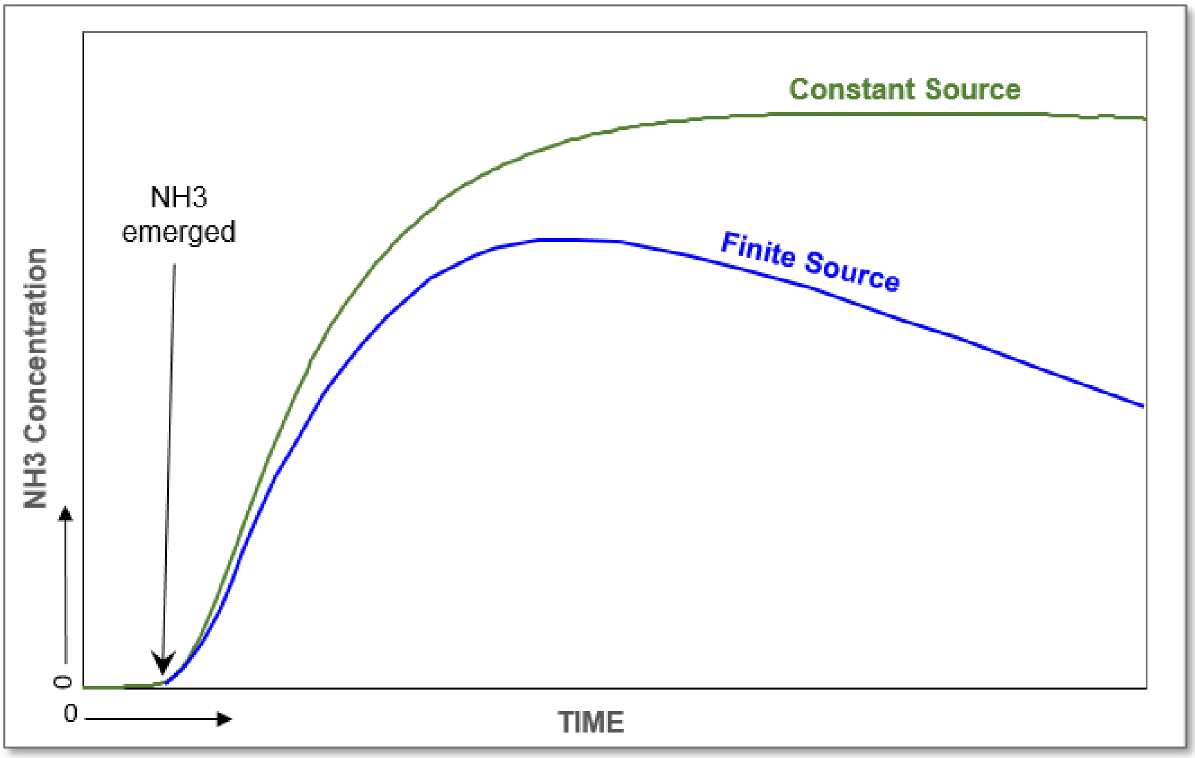


Figure BD:9 Typical Solute Breakthrough Curve for Finite and Constant Source

The predicted maximum ammonia (as nitrogen) concentrations for the finite and constant ammonia source scenarios by design package are summarised in Table BD:5. As illustrated in Figure BD:9, depletion of the ammonia concentrations will occur after the maximum concentrations reach the tunnels for the finite source scenario, while they will remain generally constant for the constant source scenario.

Concentration time plots for selected points (model nodes) along the mainline tunnel, ventilation and access tunnels for the finite and constant ammonia scenarios are included in Attachment 2.

Table BD:5 Maximum Ammonia (as N) Concentrations Predicted for Finite and Constant Source Base Scenarios – St Peters Area

Package ID		Maximum predicted ammonia (as N) in groundwater inflow (mg/L)	
		Finite Source Scenario	Constant Source Scenario
M120	PS09	420	480
	PS10	330	370
M110	PS09	25	30
	PS10	25	30
Ventilation Tunnel	PS21	620	740
SPI Ramps	M190	550	680
	M180	285	320

Results of the sensitivity analysis undertaken indicate that in addition to ammonia concentration, ammonia arrival time also depends noticeable on the dispersivity and porosity values adopted in the model, while distribution of maximum in-tunnel concentrations depends predominantly on dispersivity values.

Geological features such as high permeability fault zones or low permeability dykes are indicated to significantly affect predicted ammonia concentration distribution, arrival time and maximum values that would impact the underground structures.

The presence of the Woollomoolo Fault zone between the mainline tunnel and the Sydney Park and Alexandria landfill is indicated to have a beneficial impact on restricting length of the mainline tunnel exposed to ammonia (as nitrogen) concentrations higher than 250 mg/L. The fault zone acts as a distinctive pathway that channels water flow towards the ventilation tunnel.

The Woolloomooloo fault zone has been modelled based on the expected permeability of this geological structure. The Base Scenario model also includes grouting of this fault where it intersects with the project alignment to allow SWTC inflow criteria to be met in this area. Should the Woolloomooloo Fault zone be less permeable than modelled, or it is cut off by a low permeability feature such as a dyke, the extent of impact of groundwater with ammonia concentrations greater than 250 mg/L on the mainline tunnel would be more extensive than predicated by the Base Scenario simulations.

In summary, the scenarios simulated by the Contaminant Transport Model demonstrate the range of outcomes, which can be predicted depending on the input parameters assumed and this variability was taken into consideration when selecting the “design case” for the durability, groundwater treatment plant and in tunnel gas risk assessments.

Conclusions

The contaminant transport modelling indicates that the construction of the project works will result in the migration of contaminants from adjacent landfills towards the tunnel. Mitigation strategies will need to be implemented to address potential impacts of these contaminants on the permanent works.

One of the key insights gained from the modelling is that the assumed geological setting and hydraulic properties of the assumed structural features can have an effect on the results of the model. The geological setting and its contribution to the project effects on the groundwater flow system and contaminant transport will therefore be difficult to confirm ahead of tunnel construction.

Attachment 1

Overview

The Base Scenario presented in Appendix BD of this report assumes grouting strategies are implemented along the alignment to achieve compliance with SWTC inflow criteria.

Four sensitivity scenarios have been considered to further assess variability in flow and contaminant transport results from the model due to changes in geology and/or modelling assumptions. The following scenarios have been considered:

- No Grouting at Woolloomooloo Fault Zone (No Grouting Option Scenario)
- Impact of Dyke at SPI (Dyke Scenario)
- Higher Dispersivity Scenario
- Low Effective Porosity Scenario

For these scenarios concentration of ammonia (as nitrogen) was assumed to be the same (1,200 mg/L) in all three Central West Pits of the Sydney Park landfill. Concentrations in other landfills and Sydney Park pits were generally the same as for the Base Scenario. Additionally, for all these scenarios constant ammonia concentration at the source landfill were adopted, i.e., constant ammonia source scenarios.

The results of these modelling scenarios are summarised below.

No Grouting Option Scenario

The No Grouting Option Scenario was explored to assess the potential to reduce extents of ammonia impacted tunnel if the Woolloomooloo Fault intersection was not grouted.

By accepting an increase in inflow, the groundwater levels within the surrounding area are predicted to reduce, with the levels in the vicinity of the St Peters ramps estimated to decrease by up to about 4 m (Figure A1-1). The under-drained length of ramps is also predicted to increase, thus reducing the length of ramp tunnel impacted by leachate.

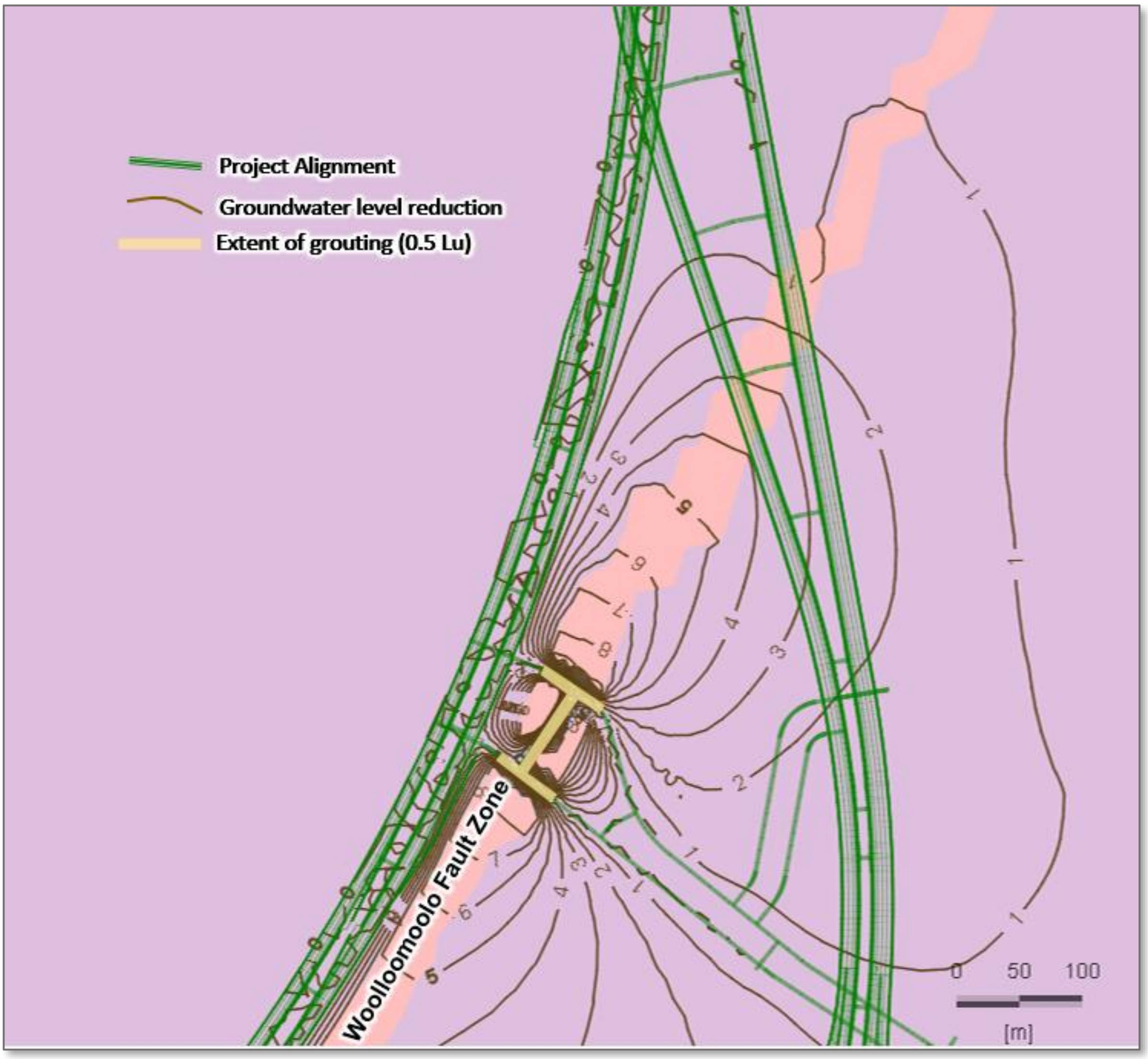


Figure A1-1: Reduction in Long-term Groundwater Levels within the Rock Aquifer without grouting at PS21, No Grouting Option Scenario

For comparison purposes Table B1 presents the predicted groundwater inflows for the No Grouting Option Scenario and Base Scenario over 100 years for the Exit Ramp M190. Note the groundwater inflows in the No Grouting Option Scenario occur further along the ramp, and as such, the contaminated groundwater pass under the ramps. The No Grouting Option Scenario predicted long-term ammonia contours for the rock aquifer are shown in Figure A1-2 for the St Peters area. The contours shown in this figure are expressed as a percentage of the concentration at the Sydney Landfill Central West Pits, which are the maximum concentrations reported within the St Peters area. These contours represent the long-term ammonia concentrations in groundwater, which are predicted to come in contact with the mainline and ventilation tunnels and collected by the tunnel drainage system. For the No Grouting Option Scenario the contaminated groundwater is predicted to pass under the SPI ramps and as such would not enter them under these conditions.

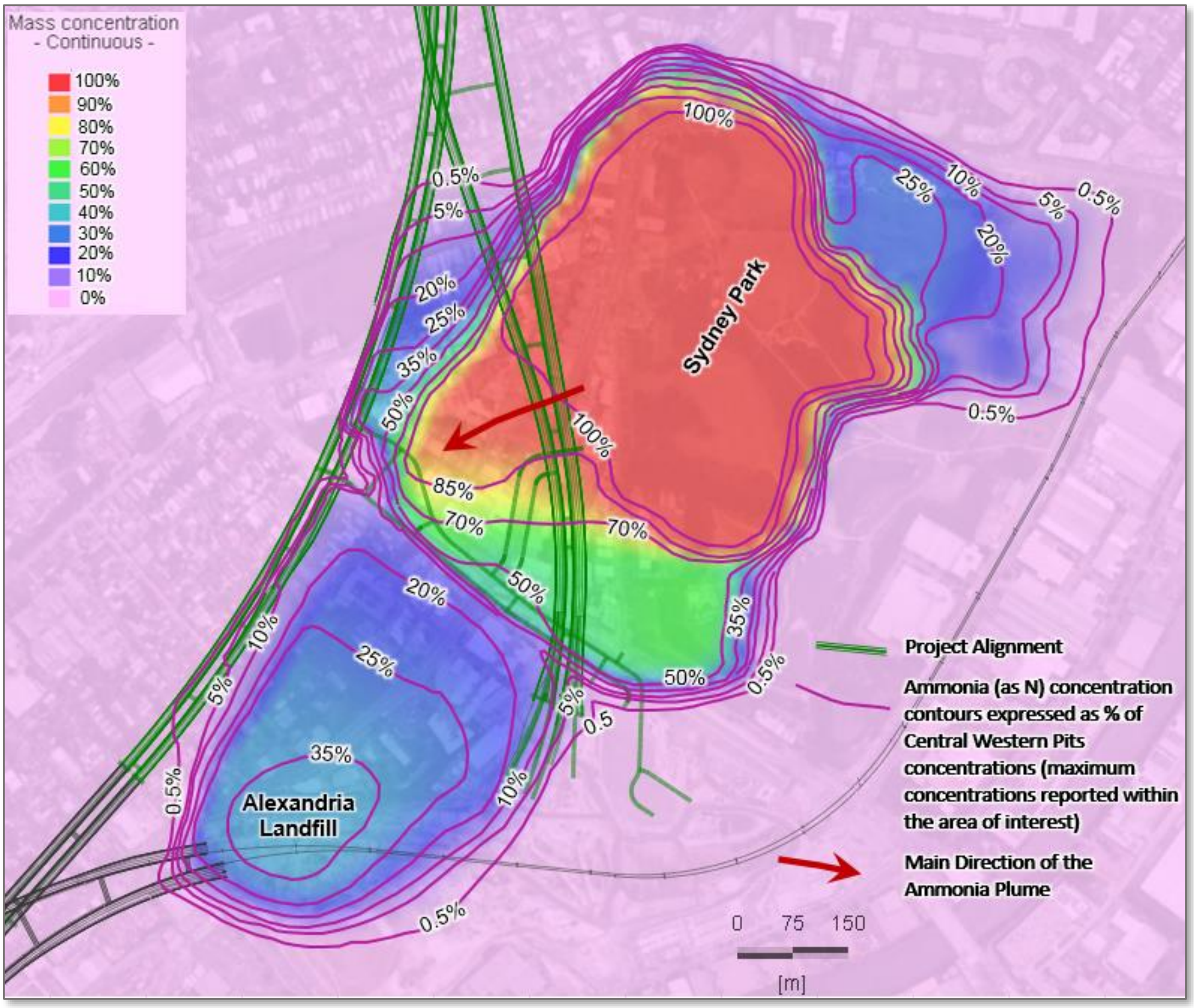


Figure A1-2: Long Term Ammonia Distribution within the Rock Aquifer at the Mainline and Ventilation Tunnels Level Relative to the Maximum Concentrations at the Sydney Park Central West Pits, No Grouting Option Scenario

The No Grouting Option Scenario results also suggested that the main direction of the highest ammonia concentrations originated from the Sydney Park Central West pits would be in a southwest direction towards the deepest sections of the PS21 Ventilation tunnels. Distribution of the highest ammonia concentrations towards the mainline tunnels is indicated to be shielded by the Woollloomoolo Fault with concentrations equal to less than 40 % of the Central West pits concentration likely to reach the mainline tunnel M120 compared to 85 % reaching the PS21 Ventilation tunnel.

The No Grouting Option Scenario indicates that not grouting the Woolloomooloo fault is likely to reduce the extent of tunnel impacted by ammonia concentrations in excess of 250mg/L.

Dyke Scenario

Dykes are commonly encountered in excavations across the Sydney basin. Typically, these structures exhibit low permeability perpendicular to their strike, with high permeability zones at the margins (ie. high flow zones parallel to the dyke). The SPI ramp pilot tunnel recently encountered a Dyke during excavation. The objective of the Dyke Scenario was to explore how the presence of a low permeability dyke (in the location of the dyke encountered within the SPI ramp pilot tunnel) could affect the local groundwater flow pattern, tunnel inflows and ammonia concentration distributions within the St Peters area.

For the purpose of this assessment the Dyke Scenario simulated a single dyke crossing the SPI ramp tunnels and positioned north of the PS21 ventilation tunnels as shown in Figure A1-3. A hydraulic conductivity of 5 x

10^{-9} m/s was adopted to the dyke and it was assumed that the dyke also cut through the Woolloomooloo Fault zone (i.e, intrusion of the dyke occurred post faulting). No grouting of the Woolloomooloo Fault zone at PS21 within the PS21 ventilation tunnel was also assumed in this modelling scenario.

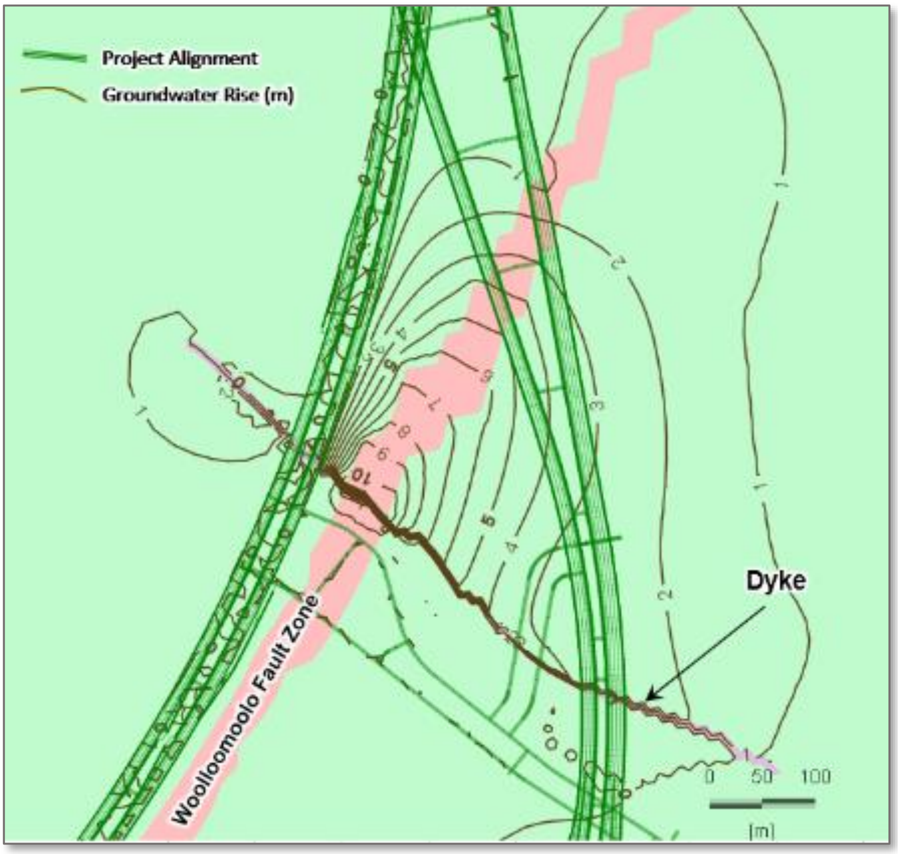


Figure A1-3: Long-term Rise in Groundwater Levels within the Rock Aquifer Predicted by the Dyke Scenario (compared to the no grout option)

The Dyke Scenario results suggest that groundwater levels would rise north of the dyke with a groundwater rise of up to about 5 m at the SPI ramps (Figure A1-3) as compared to the No Grouting Option Scenario and up to 1.0 m with respect to the Base Scenario. This in turn resulted in an increased groundwater inflow into the SPI ramps. The inflow criteria of 1L/s/km would still be achieved under this scenario.

In addition to the high inflows, the length over which the inflows would be expected to occur would be longer in the Dyke Scenario when compared to both the No Grouting Option Scenario and the Base Scenario. The extent of the predicted inflows for the Dyke Scenario is shown in yellow shading in Figure A1-4, along with the predicted long-term ammonia (as nitrogen) concentration distribution for the rock aquifer.

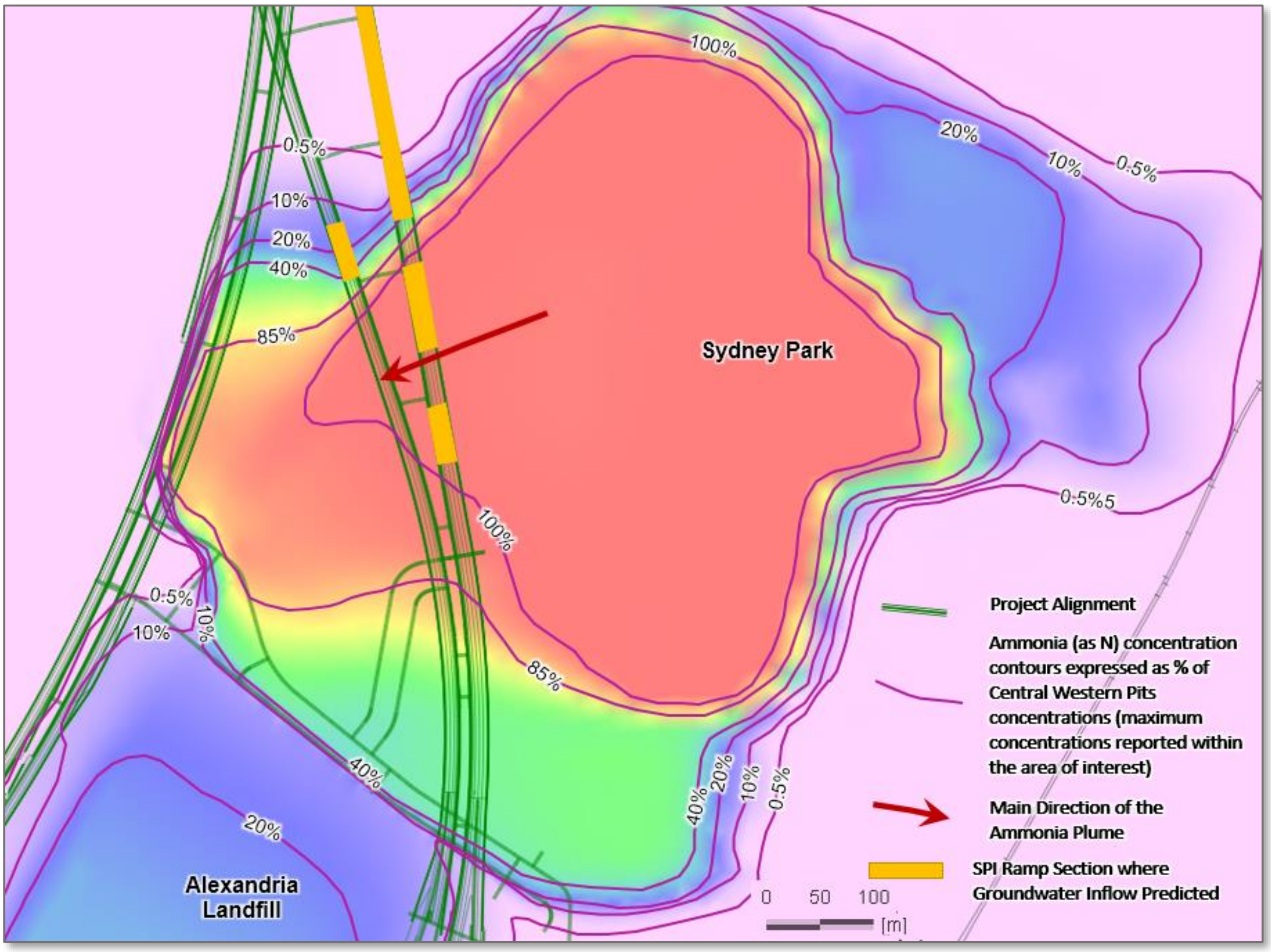


Figure A1-4: Long Term Ammonia (as Nitrogen) Concentration Distribution within the Rock Aquifer at the Mainline and Ventilation Tunnels Level, Dyke Scenario

As shown in Figure A1-4 groundwater with high ammonia (as nitrogen) concentrations are predicted to be push further towards Mainline Carriageway tunnel M120 as the groundwater flow towards the PS21 tunnels would be restricted by the presence of a low permeability dyke. Ammonia concentrations equal to about 85 % of the Central West pits concentration are predicted by this scenario likely to reach the mainline tunnel M120 compared to 40 % predicted by the No Grouting Option Scenario. The spread of the ammonia concentrations equal to 85 % of the Central West pits concentration is also indicated to be considerably wider than for the No Grouting Option Scenario.

