TECHNICAL REPORT

Sydney Metro West – Western Tunnelling Package

Hydrogeological Interpretive Report (including Groundwater Modelling)

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Abbreviations and Terms

Abbreviation / Term	Definition
µS/cm	MicroSiemens per centimetre, a unit electrical conductivity
3D	Three-dimensional
ASh	Ashfield Shale
ASS	Acid Sulfate Soils
Ch	Chainage
EIS	Environmental Impact Statement
GIR	Geotechnical Interpretative Report for Detailed Design

Abbreviation / Term	Definition
GLC	Gamuda Laing O'Rourke Consortium
Groundwater table	Represents the phreatic groundwater surface (groundwater table) connected to the atmosphere. Referred to herein as the groundwater level or the groundwater table.
Groundwater level	The observed groundwater level recorded in m AHD or mbgl in a standpipe piezometer.
Groundwater potentiometric surface	Represents the groundwater level in a confined or semi-confined aquifer. Referred to herein as the groundwater level or potentiometric surface.
HIR	Hydrogeological Interpretative Report for Detailed Design (This document)
HS	Hawkesbury Sandstone
km	kilometre
Kv/kh	Vertical to horizontal hydraulic conductivity ratio
L/s	Litres per second
m	metres
mbgl	Metres below ground level
m/day	Metres per day, a unit of permeability or hydraulic conductivity
mAHD	Metres Australia Height Datum
MF	Mittagong Formation
MSF	Clyde Maintenance and Stability Facility
PASS	Potentially Acid Sulfate Soils
RSF	Rosehill Services Facility
Screen Interval	Refers to the slotted section and filter pack interval of a standpipe piezometer
SI	Site Investigation
SM	Sydney Metro
SMW	Sydney Metro West
SMW-GIR	Sydney Metro West Geotechnical Interpretation Report provided for Tender stage assessment by Sydney Metro
Ss	Specific Storage
Standpipe piezometer	An installation within a borehole that is open to allow measurement of groundwater level. Also referred to as standpipe, monitoring well, monitoring bore.
Sy	Specific Yield
TBM	Tunnel Boring Machine
TDS	Total Dissolved Solids
TS-GIR	Tender Submission Geotechnical Interpretation Report
TS-HIR	Tender Submission Hydrogeological Interpretation Report

Abbreviation / Term	Definition
uL	Lugeon, 1 Lugeon = 10^{-7} m/sec
VWP	Vibrating Wire Piezometer. Fully grouted installation with sensors to measure groundwater pressure.
WCS	Water Conveyancing Structures
WTP	Western Tunnelling Package

EXECUTIVE SUMMARY

This Hydrogeological Interpretive Report (HIR) has been prepared on behalf of the Gamuda Laing O'Rourke Consortium (GLC) to address groundwater related aspects of the General Specification (Sydney Metro, Feb 2022a) and the Particular Specification (Sydney Metro, Feb 2022b). These requirements broadly include the interpretation of available hydrogeological information (such as geological conditions, rock mass and soil hydraulic parameters and groundwater elevations) to inform design decisions relating to:

- Groundwater inflows to subsurface infrastructure
- Groundwater drawdown due to temporarily or permanently drained subsurface infrastructure

The General Specification (Sydney Metro 2022a) also requires the assessment of the Project against the planning approvals. For this assessment this is interpreted to be the project conditions of approval (CoA) that relate to groundwater and the revised environmental mitigation measures (REMM) proposed in the SMW Amendment Report (Sydney Metro 2020c).

This report includes the revised groundwater modelling to address the groundwater CoA (D122).

The information provided in this HIR also provides information that supports the assessments for other disciplines within GLC that inform the design. This includes:

- Ground settlement
- Infrastructure durability
- Contamination (including exposure of acid sulfate soils)

Rosehill Service Facility (RSF) detailed hydrogeological interpretation is presented in SMWSTWTP-GLO-RSH-SF500-EN-RPT-000001 (Groundwater Modelling Report – Rosehill Service Facility).

This assessment has focused on Stage 1 works, which includes handing the following infrastructure over to Sydney Metro and/or their follow-on fit out contractors:

- Temporarily drained excavations at Westmead and Paramatta Stations.
- Permanently drained infrastructure for the Clyde Portal and Clyde Dive
- Undrained infrastructure for running tunnels, caverns, nozzles, junction, spur tunnels, stub tunnels and cross-passages.

Groundwater modelling works have been undertaken to estimate groundwater inflows to infrastructure and the associated drawdown. A summary of the groundwater modelling works is provided in Section 3.2.

Groundwater drawdown results

The modelled drawdown results have been compared against the CoA and the REMM for groundwater. The locations in the report where each COA and REMM has been discussed in this report is summarised in the conditions of approval and revised environmental mitigation measure tables below.

Conditions of approval for groundwater

CoA	COA Description	Report reference
D121	“Make-good” provisions for groundwater users must be provided in the event of a material decline in water supply levels, quality or quantity from	Attachment 2, Section 3.3.2 and Table 12.

	registered existing bores associated with groundwater changes from construction.	
D122	The Proponent must submit a revised Groundwater Modelling Report in association with Stage 1 of the Critical State Significant Infrastructure (CSSI) to the Planning Secretary for information before bulk excavation at the relevant construction location. The Groundwater Modelling Report must include:	This report documents the groundwater modelling works and scope to meet this requirement.
	(a) For each construction site where excavation will be undertaken, cumulative (additive) impacts from nearby developments, parallel transport projects and nearby excavation associated with the CSSI.	Attachment 2, Section 3.3.2 and Table 12.
	(b) Predicted incidental groundwater take (dewatering) including cumulative project effects.	Attachment 2, Section 3.3.2 and Table 12.
	(c) Potential impacts for all latter stages of the CSSI or detail and demonstrate why these later stages of the CSSI will not have lasting impacts to the groundwater system, ongoing groundwater incidental take and groundwater level drawdown effects.	Attachment 2, Section 3.3.2 and Table 12.
	(d) Actions required after Stage 1 to minimise the risk of inflows (including in the event latter stages of the CSSI are delayed or do not progress) and a strategy for accounting for any water taken beyond the life of the operation of the CSSI.	Attachment 2, Section 3.3.2 and Table 12.
	(e) Saltwater intrusion modelling analysis, from estuarine and saline groundwater in shale, into The Bays metro station site and other relevant metro station sties.	Attachment 2, Section 3.3.2 and Table 12.
	(f) A schematic of the conceptual hydrogeological model.	Attachment 2, Section 3.3.2 and Table 12.

Revised environmental mitigation measures for groundwater

Reference	Impact	Mitigation measure	Report reference
GW1	Loss of groundwater available to existing groundwater	Site inspection would be carried out on private domestic supply bore GW305646 to confirm the current viability of that bore. If found to be viable and predicted to be significantly impacted, make good measures would be implemented if a loss of yield were to occur.	Not applicable
GW2	Potential reduced baseflow to Toongabbie Creek, Domain Creek, A'Becketts Creek, Duck Creek, Haslams Creek, Powells Creek and the Mason Park wetlands, Bicentennial Park wetlands, Brickpit and Powells Creek Reserve. Requirements for	A review of additional geotechnical and hydrogeology data would be undertaken to confirm the geological and groundwater conditions and determine, based on these local conditions, whether predicted groundwater drawdown from Stage 1 is likely to occur in the vicinity of these creeks. Where the additional data review shows local conditions and predicted groundwater drawdown are likely to cause surface water/groundwater interaction, then additional site investigations (in accordance with GW3) would be undertaken for those creeks or surface water bodies.	Attachment 2, Section 3.3.2 and Table 13.

Reference	Impact	Mitigation measure	Report reference
	baseline monitoring of hydrological attributes		
GW3	Potential reduced baseflow to Toongabbie Creek, Domain Creek, A'Becketts Creek, Duck Creek, Haslams Creek, Powells Creek and the Mason Park wetlands, Bicentennial Park wetlands, Brickpit and Powells Creek Reserve. Requirements for baseline monitoring of hydrological attributes	Additional site investigations would be carried out at creeks or surface water bodies where the additional data review in GW2 shows there is a likely surface water / groundwater interaction. This would involve baseline monitoring of creek flows (streamflow gauging) prior to construction, and baseflow streamflow analysis to confirm the existing groundwater baseflow contribution to streamflow for each creek. Where a significant reduction in baseflow is predicted due to Stage 1, design responses would be implemented at station and shaft excavations to reduce potential baseflow loss.	Attachment 2, Section 3.3.2 and Table 13.
GW4	Requirements for baseline monitoring of hydrological attributes migration of contaminants in groundwater and reduction in beneficial uses of aquifers	Monitoring of groundwater levels and quality of the site area would occur before, during and after construction. This would also include monitoring of potential contaminants of concern. Groundwater level data would be regularly reviewed during and after construction by a qualified hydrogeologist. Groundwater monitoring data would be provided to the NSW Environment Protection Authority and Department of Planning, Industry and Environment and the Natural Resources Access Regulator for information.	Attachment 2, Section 3.3.2 and Table 13.
GW5	Ground movement and settlement	A detailed geotechnical and hydrogeological model for Stage 1 would be developed and progressively updated during design and construction. The detailed geotechnical and hydrogeological model would include: – Assessment of the potential for damage to structures, services, basements and other sub-surface elements through settlement or strain – Predicted groundwater inflows, groundwater take and changes to groundwater levels including at nearby water supply works. – Where building damage risk is rated as moderate or higher (as per the CIRIA 1996 risk-based criteria), a structural assessment of the affected buildings/structures would be carried out and specific measures implemented to address the risk of damage. – Where a significant exceedance of target changes to groundwater levels are predicted at surrounding land uses and nearby water supply works, an appropriate groundwater monitoring program would be	Attachment 2, Section 3.3.1, Section 3.3.2 and Table 13.

Reference	Impact	Mitigation measure	Report reference
		developed and implemented. The program would aim to confirm no adverse impacts on groundwater levels or to appropriately manage any impacts. Monitoring at any specific location would be subject to the status of the water supply work and agreement with the landowner.	
GW6	Ground movement and settlement	Condition surveys of buildings and structures in the vicinity of the tunnel and excavations would be carried out prior to the commencement of excavation at each site.	Attachment 2, Section 3.3.2 and Table 13.

Groundwater inflow results

The interpreted groundwater inflows are presented in Section 3.3.1 and Attachment 1.

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1 INTRODUCTION

1.1 Project Overview

Sydney Metro West (SMW) is a new underground railway connecting Greater Parramatta and the Sydney CBD. A linchpin of the Future Transport 2056 strategy and the Greater Sydney Commission's 'Metropolis of Three Cities Vision', it will provide fast connections between greater Sydney's two major business centres as well as providing better access to the growing business and entertainment precincts in Olympic Park and Pyrmont, the health and medical research hub at Westmead and the future business and tourism site at The Bays.

The Western Tunnelling Package (WTP) is an enabling package for SMW. It involves 9 km of twin railway tunnels (including cross-passages) between Sydney Olympic Park and Westmead as well as:

- Westmead Station box excavation, including temporary support, stub tunnels, partially mined station cavern and crossover cavern including permanent lining and support.
- Parramatta Station, including excavation of station box and associated support.
- Clyde Maintenance and Stabling Facility (MSF), including permanent dive structure, portal, spur tunnels with access shaft, spur tunnel junction (also called Clyde Junction), water conveyancing structures (creek diversion and retention basin), bulk earthworks, civil structures, utilities corridor and road crossing.
- A precast segment manufacturing facility at Eastern Creek.
- Demolition and site clearance works.

Data for this work includes the documents listed below:

- Tender phase site investigation information, geotechnical and environmental reports up to tender information documents Schedule A22, Addendum 46 and information provided by Sydney Metro after tender but prior to 1 April 2022.
- The latest Geotechnical Interpretive Report (GIR) generated by the GHD/SMEC Design Joint Venture team.