The High Dispersivity Scenario

The high dispersivity scenario assumed a longitudinal dispersity of 50 m and a transverse dispersivity of 5 m, which were five times greater than the values adopted for the No Grouted Option Scenario. All other model material properties remained consistent with the No Grouted Option Scenario. The results of this sensitivity scenario indicated that:

- arrival time of the solutes is faster, and stabilisation of the concentrations will occur earlier
- contaminant concentrations in the groundwater inflow into the tunnel sections closer to the source could be up to 30% lower
- the increased spread of contamination results in contaminant concentrations in the groundwater inflow into the tunnel sections further away from the source increasing by up to one order of magnitude.
- Increased contaminant travel speeds result in an overall increase in the contaminant concentrations impacting the tunnel within the design life. For example, concentrations at the PS21 Ventilation Tunnels increased by about 15%.

The Low Effective Porosity Scenario

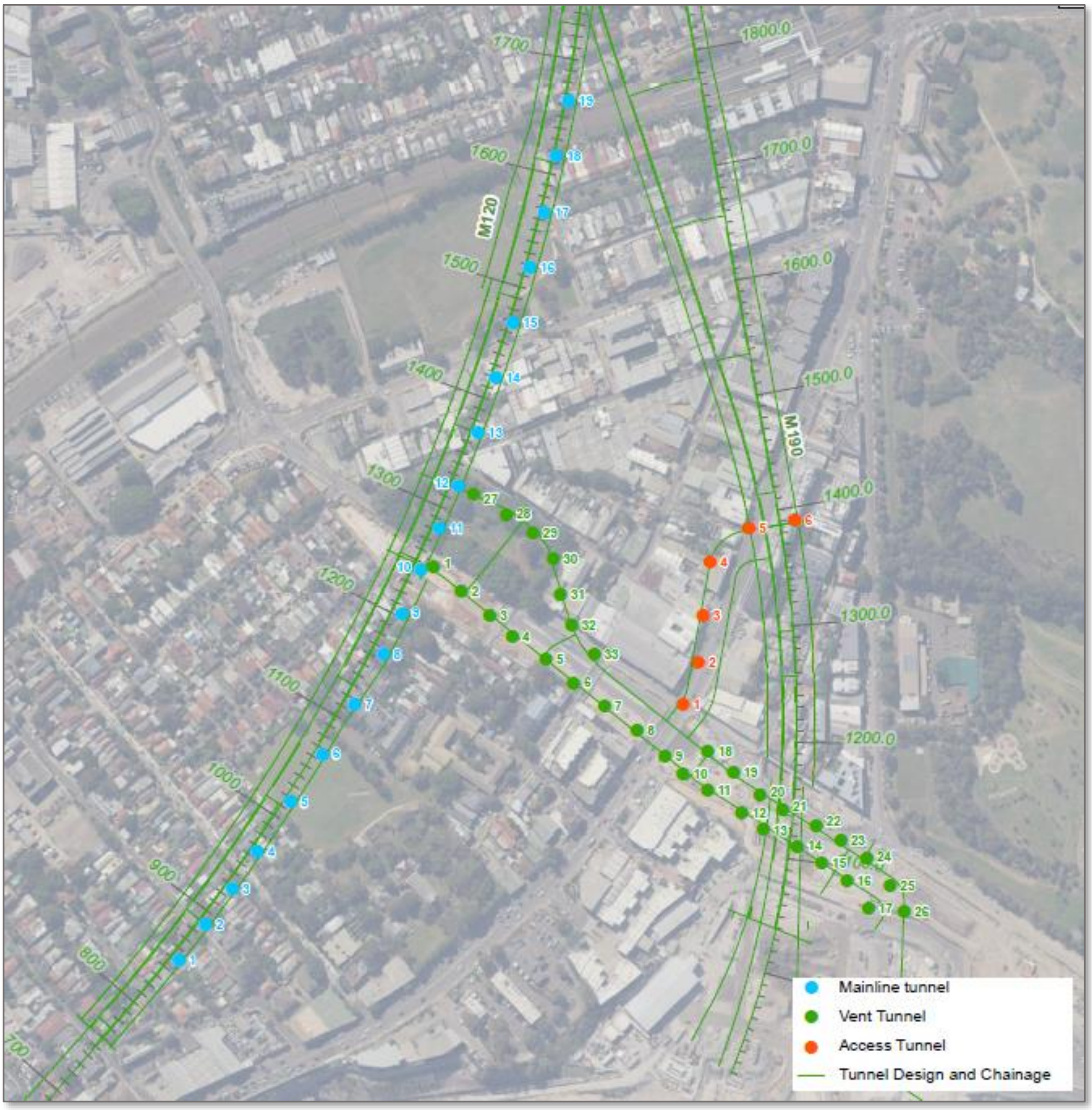
The low effective porosity scenario included reduced effective porosity of the rock aquifer as follows:

- Hawkesbury Sandstone from 0.01 to 0.005
- Ashfield Shale from 0.05 to 0.01
- Fault zones from 0.04 to 0.008.

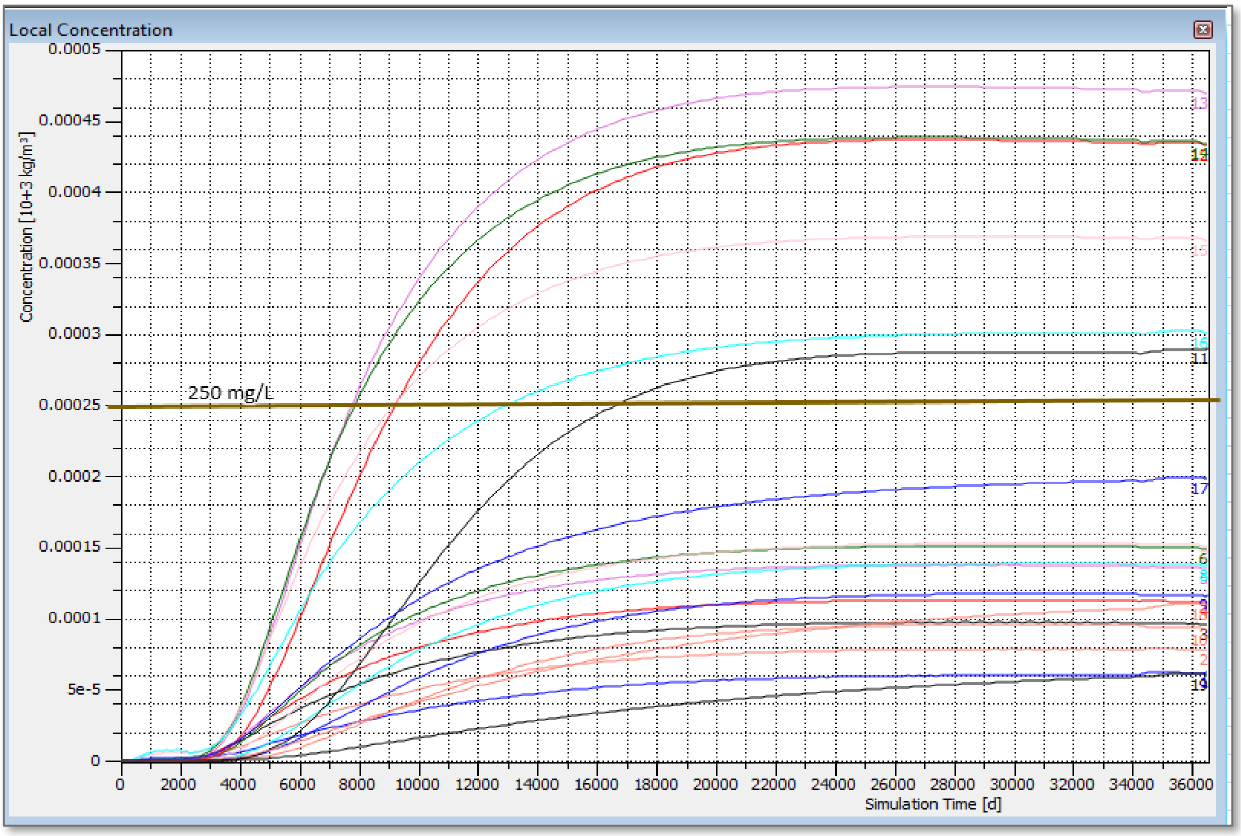
All other model inputs were consistent with the constant source no grouting option. The results from the Low Porosity Sensitivity Scenarios indicated the following:

- the solute will travel faster with arrival time generally decreasing to about 3 months after construction for the sections closer to sources and to about 7 years further away
- stabilisation of the concentrations is likely to happen faster, generally 10 years to 15 years after construction
- long term concentrations at the distant tunnel sections are likely to be greater than those predicted by the no grout option.

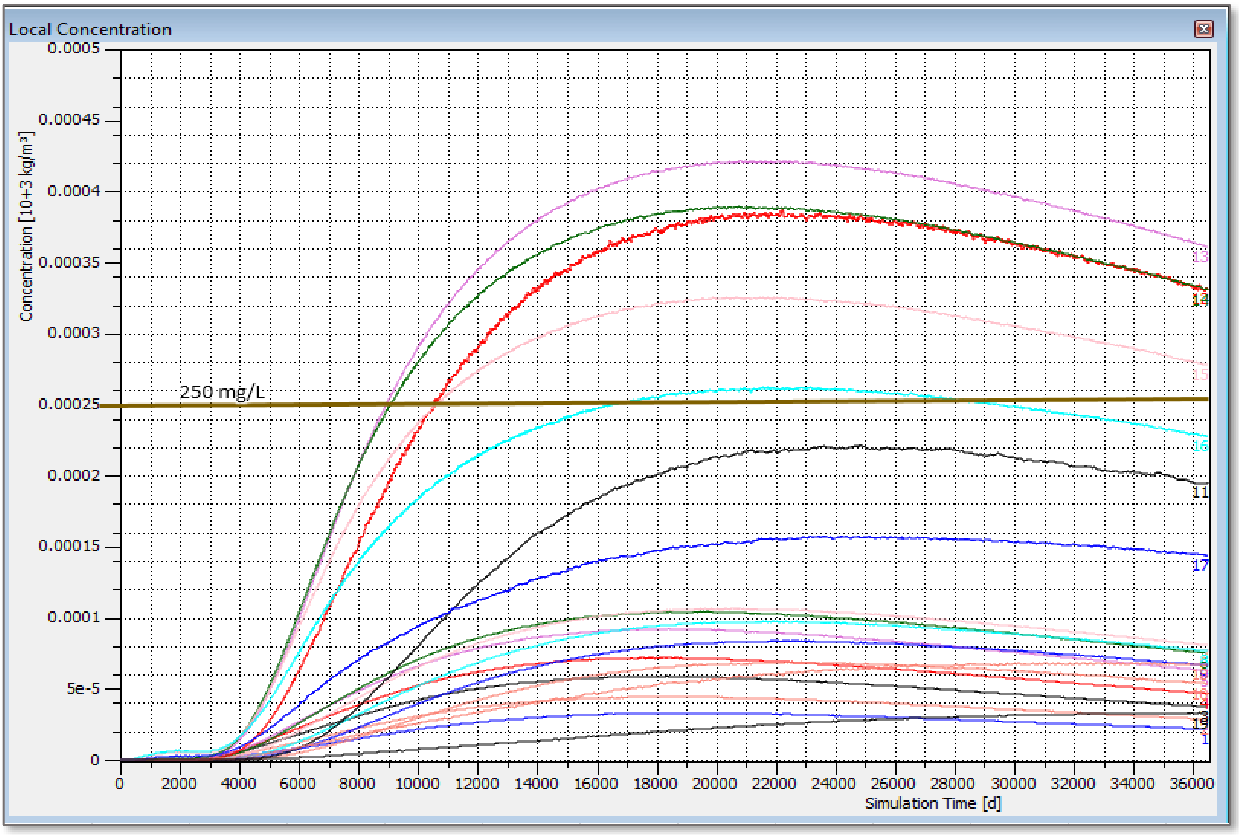
Attachment 2



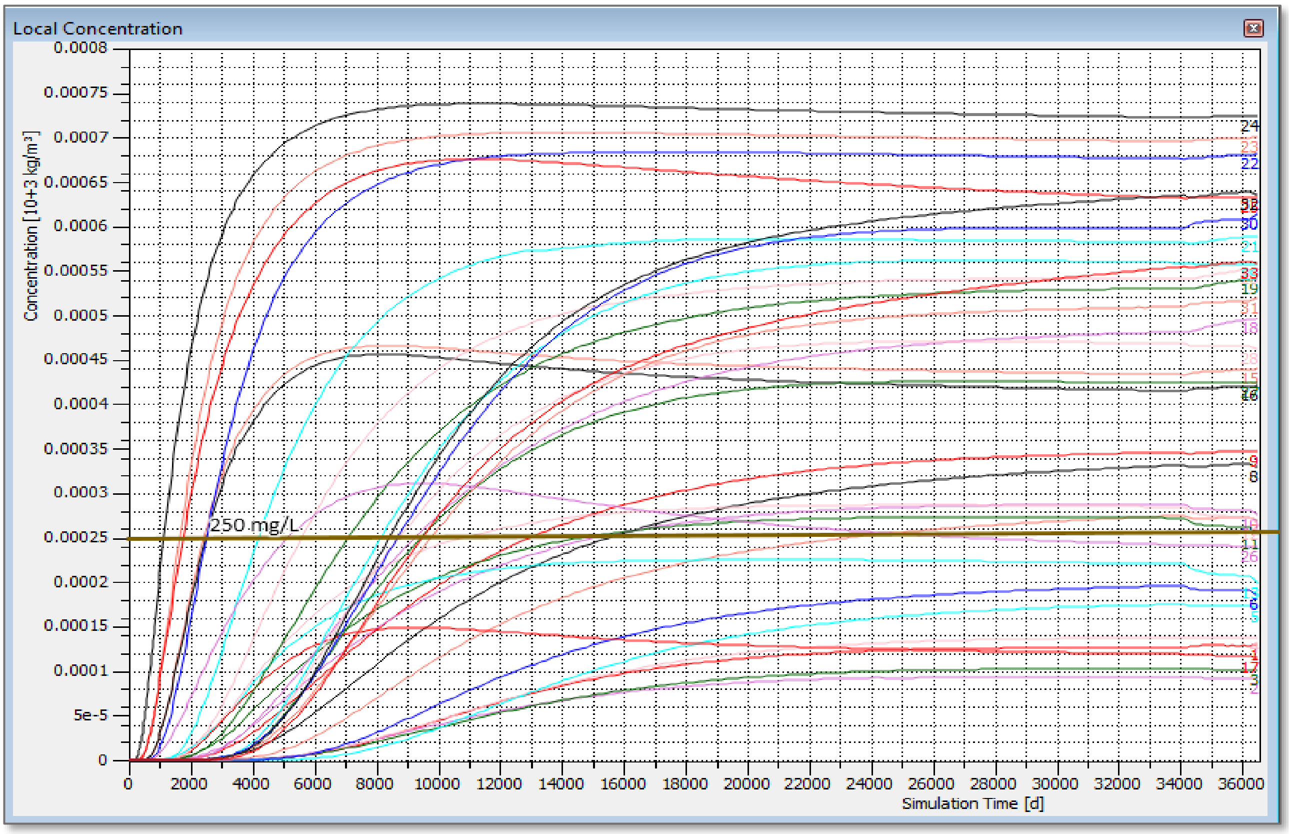
Attachment 2 Figure 1: Location of the Model Nodes for Ammonia Concentration Time plots



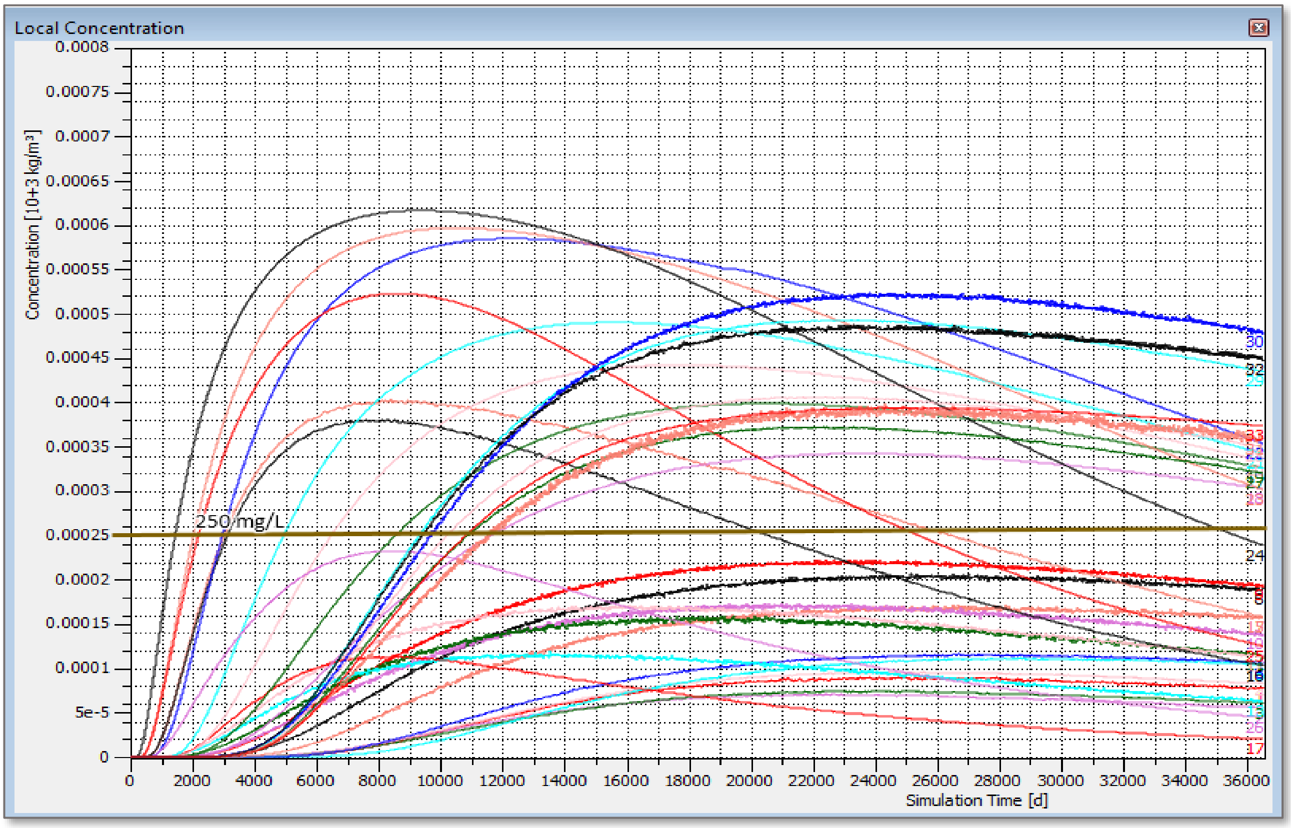
Attachment 2 Figure 2: Mainline Tunnel Ammonia Concentration Time plots – Constant Source



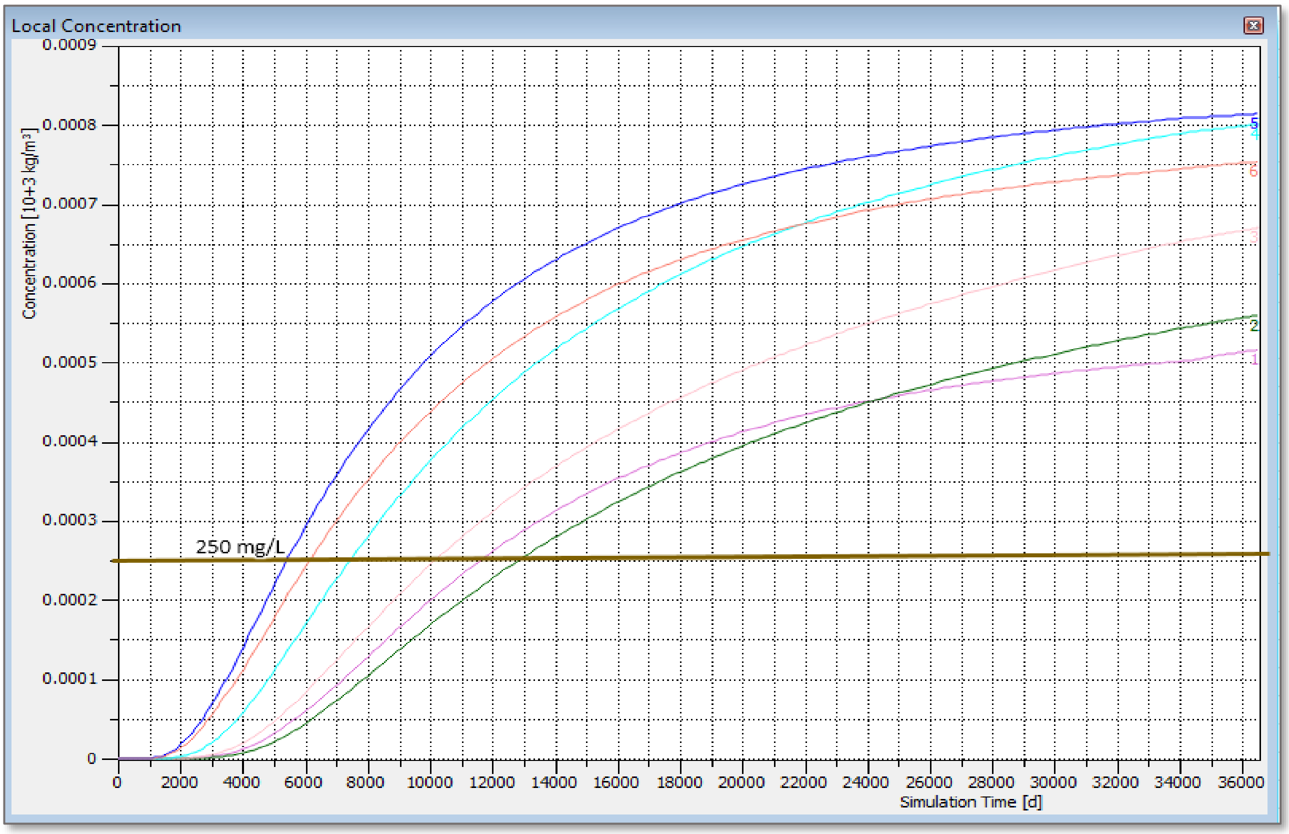
Attachment 2 Figure 3: Mainline Tunnel Ammonia Concentration Time plots – Finite Source



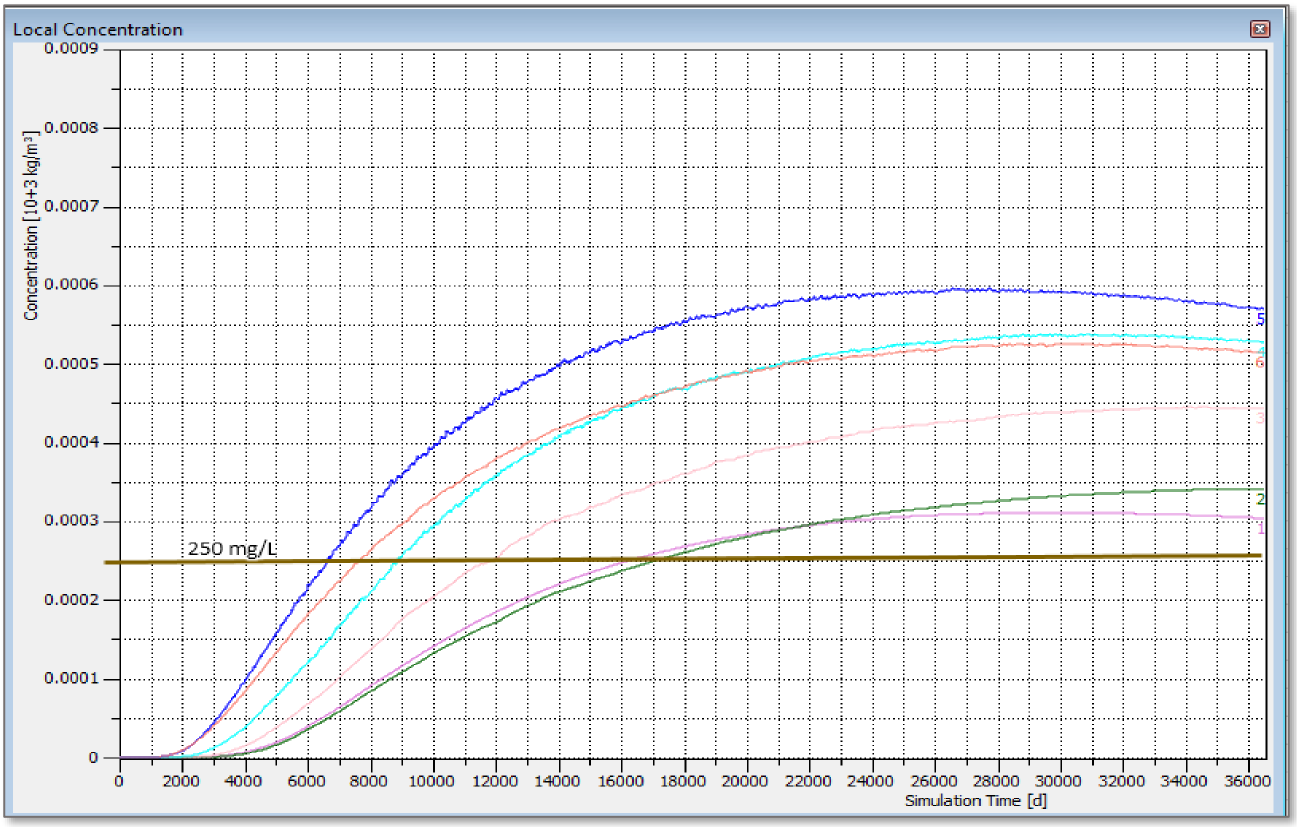
Attachment 2 Figure 4: Vent Tunnel Ammonia Concentration Time plots – Constant Source



Attachment 2 Figure 5: Vent Tunnel Ammonia Concentration Time plots – Finite Source



Attachment 2 Figure 6: Access Tunnel Ammonia Concentration Time plots – Constant Source



Attachment 2 Figure 7: Access Tunnel Ammonia Concentration Time plots – Finite Source

Appendix BE

Groundwater Modelling Peer Review

Jacobs

Memorandum

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Subject	**Groundwater Modelling Review - Final Design Stage**	**Project Name**	M4-M5 Link Tunnels
Attention	Sven Padina	**Project No.**	IA154801
From	Brian Barnett		
Date	20 July 2020		
Copies to			

1. Introduction

I am a groundwater modeller and hydrogeologist with 40 years of experience in the groundwater and geothermal industries in Australia, New Zealand and Japan. My CV is attached to this report as Attachment 1.