1.2 Purpose of this Report

This Hydrogeological Interpretive Report (HIR) has been prepared on behalf of the Gamuda Laing O'Rourke Consortium (GLC) to address groundwater related aspects of the General Specification (Sydney Metro, Feb 2022a) and the Particular Specification (Sydney Metro, Feb 2022b). These requirements are summarised below:

- The hydrogeological conditions and parameters along the alignment as a basis for the selection of design parameters such as grouting versus non-grouting, proposed excavation methods and dewatering options.
- Assessment of groundwater levels and inflows during the delivery of the works and at handover, including the preliminary comment on proposed methods of investigation, management, and control of these effects.
- A schedule of key geotechnical and hydrogeological features and the proposed treatment of these key geotechnical and hydrogeological features during the delivery of the tunnelling contractor's activities.

The assessment and management of contaminated groundwater (including acid sulphate soils and rock) and the associated durability issues are the responsibility of the GLC contamination and durability disciplines. The assessment and management of ground settlement is the responsibility of the GLC predicted effects discipline. The information provided in this HIR will directly inform those works.

Rosehill Service Facility (RSF) detailed hydrogeological interpretation is being completed by Aurecon for GLC and is outside the scope of this assessment.

This assessment focuses on the temporary works, which is estimated to approximate 2 to 3 years, after which the infrastructure is ‘handed over’ (i.e. handover) to Sydney Metro (SM) for follow-on works. The running tunnels (including cross-passages), nozzles, caverns, spur tunnels, spur tunnel junction, dive and portal will be handed over as undrained infrastructure whereas the stations boxes, the portal and Clyde Dive will be handed over as drained infrastructure. The portal and Clyde Dive will be handed over as permanently drained infrastructure whereas the station boxes will be handed over as temporarily drained infrastructure. The water conveyancing structures at Clyde may be handed over as drained or undrained. The responsibility for follow-on works is the responsibility of Sydney Metro and their fit-out subcontractors.

The content of this HIR is specified in Section 3.8.1.3 Site Investigations, Geotechnical Interpretive Report of Volume 4A (General Specification) Sydney Metro West (Sydney Metro 2022a). The key components listed in this section that are relevant to hydrogeology include:

- The hydrogeology inputs form part of the geotechnical reporting and as such the hydrogeological assessment uses the geotechnical interpretation of the geological conditions along the alignment.
- Descriptions of the hydraulic conductivity of existing materials.
- Interpretation of the groundwater elevation data.
- Hydrogeological assessment of the in-situ testing data at key infrastructure features, including:
  - Underground stations and affected water crossings, including the expected impact on the groundwater regime
  - Groundwater levels and the expected groundwater conditions, including estimates of inflows and pumping rates
  - The influence of groundwater with regard to methods of excavation and installation of ground support.
- A detailed assessment of the design groundwater levels to be adopted in design, including areas where perched groundwater may be present. For the purposes of the interpretation of groundwater inflows and drawdown the observed groundwater elevations have been used. For design of the infrastructure, ground surface or water surface (where surface water is present) has been recommended for use. Climate change has not been included in the assessment of design water levels as the assessment focuses on design up until handover to the follow-on contractors.

The general specification (Sydney Metro 2022a) also requires the assessment of the Project against the planning approvals. For this assessment this is interpreted to be the project conditions of approval that relate to groundwater and the mitigation and management measures proposed in the SMW Amendment Report (Sydney Metro 2020c). For the purposes of this report the results have been compared against the conditions of approval and mitigation measures for the purpose of highlighting departures that require further consideration in following stages of the project.

1.3 Alignment changes

There have been some significant adjustments to the alignment relative to the initial concept design modelled during tender WTP4.3). These include:

- The whole of alignment coordinated to Map Grid of Australia (MGA 2020) datum since tender submission, which was in Geocentric Datum 1994 (GDA94).
- Removal of the Silverwater Services facility.
- Realignment of the running tunnels to the south, between the former Silverwater Services Facility and Clyde Junction.
- Repositioning of the Portal and Clyde Dive approximately 60 m to the south.
- Realignment of the spur tunnels running between the Clyde Dive and the Clyde Junction to the north.
- Installation of a new access shaft near Rosehill Racecourse immediately northeast of the spur tunnels to facilitate earlier commencement of construction of the spur tunnels.
- Increasing the depth of Parramatta Station excavation and nozzles by approximately 1 m and extending the D-Wall depth to approximately 1 m beyond the base of the excavation.
- Shifting the station box and associated infrastructure at Westmead Station approximately 60 m to the south, removal of the adit and shortening of the stub tunnels from approximately 250 m to 90 m.
- Lengthening of the Rosehill Services Facility and repositioning to the north east by approximately 180 m (not part of this assessment).

The current design alignment is WTP5.3B (dated 16 March 2022) and has been used for this assessment.

1.4 Reporting History

Table 1 presents the history of relevant reports for this project and the development of this HIR.

Table 1: Summary of abbreviations and terms

Abbreviation in this Report*	Description	Document Reference
	Groundwater monitoring report, 00013/11180 Sydney Metro West Geotechnical Investigation	1791865-003-R-GWMR-RevA (GDP, 2018
	Additional Groundwater Sampling - 00013/11180 Sydney Metro West Geotechnical Investigation	1791865-010-R-Additional Groundwater Sampling RevA (GDP, 2019)
	Sydney Metro West – Scoping and & Definition Design Services, Tunnels, Dive and Underground Structures Report – Westmead to the Bays	SMW_10-CCM-TW-ZZ-RP-GE-000002 (SMW, 2020a)
SMW-GIR	Sydney Metro West – Scoping and & Definition Design Services, Geotechnical Interpretive Report – Westmead to the Bays, Concept Design.	SMW_10-CCM-TW-ZZ-RP-GE-000001 (SMW, 2020b)
Tender EIS	Westmead to the Bays and Sydney CBD, Environmental Impact Statement – Concept and Stage 1. Technical Paper 7: Groundwater Assessment	IA 199800-GW-RP-Stage1 Jacobs (2020)
SMW EIS	Sydney Metro West Environmental Impact Statement, Chapter 20	Jacobs and Arcadis (2020)

Abbreviation in this Report*	Description	Document Reference
SMW Amendment Report	Sydney Metro West, Westmead to The Bays and Sydney CBD Amendment Report Concept and Stage 1 2020	Sydney Metro (2020c)
	Geotechnical Factual Report 00013.11180 Sydney Metro West Geotechnical Investigation	1791865-001-R-GDR-Rev0 (GDP, 2020a)
	Contamination Factual Report - Downer EDI, Unwin Street, Rosehill	1791865-019-R-Rev0 (GDP, 2020b)
	Groundwater Monitoring Report Stage 2 Locations. Sydney Metro West Geotechnical Investigation.	1791865-023-R-GMW Stage 2 Rev 0 (GDP, 2021)
TS-GIR	Geotechnical Interpretive Report (Tender Submission)	(GALC, 2021b)
TS-HIR	Hydrogeological Interpretive Report (Tender Submission)	SMSMW210-GALC-SWDSW000-GE-TME-000001000 – Rev A. (GALC, 2021a)
	Groundwater monitoring report – Stage 3 Locations, 00013/11180 Sydney Metro West	1791865-026-R-GWM Stage 3 RevB (GDP, 2021)
	00013/11198 Sydney Metro West Geotechnical Investigation – Western Tunnelling Package – Interim Geotechnical Data Report	20446669-001-R-GDR-RevB_WTP (SMW, 2022)
	00013/11180 Sydney Metro West. Groundwater Monitoring Report – Western Tunnelling Package Locations	20446669-003-R-GMWR-Rev A (GDP, 2022)
	Westmead Station - Stage 1 Design Report Hydrogeology Technical Memorandum Rev A	SMWSTWTP-GLO-WMD-SN650-GE-MEM-010102 (GHD and SMEC 2022a)
	Parramatta Station - Stage 1 Design Report Hydrogeology Technical Memorandum Rev A	SMWSTWTP-GLO-PTA-SN600-GE-MEM-010102 (GHD and SMEC 2022b)
	Clyde – Stage 1 Design Technical Memorandum RevA.1	SMWSTWTP-GLO-TJ550-GE-MEM-010101(GHD and SMEC 2022c)
GIR	Sydney Metro West – Western Tunnelling Package Geotechnical Interpretive Report (Detailed Design)	SMWSTWTP-GLO-SWD-SW000-GE-RPT-010101 (GHD and SMEC 2022e)

*where referred to in this report

1.5 Scope

The scope of this report is as follows:

- Review of the supplied SMW-GIR, historic and tender phase site investigation information (as above in Table 1). As noted previously, the review included selected data provided by SM prior to 1 April 2021 which includes tender information documents up to Schedule A22 Addendum 46.
- Development of a two-dimensional (2D) hydrogeological model along the alignment, highlighting key hydrogeological features and zones of hydrogeological uncertainty (Section 2.0).
- Interpretation of groundwater inflows to relevant water conveyancing structures, portal, dive, station, tunnel and cross-passage infrastructure (Section 3.0).
- Interpretation of predicted groundwater drawdown associated with inflows to station infrastructure (i.e., stations, shafts, caverns, dive, etc) and the running tunnels/cross-passages. The drawdown associated with the TBM and the cross-passage development is considered to be highly localised and temporary and as such the drawdown associated with these features was not modelled. This was a similar approach to that adopted for the Tender Phase EIS groundwater impact assessment (Jacobs 2020) (Section 3.0).Comment on the potential management requirements to mitigate any departures from specification, which has primarily included further assessment of wall treatment to reduce inflows at Parramatta Station (Section 4.0).

Where necessary the report makes reference to Stage 1 design technical memorandums that have been developed for key infrastructure at Westmead (SMWSTWTP-GLO-WMD-SN650-GE-MEM-010102 Rev A), Parramatta (SMWSTWTP-GLO-PTA-SN600-GE-MEM-010102 Rev A) and Clyde (SMWSTWTP-GLO-TJ550-GE-MEM-010101 Rev A.1). The revision dates on the technical memorandums documents should be noted to ensure that the latest revisions, capturing the latest information, are being considered.

1.6 Hydrogeological Model Development

The hydrogeological model presented in Attachment 1 of this report has been developed as follows:

- An appreciation of the regional setting for the Project has been developed through the review of the SMW-GIR, and geotechnical and hydrogeological information provided in the tender information documents (refer to Table 1) up to schedule A22 and information received to 1 April 2022 to understand the key hydrogeological and topographic features of the Project.
- The TS-GIR has been used as the basis of the geological, lithological and geotechnical understanding of the Project alignment, although, due to concurrent development of the geotechnical long section for the WTP5.3B alignment with this hydrogeological package of work, the interpreted geotechnical long section for the WTP4.3 alignment has been adopted for the hydrogeological model.
- Further review and assessment of subsurface data has been undertaken (using data provided by Sydney Metro up until 1 April 2022) to provide an understanding of the hydrogeological systems. This includes review and analysis of geotechnical borehole logs, standpipes piezometers and vibrating wire piezometers (VWPs) and construction drawings. Assessment of water pressure (packer) test results, hydraulic properties and design parameters has allowed grouping of units with similar hydrogeological characteristics. These may differ from the stratigraphic rock mass units presented in the GIR and are discussed as applicable.
- The development of the hydrogeological model was performed through an iterative process of assigning hydrogeological classification at selected relevant individual test sites (i.e.

classification of individual borehole data), grouping of zones of similar properties, and a broad interpretation of regional features (i.e. the groundwater table) which have been overlain on the TS-GIR 2D geological model (WTP4.3 alignment).

1.7 Limitations

This report has been prepared by GLC for SM and may only be used and relied on by SM for the purpose agreed between GLC and SM as set out in Section 1 of this report.

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

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

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

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

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

Additional site specific limitations are provided in the technical memorandums that have been developed for key infrastructure at Westmead (SMWSTWTP-GLO-WMD-SN650-GE-MEM-010102 Rev A), Parramatta (SMWSTWTP-GLO-PTA-SN600-GE-MEM-010102 Rev A) and Clyde (SMWSTWTP-GLO-TJ550-GE-MEM-010101 Rev A.1).