This Memorandum has been prepared as a formal peer review report of the Final Design (FD) Stage M4-M5 Link Tunnels Hydrogeological Design Report (the Report) and the Hydrogeological Numerical Modelling Report which is included as Appendix B (the Appendix) of the Report. For addition context and background, I was provided with a document entitled *Main Tunnel Works, Scope of Works and Technical Criteria – Appendix C.2 Project Company Documentation Schedule*, by Sydney Motorway Corporation Pty Limited of June 2018. I understand this document outlines the reporting standards and requirements for the various types of reports prepared for this project. I have referred to Section 1.12 Geotechnical and Hydrogeological Reporting as defining the reporting requirements for the Report and the Appendix.

My initial review was undertaken on 4th and 6th of June 2020. This was followed by discussions with the groundwater modeller, Irena Krusic-Hrustanpasic, to help clarify various aspects of the work and to resolve various issues raised in my initial review.

As part of the review I have prepared a log of issues spreadsheet that has acted as a record of review comments. Most of these issues were created as a result of my initial review and these were re-assessed following discussions with Ms Krusic-Hrustanpasic. The log of issues and responses are included in this report as Attachment 2.

2. Unresolved Issue from Substantial Detailed Design Stage - Reporting the time to steady state

In a previous review of the project groundwater modelling undertaken at Substantial Detailed Design (SDD) Stage I raised the following issue:

The SWTC Appendix C2, Section 1.12 defines the reporting requirements for Hydrogeological Interpretive Reports. At item iv) on Page 10, it is noted that the report should include "provision of a predictive model of the time to reach steady state conditions and the predicted effects of steady state condition.". The report does not address this issue.

At that stage I noted that the issue had not been resolved and had been deferred to the Final Design (FD) Stage modelling. I note that Section 10.3.1 of the Report (and Section 8.2 of the Appendix) includes a statement that modelled inflow rates approach a steady state after 85 – 90 years. Accordingly, I am happy that the requirement has been met.

3. General commentary

The model has been calibrated in steady state using groundwater heads measured in monitoring bores within the model domain and an acceptable level of calibration has been attained for the steady state approach.

A transient calibration was carried out in the FD Stage modelling utilising pumping and head observations from a 27 day pumping test carried out at a site near Hawthorne Canal. The work provides further constraints on model parameters within that region influenced by drawdown responses during the test. The calibration has resulted in local scale heterogeneity being added to the model in the vicinity of the test. It is recognised that the Hawthorne Canal site is of particular significance due to the presence of an alluvial palaeochannel with elevated permeabilities in the underlying rock. In this regard the additional transient calibration represents an improvement in the confidence with which the model can be used in predictive analysis.

Modelling has also helped identify and partially quantify areas of concern at:

- Hawthorne Canal, where elevated inflows are expected as the project intersects a high permeability rock zone beneath the canal,
- At Alexandra and Sydney Park, where contaminated groundwater is expected to seep from nearby landfills towards project elements at this location,
- Iron Cove Bay, where the project is in proximity to the harbour and there is a potential for the migration of seawater towards the tunnel.

Drawdown propagation at post construction steady state indicates substantial levels of drawdown are expected to occur as a result of the project. This outcome is consistent with the fact that the tunnel and associated structures will be drained in the long term.

The modelling report (the Appendix) includes details of a sensitivity analysis that has been performed to help illustrate the level of variability in groundwater inflows and associated drawdown that would result from the adoption of different model parameters. In this case the sensitivity analysis involved ten predictive model runs each with a nominal variation of a single parameter or boundary condition from its calibrated value. Estimated groundwater inflow rates and drawdown are presented for each of the ten runs providing a range of potential outcomes. The analysis provides useful insights into the sensitivity of the predictions to various parameters and boundary condition assumptions and has been used to address predictive uncertainty.

My detailed review comments are documented in the Log of Issues included as Attachment 2. Many of these issues related to how the work was reported and these have been satisfactorily resolved.

4. Assessment against guidelines checklist

An assessment of the modelling against the Australian Groundwater Modelling Guidelines compliance checklist is presented in Table 4.1.

Table 4.1: Compliance checklist

Question	*Yes/No*	*Comments*
1. Are the model objectives and model confidence level classification clearly stated?	yes	
2. Are the objectives satisfied?	yes	
3. Is the conceptual model consistent with objectives and confidence level classification?	yes	Confidence level is set in project SWTC and limited by duration of available groundwater monitoring data
4. Is the conceptual model based on all available data, presented clearly and reviewed by an appropriate reviewer?	yes	
5. Does the model design conform to best practice?	yes	
6. Is the model calibration satisfactory?	yes	
7. Are the calibrated parameter values and estimated fluxes plausible?	yes	
8. Do the model predictions conform to best practice?	yes	
9. Is the uncertainty associated with the predictions reported?	yes	A sensitivity analysis has been undertaken and this provides useful insights into potential variability of outcomes.
10. Is the model fit for purpose?	yes	

The checklist suggests that the work is generally compliant with the Guidelines. Predictive uncertainty has been addressed and reported through the implementation of a sensitivity analysis that illustrates potential variability in key modelling outcomes. The approach may be considered appropriate for a Class 2 Confidence Level Classification. The limited availability of data has meant that the model can strictly only achieve a Class 2 rating. However, it is noted that there have been a number of tunnelling projects undertaken within the metropolitan Sydney environment and the experience gained from modelling, constructing and operating these facilities adds confidence to the model and its predictions. Indeed, it could well be argued that such experience could elevate the confidence associated with model predictions above those normally associated with a Class 2 model.

5. Conclusion

The modelling has been undertaken in a manner that is consistent with current industry standards and is generally consistent with the recommended approach and Guiding Principles included in the Australian Groundwater Modelling Guidelines [1]. The work is consistent with similar infrastructure modelling investigations that I have been involved with (as a modeller and as reviewer) in recent years.

In light of the above discussion I consider the FD model to be of Class 2 Confidence Level Classification and is fit for purpose for assessing tunnel inflows and associated drawdown impacts.

[1] Barnett B, Townley LR, Post V, Evans RE, Hunt RJ, Peeters L, Richardson S, Werner AD, Knapton A and Boronkay A 2012. Australian groundwater modelling guidelines. Waterlines Report #82, National Water Commission, Canberra.

Attachment 1 Brian Barnett CV

JACOBS

Brian Barnett

PRINCIPAL GROUNDWATER MODELLER

Brian Barnett has forty years' experience in groundwater resource assessment, groundwater modelling, hydrogeology and geothermal reservoir engineering. He has acquired a broad range of modelling experience through numerous technical investigations of groundwater resources, mine dewatering and water management, impacts of land use change and contaminant transport. He was SKM's Practice Leader in Groundwater Modelling and leads the Australian groundwater modelling team in Jacobs. He is responsible for ensuring the highest technical standards in all aspects of numerical modelling of groundwater flow and solute transport throughout the company. Brian is a principal author and editor of the Australian Groundwater Modelling Guidelines. Published in 2012, the Guidelines have been accepted throughout Australia as a benchmark defining best industry practice.

EDUCATION/QUALIFICATIONS

Bachelor of Engineering (Civil) Honours, University of Auckland, 1980.

MEMBERSHIPS AND AFFILIATIONS

Member of the International Association of Hydrogeologists

Areas of Expertise

- Groundwater Modelling.
- Solute Transport Modelling
- Hydrogeology
- Geothermal Reservoir Engineering

Relevant Project Experience

Westgate Road Tunnel

Client: Transurban

Title: Groundwater Modeller.

Start/End Dates: March 2017 to Present

Scope/Description: The Westgate Tunnel is a major road construction project in Melbourne that is designed to provide a major western arterial route and alternative to the existing and heavily used Westgate Bridge. The project includes the construction of dual road tunnels beneath existing infrastructure to the west of the CBD. Tunnelling activities are planned in the region of the Yarra delta where vertical superposition of highly permeable aquifers and highly compressive silts provide unique and potentially hazardous construction conditions. Groundwater modelling is being used to assess tunnel inflows and associated drawdown impacts.

Responsibilities: Supervision of all groundwater modelling components of the investigation.

Melbourne Metro Rail Tunnel

Client: Melbourne Metro Rail Authority

Title: Groundwater Modeller.

Start/End Dates: March 2015 to Present

Scope/Description: The Melbourne Metro Rail Tunnel is a major infrastructure project that involves the construction of a rail tunnel and five underground stations beneath the central business district of Melbourne. One of the key project risks is the potential for subsidence arising from groundwater drawdown that will occur during construction and operation of the tunnel. Groundwater modelling has been used to assess the potential impacts.

Responsibilities: Review of the groundwater modelling components of the investigation.

Australian Groundwater Modelling Guidelines

Client: National Water Commission

Title: Project Manager.

Start/End Dates: March 2011 to June 2012

Scope/Description: The Australian Groundwater Modelling Guidelines was produced by SKM (now trading as Jacobs) and a team of leading groundwater modelling exponents drawn from the private and public sector including consultants, academics and regulators. Brian took a leading role in managing the project and the team and in editing and writing the document. It has been widely adopted throughout Australia as the benchmark for best industry practice for groundwater modelling in Australia. The Guidelines were published by the National Water Commission in June 2012.

Responsibilities: Project manager, co-editor and principal author.

Groundwater Model of the Myalup Irrigated Agriculture Precinct

Client: Department of Water and Environmental Regulation (WA)

Title: Project Manager and Lead Modeller.

Start/End Dates: October 2017 to Present

Scope/Description: Development of a three dimensional groundwater flow model to assess the sustainable yield of a coastal aquifer system south of Perth, Western Australia. The work is being undertaken to support water allocation planning by the DWER. A FEFLOW regional model is being constructed, calibrated and run in predictive mode to assess the long term impacts of various levels of future groundwater extraction. In addition, a series of two dimensional vertical slice models are also being constructed and used to assess density dependent solute transport in the coastal setting as required to simulate saltwater intrusion that may arise from on-going future groundwater development.

Responsibilities: Project manager and lead modeller.

North Stockton Sandbeds Groundwater Model

Client: Hunter Water Corporation

Start/End Dates: 2003 to 2004

Scope/Description: The North Stockton Sandbeds is a coastal aquifer near Newcastle, NSW. Hunter Water commissioned SKM to investigated its potential to provide groundwater for municipal water supply during periods of drought. Groundwater flow models were developed in the Modflow finite difference modelling code FEFLOW finite element code. A series of two dimensional vertical slice models were developed in FEFLOW density dependant solute transport mode to assess potential impacts on the seawater freshwater interface of various groundwater extraction scenarios.

Responsibilities: Project manager, Lead modeller

Tomago Sandbeds – Groundwater Modelling.

Client: Hunter Water Corporation.

Brian Barnett

PRINCIPAL GROUNDWATER MODELLER

Start/End Dates: October 2003 to December, 2010

Scope/Description: Groundwater modelling services were provided to Hunter Water Corporation over a period of eight years involving a number of significant modelling assignments. The work has included:

1. Development of a groundwater flow and solute transport model of the North Stockton Sandbeds (adjacent to the Tomago Sandbeds) in 2002 - 2003.

2. Development of a groundwater model of the Tomago Sandbeds to assist with assessment of impacts on GDE's in 2005.

3. Development of a local scale model of the Tanilba Bay WWTW on the Tomago Sandbeds in 2008.

4. Upgrading of the Tomago Sandbeds Gorundwater model in cooperation with Hunter Water staff in 2010 to assist with water resource planning.

Responsibilities: Project manager and lead modeller for all projects.

Murray Darling Basin Sustainable Yields Project

Client: CSIRO

Title: Groundwater Modeller

Start/End Dates: 2007 to 2008

Scope/Description: Brian was groundwater modelling team leader for a major project covering groundwater resources in Queensland, New South Wales, Victoria and South Australia. SKM was contracted by CSIRO in 2007 to undertake the groundwater resource assessment for the entire Murray Darling Basin. The project involved the numerical modelling of all major fresh water aquifers in the basin. Twelve finite difference numerical models were run for the study. Results were used to quantify the available groundwater resources of the basin and to assess the impacts of future climate change and impacts of groundwater development on river flows

Responsibilities: Leader of the groundwater modelling team that included eight modellers working in SKM's Melbourne, Adelaide and Sydney offices.

Upper Macquarie groundwater model, NSW

Client: New South Wales Office of Water (NOW)

Title: Groundwater Modeller.

Start/End Dates: April 2009 to June, 2010

Scope/Description: Development of a numerical groundwater model of the Upper Macquarie Catchment Groundwater Management Area. The model was developed to assist NOW in their on-going management of the water resources of the Upper Macquarie alluvial aquifer. Groundwater is used extensively for municipal water supply (including the city of Dubbo) and for irrigation.

Responsibilities: Project manager and supervising modeller responsible for the design and construction of the model and the calibration and predictive analysis and uncertainty analysis.

Frieda River Mine Dewatering Investigations, Papua New Guinea.

Client: Xstrata Copper.

Brian Barnett

PRINCIPAL GROUNDWATER MODELLER

Title: Groundwater Modeller.

Start/End Dates: October 2012 to March, 2013

Scope/Description: Groundwater modelling of a proposed copper mine in Papua New Guinea highlands. Groundwater models using the finite element FEFLOW modelling code were used to estimate the dewatering pumping requirement for the mine and to provide an assessment of the environmental impacts that may accompany mine dewatering and operation of water storage and tailings storage facilities.

Responsibilities: Lead modeller responsible for the design and construction of the model and its use in predictive scenarios.

Millstream Aquifer Model, WA.

Client: Western Australia Department of Water.

Title: Groundwater Modeller.

Start/End Dates: October 2008 to February, 2010

Scope/Description: Groundwater modelling of an inland aquifer in the Pilbara area of Western Australia. The aquifer is used for municipal water supply purposes and the project was aimed at helping to determine sustainable extraction rates from the aquifer. A principal constraint on future development is the requirement to protect and maintain iconic groundwater dependent river pools and springs.

Responsibilities: Project Manager and lead modeller responsible for the design and construction of the model and its use in predictive scenarios.

Collie Coal Basin – Groundwater Model, WA

Client: Western Australia Department of Water.

Title: Groundwater Modeller.

Start/End Dates: June 2009 to February, 2010

Scope/Description: The groundwater resources of the Collie Basin are heavily impacted by many years of coal mining and power generation. A groundwater model of the basin was developed and calibrated and used to assess future impacts that may arise from expanded coal mining and increased water extraction for dewatering and power station cooling.

Responsibilities: Project Manager and supervising modeller

Barwon Downs – Groundwater Model, Victoria

Client: Barwon Region Water Authority.

Title: Groundwater Modeller.

Start/End Dates: 2003 to present

Scope/Description: Brian has worked for a number of years with Barwon Water on the development and use of a complex groundwater flow model of the Barwon Downs Graben in Western Victoria. The Graben hosts deep confined aquifers that are used for water supply for the City of Geelong and surrounding urban centres. Work has continued for a number of years and has progressed from initial model design and development through various stages of upgrade and refinement. The work has been instrumental in allowing Barwon Water to secure ongoing groundwater extraction licenses for the borefield.

Responsibilities: Lead modeller

Lower De Grey and Lower Robe Groundwater Models, WA

Client: Western Australia Department of Water.

Title: Groundwater Modeller.

Start/End Dates: 2009 to 2010

Scope/Description: Groundwater models of two coastal alluvial aquifer systems in the Pilbara Region of Western Australia were develop and calibrated for the WA government. The work was aimed at defining the sustainable extraction limits for potential water supply borefields that may in future be used to supplement the Pilbara water supply.

Responsibilities: Project Manager and Supervising Modeller

Kulwin and WRP mineral sand mine groundwater models, Victoria.

Client: Iluka Resources Ltd.

Title: Groundwater Modeller.

Start/End Dates: 2002 to 2009

Scope/Description: Groundwater models of two mineral sand mines in northern Victoria were developed and calibrated to assist in the design of mine dewatering and water disposal facilities required to support a dry mining operation. The modelling work was instrumental in establishing the feasibility of mining these deposits that are deep below the water table and in securing the required environmental approvals and water licenses for the project.

Responsibilities: Project Manager and Lead Modeller.

Career Summary

May 2000 to present – Senior/Principal Groundwater Modeller at SKM/Jacobs (Melbourne, Australia). Responsible for leading the Australian groundwater modelling practice in SKM and Jacobs (Jacobs acquired SKM in Dec. 2014).

March 1997 to May 2000 – Senior Geothermal Reservoir Engineer and Groundwater Modeller at Kingston Morrison (Auckland New Zealand). Responsible for geothermal reservoir engineering assessments including numerical reservoir modelling of high temperature, two phase fluid reservoirs used for geothermal power generation. Also responsible for hydrogeological investigations including groundwater modelling.

1991 to 1997 – Geothermal Reservoir Engineer at Sumiko Consultants (Tokyo, Japan). Responsible for geothermal reservoir engineering assessments including numerical reservoir modelling using the TOUGH2 code to simulate high temperature two phase reservoirs in Japan.

1981 to 1991 – Geothermal Reservoir Engineer at Geothermal Energy New Zealand Ltd. (Auckland, New Zealand). Responsible for geothermal field measurements and reservoir assessments in Indonesia, Japan, Greece and Kenya.

1979 to 1981 – Groundwater Engineer at the Hawkes Bay Regional Water Board (Napier, New Zealand). Responsible for hydrogeological investigations including aquifer tests and water quality assessments for an environmental regulator.

Attachment 2 – Log of Issues

PROJECT PACKAGE NUMBER

M4M5-JAJV-PRW-GEO-GW02-RPT-0005

PROJECT PACKAGE TITLE

Hydrogeological Design Report

Design Report: M4M5-JAJV-PRW-GEO-GW02-RPT-0005

Drawing List: N/A

DATE ISSUED	SUBMISSION
21/01/2019	DCD
16/12/2019	SDD

DESIGN ISSUE CATEGORY LEGEND

1 Minor Issue - does not require a detailed response to the verifier

2 Moderate Issue - requires response from the designer

3 Significant design issue to be resolve

RESPONSE STATUS LEGEND

O Open Comment

C Closed Comment

W Withdrawn Comment

N Action for Next Phase

No.	Phase	Discipline	Doc, Rev (Page Label)	Reviewer Name (Author)	Initial Comment Date	Reviewer Initial Comment	Issue Category	Designer Response	Initial Response Date	Response Status	Reviewer Comment Closeout	Date Closed
Final Design Stage - Hydrogeological Design Report Rev. C2												
75	FD	GT	Section 7.6	Brian Barnett	4/06/2020	The paragraph immediately below Figure 7-8 on page 46 suggests that the pumping test results reflect the unfractured rock mass permeability. I don't follow the logic here. Just because the analytical method assumed isotropic radial flow, it doesn't mean that the result is not influenced by the presence of fractures. Results are unlikely to be indicative of the competent rock mass.	2	The text was edited to refer to bulk hydraulic conductivity.	11-Jun-2020		Resolved	15-Jun-20
76	FD	GT	Section 7.6	Brian Barnett	4/06/2020	Page 46. Please standardise the number formats using superscripts for exponents.	1	Noted. Has been corrected.	11-Jun-2020		Resolved	15-Jun-20
77	FD	GT	Section 7.7, page 47	Brian Barnett	4/06/2020	In the paragraph immediately folowing Table 7-5, I suggest inclusion of definitions of total porosity and effective porosity so we can understand the significance of the distinction.	1	Noted. Definition added.	11-Jun-2020		Resolved	15-Jun-20
78	FD	GT	Section 7.7, page 47	Brian Barnett	4/06/2020	In the second paragraph following Table 7-5 make clear that these values were adopted in the numerical groundwater model.	1	This is stated in the third paragraph after Table 7-5.	11-Jun-2020		Resolved	15-Jun-20
79	FD	GT	Section 8.1.2, page 48	Brian Barnett	4/06/2020	Please clarify whether planned waterway naturalisation is accounted for in the numerical model predictions of the long term operational phase impacts?	1	Planned waterway naturalisation was not included in the model as there are no information when this may happen and to what extent.	11-Jun-2020		Resolved	15-Jun-20
80	FD	GT	Section 8.2, page 50	Brian Barnett	4/06/2020	Quotes from WSP are of questionable value. If these quotes are to remain there should be more context because at the moment I am struggling to understand how such claims can be made.	3	WSP quote removed.	11-Jun-2020		Resolved	15-Jun-20
81	FD	GT	Section 8.4, page 53	Brian Barnett	4/06/2020	The paragraph that starts "Published experiences…" Should include references to the published material.	1	The text was updated to include references to the published material.	11-Jun-2020		Reolved	15-Jun-20
82	FD	GT	Section 8.5.2, page 55	Brian Barnett	4/06/2020	Sentence immediately before Figure 8-4 - I'm not sure what this observation has to do with Vertical Head Gradients? If it is important then you should provide figures that show the measured tidal responses.	1	The sentence has been removed.	11-Jun-2020		Resolved	15-Jun-20
83	FD	GT	Section 10.2 page 69	Brian Barnett	4/06/2020	Include a comment as to why the model was not used to assess construction inflows and impacts.	3	Understand this comment has been withdrawn.	11-Jun-2020		Withdrawn	15-Jun-20
84	FD	GT	Section 10.3.1 page 70	Brian Barnett	4/06/2020	Incomplete sentence starts the 3rd paragraph in this Section.	1	The figure references were missing. This has been corrected.	11-Jun-2020		Resolved	15-Jun-20
85	FD	GT	Section 10.3.1 page 75	Brian Barnett	4/06/2020	Is there any reason for separating into two tunnels. Normally a single inflow estimate that treats the tunnel as one structure would be appropriate?	1	It is a project requirement to report inflows for each carriageway separately.	11-Jun-2020		Withdrawn	15-Jun-20
86	FD	GT	Section 10.4.4 page 84	Brian Barnett	4/06/2020	In figures such as Fig 10-12, it should be possible to put time markers on the flow lines. This will provide useful information on the time taken before impacts can be expected.	1	The time markers on the flow line would show arrival of the maximum concentrations due to pure advection flow. This could be misleading as saline impacted groundwater would be expected to arrive earlier due to dispersion and density gradient driven flow. For tunnel design purposes it is more important to understand where intrusion will happen over the design life of the structure.	11-Jun-2020		Resolved	15-Jun-20
87	FD	GT	Section 10.4.6 page 87	Brian Barnett	4/06/2020	The first paragraph on Page 87 includes a quote from AECOM 2017. While I appreciate that this is a quote, it doesn't really stand up to scrutiny and is not supported by basic hydrogeological principles. I'd prefer that these quotes (also applies to comment 80 above) were removed completely from the document.	3	The quote has been removed and some additional words were added with respect to referencing the EIS.	11-Jun-2020		Resolved	15-Jun-20
88	FD	GT	Section 11.10 page 95	Brian Barnett	4/06/2020	Final dot point of Section 10.11 - further reporting detail is warranted here. I am interested in understanding the plan for monitoring of inflows and whether or not estimates of inflow per kilometre can be obtained. If not then compliance, with inflow criterion is difficult/impossible to assess.	2	Refer to M4-M5 Link Tunnels CEMP: Groundwater Monitoring Program for further details regarding monitoring to be undertaken.	11-Jun-2020		Resolved	15-Jun-20

Final Design Stage - Hydrogeological Design Report Rev. C2 Appendix B Groundwater Modelling Report												
89	FD	GT	Appendix B, Section 4.4.2	Brian Barnett	4/06/2020	This section describes groundwater levels. It should include a figure showing potentiometric surface map - perhaps refer to Figure 4-5.	1	Agree. Figure 4-5 moved into Section 4.4.2 and renamed Figure 4-3.	11-Jun-2020		Resolved	15-Jun-20
90	FD	GT	Appendix B, Section 4.4.3, page 11	Brian Barnett	4/06/2020	Second dot point under Figure 4-4 concludes recharge rates are dependent on rainfall intensity and duration. This is not supported by the data presented. Information on duration of the rainfall events are not reported.	1	Groundwater levels did not respond to the August 2019 rainfall in any of the VWPs. The total rainfall at that time was 40 mm and occurred over one day. However groundwater levels in three out of five VWPs responded to the higher intensity rainfall event (total of 102 mm) that occurred over 3 days in September (17 Sep to 19 Sep), We believe this support our comment. Text clarified to support conclusions made.	11-Jun-2020		Resolved	15-Jun-20
91	FD	GT	Appendix B, Section 4.4.4, page 11	Brian Barnett	5/06/2020	Evapotranspiration is often a significant groundwater discharge mechanism - the report should address whether ET is a significant component of the water budget. This is particularly relevant since Section 6.4.2 discusses the effects of ET on net recharge rates.	1	Additional text added as follows: Evapotranspiration was also considered to be significant component of the water budget and source of water loss from the groundwater system.	11-Jun-2020		Resolved	15-Jun-20
92	FD	GT	Appendix B, Section 5.2, page 15-16	Brian Barnett	5/06/2020	The paragraph below Figure 5-2 should include additional context by stating that the base of the model, at -120 mAHD, is set as a no-flow boundary.	1	The text below Figure 5-2 and in Section 5.3 has been updated to clarify the base of the model was set as a no-flow boundary.	11-Jun-2020		Resolved	15-Jun-20
93	FD	GT	Appendix B, Section 5.3, page 20	Brian Barnett	5/06/2020	The first sentence following Figure 5-6 should read *"Heads assigned to the Cauchy boundary conditions for creeks and canals…"*. The last sentence of the second paragraph below Figure 5-6 refers to M4 and M5 "***waterways"***. I assume this should be motorways?	1	This was a typo that has been corrected.	11-Jun-2020		Resolved	15-Jun-20
94	FD	GT	Appendix B, Section 6.1, page 22	Brian Barnett	5/06/2020	The first paragraph should be moved to below the dot points. I want to understand the calibration strategy before reading how calibration was achieved.	1	Paragraph was moved below dot points.	11-Jun-2020		Resolved	15-Jun-20
95	FD	GT	Appendix B, Section 6.1, page 22	Brian Barnett	5/06/2020	The final paragraph of Section 6.1 doesn't make sense. *"i.e. defined by nearest nondependent model slice above."* What is defined by the nearest nondependent model slice above? Since all model slices are dependent except for the top slice then all model slices are defined by the top slice. I have no idea what this means.	2	The text in brackets has been removed.	11-Jun-2020		Resolved	15-Jun-20
96	FD	GT	Appendix B, Section 6.2.2, page 23	Brian Barnett	5/06/2020	Last paragraph on page 26 discusses porosity. Feflow phreatic surface option requires specific yield value and not porosity. Suggest the paragraph refer to specific yield.	1	The text was edited as model was calibrated to drainable porosity, i.e., specific yield.	11-Jun-2020		Resolved	15-Jun-20
97	FD	GT	Appendix B, Section 6.2.3, page 24	Brian Barnett	5/06/2020	In Figure 6-1 please clarify whether the drawdown contours are predicted by the model or contoured from observations.	2	Clarification added.	11-Jun-2020		Resolved	15-Jun-20
98	FD	GT	Appendix B, Section 6.2.3, page 24-25	Brian Barnett	5/06/2020	Add tunnel alignment to Figures 6-1 and 6-2	2	The tunnel locations have not been added to the Figures 6-1 and 6-2 as the calibration is related to pre-excavation conditions.	11-Jun-2020		Resolved	15-Jun-20
99	FD	GT	Appendix B, Section 6.2.3	Brian Barnett	5/06/2020	While it is clear that the pumping test calibration requires the introduction of heterogeneity in aquifer properties it is likley that a similar calibration could be attained with different parameter values and zonations. I think this issue should be acknowledged as A) the zones do not appear to be aligned with geological features that would help support the chosen zonation and B) the choice of zone shape and extent may have a significant influence on the predictive model outcomes.	3	Agree that there is no unique solution for any model calibration. The success of calibration process depends heavily on the monitoring data available and distribution of monitoring points across the model domain, which could considerably limit alternative solutions. Drawdown data recorded during the pumping test indicated an elongation of the groundwater drawdown cone of depression in an easterly/south easterly direction where quite a good coverage of monitoring points existed. Initially calibration of the model considered a uniform hydraulic conductivity (as derived from the analytical solutions) which was extended radially away from the pumping well. This solution resulted in a significant underestimation of the groundwater drawdown at the distant wells, particularly at monitoring wells LSB-MT-BH014a, HB_BH14 and LSB-GW-HB-BH012 and the observed elongation could be replicated. To achieve the observed elongation, a narrow zone of a higher hydraulic conductivity in the direction of drawdown elongation needed to be introduced. Direction and distribution of this zone was tested through a number of calibration runs until a satisfactory solution was achieved. Although the adopted solution is not an unique solution, the calibration process showed that alternative solutions were constrained by the shape of groundwater drawdown cone observed.	11-Jun-2020		Resolved	15-Jun-20

100	FD	GT	Appendix B, Section 6.4.1, page 32	Brian Barnett	5/06/2020	Table 6-4 and accompanying description refers to porosity when the model uses specific yield.	1	Text was corrected to refer to specific yield.	11-Jun-2020		Resolved	15-Jun-20
101	FD	GT	Appendix B, Section 7.3, page 39	Brian Barnett	5/06/2020	Predictive model uses Unsaturated/saturated option whereas calibration uses Phreatic Option. Some explanation is required as to why this approach was adopted and whether calibration has been repeated with the Unsaturated/saturated option.	3	The unsaturated/saturated option was used in order to achieve a more stable model and to improve the water budget for the predictive runs. Difference between unsaturated/saturated and phreatic model options is related only to treatment of the transient water table response in the model. Considering that the inflow into tunnels and development of the groundwater drawdown will be driven by the deeper groundwater system in the long term, these differences are not expected to significantly affect the model's inflow and drawdown predictions. An additional pumping test simulation, however, has been undertaken using the unsaturated/saturate option and the results from this model simulation indicate there no changes to the groundwater response within the deep rock aquifer, while changes in the alluvial aquifer were not significantly different from the phreatic model.	11-Jun-2020		Resolved	15-Jun-20
102	FD	GT	Appendix B, Section 8.1	Brian Barnett	5/06/2020	figures 8-1 to 8-5 and 8-11 to 8-13 show results for the operational phase - how long after the tunnel construction do the results represent?	2	The operational phase results are for the period 100 years after tunnel opening. A note clarifying timing is added in the report and additional figures added to show the inflows shortly after tunnel opening.	11-Jun-2020		Resolved	15-Jun-20
103	FD	GT	Appendix B, Section 8.4	Brian Barnett	5/06/2020	Figures 8-14 and 8-15 would be improved if time markers were added to the particle traces.	2	See response to Comment # 86.	11-Jun-2020		Resolved	15-Jun-20
104	FD	GT	Appendix B, Section 8.5, page 55	Brian Barnett	5/06/2020	The statement *"Although the predicted percentage reduction in groundwater contribution to baseflow in some cases is large, this reduction represents a small reduction in the overall stream flow, as the baseflow simulated in the model only represents the occasions when the groundwater reaches ground level and enters the waterbody."* in the first paragraph of Section 8.5 is misleading and inappropriate. Large reductions in baseflow are not mitigated by the fact that they occur when groundwater heads are above stream level. If the predicted change in base flow in the model is small with respect to stream flow then it is because there is significant baseflow contribution from stream reaches outside the model domain.	3	The text has been updated and statement removed	11-Jun-2020		Resolved	15-Jun-20
105	FD	GT	Appendix B, Section 8.5, page 55	Brian Barnett	5/06/2020	In paragraph 2 of Section 8.5, it is hard to accept the statement *"it is likely that the majority of the stream flow is derived from stormwater runoff".* Runoff events in an urban environment are usually of limited duration and will not sustain permanent flows. If the creeks are no more than stormwater drains then they should be described as such.	3	See response to Comment # 104 above	11-Jun-2020		Resolved	15-Jun-20
106	FD	GT	Appendix B, Section 9.	Brian Barnett	5/06/2020	While the sensitivity analysis described in the section provides valuable information on the range of predictive outcomes that should be allowed for, it should be noted that it is not a calibration constrained uncertainty analysis. It is difficult to appreciate how many of these scenarios would produce acceptable calibration results.	1	The text has been edited, to make clear that the reference to Modelling Guidelines is related to sensitivity analyses and not the uncertainty analysis.	11-Jun-2020		Resolved	15-Jun-20
107	FD	GT	Appendix B, Section 9, page 59	Brian Barnett	5/06/2020	In table 6-1, a more accurate definition of the impact of each scenario on the 20 m drawdown contour would be the calculated area within the contour.	1	Agree that this would be beneficial, however this could also be misleading when difference in the drawdown extent are within localised areas. In some cases where groundwater drawdown extends uniformly further away than the base case, the area of the 20 m contour extent may not be distinguished clearly from the area where only localised higher extent drawdown occurred (variable hydraulic conductivities for example).	11-Jun-2020		Resolved	15-Jun-20

NSW GOVERNMENT | Department of Industry

Contact: Ellie Randall
Email: ellie.randall@nrar.nsw.gov.au

Courtney Moran
Senior Environment Officer
Infrastructure & Place

Our ref: OUT20/10329

email: Courtney.Moran@transport.nsw.gov.au

Dear Courtney,

27 August 2020

M4-M5 Link Tunnels - Hydrogeological Design Report Review

Thank you for giving the Department of Planning, Industry and Environment – Water (DPIE-Water) the opportunity to review the Hydrogeological Design Report for M4-M5 Link Tunnels. DPIE-Water has reviewed the report and provides the following comments:

1. The conceptual and numerical models must be updated when 24 months of groundwater monitoring data are available.
2. Groundwater level and quality monitoring is to continue throughout the construction and for 10 years minimum during operational phase of the M4-M5 Link project.
3. The proponent is to continue mitigation measures throughout construction to ensure the inflows are below 1 L/s/km.
4. DPIE-Water anticipates on-going communication on the development of the next model update, the proponent is to provide a 'clear plan' for updating the model to DPIE-Water.
5. The next update of the modelling and reporting must include revision of the groundwater modelling objectives to include assessment of cumulative effects of all aspects of other infrastructure projects that intercept the model domain. The 24 months groundwater monitoring data update to the model and report must emphasise on:
    - Considering and addressing DPIE-Water comments on earlier versions of the model and report
    - verification and validation against groundwater monitoring data
    - in predicted high inflow areas
    - for predicted impacts; and
    - effectiveness of the construction / mitigation methods used include a proper third-party independent peer review for the updated model, which should be a full review.

Hydrogeologists from the Department of Planning, Industry and Environment can be made available to discuss any matters of concern or ongoing issues.

Should you have any further queries in relation to this submission please do not hesitate to contact the Natural Resources Access Regulator's Service Support Team at nrar.servicedesk@industry.nsw.gov.au.

Yours sincerely

alonlollar

Alison Collaros
Licensing and Approvals Manager (East)
Natural Resources Access Regulator
Department of Planning, Industry and Environment

Meeting Minutes

Meeting Name	Groundwater Modelling Report – Consultation with DPI	**Date**	9 July 2020
Project	**WestConnex M4–M5 Link – Main Tunnel Works**	**Time**	1400hrs to 1600hrs
Venue	Microsoft Team	**Our Ref**	WCX-M4M5Link-Memo-GEO099

Attendees

Company	Name	Company	Name
DPIE	Richard Green	Golder	Irena Krusic-Hrustanpansic
DPIE	Llyle Sawyer	JAJV	Sven Padina
DPIE	Hisham Zarour	LSBJV	Anna Burke
TfNSW	Sam Sader	LSBJV	Grant Sainsbury
TfNSW	Courtney Moran	LSBJV	Martin Knight
M4-M5 Link Group	Jack McGovern		

Apologies

Company	Name	Company	Name
NRAR	Ellie Randall		

Distribution

Attendees plus following:

Company	Name	Company	Name

Items

No.	Description	Action by /Status	Date
1.	**Introduction**		
1.1	Subsequent to submission of the Final Design (FD) WCX3A Groundwater Modelling Report and in compliance with the Minister's Conditions of Approval (specifically CoA E193 and CoA 192/194), this meeting has been arranged to discuss the following: 1. General changes and additions from DCD to FD modelling; 2. Discussion on our response to comments 1 to 7 coming from the DPIE review of the DCD report; 3. Review the deliverables timetable to ensure compliance with CoA.	Note	-
2.	**Item 1: Presentation**		
2.1	Attached for information is the Power Point Presentation. The following were covered: • MCoA requirements E192, E193 and E194 and SWTC requirements; • Model Domain; • Regional Model Development and Reporting; • Groundwater model inputs; • Development from DCD to SDD groundwater model; • Development from SDD to FD groundwater model; • Model inflow predictions – opening; • Model inflow predictions – year 3; • Model inflow predictions – steady state; • DPIE Water Review; • DPIE Water Consultation Process. The major discussion topic to come out of the presentation are discussed below.	Note	-
2.2	The main developments from DCD →SD →FD are: • Refinement of groundwater inflows and drawdown predictions; • Transient model calibration using the Hawthorn Canal Pumping Test data; • Final steady state calibration using accumulated water level data; • Incorporation of the tunnel construction sequence; • Finalising inflow and drawdown assessments; • Finalising for saltwater migration assessments from Iron Cove and Hawthorne Canal; • Assessment of potential stream groundwater baseflow changes; • Completion of detailed sensitivity and uncertainty analysis; • FD model designed to meet class two criteria with elements of class 3.	Note	-

Items

No.	Description	Action by /Status	Date
2.3	Model inflow predictions: • At opening the model predicts two areas of minor inflow exceedences, one at Hawthorne Canal and another as an accumulation of inflows at the Luna Park and Johnstons Creek faults; • Within year 3 all inflows are predicted to be less than the SWTC requirement of 1 litre per km of tunnel; • At steady state tunnel inflow is well below the SWTC limits; • Groundwater inflow is being currently assessed during construction works and results are provided in the monthly geotechnical report.	Note	-
2.4	DPIE Water Review. A response has been provided to the seven comments made by DPIE on the SDD report. LSBJV / JAJV consider the FD report now fully addresses and closes out all the DPIE-Water review comments.	Note	-
2.5	DPIE Water Consultation Process. • 50% of the tunnel has been excavated and the last primary support package will be issued IFC on 30-July 2020;	Note	-
3	**Technical Review**		
3.1	Questions raised by DPIE and response from Golder as follows: • Are there any benefits from the model? How are the results from the model used? → Amongst others, the model informed us of water bearing features encountered during construction at Wattle Street, that required in-tunnel grouting, and the WFZ at SPI; • Parameter values – have and how much final parameter values changed from DCD to FD (K, Sy, Ss recharge)? → Golder gave some examples for hydraulic conductivity values and talked about recharge. • Has evapotranspiration been applied in the model? Golder explained that evapotranspiration has been applied in the model. Feflow is coded to not include evapotranspiration but net recharge that reach groundwater (infiltration from rainfall minus evapotraspiration). Other modelling codes such as Modflow required both, infiltration from rainfall and evapotranspiration as separate inputs and then net recharge is calculated by the code.	Note	-
4	**Schedule of Deliverables**		
	DPIE will provide comments on the FD report by 31-July 2020 to NRAR	DPIE / NRAR	31-July 2020

Jacobs Aurecon Joint Venture
Building 3, Level 7
189 O'Riordan Street,
Mascot NSW 2020

Jacobs Aurecon Joint Venture
WestConnex M4-M5 Link – Main Tunnel Works

Memorandum

To	Martin Knight (LSBJV)	**Date** 24 July 2020
Copies	Brian Griffiths (JAJV) Saina Emami (JAJV)	**Document ID** M4M5-JAJV-PRW-GEO-GW02-MEM-0092
From	Sven Padina	**Revision** A
Subject	GW02 - Peer Review of FD groundwater Model	

Martin

Attached for you information is the Peer Review report from Brian Barnett of the GW02 Final Design Report groundwater modelling.

Regards

Sven

Sven Padina

Principal Geotechnical Engineer | WestConnex M4-M5 Link - Main Tunnel Works | Jacobs

T +61 436 119 973 | **E** **sven.padina@jacobs.com**

Building 3, Level 7, 189 O'Riordan Street, Mascot NSW 2020

Jacobs Aurecon Joint Venture

Attached Jacobs Memo 20/07/20 – Groundwater Modelling Review – Final Design Stage

Jacobs

Memorandum

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Subject	**Groundwater Modelling Review - Final Design Stage**	**Project Name**	M4-M5 Link Tunnels
Attention	Sven Padina	**Project No.**	IA154801
From	Brian Barnett		
Date	20 July 2020		
Copies to			

1. Introduction

I am a groundwater modeller and hydrogeologist with 40 years of experience in the groundwater and geothermal industries in Australia, New Zealand and Japan. My CV is attached to this report as Attachment 1.

This Memorandum has been prepared as a formal peer review report of the Final Design (FD) Stage M4-M5 Link Tunnels Hydrogeological Design Report (the Report) and the Hydrogeological Numerical Modelling Report which is included as Appendix B (the Appendix) of the Report. For addition context and background, I was provided with a document entitled *Main Tunnel Works, Scope of Works and Technical Criteria – Appendix C.2 Project Company Documentation Schedule*, by Sydney Motorway Corporation Pty Limited of June 2018. I understand this document outlines the reporting standards and requirements for the various types of reports prepared for this project. I have referred to Section 1.12 Geotechnical and Hydrogeological Reporting as defining the reporting requirements for the Report and the Appendix.

My initial review was undertaken on 4th and 6th of June 2020. This was followed by discussions with the groundwater modeller, Irena Krusic-Hrustanpasic, to help clarify various aspects of the work and to resolve various issues raised in my initial review.

As part of the review I have prepared a log of issues spreadsheet that has acted as a record of review comments. Most of these issues were created as a result of my initial review and these were re-assessed following discussions with Ms Krusic-Hrustanpasic. The log of issues and responses are included in this report as Attachment 2.

2. Unresolved Issue from Substantial Detailed Design Stage - Reporting the time to steady state

In a previous review of the project groundwater modelling undertaken at Substantial Detailed Design (SDD) Stage I raised the following issue:

The SWTC Appendix C2, Section 1.12 defines the reporting requirements for Hydrogeological Interpretive Reports. At item iv) on Page 10, it is noted that the report should include "provision of a predictive model of the time to reach steady state conditions and the predicted effects of steady state condition.". The report does not address this issue.

At that stage I noted that the issue had not been resolved and had been deferred to the Final Design (FD) Stage modelling. I note that Section 10.3.1 of the Report (and Section 8.2 of the Appendix) includes a statement that modelled inflow rates approach a steady state after 85 – 90 years. Accordingly, I am happy that the requirement has been met.

3. General commentary

The model has been calibrated in steady state using groundwater heads measured in monitoring bores within the model domain and an acceptable level of calibration has been attained for the steady state approach.

A transient calibration was carried out in the FD Stage modelling utilising pumping and head observations from a 27 day pumping test carried out at a site near Hawthorne Canal. The work provides further constraints on model parameters within that region influenced by drawdown responses during the test. The calibration has resulted in local scale heterogeneity being added to the model in the vicinity of the test. It is recognised that the Hawthorne Canal site is of particular significance due to the presence of an alluvial palaeochannel with elevated permeabilities in the underlying rock. In this regard the additional transient calibration represents an improvement in the confidence with which the model can be used in predictive analysis.

Modelling has also helped identify and partially quantify areas of concern at:

- Hawthorne Canal, where elevated inflows are expected as the project intersects a high permeability rock zone beneath the canal,
- At Alexandra and Sydney Park, where contaminated groundwater is expected to seep from nearby landfills towards project elements at this location,
- Iron Cove Bay, where the project is in proximity to the harbour and there is a potential for the migration of seawater towards the tunnel.

Drawdown propagation at post construction steady state indicates substantial levels of drawdown are expected to occur as a result of the project. This outcome is consistent with the fact that the tunnel and associated structures will be drained in the long term.

The modelling report (the Appendix) includes details of a sensitivity analysis that has been performed to help illustrate the level of variability in groundwater inflows and associated drawdown that would result from the adoption of different model parameters. In this case the sensitivity analysis involved ten predictive model runs each with a nominal variation of a single parameter or boundary condition from its calibrated value. Estimated groundwater inflow rates and drawdown are presented for each of the ten runs providing a range of potential outcomes. The analysis provides useful insights into the sensitivity of the predictions to various parameters and boundary condition assumptions and has been used to address predictive uncertainty.

My detailed review comments are documented in the Log of Issues included as Attachment 2. Many of these issues related to how the work was reported and these have been satisfactorily resolved.

4. Assessment against guidelines checklist

An assessment of the modelling against the Australian Groundwater Modelling Guidelines compliance checklist is presented in Table 4.1.

Table 4.1: Compliance checklist

Question	*Yes/No*	*Comments*
1. Are the model objectives and model confidence level classification clearly stated?	yes	
2. Are the objectives satisfied?	yes	
3. Is the conceptual model consistent with objectives and confidence level classification?	yes	Confidence level is set in project SWTC and limited by duration of available groundwater monitoring data
4. Is the conceptual model based on all available data, presented clearly and reviewed by an appropriate reviewer?	yes	
5. Does the model design conform to best practice?	yes	
6. Is the model calibration satisfactory?	yes	
7. Are the calibrated parameter values and estimated fluxes plausible?	yes	
8. Do the model predictions conform to best practice?	yes	
9. Is the uncertainty associated with the predictions reported?	yes	A sensitivity analysis has been undertaken and this provides useful insights into potential variability of outcomes.
10. Is the model fit for purpose?	yes	

The checklist suggests that the work is generally compliant with the Guidelines. Predictive uncertainty has been addressed and reported through the implementation of a sensitivity analysis that illustrates potential variability in key modelling outcomes. The approach may be considered appropriate for a Class 2 Confidence Level Classification. The limited availability of data has meant that the model can strictly only achieve a Class 2 rating. However, it is noted that there have been a number of tunnelling projects undertaken within the metropolitan Sydney environment and the experience gained from modelling, constructing and operating these facilities adds confidence to the model and its predictions. Indeed, it could well be argued that such experience could elevate the confidence associated with model predictions above those normally associated with a Class 2 model.

5. Conclusion

The modelling has been undertaken in a manner that is consistent with current industry standards and is generally consistent with the recommended approach and Guiding Principles included in the Australian Groundwater Modelling Guidelines [1]. The work is consistent with similar infrastructure modelling investigations that I have been involved with (as a modeller and as reviewer) in recent years.

In light of the above discussion I consider the FD model to be of Class 2 Confidence Level Classification and is fit for purpose for assessing tunnel inflows and associated drawdown impacts.

[1] Barnett B, Townley LR, Post V, Evans RE, Hunt RJ, Peeters L, Richardson S, Werner AD, Knapton A and Boronkay A 2012. Australian groundwater modelling guidelines. Waterlines Report #82, National Water Commission, Canberra.

Attachment 1 Brian Barnett CV

Brian Barnett

PRINCIPAL GROUNDWATER MODELLER

Brian Barnett has forty years' experience in groundwater resource assessment, groundwater modelling, hydrogeology and geothermal reservoir engineering. He has acquired a broad range of modelling experience through numerous technical investigations of groundwater resources, mine dewatering and water management, impacts of land use change and contaminant transport. He was SKM's Practice Leader in Groundwater Modelling and leads the Australian groundwater modelling team in Jacobs. He is responsible for ensuring the highest technical standards in all aspects of numerical modelling of groundwater flow and solute transport throughout the company. Brian is a principal author and editor of the Australian Groundwater Modelling Guidelines. Published in 2012, the Guidelines have been accepted throughout Australia as a benchmark defining best industry practice.

EDUCATION/QUALIFICATIONS

Bachelor of Engineering (Civil) Honours, University of Auckland, 1980.

MEMBERSHIPS AND AFFILIATIONS

Member of the International Association of Hydrogeologists

Areas of Expertise

- Groundwater Modelling.
- Solute Transport Modelling
- Hydrogeology
- Geothermal Reservoir Engineering

Relevant Project Experience

Westgate Road Tunnel

Client: Transurban

Title: Groundwater Modeller.

Start/End Dates: March 2017 to Present

Scope/Description: The Westgate Tunnel is a major road construction project in Melbourne that is designed to provide a major western arterial route and alternative to the existing and heavily used Westgate Bridge. The project includes the construction of dual road tunnels beneath existing infrastructure to the west of the CBD. Tunnelling activities are planned in the region of the Yarra delta where vertical superposition of highly permeable aquifers and highly compressive silts provide unique and potentially hazardous construction conditions. Groundwater modelling is being used to assess tunnel inflows and associated drawdown impacts.

Responsibilities: Supervision of all groundwater modelling components of the investigation.

Melbourne Metro Rail Tunnel

Client: Melbourne Metro Rail Authority

Title: Groundwater Modeller.

Start/End Dates: March 2015 to Present

Scope/Description: The Melbourne Metro Rail Tunnel is a major infrastructure project that involves the construction of a rail tunnel and five underground stations beneath the central business district of Melbourne. One of the key project risks is the potential for subsidence arising from groundwater drawdown that will occur during construction and operation of the tunnel. Groundwater modelling has been used to assess the potential impacts.

Responsibilities: Review of the groundwater modelling components of the investigation.

Australian Groundwater Modelling Guidelines

Client: National Water Commission

Title: Project Manager.

Start/End Dates: March 2011 to June 2012

Scope/Description: The Australian Groundwater Modelling Guidelines was produced by SKM (now trading as Jacobs) and a team of leading groundwater modelling exponents drawn from the private and public sector including consultants, academics and regulators. Brian took a leading role in managing the project and the team and in editing and writing the document. It has been widely adopted throughout Australia as the benchmark for best industry practice for groundwater modelling in Australia. The Guidelines were published by the National Water Commission in June 2012.

Responsibilities: Project manager, co-editor and principal author.

Groundwater Model of the Myalup Irrigated Agriculture Precinct

Client: Department of Water and Environmental Regulation (WA)

Title: Project Manager and Lead Modeller.

Start/End Dates: October 2017 to Present

Scope/Description: Development of a three dimensional groundwater flow model to assess the sustainable yield of a coastal aquifer system south of Perth, Western Australia. The work is being undertaken to support water allocation planning by the DWER. A FEFLOW regional model is being constructed, calibrated and run in predictive mode to assess the long term impacts of various levels of future groundwater extraction. In addition, a series of two dimensional vertical slice models are also being constructed and used to assess density dependent solute transport in the coastal setting as required to simulate saltwater intrusion that may arise from on-going future groundwater development.

Responsibilities: Project manager and lead modeller.

North Stockton Sandbeds Groundwater Model

Client: Hunter Water Corporation

Start/End Dates: 2003 to 2004

Scope/Description: The North Stockton Sandbeds is a coastal aquifer near Newcastle, NSW. Hunter Water commissioned SKM to investigated its potential to provide groundwater for municipal water supply during periods of drought. Groundwater flow models were developed in the Modflow finite difference modelling code FEFLOW finite element code. A series of two dimensional vertical slice models were developed in FEFLOW density dependant solute transport mode to assess potential impacts on the seawater freshwater interface of various groundwater extraction scenarios.

Responsibilities: Project manager, Lead modeller

Tomago Sandbeds – Groundwater Modelling.