2 HYDROGEOLOGICAL MODEL

2.1 Climate and Rainfall

The Project falls within the catchment of the Parramatta River and Sydney Harbour. Review of the Bureau of Meteorology (BOM http://www.bom.gov.au/climate/data/, date access 21 April 2022) rainfall and temperature data in April 2022 at the Sydney Observatory climate station (BOM Station 66062, closed July 2021) and the Parramatta North (BOM station number 66124) climate station show a mean annual rainfall of 1,213mm and 968 mm respectively with mean maximum temperature ranges from 17°C to 28°C.

Sydney's climate is characterised as temperate, having no dry season with rainfall predominance throughout the autumn and winter periods. Rainfall that infiltrated the ground contributes recharge to groundwater. Evaporation data derived from BOM at station 66062 (Sydney Observatory) presented in the WestConnex M4-M5 EIS (AECOM, 2017) shows evaporation during the winter months ranges from 55 mm to 90 mm, 110 mm to 160 mm for the spring months and 160 mm to 180 mm in the summer months.

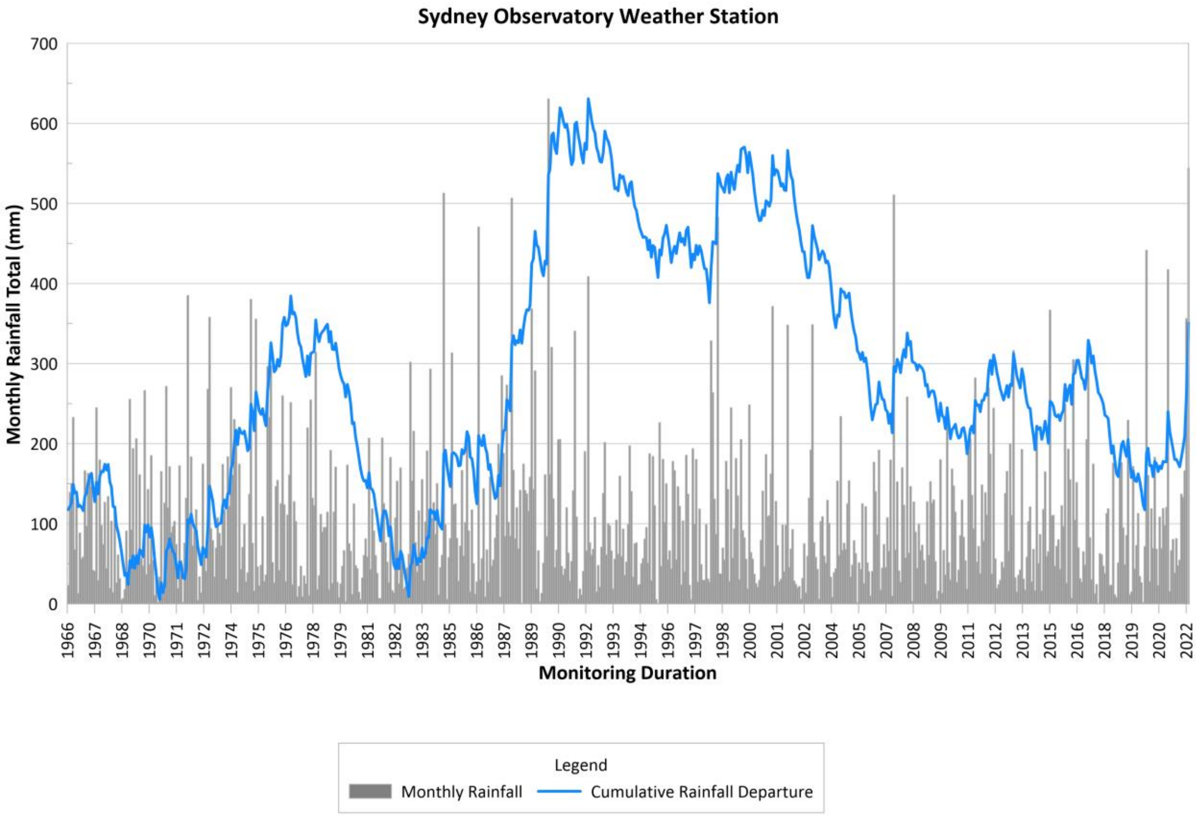
The long-term data has been collated to assess the cumulative rainfall departure (CRD) and the long-term weather trends after the method of Ferdowsian et al (2001) for Sydney Observatory, combined from the new station 66214 and the closed station 66062 and Parramatta North weather station. The CRD compares the cumulative monthly rainfall with the long-term monthly average to establish a trend in terms of periods of below average or above average rainfall conditions, represented by the slope of the line on a graph (Figure 1). Where a water table aquifer is responding to long term weather conditions the hydrograph will tend to follow the CRD with increases in level during above average rainfall conditions and decreases during periods of below average conditions.

The investigation and monitoring period for the concept SWM alignment, assumed to be from 2017 to 2021, occurred during a period of generally below average rainfall conditions after a period of typically average rainfall conditions between 2004 and 2017. Between 2021 and 2022 Sydney experienced significant rainfall periods with major flooding during a *la nina* period. This is shown by the sharp increase in the trend line for above average rainfall conditions on Figure 1.

It is expected that local groundwater observations would demonstrate a similar trend to the long-term hydrograph if long term weather conditions are the main driving factors for groundwater recharge. In other parts of the alignment, groundwater levels may be influenced by extraction and construction activities.

For example:

- Groundwater is assumed to be extracted for irrigation at Rosehill Racecourse. Abstractive groundwater use would likely be seasonal, to make up for rainfall deficits over the summer period.



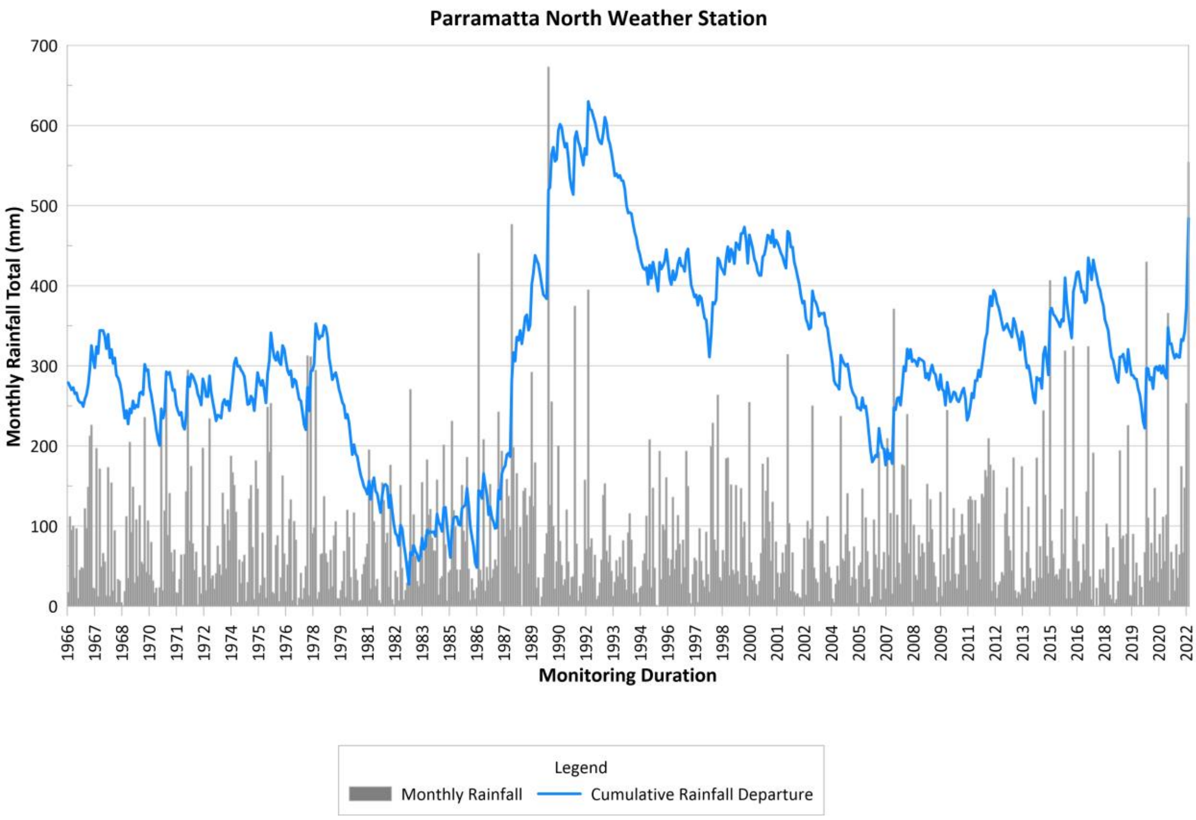


Figure 1: Cumulative rainfall departure graph with monthly rainfall from 1966 to 2022 at Sydney Observatory weather station (top) and Parramatta North weather station (bottom)

- At Paramatta, there are a number of existing buildings with deep (multi-level) basements, and sites currently under construction that will incorporate deep excavations. Where there are existing drained basements, these may result in a local depression in regional water levels. Construction dewatering may provide localised, short term reductions in groundwater levels. Groundwater observations during the investigation period are limited in terms of both the spatial extent and monitoring duration. The effects of the excavation, local basement dewatering and groundwater extraction have been considered for this assessment to infer groundwater patterns across the alignment.

2.2 Regional Topography and Drainage

Elevations range from 140 mAHD in the north-west of the catchment to sea level in the east. Across the length of the proposed alignment this ranges from approximately 45 mAHD to sea level, however, due to variations in proximity to major rivers and creeks, the surface topography local to the alignment is highly undulating with an overall drainage gradient to the north and east towards the Parramatta River and Sydney Harbour.

Most of the waterways are urbanised with those closer to the coast and Sydney Harbour being tidal. The majority of the Project footprint is heavily urbanised, and run-off is drained by the stormwater network. Primarily surface water features in the Project footprint include the Parramatta River, Duck River, and local minor creeks including some infilled creeks. Some waterways have been highly modified.

2.3 Regional Geology

The regional geology is described in detail within Section 3 of the WTP-GIR. It is summarised in this section with an emphasis on the hydrogeological characteristics.

The information contained within the SMW-GIR is substantially a literature and data review of published and available information relevant to the alignment. Data from the tender phase site investigation reviewed to date, supports the SMW-GIR understanding with minor adjustments identified at specific locations. These are presented in the WTP-GIR.

The alignment is situated within the geological area known as the Sydney Basin. The Sydney Basin is characterised by a sub-horizontally layered Permian - Triassic age sedimentary sequence of rocks. The published 1:100,000 scale series geological map for Sydney Sheet 9130 (Herbert, 1983) indicates that the stratigraphic units expected in the Project area comprise:

- Fill of highly variable nature – typically associated with reclaimed areas adjacent to Sydney Harbour-Parramatta River system and some parklands.
- Transported soils (alluvial sediments). These comprise Holocene aged sediments which are typically under-consolidated as wells as older Pleistocene aged sediments which are typically over-consolidated sandy clay soils.
- Igneous intrusions.
- Residual soils.
- Ashfield Shale (of the Wianamatta Group).
- Mittagong Formation.
- Hawkesbury Sandstone.

The surface expression of the geology is presented in Figure 2, duplicated from the SMW-GIR. This includes the interpreted position of igneous dykes and structural geological features as described in published geological mapping and in Och et al (2009), with modification based on additional Project data and presented within Sydney Metro (SWM-GIR) supplied information documents. The geological and hydrogeological properties of the different units are well-understood and documented in published literature.