Client: Hunter Water Corporation.

Brian Barnett
PRINCIPAL GROUNDWATER MODELLER

Start/End Dates: October 2003 to December, 2010

Scope/Description: Groundwater modelling services were provided to Hunter Water Corporation over a period of eight years involving a number of significant modelling assignments. The work has included:

1. Development of a groundwater flow and solute transport model of the North Stockton Sandbeds (adjacent to the Tomago Sandbeds) in 2002 - 2003.

2. Development of a groundwater model of the Tomago Sandbeds to assist with assessment of impacts on GDE's in 2005.

3. Development of a local scale model of the Tanilba Bay WWTW on the Tomago Sandbeds in 2008.

4. Upgrading of the Tomago Sandbeds Gorundwater model in cooperation with Hunter Water staff in 2010 to assist with water resource planning.

Responsibilities: Project manager and lead modeller for all projects.

Murray Darling Basin Sustainable Yields Project

Client: CSIRO

Title: Groundwater Modeller

Start/End Dates: 2007 to 2008

Scope/Description: Brian was groundwater modelling team leader for a major project covering groundwater resources in Queensland, New South Wales, Victoria and South Australia. SKM was contracted by CSIRO in 2007 to undertake the groundwater resource assessment for the entire Murray Darling Basin. The project involved the numerical modelling of all major fresh water aquifers in the basin. Twelve finite difference numerical models were run for the study. Results were used to quantify the available groundwater resources of the basin and to assess the impacts of future climate change and impacts of groundwater development on river flows

Responsibilities: Leader of the groundwater modelling team that included eight modellers working in SKM's Melbourne, Adelaide and Sydney offices.

Upper Macquarie groundwater model, NSW

Client: New South Wales Office of Water (NOW)

Title: Groundwater Modeller.

Start/End Dates: April 2009 to June, 2010

Scope/Description: Development of a numerical groundwater model of the Upper Macquarie Catchment Groundwater Management Area. The model was developed to assist NOW in their on-going management of the water resources of the Upper Macquarie alluvial aquifer. Groundwater is used extensively for municipal water supply (including the city of Dubbo) and for irrigation.

Responsibilities: Project manager and supervising modeller responsible for the design and construction of the model and the calibration and predictive analysis and uncertainty analysis.

Frieda River Mine Dewatering Investigations, Papua New Guinea.

Client: Xstrata Copper.

Brian Barnett

PRINCIPAL GROUNDWATER MODELLER

Title: Groundwater Modeller.

Start/End Dates: October 2012 to March, 2013

Scope/Description: Groundwater modelling of a proposed copper mine in Papua New Guinea highlands. Groundwater models using the finite element FEFLOW modelling code were used to estimate the dewatering pumping requirement for the mine and to provide an assessment of the environmental impacts that may accompany mine dewatering and operation of water storage and tailings storage facilities.

Responsibilities: Lead modeller responsible for the design and construction of the model and its use in predictive scenarios.

Millstream Aquifer Model, WA.

Client: Western Australia Department of Water.

Title: Groundwater Modeller.

Start/End Dates: October 2008 to February, 2010

Scope/Description: Groundwater modelling of an inland aquifer in the Pilbara area of Western Australia. The aquifer is used for municipal water supply purposes and the project was aimed at helping to determine sustainable extraction rates from the aquifer. A principal constraint on future development is the requirement to protect and maintain iconic groundwater dependent river pools and springs.

Responsibilities: Project Manager and lead modeller responsible for the design and construction of the model and its use in predictive scenarios.

Collie Coal Basin – Groundwater Model, WA

Client: Western Australia Department of Water.

Title: Groundwater Modeller.

Start/End Dates: June 2009 to February, 2010

Scope/Description: The groundwater resources of the Collie Basin are heavily impacted by many years of coal mining and power generation. A groundwater model of the basin was developed and calibrated and used to assess future impacts that may arise from expanded coal mining and increased water extraction for dewatering and power station cooling.

Responsibilities: Project Manager and supervising modeller

Barwon Downs – Groundwater Model, Victoria

Client: Barwon Region Water Authority.

Title: Groundwater Modeller.

Start/End Dates: 2003 to present

Scope/Description: Brian has worked for a number of years with Barwon Water on the development and use of a complex groundwater flow model of the Barwon Downs Graben in Western Victoria. The Graben hosts deep confined aquifers that are used for water supply for the City of Geelong and surrounding urban centres. Work has continued for a number of years and has progressed from initial model design and development through various stages of upgrade and refinement. The work has been instrumental in allowing Barwon Water to secure ongoing groundwater extraction licenses for the borefield.

Responsibilities: Lead modeller

Lower De Grey and Lower Robe Groundwater Models, WA

Brian Barnett

PRINCIPAL GROUNDWATER MODELLER

Client: Western Australia Department of Water.

Title: Groundwater Modeller.

Start/End Dates: 2009 to 2010

Scope/Description: Groundwater models of two coastal alluvial aquifer systems in the Pilbara Region of Western Australia were develop and calibrated for the WA government. The work was aimed at defining the sustainable extraction limits for potential water supply borefields that may in future be used to supplement the Pilbara water supply.

Responsibilities: Project Manager and Supervising Modeller

Kulwin and WRP mineral sand mine groundwater models, Victoria.

Client: Iluka Resources Ltd.

Title: Groundwater Modeller.

Start/End Dates: 2002 to 2009

Scope/Description: Groundwater models of two mineral sand mines in northern Victoria were developed and calibrated to assist in the design of mine dewatering and water disposal facilities required to support a dry mining operation. The modelling work was instrumental in establishing the feasibility of mining these deposits that are deep below the water table and in securing the required environmental approvals and water licenses for the project.

Responsibilities: Project Manager and Lead Modeller.

Career Summary

May 2000 to present – Senior/Principal Groundwater Modeller at SKM/Jacobs (Melbourne, Australia). Responsible for leading the Australian groundwater modelling practice in SKM and Jacobs (Jacobs acquired SKM in Dec. 2014).

March 1997 to May 2000 – Senior Geothermal Reservoir Engineer and Groundwater Modeller at Kingston Morrison (Auckland New Zealand). Responsible for geothermal reservoir engineering assessments including numerical reservoir modelling of high temperature, two phase fluid reservoirs used for geothermal power generation. Also responsible for hydrogeological investigations including groundwater modelling.

1991 to 1997 – Geothermal Reservoir Engineer at Sumiko Consultants (Tokyo, Japan). Responsible for geothermal reservoir engineering assessments including numerical reservoir modelling using the TOUGH2 code to simulate high temperature two phase reservoirs in Japan.

1981 to 1991 – Geothermal Reservoir Engineer at Geothermal Energy New Zealand Ltd. (Auckland, New Zealand). Responsible for geothermal field measurements and reservoir assessments in Indonesia, Japan, Greece and Kenya.

1979 to 1981 – Groundwater Engineer at the Hawkes Bay Regional Water Board (Napier, New Zealand). Responsible for hydrogeological investigations including aquifer tests and water quality assessments for an environmental regulator.

Attachment 2 – Log of Issues

PROJECT PACKAGE NUMBER
M4M5-JAJV-PRW-GEO-GW02-RPT-0005
PROJECT PACKAGE TITLE
Hydrogeological Design Report
Design Report: M4M5-JAJV-PRW-GEO-GW02-RPT-0005
Drawing List: N/A

DATE ISSUED	SUBMISSION
21/01/2019	DCD
16/12/2019	SDD

DESIGN ISSUE CATEGORY LEGEND

1 Minor Issue - does not require a detailed response to the verifier
2 Moderate Issue - requires response from the designer
3 Significant design issue to be resolve

RESPONSE STATUS LEGEND

O Open Comment
C Closed Comment
W Withdrawn Comment
N Action for Next Phase

No.	Phase	Discipline	Doc, Rev (Page Label)	Reviewer Name (Author)	Initial Comment Date	Reviewer Initial Comment	Issue Category	Designer Response	Initial Response Date	Response Status	Reviewer Comment Closeout	Date Closed
Final Design Stage - Hydrogeological Design Report Rev. C2												
75	FD	GT	Section 7.6	Brian Barnett	4/06/2020	The paragraph immediately below Figure 7-8 on page 46 suggests that the pumping test results reflect the unfractured rock mass permeability. I don't follow the logic here. Just because the analytical method assumed isotropic radial flow, it doesn't mean that the result is not influenced by the presence of fractures. Results are unlikely to be indicative of the competent rock mass.	2	The text was edited to refer to bulk hydraulic conductivity.	11-Jun-2020		Resolved	15-Jun-20
76	FD	GT	Section 7.6	Brian Barnett	4/06/2020	Page 46. Please standardise the number formats using superscripts for exponents.	1	Noted. Has been corrected.	11-Jun-2020		Resolved	15-Jun-20
77	FD	GT	Section 7.7, page 47	Brian Barnett	4/06/2020	In the paragraph immediately folowing Table 7-5, I suggest inclusion of definitions of total porosity and effective porosity so we can understand the significance of the distinction.	1	Noted. Definition added.	11-Jun-2020		Resolved	15-Jun-20
78	FD	GT	Section 7.7, page 47	Brian Barnett	4/06/2020	In the second paragraph following Table 7-5 make clear that these values were adopted in the numerical groundwater model.	1	This is stated in the third paragraph after Table 7-5.	11-Jun-2020		Resolved	15-Jun-20
79	FD	GT	Section 8.1.2, page 48	Brian Barnett	4/06/2020	Please clarify whether planned waterway naturalisation is accounted for in the numerical model predictions of the long term operational phase impacts?	1	Planned waterway naturalisation was not included in the model as there are no information when this may happen and to what extent.	11-Jun-2020		Resolved	15-Jun-20
80	FD	GT	Section 8.2, page 50	Brian Barnett	4/06/2020	Quotes from WSP are of questionable value. If these quotes are to remain there should be more context because at the moment I am struggling to understand how such claims can be made.	3	WSP quote removed.	11-Jun-2020		Resolved	15-Jun-20
81	FD	GT	Section 8.4, page 53	Brian Barnett	4/06/2020	The paragraph that starts "Published experiences…" Should include references to the published material.	1	The text was updated to include references to the published material.	11-Jun-2020		Reolved	15-Jun-20
82	FD	GT	Section 8.5.2, page 55	Brian Barnett	4/06/2020	Sentence immediately before Figure 8-4 - I'm not sure what this observation has to do with Vertical Head Gradients? If it is important then you should provide figures that show the measured tidal responses.	1	The sentence has been removed.	11-Jun-2020		Resolved	15-Jun-20
83	FD	GT	Section 10.2 page 69	Brian Barnett	4/06/2020	Include a comment as to why the model was not used to assess construction inflows and impacts.	3	Understand this comment has been withdrawn.	11-Jun-2020		Withdrawn	15-Jun-20
84	FD	GT	Section 10.3.1 page 70	Brian Barnett	4/06/2020	Incomplete sentence starts the 3rd paragraph in this Section.	1	The figure references were missing. This has been corrected.	11-Jun-2020		Resolved	15-Jun-20
85	FD	GT	Section 10.3.1 page 75	Brian Barnett	4/06/2020	Is there any reason for separating into two tunnels. Normally a single inflow estimate that treats the tunnel as one structure would be appropriate?	1	It is a project requirement to report inflows for each carriageway separately.	11-Jun-2020		Withdrawn	15-Jun-20
86	FD	GT	Section 10.4.4 page 84	Brian Barnett	4/06/2020	In figures such as Fig 10-12, it should be possible to put time markers on the flow lines. This will provide useful information on the time taken before impacts can be expected.	1	The time markers on the flow line would show arrival of the maximum concentrations due to pure advection flow. This could be misleading as saline impacted groundwater would be expected to arrive earlier due to dispersion and density gradient driven flow. For tunnel design purposes it is more important to understand where intrusion will happen over the design life of the structure.	11-Jun-2020		Resolved	15-Jun-20
87	FD	GT	Section 10.4.6 page 87	Brian Barnett	4/06/2020	The first paragraph on Page 87 includes a quote from AECOM 2017. While I appreciate that this is a quote, it doesn't really stand up to scrutiny and is not supported by basic hydrogeological principles. I'd prefer that these quotes (also applies to comment 80 above) were removed completely from the document.	3	The quote has been removed and some additional words were added with respect to referencing the EIS.	11-Jun-2020		Resolved	15-Jun-20
88	FD	GT	Section 11.10 page 95	Brian Barnett	4/06/2020	Final dot point of Section 10.11 - further reporting detail is warranted here. I am interested in understanding the plan for monitoring of inflows and whether or not estimates of inflow per kilometre can be obtained. If not then compliance, with inflow criterion is difficult/impossible to assess.	2	Refer to M4-M5 Link Tunnels CEMP: Groundwater Monitoring Program for further details regarding monitoring to be undertaken.	11-Jun-2020		Resolved	15-Jun-20

Final Design Stage - Hydrogeological Design Report Rev. C2 Appendix B Groundwater Modelling Report												
89	FD	GT	Appendix B, Section 4.4.2	Brian Barnett	4/06/2020	This section describes groundwater levels. It should include a figure showing potentiometric surface map - perhaps refer to Figure 4-5.	1	Agree. Figure 4-5 moved into Section 4.4.2 and renamed Figure 4-3.	11-Jun-2020		Resolved	15-Jun-20
90	FD	GT	Appendix B, Section 4.4.3, page 11	Brian Barnett	4/06/2020	Second dot point under Figure 4-4 concludes recharge rates are dependent on rainfall intensity and duration. This is not supported by the data presented. Information on duration of the rainfall events are not reported.	1	Groundwater levels did not respond to the August 2019 rainfall in any of the VWPs. The total rainfall at that time was 40 mm and occurred over one day. However groundwater levels in three out of five VWPs responded to the higher intensity rainfall event (total of 102 mm) that occurred over 3 days in September (17 Sep to 19 Sep), We believe this support our comment. Text clarified to support conclusions made.	11-Jun-2020		Resolved	15-Jun-20
91	FD	GT	Appendix B, Section 4.4.4, page 11	Brian Barnett	5/06/2020	Evapotranspiration is often a significant groundwater discharge mechanism - the report should address whether ET is a significant component of the water budget. This is particularly relevant since Section 6.4.2 discusses the effects of ET on net recharge rates.	1	Additional text added as follows: Evapotranspiration was also considered to be significant component of the water budget and source of water loss from the groundwater system.	11-Jun-2020		Resolved	15-Jun-20
92	FD	GT	Appendix B, Section 5.2, page 15-16	Brian Barnett	5/06/2020	The paragraph below Figure 5-2 should include additional context by stating that the base of the model, at -120 mAHD, is set as a no-flow boundary.	1	The text below Figure 5-2 and in Section 5.3 has been updated to clarify the base of the model was set as a no-flow boundary.	11-Jun-2020		Resolved	15-Jun-20
93	FD	GT	Appendix B, Section 5.3, page 20	Brian Barnett	5/06/2020	The first sentence following Figure 5-6 should read *"Heads assigned to the Cauchy boundary conditions for creeks and canals…"*. The last sentence of the second paragraph below Figure 5-6 refers to M4 and M5 "***waterways***". I assume this should be motorways?	1	This was a typo that has been corrected.	11-Jun-2020		Resolved	15-Jun-20
94	FD	GT	Appendix B, Section 6.1, page 22	Brian Barnett	5/06/2020	The first paragraph should be moved to below the dot points. I want to understand the calibration strategy before reading how calibration was achieved.	1	Paragraph was moved below dot points.	11-Jun-2020		Resolved	15-Jun-20
95	FD	GT	Appendix B, Section 6.1, page 22	Brian Barnett	5/06/2020	The final paragraph of Section 6.1 doesn't make sense. *"i.e. defined by nearest nondependent model slice above."* What is defined by the nearest nondependent model slice above? Since all model slices are dependent except for the top slice then all model slices are defined by the top slice. I have no idea what this means.	2	The text in brackets has been removed.	11-Jun-2020		Resolved	15-Jun-20
96	FD	GT	Appendix B, Section 6.2.2, page 23	Brian Barnett	5/06/2020	Last paragraph on page 26 discusses porosity. Feflow phreatic surface option requires specific yield value and not porosity. Suggest the paragraph refer to specific yield.	1	The text was edited as model was calibrated to drainable porosity, i.e., specific yield.	11-Jun-2020		Resolved	15-Jun-20
97	FD	GT	Appendix B, Section 6.2.3, page 24	Brian Barnett	5/06/2020	In Figure 6-1 please clarify whether the drawdown contours are predicted by the model or contoured from observations.	2	Clarification added.	11-Jun-2020		Resolved	15-Jun-20
98	FD	GT	Appendix B, Section 6.2.3, page 24-25	Brian Barnett	5/06/2020	Add tunnel alignment to Figures 6-1 and 6-2	2	The tunnel locations have not been added to the Figures 6-1 and 6-2 as the calibration is related to pre-excavation conditions.	11-Jun-2020		Resolved	15-Jun-20
99	FD	GT	Appendix B, Section 6.2.3	Brian Barnett	5/06/2020	While it is clear that the pumping test calibration requires the introduction of heterogeneity in aquifer properties it is likley that a similar calibration could be attained with different parameter values and zonations. I think this issue should be acknowledged as A) the zones do not appear to be aligned with geological features that would help support the chosen zonation and B) the choice of zone shape and extent may have a significant influence on the predictive model outcomes.	3	Agree that there is no unique solution for any model calibration. The success of calibration process depends heavily on the monitoring data available and distribution of monitoring points across the model domain, which could considerably limit alternative solutions. Drawdown data recorded during the pumping test indicated an elongation of the groundwater drawdown cone of depression in an easterly/south easterly direction where quite a good coverage of monitoring points existed. Initially calibration of the model considered a uniform hydraulic conductivity (as derived from the analytical solutions) which was extended radially away from the pumping well. This solution resulted in a significant underestimation of the groundwater drawdown at the distant wells, particularly at monitoring wells LSB-MT-BH014a, HB_BH14 and LSB-GW-HB-BH012 and the observed elongation could be replicated. To achieve the observed elongation, a narrow zone of a higher hydraulic conductivity in the direction of drawdown elongation needed to be introduced. Direction and distribution of this zone was tested through a number of calibration runs until a satisfactory solution was achieved. Although the adopted solution is not an unique solution, the calibration process showed that alternative solutions were constrained by the shape of groundwater drawdown cone observed.	11-Jun-2020		Resolved	15-Jun-20

100	FD	GT	Appendix B, Section 6.4.1, page 32	Brian Barnett	5/06/2020	Table 6-4 and accompanying description refers to porosity when the model uses specific yield.	1	Text was corrected to refer to specific yield.	11-Jun-2020		Resolved	15-Jun-20
101	FD	GT	Appendix B, Section 7.3, page 39	Brian Barnett	5/06/2020	Predictive model uses Unsaturated/saturated option whereas calibration uses Phreatic Option. Some explanation is required as to why this approach was adopted and whether calibration has been repeated with the Unsaturated/saturated option.	3	The unsaturated/saturated option was used in order to achieve a more stable model and to improve the water budget for the predictive runs. Difference between unsaturated/saturated and phreatic model options is related only to treatment of the transient water table response in the model. Considering that the inflow into tunnels and development of the groundwater drawdown will be driven by the deeper groundwater system in the long term, these differences are not expected to significantly affect the model's inflow and drawdown predictions. An additional pumping test simulation, however, has been undertaken using the unsaturated/saturate option and the results from this model simulation indicate there no changes to the groundwater response within the deep rock aquifer, while changes in the alluvial aquifer were not significantly different from the phreatic model.	11-Jun-2020		Resolved	15-Jun-20
102	FD	GT	Appendix B, Section 8.1	Brian Barnett	5/06/2020	figures 8-1 to 8-5 and 8-11 to 8-13 show results for the operational phase - how long after the tunnel construction do the results represent?	2	The operational phase results are for the period 100 years after tunnel opening. A note clarifying timing is added in the report and additional figures added to show the inflows shortly after tunnel opening.	11-Jun-2020		Resolved	15-Jun-20
103	FD	GT	Appendix B, Section 8.4	Brian Barnett	5/06/2020	Figures 8-14 and 8-15 would be improved if time markers were added to the particle traces.	2	See response to Comment # 86.	11-Jun-2020		Resolved	15-Jun-20
104	FD	GT	Appendix B, Section 8.5, page 55	Brian Barnett	5/06/2020	The statement *"Although the predicted percentage reduction in groundwater contribution to baseflow in some cases is large, this reduction represents a small reduction in the overall stream flow, as the baseflow simulated in the model only represents the occasions when the groundwater reaches ground level and enters the waterbody."* in the first paragraph of Section 8.5 is misleading and inappropriate. Large reductions in baseflow are not mitigated by the fact that they occur when groundwater heads are above stream level. If the predicted change in base flow in the model is small with respect to stream flow then it is because there is significant baseflow contribution from stream reaches outside the model domain.	3	The text has been updated and statement removed	11-Jun-2020		Resolved	15-Jun-20
105	FD	GT	Appendix B, Section 8.5, page 55	Brian Barnett	5/06/2020	In paragraph 2 of Section 8.5, it is hard to accept the statement *"it is likely that the majority of the stream flow is derived from stormwater runoff"*. Runoff events in an urban environment are usually of limited duration and will not sustain permanent flows. If the creeks are no more than stormwater drains then they should be described as such.	3	See response to Comment # 104 above	11-Jun-2020		Resolved	15-Jun-20
106	FD	GT	Appendix B, Section 9.	Brian Barnett	5/06/2020	While the sensitivity analysis described in the section provides valuable information on the range of predictive outcomes that should be allowed for, it should be noted that it is not a calibration constrained uncertainty analysis. It is difficult to appreciate how many of these scenarios would produce acceptable calibration results.	1	The text has been edited, to make clear that the reference to Modelling Guidelines is related to sensitivity analyses and not the uncertainty analysis.	11-Jun-2020		Resolved	15-Jun-20
107	FD	GT	Appendix B, Section 9, page 59	Brian Barnett	5/06/2020	In table 6-1, a more accurate definition of the impact of each scenario on the 20 m drawdown contour would be the calculated area within the contour.	1	Agree that this would be beneficial, however this could also be misleading when difference in the drawdown extent are within localised areas. In some cases where groundwater drawdown extends uniformly further away than the base case, the area of the 20 m contour extent may not be distinguished clearly from the area where only localised higher extent drawdown occurred (variable hydraulic conductivities for example).	11-Jun-2020		Resolved	15-Jun-20

Jacobs Aurecon Joint Venture
Building 3, Level 7
189 O'Riordan Street,
Mascot NSW 2020

Jacobs Aurecon Joint Venture
WestConnex M4-M5 Link – Main Tunnel Works

Memorandum

To	Grant Sainsbery (LSBJV), Martin Knight (LSBJV)	**Date** 30 June 2020
Copies	Brian Griffiths (JAJV)	**Document ID** M4M5-JAJV-PRW-GEO-GW02-MEM-0088
From	Sven Padina	**Revision** A
Subject	Response to DPIE Water review of GW02 DCD groundwater modelling report	

In response to the DPIE-Water review of the Detailed Concept Design (DCD) groundwater modelling report (design package GW02), letter dated 22 June 2020, doc ref OUT20/7501, Table 1 below presents both the DPIE-Water review comment, together with JAJV response to close out each comment. For reference the DPIE-Water letter is also attached.

It is important to note that since issue of the DCD report to DPIE-Water in Aug 2019, project wide groundwater modelling and reporting has been substantially progressed and developed as presented in the Substantial Detailed Design (SDD) report, dated 20/12/019, and now the Final Design (FD) report, dated 19/6/20.

We consider the FD report now fully addresses and closes out all the DPIE-Water DCD report review comments.

Table 1: DPIE Water GWO2 DCD Report review comments and JAJV close out responses

Item	DPIE-Water Review Comment	JAJV Response
1	The next stage of modelling must include revision of the groundwater modelling objectives to include assessment of cumulative effects of all infrastructure projects that intercept the model domain.	The FD model now includes the New M5 and M4 East tunnels, both of which have a significant influence on the groundwater system and model predictions at either end of the M4-M5 Link because of their extent and drained nature. The Sydney Metro and Sydney Water Pressure tunnels have not been included because they are sealed or steel lined structures and their influence on the model predictions would therefore be small. Refer to Section 6.2 of the Hydrogeological Design Report (HDR) for further details.
2	The next version of the conceptual and numerical models must be updated using recently collected data as well as all available information.	The FD Model was informed by the most recent data from the LSBJV detailed design site investigation and Hawthorne Canal Pumping Test, observations from construction of the initial tunnels, as well as the available third-party data included in the factual report (GW01). Refer to Sections 4.4 and Section 6.0 in the FD Groundwater Numerical Modelling Report.