The geology of the alignment is dominated by Triassic-aged Ashfield Shale and Hawkesbury Sandstone, which also influences the topography and drainage. The Project lies within the Cumberland Plain which forms a relatively flat centre of the Sydney Basin. It is characterised by gently undulating hills and slopes underlain by siltstones and shales of the Wianamatta Group. The Wianamatta Group has been further differentiated into the Ashfield Shale, Bringelly Shale and Minchinbury Sandstone. The latter two units are noted in Figure 2, however, as they do not fall close to the Project alignment, they are not discussed further.

The shales of the Wianamatta Group rarely form as outcrops and weather quickly on exposure, creating deep soil profiles typically comprising high plasticity clays and silty clays. Below the shales and exposed at surface close to Sydney Harbour, is the Hawkesbury Sandstone. It typically comprises medium to coarse grained, cross-bedded to massive sandstone, with siltstone beds and rare siltstone breccia, with outcrops scattered across the basin. Sub-vertical outcrop exposures are common.

The Project area has been subject to a number of inter-glacial cycles. A period of downcutting occurred during a glacial maximum approximately 120,000 years ago when sea levels were about 100 m lower than present. This resulted in the incision of deep valley systems across the eastern margin of the basin. Subsequent sea level rises in the Pleistocene age resulted in the infilling of the valleys, including the Parramatta River and associated bays. A second period of lower sea levels eroded much of the Pleistocene sediments. Sea level rise in the Holocene age resulted in sea level rising to its present position. This resulted in further infilling of the Parramatta River system with Holocene sediments.

These palaeochannels could contain steep flanking slopes with stepped or terraced profiles (influenced by a combination of bed thickness and prevailing sub-vertical joint patterns), remnant wave cut shorelines and rocky / cliffed valley sides. Preservation of residual soils in such an environment would be rare. This palaeo-topography would be preserved beneath the Pleistocene and Holocene sediments and forms an unconformable soil-rock interface.

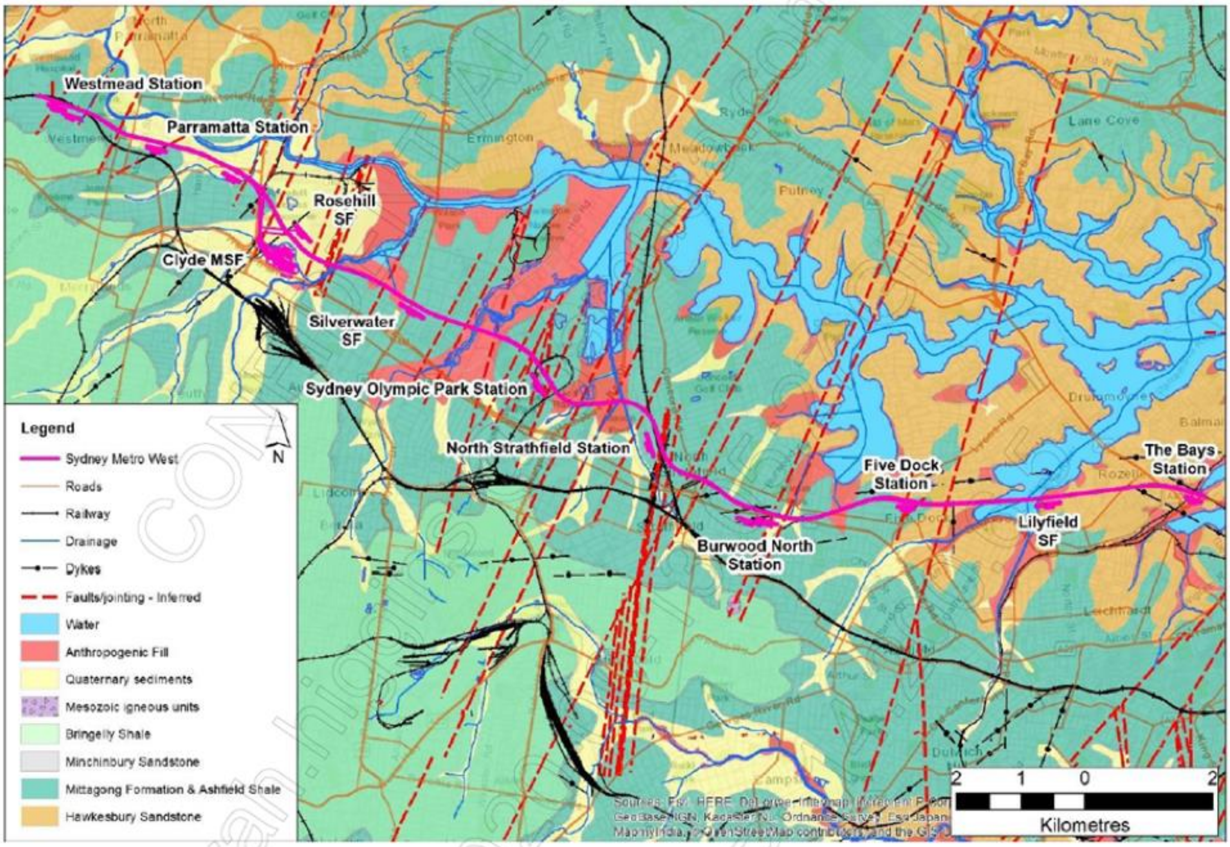


Figure 2: Overview of Project area geology (extract from Concept Design GIR)

2.4 Project Hydrogeology

2.4.1 Fill Materials

Thin layers of anthropogenic filling (generally less than 1 m thick) is common in urban areas, associated with minor modifications to the topography, landscaping and pavement construction. It can be highly variable in composition and consistency. Thicker deposits of fill are expected towards the mouths of infilled channels, backfilled quarries, landfills and land reclamation areas. Filling and excavation have modified the environment intensely over the last 200 years. This has included reclamation works along the foreshore areas and swamps, development of deep basement and tunnel excavations, quarry activities and fill placement from various major projects.

Reclamation works have occurred within the bays of the Parramatta River since the mid-19th century. The fill is highly variable in composition and quality, often accompanied by poor record keeping and uncontrolled placement. The thickest areas of fill directly intercepted by proposed excavations along the alignment are at the Clyde Portal, Clyde MSF and water conveyance structures (WCS) and the RSF.

Typically, the hydraulic properties of fill materials are dominated by the fill composition and level of compaction. Results of hydraulic permeability tests of fill material along the Project alignment show a hydraulic conductivity range of less than 0.1 m/day to 120 m/day, noting that the higher values are from whole project testing, specifically at the Bays area, to the east of the WTP.

Transient groundwater may enter fill material after rainfall. Discharge is typically towards nearby topographic surface drainage features and depressions. Shallow groundwater may be encountered in fill materials in reclaimed areas and infilled channels.

2.4.2 Transported (Alluvial) Soils

Deposits of alluvial sediments occur along the alignment within various gully and valley features, most notably at:

- Westmead, where alluvium is associated with Domain Creek, which is a tributary of the Parramatta River.
- Parramatta, where a broad area of alluvium is associated with the Parramatta River floodplain, extending from the east of the Clyde MSF to Parramatta.
- Clyde MSF and RSF, where a broad area of alluvium is associated with Duck River and Duck Creek catchments.
- Haslams Creek, which is a southern tributary of the Parramatta River. The associated floodplain deposits are west of Sydney Olympic Park and are overlain by reclamation fill.

The alluvial deposits in this area include a range of interbedded clay and sand soils. The alluvial soils are typically recent Holocene era soils which include prevalent Acid Sulfate Soils (ASS) or potential ASS (PASS). The alluvial deposits located beneath the southern banks of the Parramatta River at Parramatta contain a thick sequence of alluvium known as the Parramatta Sand Body (Parramatta Sands). At the proposed Parramatta Station the sand body is about 10 m thick. It is a heritage-listed archaeological site due to its formation as an alluvial terrace where significant Aboriginal archaeological artefacts are thought to be preserved between 1.5 m to 2 m below ground surface. Hydraulic conductivity from alluvial material around Parramatta Station ranges from $1x10^{-5}$ m/day to 0.1 m/day.

Typical hydraulic conductivity for alluvial material are between 0.01 m/day and 1 m/day and the horizontal conductivity is typically higher than the vertical. Groundwater within the alluvium can be a source of either recharge or discharge depending on whether upward or downward hydraulic gradients are present. Recharge to the alluvium is via direct rainfall recharge and stormwater runoff, or via overbanking or high flow events from waterways (Parramatta River).

2.4.3 Residual Soils

Residual soils derived from the Ashfield Shale are typically medium and high plasticity clays. These clay soils are more resistant to erosion, and regionally observed at depths of 3 m to 10 m. Residual soils derived from Hawkesbury Sandstone are typically of sandy clay or clayey sand compositions, that provide limited resistance to natural erosion. As such, the residual soil profile formed from exposed Hawkesbury Sandstone is typically of limited depth (>2 m) or absent.

Hydraulic conductivity ranges of typical soil types within the alignment are summarised from the SMW-GIR and published literature ranges as:

- Clean gravels – greater than 60 m/day
- Sand and gravel mixtures – 0.8 m/day to 86 m/day
- Very fine sands, silty sands – 0.008 m/day to 0.8 m/day
- Silts, interlaminated silts/clays – 0.001 m/day to 0.008 m/day
- Clays – less than 0.001 m/day.

In topographically elevated areas, groundwater may be present in residual soils after rainfall, however, it is transient in nature. Discharge typically occurs under gravity flow, towards adjacent, lower lying topographic surface drainage features. This groundwater may also be 'discharged' via evapotranspiration effects and deeper drainage.

Residual soils of low permeability located between overlying alluvial aquifers and underlying bedrock aquifers will tend to act as an aquitard that limits hydraulic connection between aquifers.

2.4.4 Ashfield Shale

The Ashfield Shale forms the lower part of the Wianamatta Group of rocks, a group of Middle Triassic age fine grained shales, sandstones and mudstones. The Ashfield Shale represents a regressive depositional episode, grading from lacustrine at its base and up to a marine or brackish facies within the upper sequence. These were low energy depositional environments which allowed for the accumulation of typically fine-grained sediments such as clay, silt and fine sand particles. At some locations shale may become carbonaceous. The Ashfield Shale unit is typically up to 50 m thick in this part of the basin and consists of four discrete siltstone and laminite subgroup members. All four subgroups overlie along the alignment, with the lower three sub-groups potentially intersected by the tunnel. The shale members comprise well-developed thinly laminated rock with wide spaced bedding at about 0.1 m to 0.5 m vertical spacing, with bedding spacing varying between members. The bedding planes are sub horizontal, dipping typically 0° to 5°, and persistent over tens to hundreds of metres. There are variations in bedding locally, particularly near inferred faults, but also through syn-depositional folding and bioturbation.

Jointing within the Ashfield Shale does not typically follow discrete or definable sets over large areas. Sedimentary 'slump' structures within the Ashfield shale have been previously interpreted to result in minor fault displacements, providing more persistent jointing. These features are typically curved on a large scale with joint faces typically tight and smooth.

Recharge is from direct rainfall infiltration in elevated areas where shale subcrops and discharge is typically towards topographic lows. Recharge rates are expected to be less than 5% of annual rainfall. The Ashfield Shale is typically considered to act as an aquitard and in zones of outcrop the unconfined groundwater table systems is often perched or intermittent. It may become partially confined by overlying residual clays but when overlain by alluvium is partially hydraulically connected to the unconfined alluvium.

The majority of groundwater flow is via fractures and joints, however, at a regional scale the Shale forms an aquitard, impeding groundwater infiltration to the underlying Mittagong and Hawkesbury Formations. In areas of low elevation, or where the base of the Shale is below the regional water table in the underlying aquifers, the groundwater levels in the Hawkesbury Sandstone and Mittagong Formation become confined, which is expected to be the prevailing condition along the WTP alignment. Where the base of the Shale is elevated relative to the regional groundwater levels in underlying aquifers, groundwater can become perched in the Shale with the occurrence of underdrainage (a drying up) in the underlying Hawkesbury Sandstone and Mittagong Formation before intersecting a regional aquifer at greater depth. This is more common in elevated topographical areas of north-eastern Sydney (such as around North Sydney and Chatswood).