Memorandum

Item	DPIE-Water Review Comment	JAJV Response
3	The next version of the model must be enhanced to meet Class 2 model confidence level according to the AGMG (2012) and as many Class 3 attributes as possible. This entails transient model calibration and predictive runs.	The FD Model is considered to meet a Class 2 Confidence Level with elements of Class 3 based on Table 2-1 of the Australian Groundwater Modelling Guidelines. The model Classification is discussed in Section 11 of the FD Groundwater Numerical Modelling Report.
4	The HDR and modelling report are required to provide an assessment of their compliance with the Ministers Conditions of Approval (MCoA), Scope of Works & Technical Criteria (SWTC) conditions, and the Environmental Management Measures as specified in Chapter 19 – Groundwater of the Environmental Impact Statement (EIS).	Compliances with MCoA and SWTC conditions and Revised Environmental Management Measures (REMM's) have been addressed through the HDR and Groundwater Numerical Modelling Report, with a compliance summary provided in Section 10 of the HDR and Appendix C and Appendix D.
5	Future issues of the modelling report must include sensitivity and uncertainty analyses as described in the AGMG (2012) and the recent uncertainty analysis guidance note (Middlemis and Peeters, 2018)#.	The uncertainty and comprehensive sensitivity analyses undertaken are documented in Section 9 of the FD Groundwater Numerical Modelling Report.
6	It is recommended to integrate the HDR and the groundwater modelling report to enhance the effectiveness of hydrogeological reporting.	The Modelling Report is provided as Appendix B of the HDR, i.e., it is an integral part of the overall hydrogeological interpretation and reporting.
7	The conceptual model, numerical model implementation and the modelling report must be progressively peer-reviewed by independent qualified experts as recommended in the AGMG (2012)*. The reviews should be annexed to the future versions of the modelling report.	A progressive peer-review by independent qualified experts was an integral part of the model development as summarised below: • The DCD and SDD modelling work was reviewed by Dr Noel Merrick of SLR Consulting. A copy of Dr Merrick's review is provided in Appendix BD of the FD Numerical Modelling Report. • The SDD and FD modelling work was also reviewed by Brian Barnett from Jacobs who authored the Australian Groundwater Modelling Guidelines, 2012. This included a review of modelling approach, results of the transient calibration, Hawthorne Canal Pumping test results and interpretation, and a final review of the FD modelling work and predictions

Attached - DPIE-Water the Hydrogeological Design Report review letter, dated 22 June 2020, doc ref OUT20/7501,

Contact: Ellie Randall
Email: ellie.randall@nrar.nsw.gov.au

Grant Sainsbery
Environment & Sustainability Manager
Lendlease Samsung Bouygues Joint Venture
WestConnex M4-M5 Link Tunnels

email: Grant.Sainsbery@m4-m5linktunnels.com.au

Our ref: OUT20/7501

Dear Grant,

22 June 2020

WestConnex M4-M5 Link Tunnels – Hydrogeological Design Report

Thank you for giving the Department of Planning, Industry and Environment – Water (DPIE-Water) the opportunity to review the Hydrogeological Design Report for the WestConnex M4-M5 Link Tunnels. DPIE-Water has reviewed the report and provides the following comments:

1. The next stage of modelling must include revision of the groundwater modelling objectives to include assessment of cumulative effects of all infrastructure projects that intercept the model domain.
2. The next version of the conceptual and numerical models must be updated using recently collected data as well as all available information.
3. The next version of the model must be enhanced to meet Class 2 model confidence level according to the AGMG (2012) and as many Class 3 attributes as possible. This entails transient model calibration and predictive runs.
4. The HDR and modelling report are required to provide an assessment of their compliance with the Ministers Conditions of Approval (MCoA), Scope of Works & Technical Criteria (SWTC) conditions, and the Environmental Management Measures as specified in Chapter 19 – Groundwater of the Environmental Impact Statement (EIS).
5. Future issues of the modelling report must include sensitivity and uncertainty analyses as described in the AGMG (2012) and the recent uncertainty analysis guidance note (Middlemis and Peeters, 2018)#.
6. It is recommended to integrate the HDR and the groundwater modelling report to enhance the effectiveness of hydrogeological reporting.
7. The conceptual model, numerical model implementation and the modelling report must be progressively peer-reviewed by independent qualified experts as recommended in the AGMG (2012) *. The reviews should be annexed to the future versions of the modelling report.

[#] Middlemis H and Peeters LJM (2018). Uncertainty analysis—Guidance for groundwater modelling within a risk management framework. A report prepared for the Independent Expert Scientific Committee on Coal Seam Gas and Large Coal Mining Development through the Department of the Environment and Energy, Commonwealth of Australia 2018.
[*] Barnett *et al.* (2012). Australian groundwater modelling guidelines, Waterlines report, National Water Commission, Canberra.

Further details on the comments above can be found in the appendix. Should you have any further queries in relation to this submission please do not hesitate to contact the Natural Resources Access Regulator's Service Support Team at nrar.servicedesk@industry.nsw.gov.au.

Yours sincerely

Alison Collaros
Licensing and Approvals Manager (East)
Natural Resources Access Regulator
Department of Planning, Industry and Environment

Appendix

The review of the Hydrogeological Design Report concludes the following:

1. The reported model is fit for purpose as a preliminary model, upon which more robust conceptual and numerical modelling can be based.
2. The reported model is not fit for the purpose of predicting impacts on the environment, infrastructure or groundwater users.
3. The reviewed reports do not provide enough geological and hydrogeological cross-sections along the tunnel alignment to support the conceptual model and provide confidence in the numerical model implementation.
4. The presented steady-state model is calibrated using limited groundwater level data.
    - Improved calibration is required for the expected transient model.
    - This includes the use of multi-level piezometers or vibrating weir piezometers groundwater level data and flux (baseflow) calibration targets.
    - Calibration data should have better spatial coverage and longer data records (24 months required under the NSW Aquifer Interference Policy (2012)).
5. No sensitivity and uncertainty analyses are provided.
6. The HDR and groundwater modelling report acknowledge limitations in hydrogeological knowledge and the groundwater model and highlights measures to enable more robust modelling, reduce limitations and uncertainty.
7. The conceptual model, numerical model and modelling report have not been progressively reviewed by independent qualified experts as required by Australian groundwater modelling guidelines (AGMG, 2012)*.
8. The target model confidence level for this type of development according to the AGMG (2012) is set to Class 2 (intermediate). However, the model confidence level is determined to be Class 1 (lowest).
9. Cumulative effects of the project and other Sydney infrastructure projects had not been assessed.

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[*] Barnett *et al.* (2012). Australian groundwater modelling guidelines, Waterlines report, National Water Commission, Canberra.

LSBJV Memorandum

WestConnex M4-M5 Link Mainline Tunnel	Project No.: 259954
Reference No.:	GEO090
Date:	02/04/2020
Subject:	Groundwater Design Key Modelling Requirements according to the MCoA
Author:	Martin KNIGHT

Background

According to the Minister's Conditions of Approval (MCoA):

E192: The Proponent must undertake further modelling of groundwater drawdown, tunnel inflows and saline water migration (using particle tracking).

E193: The results of the groundwater modelling must be documented in a Groundwater Modelling Report. The Groundwater Modelling Report must be finalised in accordance with the Australian Groundwater Modelling Guidelines (National Water Commission, 2012) and prepared in consultation with DPI (Water).

E194: The groundwater model must be updated once 24 months of groundwater monitoring data are available and the results of the modelling provided to the Secretary and DPI (Water) in an updated Groundwater Modelling Report.

When preparing the Groundwater Design, the Contractor (LSBJV) and their Consultant (JAJV) have attempted to engage DPI (Water) – now DPIE – in consultation according to the conditions set out above, particularly condition E193. Despite exhaustive attempts from LSBJV and JAJV, no comments have been received. Finally, on 23rd March 2020 DPIE advised LSBJV that they would not be able to progress the groundwater model review due to drought and changes in personnel. Therefore, it is not possible to resolve this matter.

This memo provides a narrative and timeline of the correspondence between the LSBJV and DPIE to demonstrate that they have made all reasonable attempts to comply with the MCoA in this regard.

Reference Documents

Reference is made to the following documents:

GT01	Geotechnical Data Report; Reference M4M5-JAJV-PRW-GEO-GT01-RPT-0005
GW01	Groundwater and In-Ground Gas Factual Report; Reference M4M5-JAJV-PRW-GEO-GW01-RPT-0005
GW02	Hydrogeological Design Report; Reference M4M5-JAJV-PRW-GEO-GW02-RPT-0005

Correspondence Summary

Date	Action
19-August 2019	Meeting between DPIE, LSBJV, JAJV, Golder and SLR (peer reviewer) to provide a presentation with a project overview. Topics discussed included the need for consultation with DPIE according to the conditions of approval E192 and E193. LSBJV provided the interim SDD GW02 groundwater modelling report as two hard copies and 1 electronic copy on a USB drive. A review timeline was agreed with the first comments from DPIE in 4-weeks and LSBJV/JAJV would provide a response in 4-6 weeks thereafter with a view to proving the FD report after completion of the pumping test.
30-August 2019	Grant Sainsbury (GS) of LSBJV provided meeting minutes and agreed actions to Richard Green (RG - principle hydrogeologist) and Ellie Randall (ER- regulator responsible for co-ordination) of DPIE.
04-Sept 2019	GS emailed ER to advise that FD GT01 and GW01 were now available to be submitted to DPIE according to the actions agreed at the meeting.
05-Sept 2019	ER asked that GT01 and GW01 be sent to her on a USB drive due to the size of the reports. GS arranged accordingly.
19-Sept 2019	GS followed up the check if DPIE received GT01 and GW01 on the USB drive. RG replied he had received it that day and would upload it to google drive to share with ER.
22-Oct 2019	GS sent an email to DPIE prompting them for comments as the 4-week review period agreed had elapsed. GS advised that in order to meet the project deadlines comments would be required by 31-October as the SDD report would be issued by 8-November. GS proposed a meeting date of 11-November to review the DPIE comments. GS also provided a high-level programme of the GW02 milestones including completion of the pumping test and FD submission.
28-Oct 2019	ER replied to GS by email advising that DPIE intended to provided comments, but that RG was out of the office for 2-weeks and would respond on his return.
28-Oct 2019	GS replied to the email from ER asking to confirm the meeting for 11-November. ER agreed, asking for a draft agenda.
4-Nov 2019	GS emailed ER asking when the comments would be available.
12-Nov 2019	Further email from GS to ER asking for progress on the comments.

20-Nov 2019	Email from GS to ER asking for the comments to be provided within the week in order to incorporate them into SDD GW02.
20-Nov 2019	ER replied to the email from GS. ER asked for an updated electronic copy of GW02 and asked if we were waiting for comments on the hard copy of the GW02 report issued at the meeting of 19-Aug 2019.
20-Nov 2019	GS replied to the email from ER. The electronic copy of GW02 was provided in the meeting of 19-August on a USB drive and we were waiting for comments on that report which included the results of the hydrogeological model.
20-Dec 2019	GW02 was submitted SDD to M4-M5 Group.
14-Jan 2020	GS emailed ER asking for a date when we would receive comments on the interim model provided in August 2019. ER replied that she would check with RG.
17-Mar 2020	GS called and emailed ER asking for the comments or confirmation that no comments would be provided. ER forwarded the email to RG and advised she would be out of the office until 30-March 2020.
23-March 2020	GS called and emailed RG. The groundwater model has progressed in the absence of comments from DPIE and we assume that no comments from DPIE will be forthcoming.
23-March 2020	RG emailed GS to notify that due to drought conditions and change of staff they have not been able to progress with their review but requested a copy of the FD report to be submitted mid-April.

Attachments

- Email correspondence between LSBJV and DPIE; and
- Presentation provided to DPIE on 19-Aug 2019.

Attachment 1

Email correspondence between LSBJV and DPIE.

Martin Knight

From: Richard Green <richard.green@dpie.nsw.gov.au>
Sent: Monday, 23 March 2020 12:36 PM
To: Grant Sainsbery; Ellie Randall; Richard Green
Cc: Padina, Sven; Martin Knight
Subject: Re: FW: Meeting with Westconnex 3A 19 August 2019

Hi Grant

As discussed on the phone today due to drought and changes in staff we have not been able to progress our draft groundwater model review to meet your current deadline. Since the report will be out in mid April regardless I suggested you send us a copy of the final. We would be interested in the implementation work being undertaken in future.

Regards
Richard

Richard Green
A / Principal Hydrogeologist

Water | Department of Planning, Industry and Environment

Level 10 | 10 Valentine Avenue | Parramatta NSW 2150
Locked Bag 5123, NSW 2124
T: +61 2 9842 8643 | F: +61 2 8838 7854 | M: 0408 418 486
E: Richard.Green@dpie.nsw.gov.au
W: https://www.dpie.nsw.gov.au/

From: Grant Sainsbery <Grant.Sainsbery@m4-m5linktunnels.com.au>
Sent: Monday, 23 March 2020 12:06 PM
To: Ellie Randall <ellie.randall@dpi.nsw.gov.au>; Richard Green <richard.green@industry.nsw.gov.au>
Cc: Padina, Sven <Sven.Padina@jacobs.com>; Martin Knight <Martin.Knight@m4-m5linktunnels.com.au>
Subject: RE: FW: Meeting with Westconnex 3A 19 August 2019

Hi Richard,

We can appreciate it that you are busy with the drought and the need to access groundwater resources across the state.

In the absence of any comments, our State Significance Infrastructure project has been forced to progress its groundwater model based upon accepted industry practices. We now assume that you won't be issuing any comments.

Our complete model will be complete next month and will be issued.

Would you like a copy for your records?

Regards,
Grant Sainsbery
Environment & Sustainability Manager
Lendlease Samsung Bouygues Joint Venture
M +61 430 395 234
Grant.Sainsbery@M4-M5Linktunnels.com.au

From: Ellie Randall <ellie.randall@dpi.nsw.gov.au>
Sent: Tuesday, 17 March 2020 11:38 AM
To: Richard Green <richard.green@industry.nsw.gov.au>
Cc: Grant Sainsbery <Grant.Sainsbery@m4-m5linktunnels.com.au>
Subject: FW: FW: Meeting with Westconnex 3A 19 August 2019

Hi Richard,

Can you please provide a response to Grant on the status of this review? I will be out of the office this afternoon, returning the 30th of March.

Kind regards

Ellie Randall | Water Regulation Officer
Natural Resources Access Regulator | Water Regulation (East)
Level 0 | 84 Crown Street | Wollongong NSW 2500
PO Box 53 Wollongong NSW 2520
T: +61 2 4275 9308 | F: +61 2 4224 9740
E: ellie.randall@nrar.nsw.gov.au
W: www.industry.nsw.gov.au

From: Grant Sainsbery <Grant.Sainsbery@m4-m5linktunnels.com.au>
Sent: Tuesday, 17 March 2020 10:17 AM
To: Ellie Randall <ellie.randall@nrar.nsw.gov.au>
Subject: RE: FW: Meeting with Westconnex 3A 19 August 2019

Hi Ellie,

Just called you. How did you go with Richard on providing comments on the prelim model?

If he is too busy with the drought, I'd appreciate confirmation that no comments will be supplied.

Thank you.

Grant Sainsbery
Environment & Sustainability Manager
Lendlease Samsung Bouygues Joint Venture
M +61 430 395 234
Grant.Sainsbery@M4-M5Linktunnels.com.au

From: Ellie Randall <ellie.randall@nrar.nsw.gov.au>
Sent: Tuesday, 14 January 2020 8:17 AM
To: Grant Sainsbery <Grant.Sainsbery@m4-m5linktunnels.com.au>
Subject: Re: FW: Meeting with Westconnex 3A 19 August 2019

Hi Grant,

I will check in with Richard Green and see where he is at with this.

Thanks

Kind regards

Ellie Randall | Water Regulation Officer

Natural Resources Access Regulator | Water Regulation (East)
Level 0 | 84 Crown Street | Wollongong NSW 2500
PO Box 53 Wollongong NSW 2520
T: +61 2 *4275 9308* | F: +61 2 *4224 9740*
E: ellie.randall@nrar.nsw.gov.au
W: www.industry.nsw.gov.au

NSW GOVERNMENT | Natural Resources Access Regulator

On Tue, Jan 14, 2020 at 8:01 AM Grant Sainsbery <Grant.Sainsbery@m4-m5linktunnels.com.au> wrote:

Hi Ellie,

Happy new year!

Just checking on a likely date for the provision of comments on the model provided in August?

Regards,
Grant Sainsbery
Environment & Sustainability Manager
Lendlease Samsung Bouygues Joint Venture
M +61 430 395 234
Grant.Sainsbery@M4-M5Linktunnels.com.au

lendlease | SAMSUNG SAMSUNG C&T | BOUYGUES CONSTRUCTION AUSTRALIA

From: Grant Sainsbery
Sent: Wednesday, 20 November 2019 2:04 PM
To: Ellie Randall <ellie.randall@nrar.nsw.gov.au>
Cc: Richard Green <richard.green@industry.nsw.gov.au>; Water Referrals <water.referrals@dpi.nsw.gov.au>; Padina, Sven <Sven.Padina@jacobs.com>; Martin Knight <Martin.Knight@m4-m5linktunnels.com.au>
Subject: RE: FW: Meeting with Westconnex 3A 19 August 2019

Hi Ellie,

An electronic copy was provided on the USB stick we left on the day.

We are hoping for some comments on the report which includes the results of the hydrogeological model.

Thanks,
Grant Sainsbery
Environment & Sustainability Manager
Lendlease Samsung Bouygues Joint Venture
M +61 430 395 234
Grant.Sainsbery@M4-M5Linktunnels.com.au

From: Ellie Randall <ellie.randall@nrar.nsw.gov.au>
Sent: Wednesday, 20 November 2019 11:37 AM
To: Grant Sainsbery <Grant.Sainsbery@m4-m5linktunnels.com.au>
Cc: Richard Green <richard.green@industry.nsw.gov.au>; Water Referrals <water.referrals@dpi.nsw.gov.au>; Padina, Sven <Sven.Padina@jacobs.com>; Martin Knight <Martin.Knight@m4-m5linktunnels.com.au>
Subject: Re: FW: Meeting with Westconnex 3A 19 August 2019

Hi Grant,

The comments on the Flood Mitigation Strategy should be with you by the end of the week (awaiting management sign off).

Grant do you have an updated electronic version modelling report? Richard has mentioned he has a hard copy version handed out at the meeting (hydrogeological design report). Was this what you required comments on?

Kind regards

Ellie Randall | Water Regulation Officer

Natural Resources Access Regulator | Water Regulation (East)
Level 0 | 84 Crown Street | Wollongong NSW 2500
PO Box 53 Wollongong NSW 2520
T: +61 2 4275 9308 | F: +61 2 4224 9740
E: ellie.randall@nrar.nsw.gov.au
W: www.industry.nsw.gov.au

On Wed, Nov 20, 2019 at 10:12 AM Grant Sainsbery <Grant.Sainsbery@m4-m5linktunnels.com.au> wrote:

Hi Ellie and Richard,

Just checking in to see if you plan on issuing comments on the Prelim model? If so, when should we plan to expect them?

Similar to the Flood Mitigation Strategy, the ability to amend things in line with comments becomes increasingly difficult as the designs and models become more developed and finalised.

We would really appreciate your comments on the Prelim model in the next week.

Thanks,
Grant Sainsbery
Environment & Sustainability Manager
Lendlease Samsung Bouygues Joint Venture
M +61 430 395 234
Grant.Sainsbery@M4-M5Linktunnels.com.au

From: Grant Sainsbery
Sent: Tuesday, 12 November 2019 4:00 PM
To: Ellie Randall <ellie.randall@nrar.nsw.gov.au>
Cc: Richard Green <richard.green@industry.nsw.gov.au>; Water Referrals <water.referrals@dpi.nsw.gov.au>; Padina, Sven <Sven.Padina@jacobs.com>; Martin Knight <Martin.Knight@m4-m5linktunnels.com.au>
Subject: RE: FW: Meeting with Westconnex 3A 19 August 2019

Hi Ellie and Richard,

Just checking on progress on the provision of comments on the prelim model?

Grant Sainsbery
Environment & Sustainability Manager
Lendlease Samsung Bouygues Joint Venture
M +61 430 395 234
Grant.Sainsbery@M4-M5Linktunnels.com.au

From: Grant Sainsbery
Sent: Monday, 4 November 2019 4:35 PM
To: Ellie Randall <ellie.randall@nrar.nsw.gov.au>
Cc: Richard Green <richard.green@industry.nsw.gov.au>; Water Referrals <water.referrals@dpi.nsw.gov.au>; Padina, Sven <Sven.Padina@jacobs.com>; Martin Knight <Martin.Knight@m4-m5linktunnels.com.au>
Subject: RE: FW: Meeting with Westconnex 3A 19 August 2019

Hi Ellie and Richard,

So that we can do some model planning, can you please advise when comments are likely on the prelim model?

Thanks,
Grant Sainsbery
Environment & Sustainability Manager
Lendlease Samsung Bouygues Joint Venture
M +61 430 395 234
Grant.Sainsbery@M4-M5Linktunnels.com.au

From: Ellie Randall <ellie.randall@nrar.nsw.gov.au>
Sent: Monday, 28 October 2019 6:21 PM
To: Grant Sainsbery <Grant.Sainsbery@m4-m5linktunnels.com.au>
Cc: Richard Green <richard.green@industry.nsw.gov.au>; Water Referrals <water.referrals@dpi.nsw.gov.au>; Padina, Sven <Sven.Padina@jacobs.com>; Martin Knight <Martin.Knight@m4-m5linktunnels.com.au>
Subject: Re: FW: Meeting with Westconnex 3A 19 August 2019

Hi Grant,

That should be fine.

Can you please send a prefer date and time and a draft agenda?

Thanks
Ellie

On Mon, 28 Oct. 2019, 17:55 Grant Sainsbery, <Grant.Sainsbery@m4-m5linktunnels.com.au> wrote:

> Hi Ellie,
>
> Thanks for the heads up.   Can we pencil in a meeting mid November?
>
> Thanks,
> Grant Sainsbery
> Environment & Sustainability Manager
> Lendlease Samsung Bouygues Joint Venture
> M +61 430 395 234
> Grant.Sainsbery@M4-M5Linktunnels.com.au
>
> From: Ellie Randall <ellie.randall@nrar.nsw.gov.au>
> Sent: Monday, 28 October 2019 1:05 PM
> To: Grant Sainsbery <Grant.Sainsbery@m4-m5linktunnels.com.au>
> Cc: richard.green@industry.nsw.gov.au; water.referrals@dpi.nsw.gov.au; Padina, Sven <Sven.Padina@jacobs.com>; Martin Knight <Martin.Knight@m4-m5linktunnels.com.au>
> Subject: Re: FW: Meeting with Westconnex 3A 19 August 2019
>
> Hi Grant,
>
> DPIE Water is planning to provide comments on the modelling. Richard Green who will be reviewing this is out of the office for the next two weeks and will provide a response upon his return.
>
> Kind regards
>
> Ellie Randall | Water Regulation Officer
>
> Natural Resources Access Regulator | Water Regulation (East)
> Level 0 | 84 Crown Street | Wollongong NSW 2500
> PO Box 53 Wollongong NSW 2520
> T: +61 2 4275 9308 | F: +61 2 4224 9740
> E: ellie.randall@nrar.nsw.gov.au
> W: www.industry.nsw.gov.au

On Tue, Oct 22, 2019 at 4:27 PM Grant Sainsbery <Grant.Sainsbery@m4-m5linktunnels.com.au> wrote:

Hi Ellie and Richard,

Does NRAR / DoI have any comments on the Prelim Model Documentation provided on 30 August?

In order for us to be able to incorporate your comments into the next version, we need your comments by 31 October please.

The development of the detailed model has continued and the next milestone will the provision of the Substantial Detailed Design (SDD) version planned for 8 November. Can we pencil in a meeting date to run through the SDD GW model in the week commencing the 11th November?

Closing out some previous actions:

- Please find attached a high level drawing show the extents of excavation carried out to date. The areas that are excavated are shown in green.
- 1L/Sec/Km Measurement - as part of tunnel mapping procedure section 6.6 (below) our tunnel geologists assign water flow categories to sections of the tunnel and then water seepage categories to individual features depending upon the inflows. This information will be used to deduce the overall water inflow for any given km of tunnel.

- Other Milestones are presented in the table below:

GW02	Project wide hydrogeological design report	SDD Submission	JAJV	**8-Nov-19**
		Start pumping test	LSBJV	23-Oct-19
		Finish pumping test	LSBJV	31-Dec-19
		FD Submission	JAJV	28-Feb-20

Please come back to me with meeting date for the week starting 11 November.

Thanks,
Grant Sainsbery
Environment & Sustainability Manager
Lendlease Samsung Bouygues Joint Venture
M +61 430 395 234
Grant.Sainsbery@M4-M5Linktunnels.com.au

From: Grant Sainsbery
Sent: Thursday, 19 September 2019 2:27 PM
To: Ellie Randall <ellie.randall@nrar.nsw.gov.au>; Richard Green <richard.green@industry.nsw.gov.au>
Subject: Re: Meeting with Westconnex 3A 19 August 2019

Terrific. Thanks Both.

Get Outlook for Android

From: Ellie Randall <ellie.randall@nrar.nsw.gov.au>
Sent: Thursday, September 19, 2019 2:20:32 PM

To: Richard Green <richard.green@industry.nsw.gov.au>
Cc: Grant Sainsbery <Grant.Sainsbery@m4-m5linktunnels.com.au>
Subject: Re: Meeting with Westconnex 3A 19 August 2019

Hi Grant and Richard,

I also received the USB drive today.

Richard I will also add this to CM9.

Cheers
Ellie Randall | Water Regulation Officer

Natural Resources Access Regulator | Water Regulation (East)
Level 0 | 84 Crown Street | Wollongong NSW 2500
PO Box 53 Wollongong NSW 2520
T: +61 2 *4275 9308* | F: +61 2 *4224 9740*
E: ellie.randall@nrar.nsw.gov.au
W: www.industry.nsw.gov.au

On Thu, Sep 19, 2019 at 2:17 PM Richard Green <richard.green@industry.nsw.gov.au> wrote:

> Hi Ellie & Grant
>
> I received the thumb drive today.
>
> I will load it onto Google drive and share it with you Ellie.
>
> Thanks Grant for the additional reports any issues we will call you.
>
> Regards
> Richard

Richard Green | A/Lead Hydrogeologist
Water Assessments - Water Science
Policy, Planning & Science | Department of Planning, Industry and Environment
Level 10 | 10 Valentine Avenue | Parramatta NSW 2150
Locked Bag 5123, NSW 2124
T: +61 2 9842 8643 | F: +61 2 8838 7854 | M: 0408 418 486
E: Richard.Green@industry.nsw.gov.au
W: https://www.dpie.nsw.gov.au/

On Thu, Sep 19, 2019 at 12:03 PM Ellie Randall <ellie.randall@nrar.nsw.gov.au> wrote:

> Hi Grant,
>
> I have not received the USB at this stage.
>
> Kind regards

Ellie Randall | Water Regulation Officer

Natural Resources Access Regulator | Water Regulation (East)
Level 0 | 84 Crown Street | Wollongong NSW 2500
PO Box 53 Wollongong NSW 2520

T: +61 2 *4275 9308* | F: +61 2 *4224 9740*
E: ellie.randall@nrar.nsw.gov.au
W: www.industry.nsw.gov.au

On Thu, Sep 19, 2019 at 11:39 AM Grant Sainsbery <Grant.Sainsbery@m4-m5linktunnels.com.au> wrote:

Hi Ellie and Richard,

Did the USB's arrive?