The Ashfield Shale is typically of low bulk hydraulic conductivity. Available hydraulic conductivity information for the alignment shows Ashfield Shale ranging from 0.0008 m/day to 0.2 m/day from a regional perspective. Yields in the Shale are typically low and less than 2 L/s.

Groundwater quality in the Ashfield Shale is highly variable and typically considered brackish or saline. Elevated salinity, low pH, sulphides and elevated heavy metals are common.

Analysis of 188 water pressure (packer) tests results in Ashfield Shale from new and historic borehole data are presented on Figure 3 relative to depth below ground. Typically, permeability decreases with depth in the Shale and results are typically within the range of 0.001 m/day to 0.1 m/day, which is marginally higher than the regional data.

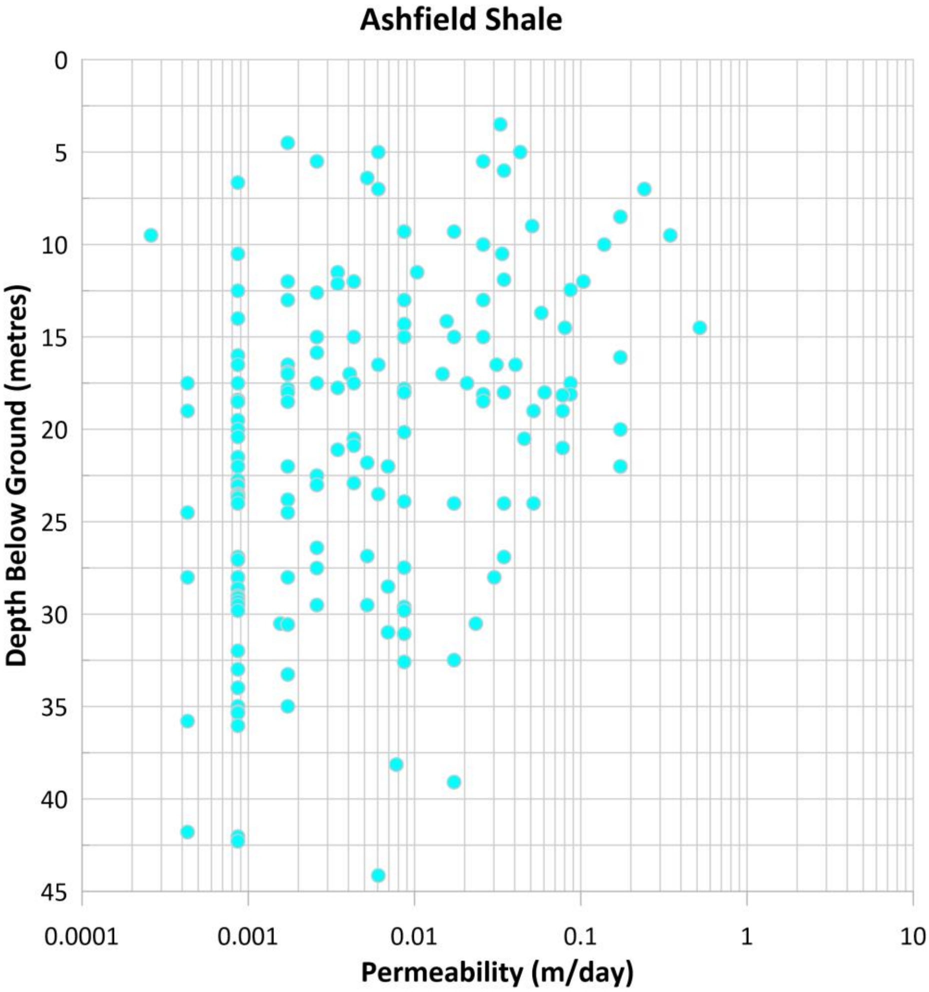


Figure 3: Analysis of packer testing in the Ashfield Shale

The distribution frequency as a percentile is compared on Figure 4 to published Ashfield Shale results of Best and Parker (2005).

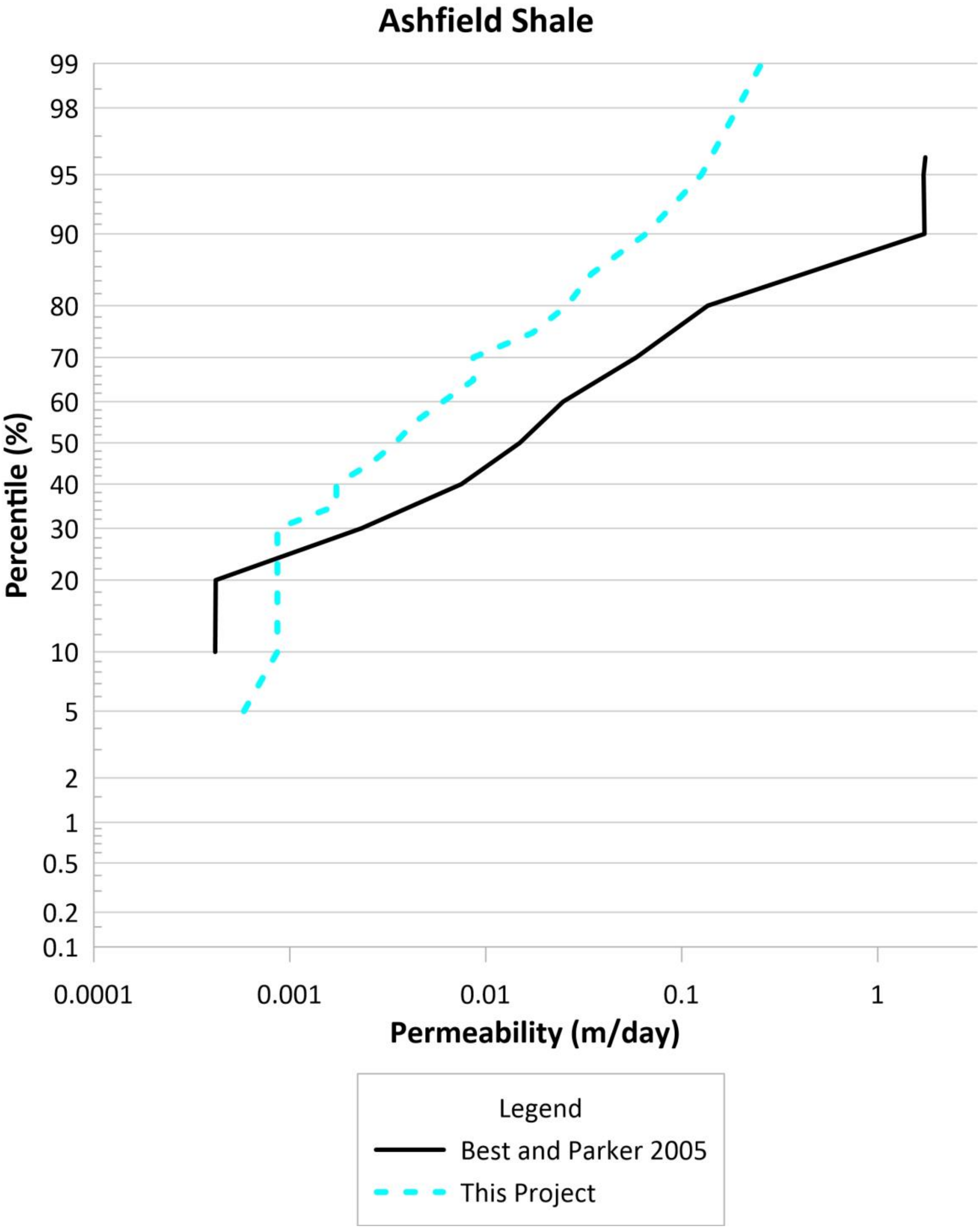


Figure 4: Permeability distribution of packer test results in Ashfield Shale

2.4.5 Mittagong Formation

Mittagong Formation separates Ashfield Shale from the underlying Hawkesbury Sandstone. It is a relatively thin unit comprising an upper, thin, very fine-grained brownish sandstone unit (typically 0.5 m thick), over a lower unit of fine-grained sandstone and interlaminated or interbedded dark grey siltstone. The lower unit is typically 1 m to 3 m thick but can be up to 10 m thick. In places the formation is completely absent, and where weathered, it is often indistinguishable from the underlying Hawkesbury Sandstone.

The formation represents the transition from the fluvial/terrestrial depositional environment of Hawkesbury Sandstone to the lacustrine depositional environment of lower Ashfield Shale. The Mittagong Formation commonly takes on characteristics of the adjacent formations with defect characteristics being largely dependent on the dominant parent lithology (siltstone or sandstone).

The Mittagong is siltier than the Hawkesbury Sandstone, but its hydraulic properties are similar and thus the two units tend to be hydraulically connected. Groundwater quality is poor due to leakage from the Ashfield Shale, and its higher clay content relative to the Hawkesbury Sandstone. Recharge is via leakage from the Ashfield shale or direct rainfall infiltration where the formation outcrops.

2.4.6 Hawkesbury Sandstone

Hawkesbury Sandstone is encountered across approximately three-quarters of the Project alignment. The formation extends across the whole of the Sydney basin and is up to approximately 290 m thick, though only the upper 50 m will be intersected by the SMW tunnel. The Hawkesbury Sandstone is often described as a medium to coarse-grained quartzose sandstone, deposited in 1 m to 3 m thick beds. Shale breccia is common at the contacts between beds, with siltstone interbeds forming a minor part of the unit. Finer and coarser-grained bands represent changes in the depositional environment.

Hawkesbury Sandstone is inferred to represent deposition by fluvial processes in a large, braided river system, with shale interbeds representing overbank and swamp type deposits. The stratigraphic units of Hawkesbury Sandstone typically comprise a repeating series attributed to three distinct facies, each representing a differing depositional process, namely:

- Massive sandstone facies
- Cross-bedded or sheet facies (well developed or indistinct/poorly developed)
- Shale/siltstone interbed facies.

The Hawkesbury Sandstone displays bedding but also contains secondary structural features such as steeply dipping joints and faults and shallower bedding shears. The sub-horizontal bedding planes are typically spaced 0.5 m to 5 m apart (commonly 1 m to 3 m apart) and dip at about 0° to °5 degrees to the NNE along the alignment. The bedding planes can be persistent over hundreds of metres. The bedding is commonly undulating or curvi-planar on a macro scale. Minor shale breccia can be present in the base of these undulating beds marking the presence of former channels.

Within the Hawkesbury Sandstone, clayey silty sand and sandy clay infill of between 10 mm and 100 mm thickness can occur on bedding contacts associated with differential weathering, stress relief and low angled thrust faulting. The bedding plane infill is variable and can include in situ weathering of a silty interbed, clay seams and clay coatings, sub-horizontal crushed seams, iron staining and limonite coatings.

The Hawkesbury Sandstone forms a regionally extensive unconfined to confined aquifer that is comprised of numerous sub-aquifers that are partially hydraulically connected. Groundwater flow is highly variable and dominated by secondary porosity and fracture flow associated with geological structures (e.g. geological faults and joints). Where fracturing and jointing are less prevalent and bedding is closely spaced or where low permeability siltstone lenses are common, groundwater migration occurs preferentially along the horizontal bedding planes and as such vertical flow is reduced relative to horizontal flow. When coupled with elevated topographic conditions a downward hydraulic head often gradient occurs. Where the interface with the Ashfield Shale is below the regional groundwater elevations in the Hawkesbury Sandstone (generally at lower elevations) the Hawkesbury Sandstone aquifer is confined. The sandstone weathers to a clayey sand residual soil profile typically less than 2 m deep. Within the upper 10 m of the profile, a duricrust may be present where iron cementations have caused the development of ferricrete or ‘coffee rock’ or silica cementation leading to silcrete. The weathering of Hawkesbury Sandstone is characterised by iron staining with orange and red colouration partly or totally penetrating the rock mass. Typically, the iron staining extends into the rock mass some 5 m to 10 m below ground surface. Sandstone areas located closer to the incised watercourses are typically more extensively weathered, which may also reflect the influence of stress relief and increased fracturing beneath, and beside the watercourses. Iron staining appears more prolific in the coarser beds of sandstone and can be concentrated along water bearing features.

Regionally the groundwater flow is eastward towards the principal discharge of the Tasman Sea. Recharge is via rainfall infiltration on fractured outcrop and through leakage from the Ashfield Shale, soil profile and alluvium. Recharge rates from rainfall are generally less than 5% of the annual rainfall. Discharge is via seepage to cuttings, creeks and waters, and evapotranspiration.

The groundwater quality is generally acidic, but of low salinity except where Ashfield Shale is locally preserved and contributes leakage into the underlying sandstone. Naturally elevated concentrations of dissolved iron and manganese occur within the Hawkesbury Sandstone, along with other dissolved metals. Dissolved iron and manganese can cause staining when discharged and oxidised and seepages may form hard ochres on the surface with prolific iron and slime bacteria sludge. There is the potential for this sludge to block drainage infrastructure if groundwater ingress is allowed to oxidise within the tunnel. Observations of other tunnels in Sydney suggest sandstone can also be geochemically altered by intrusive dykes. This can result in further development of iron and sulphide sludge forming bacterial growths associated with inflows, and elevated risk of clogging of drains.

Typically, the hydraulic conductivity of Hawkesbury Sandstone is low, in the order of $1x10^{-3}$ to $1x10^{-1}$ m/day and fracture related storage is less than 2%, though matrix storage can be higher. Yields of individual bores that do not intersect major fractures or fissures are commonly less than 2 L/s. Yields can be higher when saturated features are intersected and increased flow to tunnels is typically associated with the intersection of such major features. Hydraulic conductivity information for the alignment shows ranges of 0.0005 m/day to 1 m/day for Hawkesbury Sandstone. By contrast the Mittagong formation ranges from $1x10^{-6}$ m/day to 0.001 m/day.

Analysis of 153 water pressure (packer) tests from the Hawkesbury and Mittagong Formation from Project borehole data and provided existing background information, are presented on Figure 5. These are presented relative to depth below ground in metres. Typically, permeability decreases with depth in the sandstone and results are typically within the range of 0.001 m/day to 1 m/day. The distribution frequency as a percentile is compared on Figure 6 to published Hawkesbury Sandstone results of Tammetta and Hewitt (2004), Hewitt (2012), WestConnex M4E (AECOM, 2017) and the M4-M5 East EIS (WestConnex, 2017).

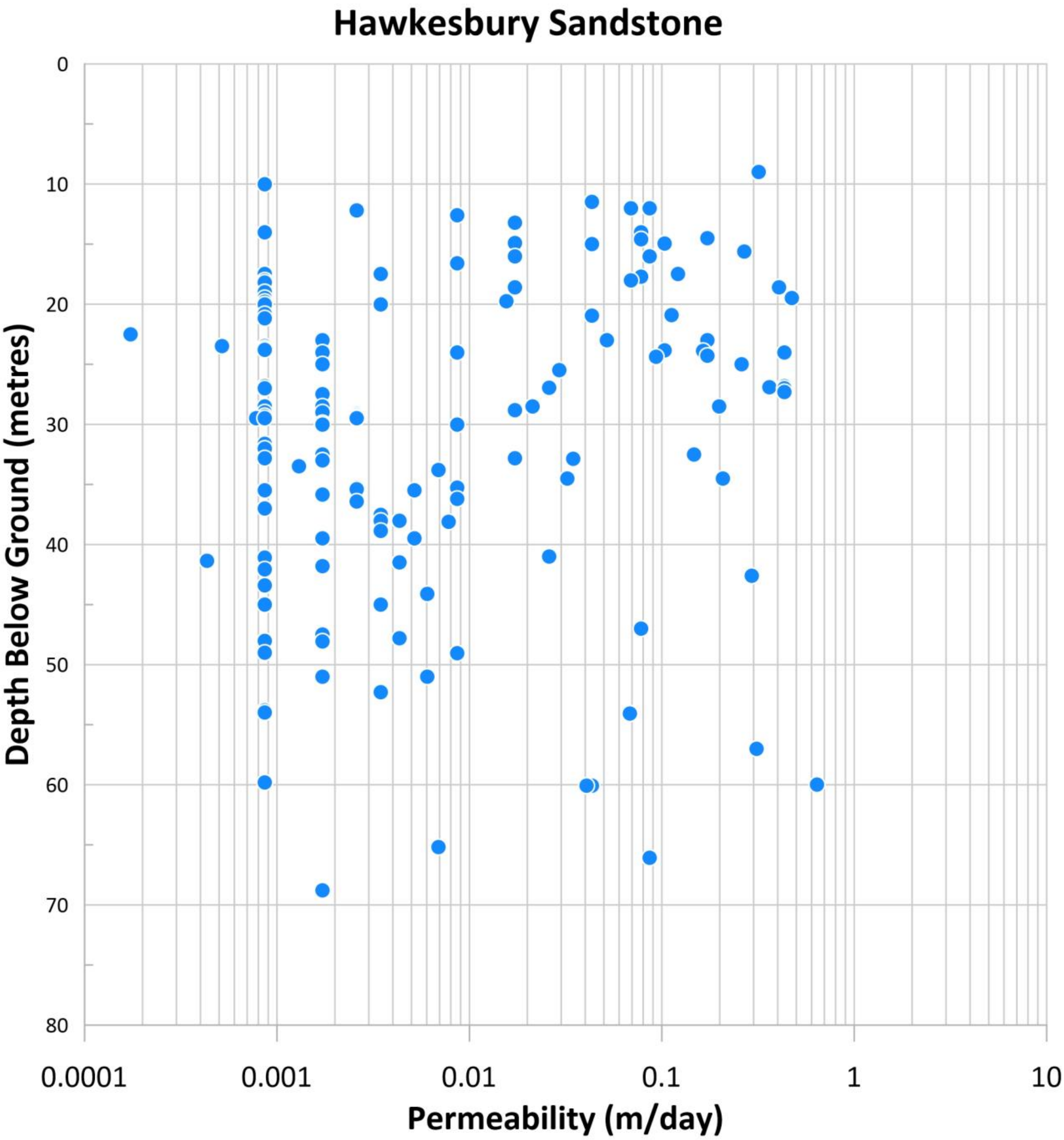


Figure 5: Analysis of packer testing in the Hawkesbury Sandstone

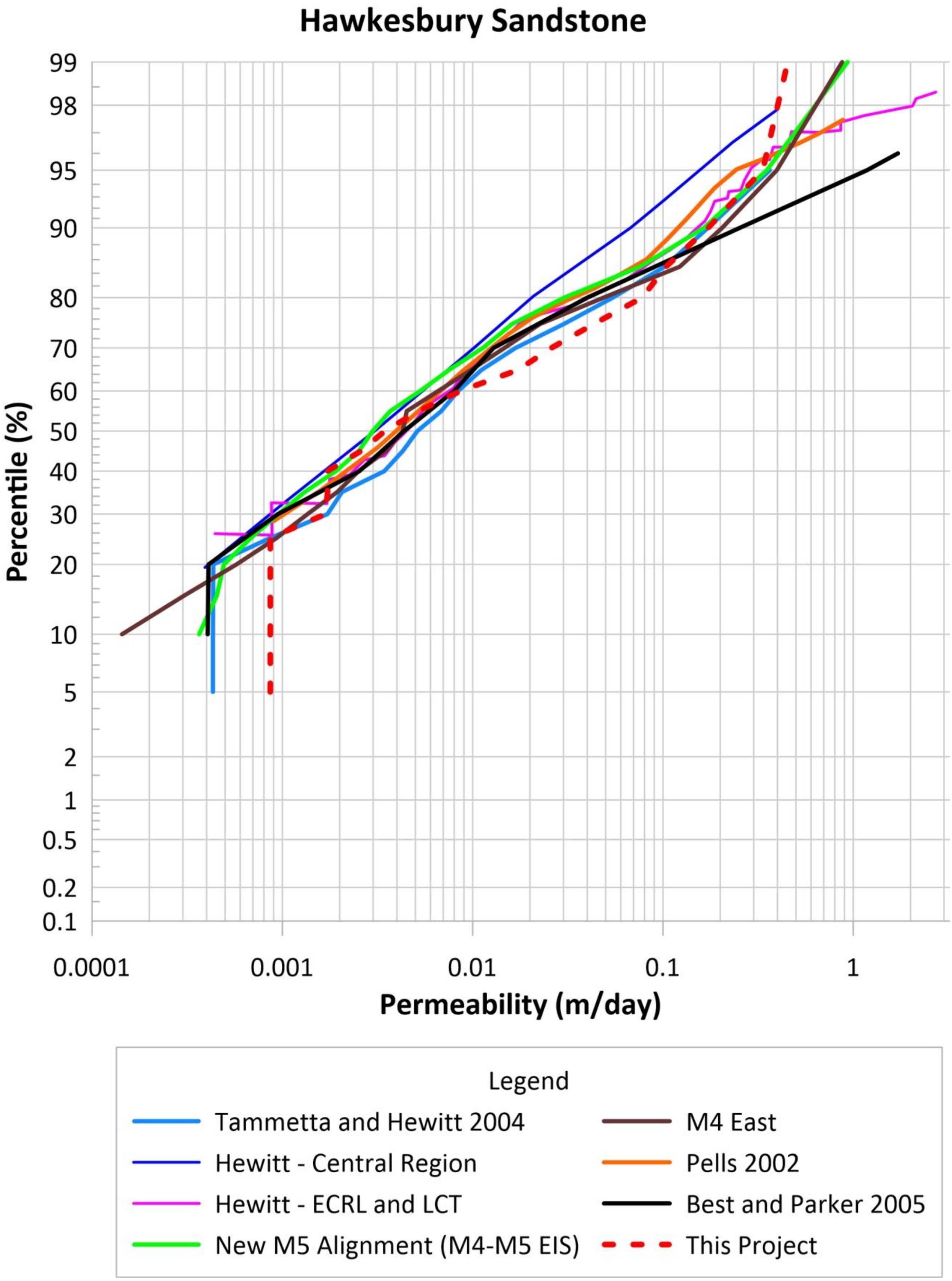


Figure 6: Permeability distribution of packer test results in Hawkesbury Sandstone and Mittagong

2.5 Key Hydrogeological Features

Along the Project alignment there are several features which from a hydrogeological perspective, are of key interest because of their potential impact upon construction:

- Dykes
- Faults, joints and joint swarms
- Acid Sulfate Soils
- Groundwater quality, and
- Existing project interactions.

Groundwater ingress to tunnelling project excavated in the Hawkesbury Sandstone is typically associated with major fractures or fault zones, however, not all structural features are saturated, open, and thus transmissive. Increased inflows can result from a higher conductivity associated with groundwater flow along such geological structures. Conversely, a reduced hydraulic conductivity can be associated with groundwater flow across these structures, and because the structure can act as a barrier to groundwater flow, it can lead to higher water pressures on one side. When intersected by a tunnel, the higher hydraulic head on the other side of the geological structure could result in higher inflows or burst inflows compared to those that were occurring before the structure was intersected.

The orientation of joints, faults and dykes influences the predominant groundwater flow directions within the Hawkesbury Sandstone. Fault planes and shears zones in the rock mass are relatively less common but may be of increased engineering significance due to their lateral extent, inherently typically lower strength, and increased hydraulic conductivity.

2.5.1 Dykes

A number of igneous dykes are known to intersect the Project alignment, while other dykes may be extrapolated to potentially intersect the alignment. A summary of dykes anticipated to intersect the alignment is presented in the GIR, noting 5 dykes are presented. The location and nature of some dykes along the alignment is reasonably well established, however, most anticipated dyke intersections have not been specifically investigated and are inferred from surrounding projects and exposures. As such, the location of dykes projected onto the alignment are indicative only.