Grant Sainsbery
Environment & Sustainability Manager
Lendlease Samsung Bouygues Joint Venture
M +61 430 395 234
Grant.Sainsbery@M4-M5Linktunnels.com.au

From: Ellie Randall <ellie.randall@nrar.nsw.gov.au>
Sent: Thursday, 5 September 2019 7:27 AM
To: Grant Sainsbery <Grant.Sainsbery@m4-m5linktunnels.com.au>
Subject: Re: Meeting with Westconnex 3A 19 August 2019

Hi Grant,

Can you please send the usb stick the below address:

Ellie Randall
Natural Resources Access Regulator
PO Box 53 Wollongong NSW 2520

If you has a second usb stick can you please send it to the Water Referrals team with a brief note at:

Water Referrals
Natural Resources Access Regulator
Locked Bag 5123
Parramatta NSW 2124

Thank you

Ellie Randall | Water Regulation Officer

Natural Resources Access Regulator | Water Regulation (East)
Level 0 | 84 Crown Street | Wollongong NSW 2500
PO Box 53 Wollongong NSW 2520
T: +61 2 *4275 9308* | F: +61 2 *4224 9740*
E: ellie.randall@nrar.nsw.gov.au
W: www.industry.nsw.gov.au

On Wed, Sep 4, 2019 at 8:46 AM Grant Sainsbery <Grant.Sainsbery@m4-m5linktunnels.com.au> wrote:

Hi Richard and Ellie,

The GT01 Geotechnical Factual report and GW01 Groundwater factual report and now ready to issue.

They are really large, so thought it best to put them on a USB and mail them to you.

What addresses should I send them to?

Thanks,
Grant Sainsbery
Environment & Sustainability Manager
Lendlease Samsung Bouygues Joint Venture
M +61 430 395 234
Grant.Sainsbery@M4-M5Linktunnels.com.au

From: Grant Sainsbery
Sent: Friday, 30 August 2019 5:11 PM
To: Richard Green <richard.green@industry.nsw.gov.au>; Ellie Randall <ellie.randall@nrar.nsw.gov.au>
Cc: Martin Knight <Martin.Knight@m4-m5linktunnels.com.au>; Padina, Sven <Sven.Padina@jacobs.com>
Subject: Meeting with Westconnex 3A 19 August 2019

HI Richard & Ellie,

Thank you for meeting with our team last week.

The intent of our meeting was to consult with DoI Water / NRAR on the groundwater model for the WestConnex 3A Project in accordance with CoA E192 and associated conditions.

Attendee's:
DoI Water / NRAR
Richard Green (DoI Water)
Hisham Zarour (DoI Water)
Ellie Randal (NRAR)

Project Team
Grant Sainsbery
Martin Knight
Sven Padina
Irena Krusic-Hrustanpasic (Golder)
Noel Merrick (SLR Peer Reviewer)
Kieran Wright

Topics Covered:

Project Team ran through a presentation covering:

1. A PowerPoint to initiate a discussion
2. Consultation Trigger – E192 and then the Groundwater Modelling Report E193
3. Modelling must be updated after 24 months – E194
4. A Project Overview was provided (Please refer to PowerPoint provided in meeting)
5. **ACTION:** Cross Sections – DoI Water requested the cross section of all of the boreholes along with the easting /northing for each borehole. (Post meeting note: This detail is provided on the borelogs and will be supplied in the next two weeks)
6. Pump Test was explained with the VWP's
7. Modelling overview
    a. FEFLOW model (Version 7.2)
    b. Fine mesh along the alignment with cell growing as you move away
    c. 3 – 10m cells
    d. 3D geological model developed using Leapfrog
    e. 11 Model slices defining 10 model layers
    f. Vertical extent -120m AHD
    g. Faults and other major fracture zones etc incorporated into the model
8. Boundary Conditions
    a. Head dept and Flux
    b. No flow boundary
    c. Base of model

9. Recharge
    a. Applied at the top slice of model
    b. Representative of the net recharge.
    c. The channels / watercourses are concrete lined limiting recharge.
10. Preliminary Model – Level 1
    a. Steady state calibration - 8.5% achieved using a total of 56 observation points
    b. No unexpected locations where inflow criteria may be exceeded.
    c. Grouting will be required around Hawthorne Canal
    d. Grouting already underway at Wattle Street Ramps. DoI Water supports the control of the water through grouting.
    e. **ACTION:** DoI Water would be interested in knowing where grouting was carried out during construction and what features were present.
    f. Some high flows zones however only Hawthorne shows where more than 1L/Sec/Km
    g. The Prelim mode goes 70% towards complying with the CoA.
11. Detailed Model will be Class 2 with aspects of Class 3.
    a. Refining the Prelim model
    b. Use Pumping test data
    c. DoI were supportive of the pump test
    d. Model extended to the west
    e. Model layers to be revised to include the most up to date 3D geological model from the recent drilling.
    f. New M5 and M4 East Tunnels and Metro drained stations to be included.
    g. Further sensitivity analysis will be completed.
12. CoA Compliance Table was presented.
13. Next Steps till end of the year.
    a. Prelim model was supplied to DoI at the meeting for review within 4 weeks. (2 hard copies and 1 pdf on USB of Hydrogeological Design Report)
    b. Modelling guidelines requires peer review to be involved on the way. Good to see that this has started.
    c. Feedback on the Prelim report and to ensure that DoI is aligned with the approach to be followed for the detailed modelling.
    d. LSBJV proposed to develop stepped program with a number of milestone meetings till the end of the year. So that at issue of final report there would be no surprises or rework/additional work.
    e. Milestone triggers for meetings.
        i. Review Prelim Model Documentation (Hard copy and Electrical Copy provided) -
        ii. Feedback timeframe - 4 to 6 weeks. Put a request int Ellie.
        iii. Pumping Test results
14. Questions from DoI Water
    a. Do you plan on manual calibration? - We plan to have a combination of PEST and Manual calibrations.
    b. Noel, do you have any concerns yet? - No concerns. The Prelim model is very thorough and beyond a Class 1 model.
    c. Info0rmal direct questions with Irena is fine. CC Grant.
    d. Otherwise formal responses between Grant and Ellie
15. **ACTION:** Project team need to ID how we will measure the 1L/S/Km
16. **ACTION:** Project Team to supply a plan showing what is currently underway and completed to date
17. **ACTION:** Project team to issue the GT01 Geotechnical Factual report and GW01 Groundwater factual report (to be reissued as FD early next week)
18. **ACTION**: Project team to propose schedules of Milestones and provisional dates to arrange future meetings

Could you please review the above, make edits as required and send back to me.

Thanks,
Grant Sainsbery
Environment & Sustainability Manager
Lendlease Samsung Bouygues Joint Venture
M +61 430 395 234
Grant.Sainsbery@M4-M5Linktunnels.com.au

Attachment 2

Presentation provided to DPIE on 19-Aug 2019

WestConnex

M4-M5 Link Tunnels

Initial DPI Groundwater Briefing

August, 2019

JAJV
Jacobs Aurecon Joint Venture
WestConnex M4-M5 Link – Main Tunnel Works

WestConnex M4-M5 Link Tunnels

lendlease

SAMSUNG C&T

BOUYGUES CONSTRUCTION AUSTRALIA

Groundwater Design – Key Modelling Requirements

MCoA requirements E192, E193 and E194:

E192 The Proponent must undertake further modeling of groundwater drawdown, tunnel inflows and saline water migration (using particle tracking).

E193 The results of the groundwater modelling must be documented in a Groundwater Modelling Report. The Groundwater Modelling Report must be finalised in accordance with the Australian Groundwater Modelling Guidelines (National Water Commission, 2012) and prepared in consultation with DPI (Water). The Groundwater Modelling Report must include, but not be limited to:

(a) justification for layer choice;

(b) specification and justification of the grid based hydraulic conductivity and storage parameters (specific yield and specific storage) assigned to each layer and/or zone with reference to those values determined from data analyses and the literature;

(c) an explanation of how groundwater flow was simulated within each model layer with reference to confined, unconfined or variably saturated flow solutions;

(d) an explanation and justification of the drain-cell conductance term(s) applied to the tunnel boundaries to limit tunnel inflows;

(e) an explanation and justification of the groundwater recharge values applied across the model domain, including around the modelled specific yield values and the water table fluctuations observed within the monitoring data in response to rainfall-fed groundwater recharge

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M4-M5 Link Tunnels

Groundwater Design – Key Modelling Requirements (cont.)

(f) details (including figures) of the expected changes in groundwater flow directions in the vicinity of landfills, groundwater wells and surface water receptors;

(g) cross-section diagrams of geology showing baseline groundwater levels in the monitoring piezometers, and for the predicted baseline condition groundwater levels in 2030 and 2100;

(h) statistical evaluation of the model's calibration;

(i) details of the groundwater monitoring data inputs (levels and quality);

(j) details of the proposed groundwater model update and validation as additional data is collected;

(k) assessment of impacts of groundwater drawdown, taking into considerations the NSW Aquifer Interference Policy (DPI, 2012), including potential impacts on licensed bores and groundwater dependent ecosystems;

(l) a comparison of the results with the modelling results detailed in the documents referred to in Condition A1; and

(m) documentation of any additional measures that would be implemented to manage and/or mitigate groundwater impacts not previously identified.

E194 The groundwater model must be updated once 24 months of groundwater monitoring data are available and the results of the modelling provided to the Secretary and DPI (Water) in an updated Groundwater Modelling Report

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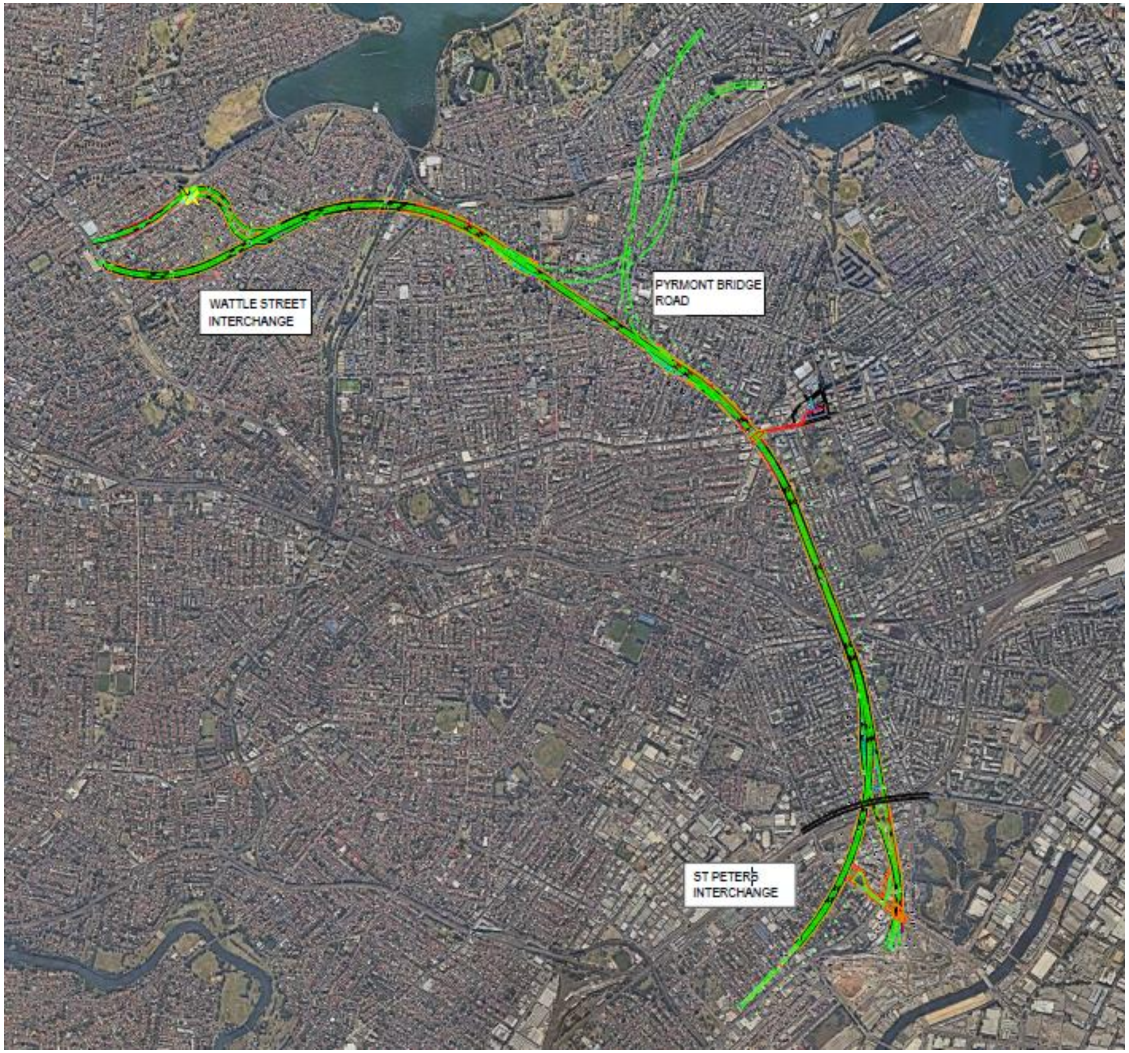
WestConnex M4 –M5 Link



WestConnex M4 –M5 Link – Project Overview

Tunnel components

- Mainline comprising 7.5 km twin tunnel linking the M4 to the M5.
- Wattle Street ramp tunnels
- St Peters Interchange (SPI) ramp tunnels
- Provision for Future Rozelle Interchange connections
- St Peters ventilation tunnels
- Pyrmont Bridge Road construction access tunnel.

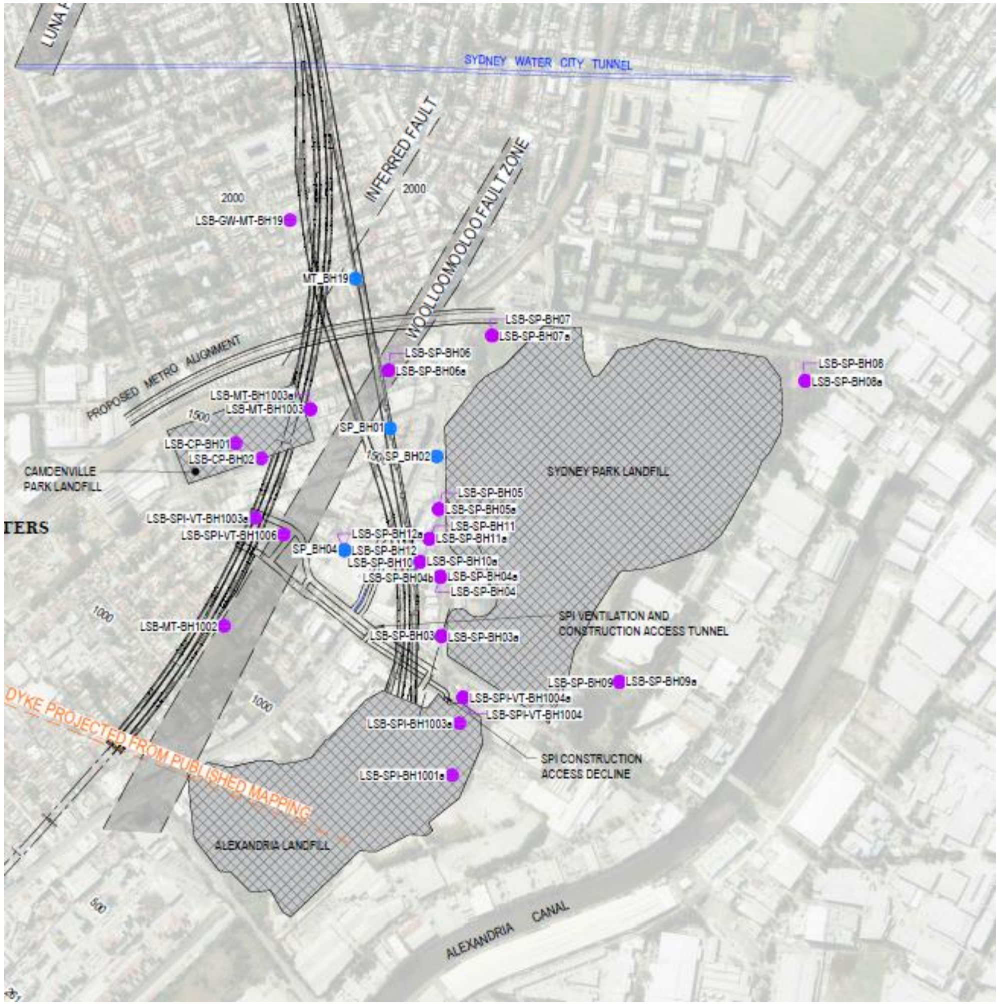
Groundwater Design – Key Requirements

- Consider a 100-year design life
- For drained tunnels, operational groundwater inflow into any tunnel must not exceed 1 litre per second in any given kilometre of tunnel.
- The proponent must undertake modelling of groundwater drawdown, tunnel inflows and saline water migration using particle tracking in consultation with DPI.

LSBJV Geotechnical Investigations

- Borehole drilling
  - 45 deep boreholes along the mainline tunnel and ramp alignments.
  - 12 boreholes around Sydney Park, 6 deep boreholes in Sydney Park
  - 13 boreholes at the SPI site.
  - 3 to 6 packer tests were carried out in most boreholes
- Groundwater sampling/observation wells
- Mainline tunnel and ramps:
  - 18 deep wells
  - 6 double well installations with one deep and one shallow screened interval
  - 1 triple well installation
- Sydney Park:
  - 6 deep wells in Park
  - 11 double well installations
  - 1 triple well installation
- SPI site
  - 1 well in the alluvium
  - 1 well in the shale bedrock.
- Re use of 11 pre-existing groundwater wells from pre-tender investigation.

Plan view of monitoring well locations



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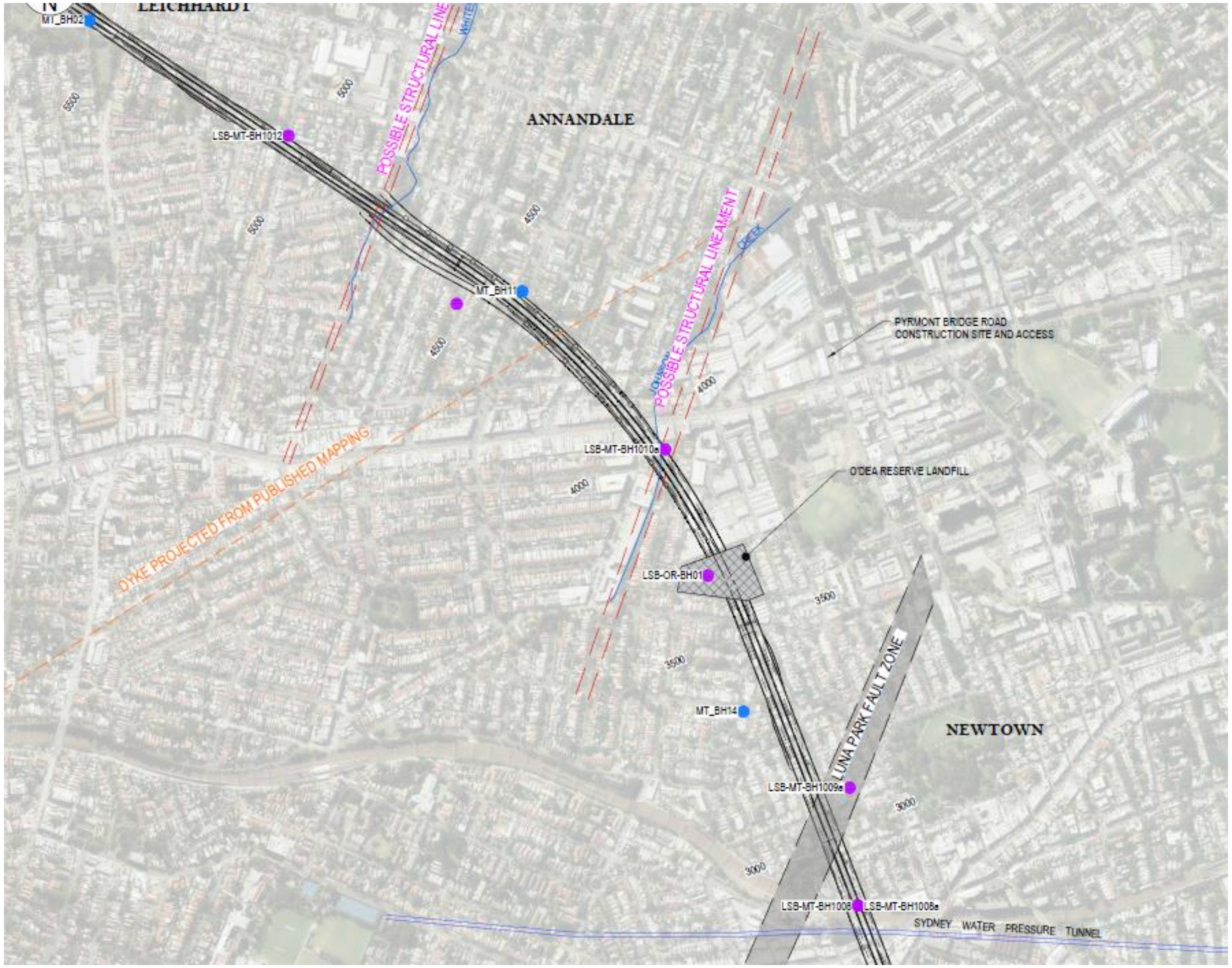
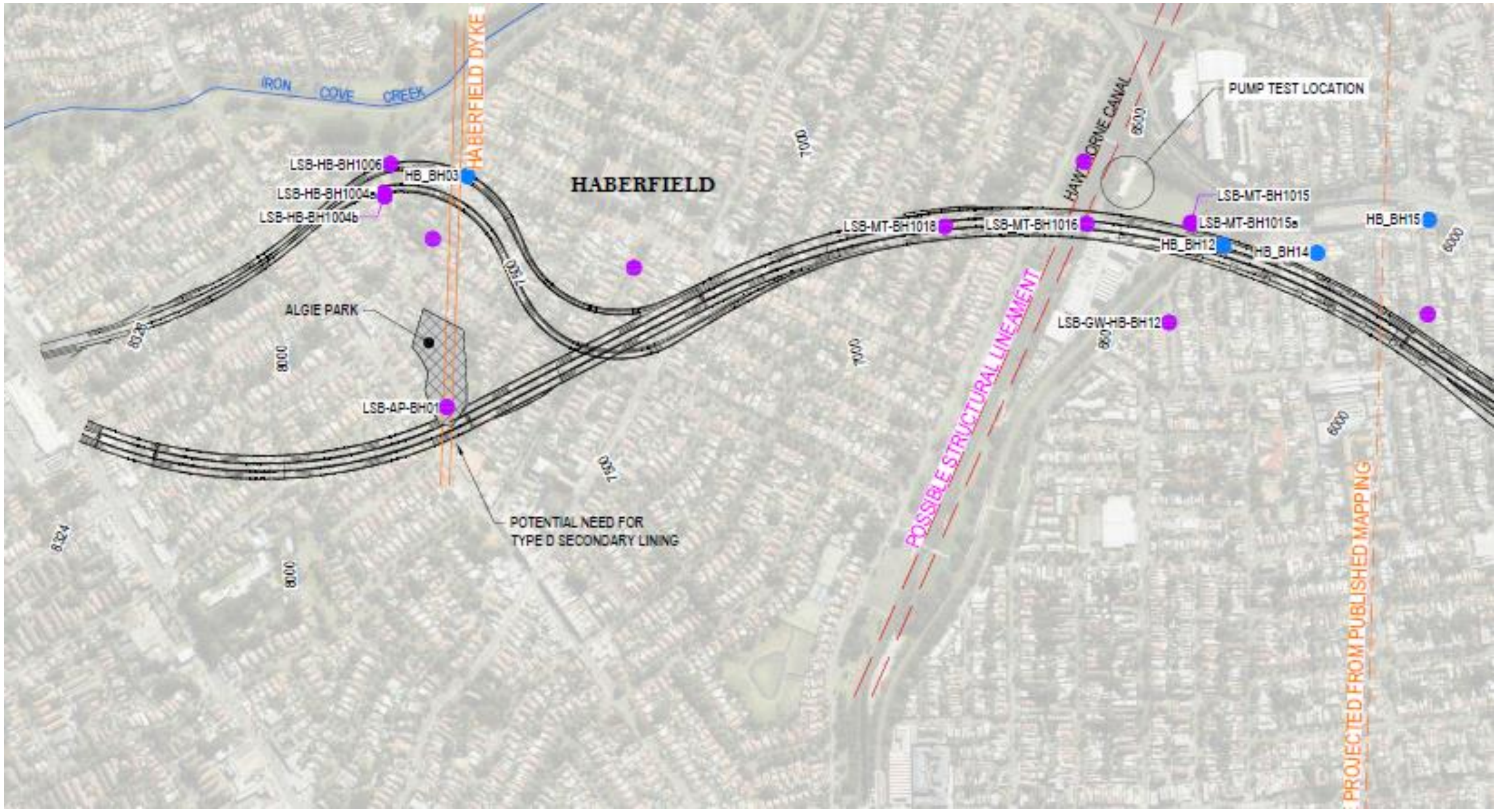
M4-M5 Link Tunnels

Plan view of monitoring well locations



Plan view of monitoring well locations



LSBJV Groundwater Investigations

Available groundwater level monitoring data

- Minimum of 12 months of water level observations from the 11 pre-existing environmental compliance groundwater level monitoring wells.
- 1 to 3 months of data logger water level monitoring data from additional LSBJV and environmental compliance well locations
- Groundwater level monitoring will continue throughout construction works

Groundwater quality testing

- Water samples for quality have been collected from 26 locations. At double well locations samples were collected from both shallow and deep wells.