Dykes in the Sydney Region are typically mafic to occasionally felsic (basaltic to doleritic) and sub-vertical. Grain sizes sometimes can be observed to increase towards the centre of the dykes due to slower rates of cooling compared to the dyke margins. While dykes commonly become fresher towards the core, this does not always occur. Occurrences of both completely weathered dykes (weathered to soil) and completely unweathered dykes are known, with the weathering relationship often related to the lithology being intruded and depth of intersection.

The dyke forming minerals are typically more susceptible to chemical weathering than sandstone host rock. Consequently, dykes often preferentially weather (i.e. degrade) to clayey soils near the surface. However, the reverse is often observed when the dykes intersect the Ashfield Shale with the contact adjacent shale more susceptible to weathering and degradation. Within the dyke, weathering and alteration can result in degradation being apparent at depth many tens of metres below the surface.

The orientation of some of the intrusions is consistent with the direction of one of the dominant jointing in the region, suggesting that the dykes often follow these pre-existing lines of weakness. Others favour other orientations suggestive of an off-shore magmatic source (Baxter-Crawford, 2018) while the dykes inferred around Haberfield to Burwood (east of SMW project) are likely associated with local diatreme intrusion.

Contact metamorphism between dykes and the Hawkesbury Sandstone can range from causing little or no effect, to heavily vitrified, leading to locally increased strength. Metamorphism of the sandstone can also result in higher strength material, increased shearing and/or faulting. Shearing is commonly encountered along the margins (i.e. contacts) of dykes or within the dyke itself, possibly reflecting magmatic episodes.

The intersection of dykes during tunnel construction can either increase or decrease groundwater ingress to the tunnel dependant on the weathering of the dyke and what unit or geological structures it cross cuts. Dykes that are unweathered and non-fractured or those that have been extremely weathered can create a hydraulic barrier to groundwater flow. These hydraulic barriers can cause differential groundwater pressure across the dyke and potential groundwater ingress to the tunnel through the fractured sandstone (running parallel and adjacent to the dyke), or limit flow where the sandstone has not been fractured. If the dyke intersects a water bearing feature (such a creek or overlying alluvial aquifers) this can provide a direct conduit for groundwater flow directly into the tunnel.

Some examples of projects within the general vicinity that have encountered specific issues in relation to dykes include:

- The Energy Australia Cable Tunnel Project was known to have problems at the TBM treatment plant with increased iron floc due to high iron concentrations of the groundwater. The Ultimo Dyke was intersected in this tunnel project.
- WestConnex M8 road project where dykes associated with the diatreme near Haberfield caused significant and immediate orange-coloured oxidation of the sandstone when excavated and resulted in additional durability treatment.

Dykes in the Sydney Region can vary markedly over relatively small distances due to geological factors such as “side-stepping” along defects, thinning out, faulting or other non-structurally controlled variations in geometry. A number of diatremes, such as the Dundas diatreme which is located about 4 km north of the alignment near Clyde, are located within a few kilometres of the alignment. Dykes are likely to radiate out from any of these diatremes and are likely to be more sheared than typical dykes.

2.5.2 Faulting/Jointing/Joint Swarms

The SMW-GIR has inferred a number of fault zones/joint swarms primarily based on surficial expressions in the Harbour and tributaries. Drilling data has been used to support the interpretations, presented in the GIR geological model. The fault zones indicate an apparent displacement in unit boundaries, and experience suggests these zones comprise a series of sub-parallel, steeply dipping to sub-vertically oriented joints over a few metres width with cumulative displacements. Hydrogeologically, the fault zones/joint swarms are important in that they may provide connectivity to surface water (where occurring at shallow depth) or promote groundwater flow. This can also enhance connection with overlying alluvials and result in higher excavation inflows and enhanced potential for settlement of alluvial material.

Rock permeability may be higher near faults/joints/joint swarms and therefore result in potentially higher groundwater inflows. Faults and joints can act as conduits to groundwater flow, however, faults may also act as barriers to groundwater flow. Increased groundwater inflows may be experienced during excavation where faults act as conduits to flow, with consequent depressurisation of the unit in the vicinity of the excavation. Excavation itself can enhance the inherent permeability of joints or brecciated zones through stress relief and dilation.

2.5.3 Acid sulfate soils

Acid sulfate soils (ASS) are naturally occurring soils, commonly associated with low lying areas of fine-grained sediments and typically occurring in lacustrine, estuarine, or swamp environments. Sediment accumulations within the harbours would also have an elevated risk of ASS. For acid sulfate soils to exist, the soils need to be saturated (anoxic) and contain sulfide minerals, the most common of which is pyrite. Potential acid sulfate soils (PASS) are water-saturated soils, rich in iron sulphide minerals, that have not yet been oxidised.

Groundwater level drawdown associated with construction excavation has the potential to de-saturate acid sulfate soils. Disturbance of PASS and exposure of the sulphide minerals to oxygen through de-saturation of the soils, results in sulfide oxidation and subsequent acidification of the soil and potentially groundwater. Acidification of groundwater can result in the mobilisation of heavy metals previously bound in the formation, leading to environmental impacts. Potential impacts of acidification and mobilisation of heavy metals include:

- Increased toxicity and loss of biodiversity in wetlands and waterways for ecosystems receiving the groundwater discharge
- Groundwater contamination for down-gradient groundwater users
- Reduced agricultural productivity
- Corrosion of concrete and steel infrastructure
- Discoloration of soil and groundwater seepage.

Management of ASS and PASS involves preventing the minerals from oxidising, or neutralising the acid released from oxidised soils by mixing those soils with a neutralising agent (generally lime). Acid drainage can also occur from rock formations that contain sulfide minerals, such as are likely to be present in the black shale units of the Ashfield Shale, and possibly in some finer grained units of the Hawkesbury Sandstone.

For the WTP alignment the characterisation and management of ASS and PASS is being considered by the GLC contamination team.

2.5.4 Groundwater quality

The quality of groundwater within the residual and alluvial soils that overlie the Ashfield Shale and Hawkesbury Sandstone is typically fresh to brackish, and may be saline in close proximity to salt water bodies. It typically has near-neutral to slightly acidic pH and concentrations of metals are generally lower than those in the underlying bedrock. Heavy industry activities and urbanisation have the potential for localised impacts on groundwater quality.

The quality within the Ashfield Shale is typically brackish to saline and acidic to near neutral (4 to 8) with a salinity ranged between 2,000 mg/L to 20,000 mg/L as Total Dissolved Solids (TDS). The quality of groundwater within the Hawkesbury Sandstone regionally is typically of low to moderate salinity, with electrical conductivity ranging between 500 µS/cm and 2,000 µS/cm (about 300 mg/L to 1,400 mg/L TDS using a 0.65 conversion factor), and pH values generally between 4.5 and 8. Generally, groundwater from this unit is a sodium-chloride type water, and high in iron. Organic compounds are not naturally associated with Ashfield Shale, Mittagong Formation or Hawkesbury Sandstone.

Groundwater in the Sydney region that has not been impacted by anthropogenic activity can contain heavy metal concentrations that are naturally above the Australian and New Zealand Environment and Conservation Council (ANZECC) water quality objectives. Elevated concentrations for some metals (e.g. iron and manganese) may be due to the leaching of natural metals from the host rock/soil.

For the WTP alignment groundwater quality, contamination and durability for structures is being considered by the GLC contamination and durability teams.

2.5.5 Groundwater Interaction with the surrounding environment

Within the WTP project alignment three areas have been identified where significant groundwater interaction may be present. These are at:

- Parramatta where basement dewatering may be occurring and near the Portal and Clyde Dive.
- Where groundwater extraction may occur from Rosehill Racecourse resulting in periodic groundwater drawdown.
- The Tender EIS (Jacobs 2020) also noted potential surface water groundwater interactions and associated terrestrial groundwater dependent ecosystems at Toongabbie Creek and Domain Creek at Westmead. It also noted interactions were highlighted at a number of other Creeks (e.g. Duck Creek near the MSF), however, these are considered to be tidal and unlikely to be adversely impacted by Project induced drawdown.

A review of publicly accessible registered groundwater bore information (i.e. BOM Groundwater Explorer Portal http://www.bom.gov.au/water/groundwater/explorer/map.shtml, date accessed 19 March 2022 and WaterNSW Real Time Data Portal https://realtimedata.waternsw.com.au/, date accessed 1 April 2022) identifies limited existing beneficial groundwater use near the Project alignment. However, this information has attached uncertainties as it may not identify all beneficial groundwater use (e.g. unregistered bores) or whether these beneficial use activities are active or whether the use is from the same beneficial aquifer and as such additional investigations (which are outside the scope of this report) may be required to confirm the presence of any potential beneficial groundwater use interactions.

2.5.6 Aquifer Interconnection

Based on the currently available information, the three main aquifers along the WTP alignment have some degree of hydraulic connection. There is vertical hydraulic connection from the unconfined alluvial aquifers to the underlying Ashfield Shale and from the Ashfield Shale to the underlying Hawkesbury Sandstone. The degree of vertical connection is expected to be reduced by historical depositional processes, degree of consolidation, lithostatic pressure and the presence of lower permeability layers which act more like aquitards

Where dykes and faults occur there may be reduced or enhanced hydraulic connection which may increase or decrease the vertical hydraulic connect between the aquifers. Where there is an absence of Ashfield Shale over the Hawkesbury Sandstone, the Hawkesbury Sandstone may be hydraulically connected to the overlying unconfined alluvial aquifers.

2.6 Local Hydrogeological Conditions

This section of the report provides a summary of the anticipated hydrogeological conditions at each of the key construction areas, which together form the Western Tunnelling Package between Sydney Olympic Park and Westmead. Attachment 1 is an interpreted hydrogeological section which shows the WTP 4.3 alignment geology, interpreted hydrogeology, groundwater levels and instrumentation details, hydraulic conductivity results and assessment of inflows and risk.

Refer to Attachment 1, Table 1-1 for a summary of the borehole position locations and other relevant hydrogeological details. Refer to the GIR for further borehole information. The technical memorandums for Westmead Station (SMWSTWTP-GLO-WMD-SN650-GE-MEM-010102 Rev A), Parramatta Station (SMWSTWTP-GLO-PTA-SN600-GE-MEM-010102 Rev A) and Clyde between the portal and Clyde Junction (SMWSTWTP-GLO-TJ550-GE-MEM-010101 Rev A.1) present the local geological and hydrogeological conditions and details of bores and testing which has been undertaken after tender submission.

The following sections in this report present a summary of the hydrogeological conditions along the project and the available hydrogeological information including the tender submission information. The tables within these sections present the screen interval for standpipe piezometers which includes the filter pack with slotted pipe section, the formation monitored and the minimum and maximum groundwater levels which have been recorded (excluding surface water ingress levels) or a relevant reported level. The groundwater level monitoring period is included within the tables. Hydraulic permeability results (i.e. water pressure (packer) tests and slug tests) are presented in tables within these sections for subsurface infrastructure excluding the running tunnels and selected results are shown on Attachment 1, Figure 1. Refer to the GIR for the complete investigation details associated with the bores.

2.6.1 Rosehill Services Facility

The HIR for Rosehill Services Facility has been prepared by Aurecon. Please refer to SMWSTWTP-GLO-RSH-SF500-EN-RPT-000001 (Groundwater Modelling Report – Rosehill Service Facility).

2.6.2 Running Tunnels – Sydney Olympic Park to Spur Junction

Geological Summary

Between Sydney Olympic Park and Haslams Creek, the running tunnels descend through the Ashfield Shale and encounter the interface with the Mittagong Formation and top of the Hawkesbury Sandstone. They pass through four inferred faults. Two faults beneath the creek displace the Mittagong Formation, such that the entire excavation is once again within Ashfield Shale.