Pumping Test

- A pumping test will be carried out at Hawthorne Canal in October

Groundwater Modelling – Regional Model Development Stages

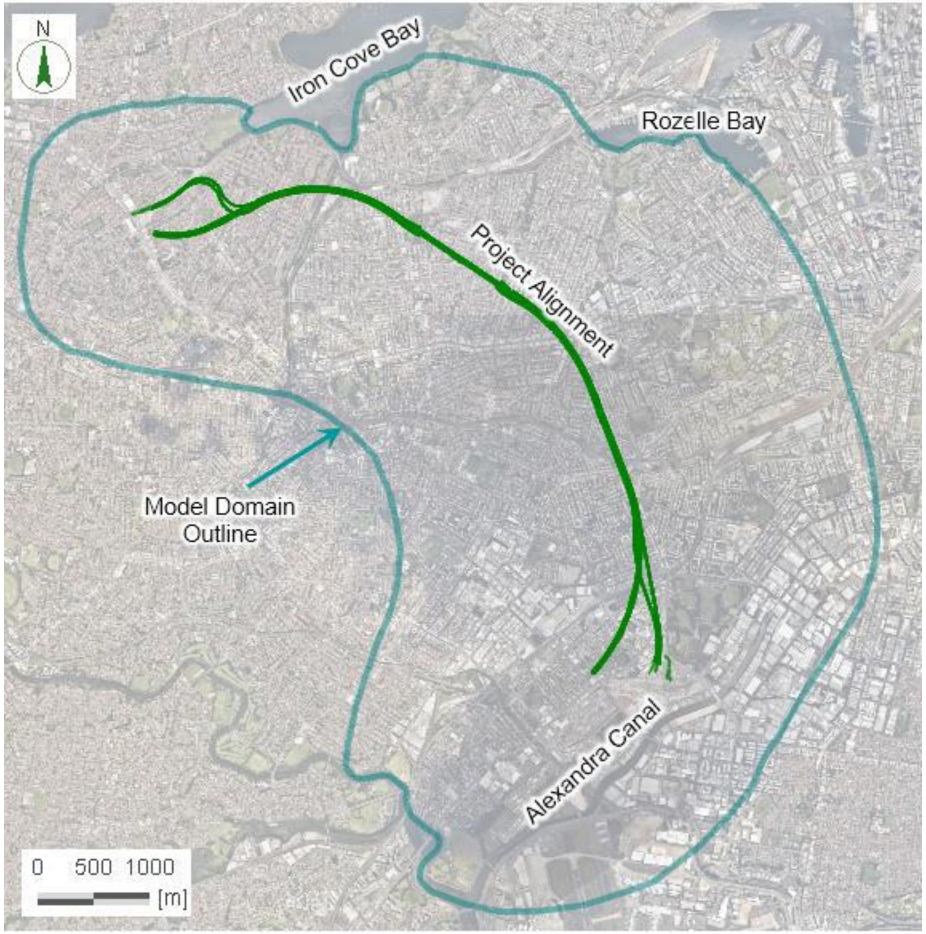
- Preliminary project wide groundwater model (completed Feb 2019)
    - Pumping test (October 2019)
- Detailed project wide groundwater model

Groundwater Modelling – Preliminary Model Objectives

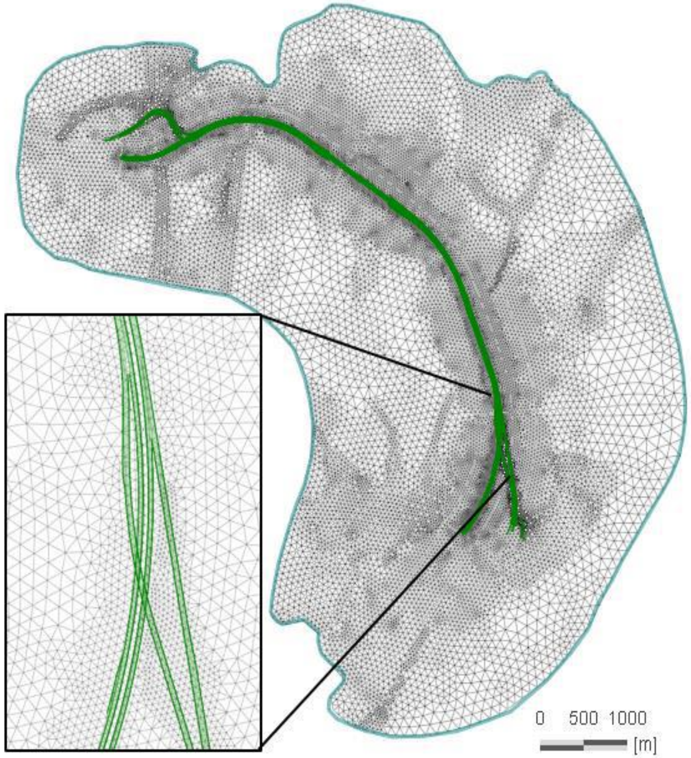
The objectives of the preliminary model were to obtain:

- Initial estimates of long-term groundwater inflows into the individual Project elements (during operational phase)
- Assess the potential long-term groundwater drawdowns
- Guide the next stage of model development work by providing insight into the main issues and uncertainty related to the data availability/collection, hydraulic properties of key hydrostratigraphic units and modelling approach

Groundwater Modelling – Set Up of Preliminary Model



FEFLOW modelling code (Version 7.2), was used to develop the model.



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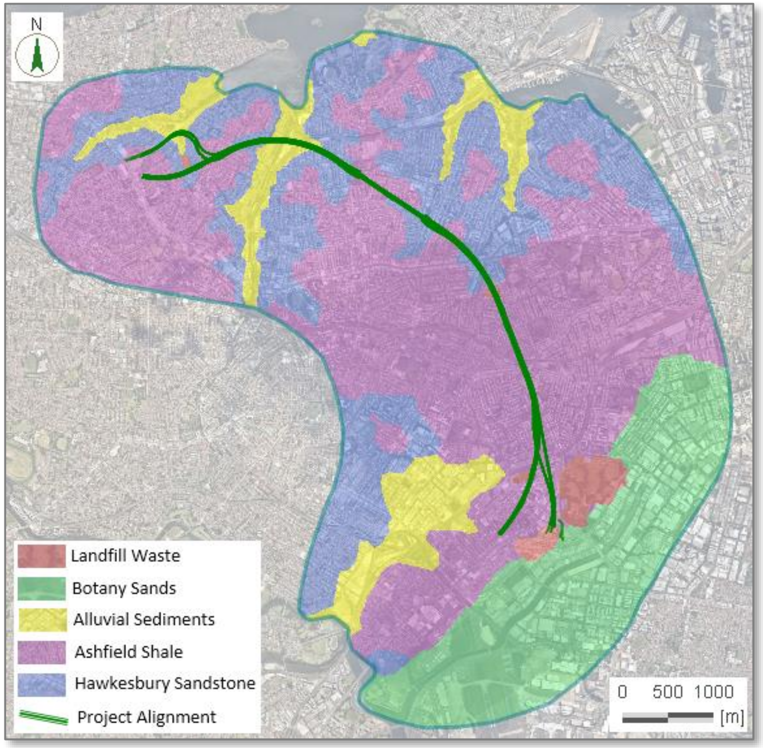
M4-M5 Link Tunnels

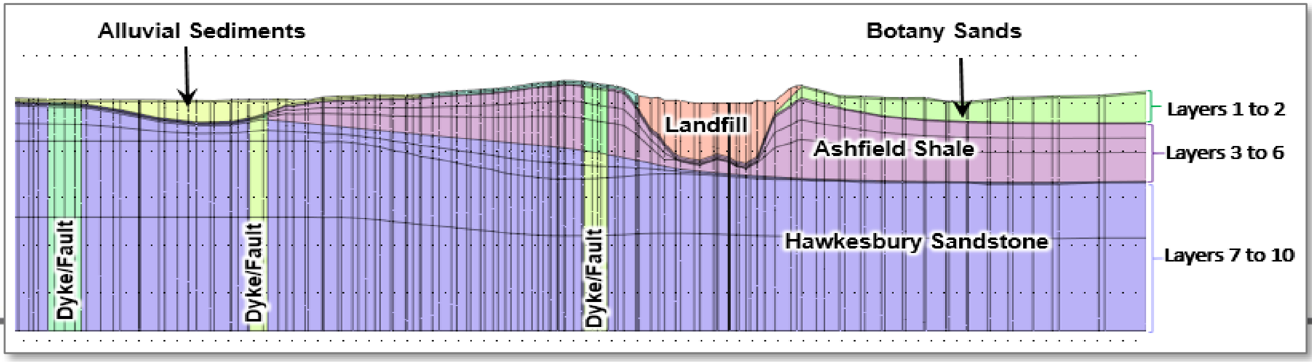
Groundwater Modelling – Set Up of Preliminary Model



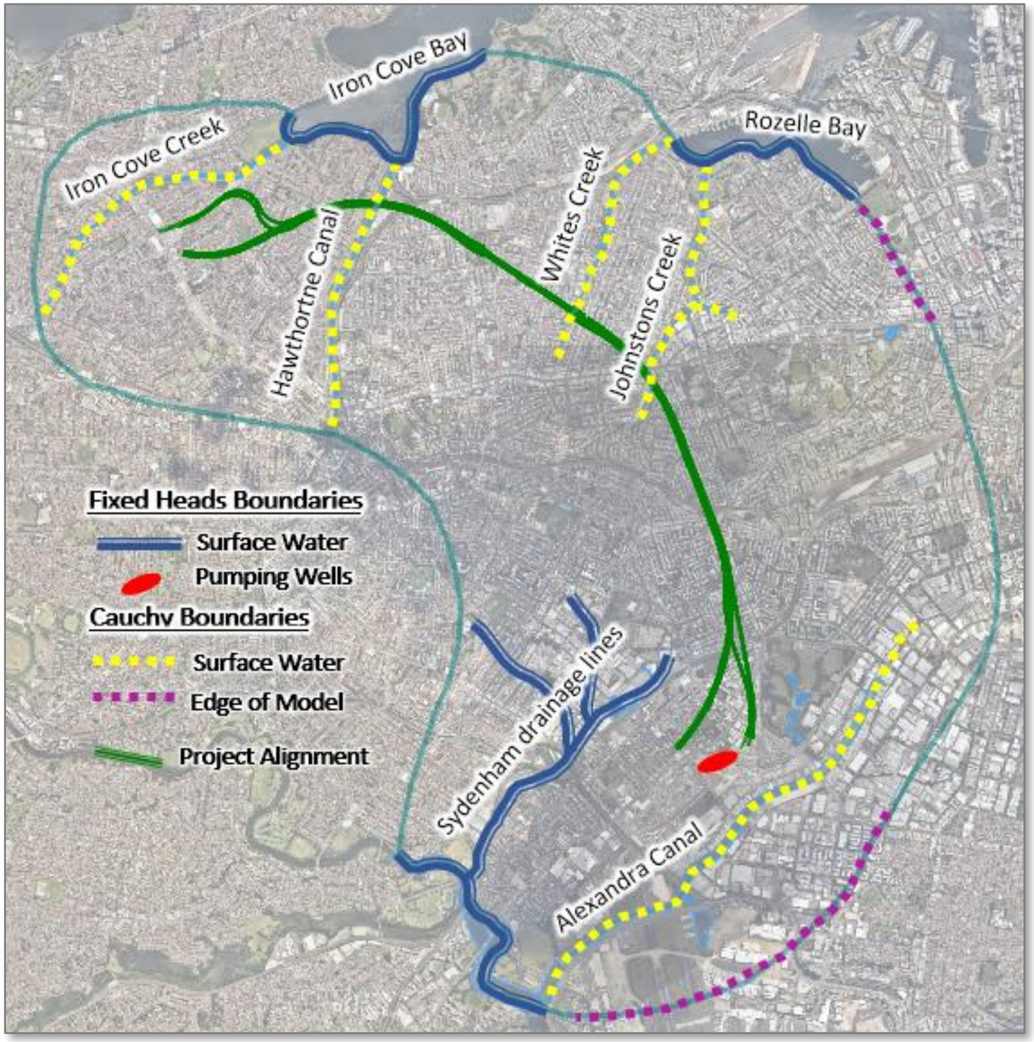
Hydrostratigraphic units based on 3D geological model (Leapfrog).

Vertical extent from topographic elevations to RL -120 m AHD:

- 11 Model Slices defining 10 Model Layers
- Major slices correspond to Quaternary Sediments, Landfill waste, Ashfield Shale and Hawkesbury Sandstone
- Refinement slices to enable tunnel simulation and vertical head gradient assessment



Groundwater Modelling – Preliminary Model Boundary Conditions



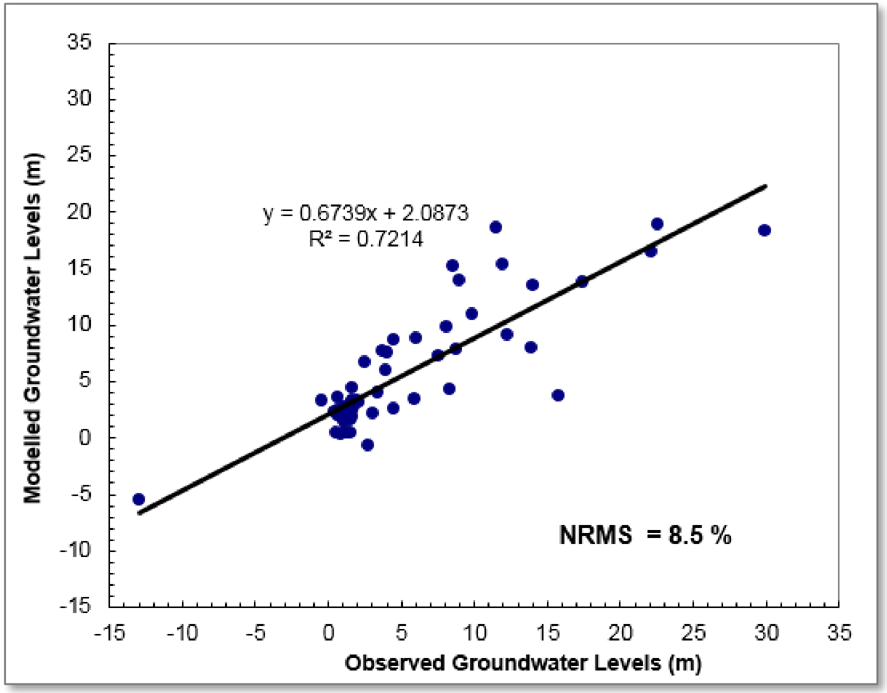
No Flow boundary:

- Western boundaries of the model domain and sections along northern and eastern boundaries
- Base of the model

Recharge:

- Applied at the top slice of the model, i.e., the ground surface.
- Representative of the net recharge (recharge from rainfall minus evapotranspiration)

Groundwater Modelling – Preliminary Model Calibration



- Model calibrated for the steady state conditions
- Normalised root mean squared error of 8.5 % achieved using a total of 56 observation points.
- A water balance error of 0.012% was achieved.

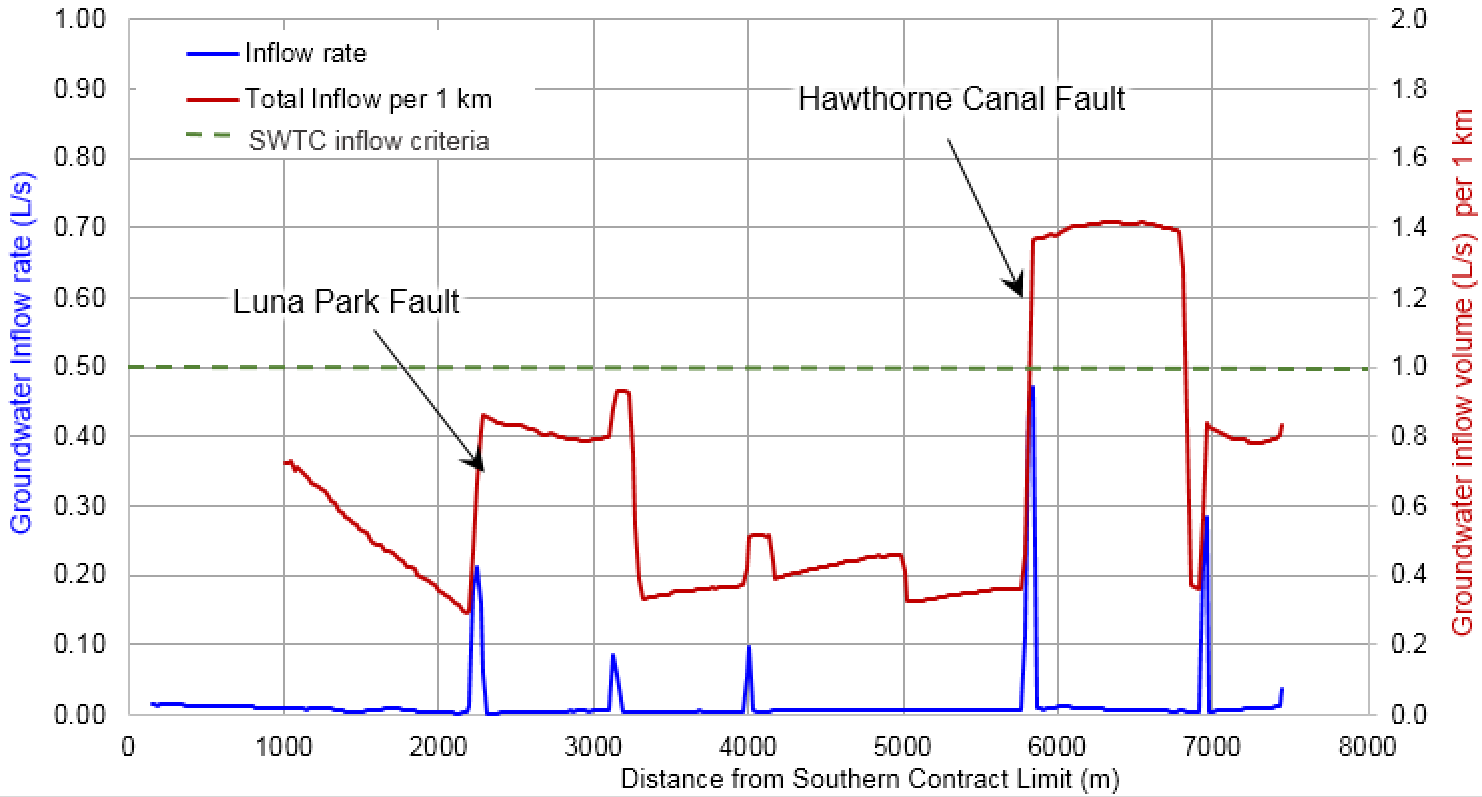
The confidence level for the Preliminary Model assessed as Class 1 when compared against the modelling criteria (Table 2-1 of the National Commission's Australian Groundwater Modelling Guidelines, Barnett et al, 2012).

Groundwater Modelling – Preliminary Model Findings

The key findings from the preliminary model were as follows;

- The predicted steady state groundwater inflows are below tunnel design criteria where no significant geological features such as faulting, etc. are present. Grouting will be required to reduce flows to allowable levels where requirements are locally indicated to be exceeded without grouting.
- No unexpected location where inflow criteria may be exceeded were identified
- There is a potential for limited salt water intrusion within the Hawthorne Canal area. However there are no sensitive receptors here.

Groundwater Modelling – Preliminary Model Inflow Predictions



Groundwater Modelling – Detailed Model Objectives

- The model will be further developed and refined based on the initial insights gained from the preliminary modelling work and results of site investigations and pumping test.
- The target confidence levels for the detailed model is Class 2 as a minimum, with elements of Class 3.

Groundwater Modelling – Detailed Model Objectives, cont.

- Main changes and considerations:
  - Model will be extended towards the west to include entire catchments of the Iron Cove Creek, Hawthorne Canal, Whites Creek, Johnstons Creek and Sydenham drainage lines.
  - Model layers will be revised to include the most up-to-date 3D geological model.
  - Further steady state and transient calibration will be undertaken to bring together all the result from LSBJV groundwater investigations, groundwater monitoring and pumping test data.
  - New M5 and M4 East Motorways and drained station excavations for Sydney Metro will be included in the model.
  - Further sensitivity analysis will be completed.

MCoA Conditions 193	Addressed in Preliminary Modelling Report
(a) justification for layer choice;	Yes
(b) specification of grid based hydraulic conductivity and storage parameters (specific yield and specific storage)	• Yes, Hydraulic conductivity parameters and distribution for each of Model Layer • Storage parameters calibrated for the steady state conditions, Detailed model will be calibrated for transient conditions
(c) Explanation how groundwater flow was simulated within each model layer	Model was develop using Feflow and does not require flow conditions to be defined for individual layers. However, model Slice 1 was defined as phreatic, the bottom Slice 11 was defined as fixed, and all other slices defined as dependent.
(d) Explanation of the drain-cell conductance term(s).	Not applicable for the Feflow modelling code.
(e) Groundwater recharge values applied across the model domain	A simplified approach was adopted for the preliminary model. This will be addressed further in the Final Modelling Report
(f) Expected changes in groundwater flow directions at landfills, groundwater wells and surface water receptors;	Yes
(g) cross-section diagrams of geology showing baseline groundwater levels in the monitoring piezometers	Will be included in the FD Modelling Report.
(h) statistical evaluation of the model's calibration;	Preliminary Model calibration and statistical evaluation included Preliminary Report. This will be addressed in further detail in Final Modelling Report
(i) groundwater monitoring data inputs	Limited discussion in Preliminary modelling Report. More details will be included in the Final Modelling Report.
(j) Proposed model update and validation as additional data is collected;	Yes
(k) Assessment of impacts of groundwater drawdown,	Yes
(l) Comparison of the results with the modelling results detailed in the documents referred to in Condition A1; and	To be addressed in the FD Modelling Report
(m) Measures to be implemented to manage and/or mitigate groundwater impacts not previously identified.	Yes

WestConnex
M4-M5 Link Tunnels

DPI Consultation Process - Discussion

- Modeling approach
- Timeline
  - Preliminary model submission to DPI (August 2019)
  - Comments on preliminary model (September 2019)
  - Detailed model and report (December 2019)
- Milestones?
- Monthly meetings with interim reviews?
- Completed by December 2019 ?
- DPI Resourcing?
- Any additional comments

CoA E193	Environment in Design Application	Design Package Application	Evidence
The results of the groundwater modelling must be documented in a Groundwater Modelling Report. The Groundwater Modelling Report must be finalised in accordance with the Australian Groundwater Modelling Guidelines (National Water Commission, 2012) and prepared in consultation with DPI Water. The Groundwater Modelling Report must include, but not be limited to:	Yes	Yes	Refer to Appendix B Groundwater Numerical Modelling Appendix H Authority Approval and Consulation Records
a) justification for layer choice	Yes	Yes	Section 5. Numerical Groundwater model Development
b) specification and justification of the grid based hydraulic conductivity and storage parameters (specific yield and specific storage) assigned to each layer and/or zone with reference to those values determined from data analyses and the literature	Yes	Yes	Section 6.4 Parameters Adopted Appendix BC Hydraulic Conductivity Distribution within Model layers
c) an explanation of how groundwater flow was simulated within each model layer with reference to confined, unconfined or variably saturated flow solutions	Yes	Yes	Section 6 Model Calibration
d) an explanation and justification of the drain-cell conductance term(s) applied to the tunnel boundaries to limit tunnel inflows	Yes	Yes	Section 3. Objectives and scope of numerical modelling
e) an explanation and justification of the groundwater recharge values applied across the model domain, including around the modelled specific yield values and the water table fluctuations observed within the monitoring data in response to rainfall-fed groundwater recharge	Yes	Yes	Section 4.4.3 Recharge Table 6.6 Net Recharge Values Adopted in the FD Model Figure 6-5 Recharge Distribution across Model Domain
f) details (including figures) of the expected changes in groundwater flow directions in the vicinity of landfills, groundwater wells and surface water receptors	Yes	Yes	Section 4.4.5 Groundwater Flow System Section 8 Model Flow Predictions Appendix BD Contaminant Transport Modelling report
g) cross-section diagrams of geology showing baseline groundwater levels in the monitoring piezometres, and for the predicted baseline condition groundwater levels in 2030 and 2100	Yes	Yes	Section 8. Model Flow Predictions Figure 8.4 + 8.5
h) statistical evaluation of the model's calibration	Yes	Yes	Section 6 Model Calibration Appendix BA Appendix BB Table 6.2 Table 6.3
i) details of the groundwater monitoring data inputs (levels and quality)	Yes	Yes	Section 4.4.2 Groundwater Levels
j) details of the proposed groundwater model update and validation as additional data is collected	Yes	Yes	Section 7. Predictive simiulation setting
k) assessment of impacts of groundwater drawdown, taking into consideration the NSW Aquifer Interference Policy (DPI, 2012), including potential impacts on licensed bores and groundwater dependent ecosystems	Yes	Yes	Section 8.3 Groundwater Drawdowns Discussed through the report in various sections
l) a comparison of the results with the modelling results detailed in the documents referred to in Condition A1	Yes	Yes	Appendix BA Simulated versus Observed Ground water drawdowns Appendix BB Observed versus Modelled
m) documentation of any additional measures that would be implemented to manage and/or mitigate groundwater impacts not previously identified	Yes	Yes	Section 7. Predictive simiulation setting