From Haslam Creek towards Clyde, the running tunnels are excavated in mixed Ashfield Shale (crown and upper walls) and Mittagong (lower walls and floor) conditions, gradually moving into mixed Mittagong and Hawkesbury excavation from Ch17.150 km. From Ch17.400 km to Ch17.700 km, the entire excavation will be in Hawkesbury Sandstone, before progressing back into mixed HS-MF and MF-ASh conditions. There may be some areas of excavation where all three units will be exposed at a single chainage. From about Ch18.100 km, the excavation chases the Ashfield-Mittagong contact, with Mittagong variably present in the floor. The variability of intersected unit is inferred to be the result of displacement by several faults that intersect the alignment.

From Ch20.180 km to the Spur Junction the running tunnels are within the Hawkesbury Sandstone and pass through four inferred faults and an inferred dyke around Ch20.600 km.

Provided boreholes in this location include:

- SMW_BH015
- SWM_ENV712
- SWM_BH120
- SWM_WTP_BH24
- SMW_BH071
- SMW_WTP_BH23
- 3103-124
- SMW_BH121
- SMW_BH031
- SMW_BH030
- SMW_WTP_BH22
- 3101-122
- SWM_WTP_BH21
- 3101-121
- SMW_BH709
- 3103-119

- SMW_WTP_BH20
- SMW_BH115
- SMW_BH060
- SMW_BH063
- SMW_WTP_BH19
- SMW_ENV042
- SMW_BH010
- SMW_WTP_BH35
- SMW_WTP_BH17
- SWMW_ENV806
- SMW_WTP_BH16
- SMW_WTP_BH15
- SMW_ENV801
- SMW_WTP_BH14
- SMW_WTP_BH13
- 3103-113
- SWM_ADD_BH01A
- SMW_ADD_BH02_w
- SMW_BH111
- SMW_BH707
- SMW_BH708 and
- SMW_BH045.

Groundwater Levels

Between Sydney Olympic Park and Haslams Creek, the groundwater table ranges from 18 mAHD at Sydney Olympic Park to near 0 mAHD at Haslams Creek, inferred from bores SMW_BH015_s, SMW_WTP_BH23_w, SMW_BH121_w, SMW_WTP_BH22_w and VWPs SMW_BH031_v and SMW_BH030_v. At SMW_WTP_BH23_w, just east of Haslams Creek, the groundwater table is around 3 mAHD (see plan view in Attachment 1).

West of Haslams Creek to Duck River the groundwater table ranges from 5.4 mAHD to 1 mAHD and near 0.4 mAHD at Duck River. Between Duck River and Duck Creek the groundwater table is near ground surface and ranges from 0.4 mAHD to 2 mAHD. From Duck Creek to the Clyde Access Shaft the groundwater table in this section of the running tunnels is inferred from SMW_BH043_w, SMW_ENV283_w and SWM_BH057_w and ranges from 1.6 mAHD to 7.3 mAHD.

From the Clyde Access Shaft to the Spur Junction the groundwater table in this section of the running tunnels is inferred from SMW_BH007_w, SMW_BH111_v, SMW_BH045_v and SMW_BH707_w. The groundwater table ranges from 1 mAHD to near 6.1 mAHD.

Hydraulic Parameters

Hydraulic conductivity results for the bores listed above range from 0.0009 m/day to 0.51 m/day in the Ashfield Shale and 0.0009 m/day to 0.475 m/day in the Hawkesbury Sandstone at the tunnel alignment. Attachment 1, Figure 1 presents selected water pressure (packer) tests results along the alignment. Refer to the GIR for details on the boreholes and water pressure (packer) tests completed in this section of running tunnels.

2.6.3 Clyde Maintenance and Stabling Facility and Water Conveyance Structures

The Clyde MSF is located in an area of low-lying land near Duck River. Surface works will require excavation below existing ground surface for the construction of the water conveyance structures (WCS) and a water retention basin. Construction of the WCS will alter the alignment of A'Becketts Creek and Duck Creek, which flows into the Duck River. They are anticipated to intersect the shallow groundwater table.

Groundwater Levels

Table 2 presents a summary of the groundwater monitoring bores in the area of the retention basin and WCS. The groundwater level in the alluvial / residual /fill material is shallow, ranging from 0.5 mAHD to 4.2 mAHD. It should be noted that the groundwater levels are from bore development records as no monitoring data has been provided and the reported groundwater levels may be influenced by residual drilling fluid and or the development process.

In this area the groundwater table is anticipated to be relatively flat, between 0.5 and 5 m below ground surface and with gradients towards the discharge points of the creeks and Duck River.

Table 2: Summary of groundwater levels at Clyde Dive

Bore Name	Ground Level (mAHD)	Screen Interval (m bgl)	Lithology Screened	Standing Water Level Range (mAHD)	Date of Recorded Level and Type
Retention Basin Area					
SMW_ENV083_w	5.03	1.5-6.0	Clay	3.75 to 4.27	13/02/2020 – Development
SMW_ENV293_w	5.47	1.2-6.0	Clay	0.76	14/10/2021 - Development
SMW_ENV089_w	4.96	3.0-6.0	Fill	-0.99 to -0.52	17/02/2020 - Development
SMW_ENV090S_w	4.57	1.0-3.0	Fill	1.62 to 1.78	13/03/2020 - Development
SMW_ENV090D_w	4.58	3.2-6.0	Clay	1.38 to 1.63	13/03/2020 - Development
SMW_ENV284_w	5.02	1.5-6.0	Clay	3.93	7/10/2021 - Development
SMW_WTP_BH19_w	5.62	34.0-40.3	Sandstone	-7^	1 Dec 2021 to 3 Mar 2022 – Hydrograph
Water Conveyance Structures					
SMW_ENV218_w	4.37	2.5-6.0	Clay	3.17	21/12/2021 - Development
SMW_ENV219_w	4.78	3.3-6.5	Clay	-0.36 to 0.67	9/12/2021 - Development
SMW_WTP_BH29_w	4.52	8.5-12.1	Clay	2.13	20/10/2021
SMW_ENV088_w	4.85	2.5-6.0	Clay	1.56 to 1.95	13/02/2022
SMW_ENV089_w	4.96	3.0-6.0	Fill	-0.99 to -0.52	17/02/2020 - Development
SMW_ENV151	3.96	2.2-6.2	Clay	2.5	September 2020 - Installation
SMW_ENV280_w	4.44	3.8-7.0	Clay	1.08 to 1.53	8/12/2021 - Development
SMW_ENV221_w	4.46	1.5-5.0	Fill	0.99	21/10/2021 - Development
SMW_ENV200_w	4.52	2.7-6.2	Clay	0.58	8/12/2021 - Development

Bore Name	Ground Level (mAHD)	Screen Interval (m bgl)	Lithology Screened	Standing Water Level Range (mAHD)	Date of Recorded Level and Type
SMW_ENV201_w	4.11	2.0-5.5	Clay and Fill	1.085	8/12/2021 - Development
SMW_ENV202_w	4.3	2.0-5.5	Fill and Silt	0.98	8/12/2021 - Development
SMW_ENV279_w	4.75	2.8-6.3	Clay	2.89	8/12/2021 - Development
SMW_ENV276_w	4.53	3.8-7.0	Clay	2.64	9/12/2021 - Development
SMW_ENV272_w	4.28	1.6-6.0	Fill and Clay	1.67	9/12/2021 - Development
SMW_ENV146	4.28	2.5-6.3	Clay	3.08	September 2020 - Installation
SMW_ENV275_w	5.0	1.5-10	Clay	2.37	22/12/2021 - Development

Hydraulic Parameters

Slug test results are available from four bores intersecting the natural clays in this area. The estimated hydraulic conductivities from the slug testing are as follows:

- SMW_ENV076 – 0.067 m/day
- SMW_ENV151 – 0.018 m/day
- SMW_ENV146 – 0.003 m/day and
- SMW_ENV045 – 0.033 m/day.

2.6.4 Clyde Portal and Dive

From the MSF area, the Portal descends northward towards the Clyde Dive from ground surface in fill and residual material through alluvial material and into Ashfield Shale. The cut and cover Dive structure, which includes the portal entrances for the road header constructed tunnels to the spur junction, is located within residual material and Ashfield Shale with the potential for fill material and some alluvium to be intersected at the southern end where it joins the portal. A fault is present within the Dive, coincident with the margin of the Duck Creek alluvial channel. This would imply a strong structural control on that alluvial channel and possible connectivity between soil and rock aquifers locally.

Groundwater Levels

Groundwater levels are presented in Table 3 for the standpipe piezometers in Ashfield Shale and the alluvial material. Monitoring data in the alluvial material for the groundwater table are limited to levels reported at the time of installation. The groundwater table, as shown on Attachment 1, Figure 1, is inferred to range from 1.6 mAHD to 7.8 mAHD with the potential for the groundwater levels in SWM_ENV077 and SMW_ENV078 to be influenced by local groundwater extraction (i.e. at Rosehill Racecourse).

Table 3: Summary of groundwater levels at Clyde Portal and Dive

Bore Name	Approximate Chainage (km)	Ground Level (mAHD)	Screen Interval (m bgl)	Lithology Screened	Standing Water Level Range (mAHD)	Groundwater Level Data Period
SMW_BH064_w	20.320	9.5	5.9-8.9	Siltstone with some clay	2.70 to 3.65	Nov 2019 to Sep 2020
SMW_BH043_w	20.410	12.78	6.5-12.5	Siltstone	6.9 to 7.8	May to Sep 2020
SMW_ENV039	20.150	6.41	7.3-10.3	Clay	1.71	Installation record
SMW_ENV077	20.200	6.03	6.0-9.0	Clay	1.99	Installation record
SMW_ENV078	20.220	6.38	8.5-14.5	Clay	0.74	Installation record

Hydraulic Parameters

Hydraulic conductivity results from water pressure (packer) tests in the Ashfield Shale at bore SMW_BH043 was 0.008 m/day (approximately 1.0 Lugeon) for the test intervals of 13 m to 18 m and 17.8 m to 24 m. Hydraulic conductivities from analysis of slug test results for SMW_ENV077 and SMW_ENV078, screened in clay, were 0.01 m/day and 0.1 m/day respectively.

2.6.5 Clyde Access Shaft

The access shaft at Clyde is located around 300 m to the north of the Clyde Dive. The shaft will encounter several metres of fill and residual soil which overlies the Ashfield Shale. The base of the shaft will be in the Hawkesbury Sandstone. The shaft will allow the entrance of the road header mining infrastructure to commence construction of the Spur Tunnels, via a short adit from this point, towards the Spur Junction before construction of the Clyde Dive and Portal is completed.

At present there are no monitoring results for groundwater levels in the bedrock at the access shaft. SMW_ADD_BH02_w had an observed groundwater level at the time of installation, in Hawkesbury Sandstone, of -0.4 mAHD. The groundwater table is inferred to range from 2 mAHD to 7 mAHD.

2.6.6 Clyde Road Header Spur Tunnels - Dive to Spur Junction

The road header mined tunnels are two separate tunnels that descend through the Ashfield Shale to the Spur Junction in the Hawkesbury Sandstone. These will be undrained at handover, and no cross-passages are proposed. A number of siltstone lenses are interpreted within the Hawkesbury Sandstone, potentially intersecting the tunnels with siltstone also inferred in the crown and shoulders of the Spur Junction excavation. Four faults and a dyke are interpreted as intersecting this portion of tunnel development. The interaction between subvertical fractured fault structures and the siltstone lenses in the Hawkesbury may contribute increased inflows to the excavation.

Groundwater Levels

The groundwater table is inferred to range from 7.2 mAHD at the Dive end of the Spur Tunnels to 1.8 mAHD at the Spur Junction. Table 4 presents a summary of the groundwater monitoring bores in the area.

Table 4: Summary of groundwater levels at Spur Tunnels

Bore Name	Approximate Chainage (km)	Ground Level (mAHD)	Screen Interval (m bgl)	Lithology Screened	Standing Water Level Range (mAHD)	Groundwater Level Data Period
SMW_BH007_s	20.800	6.49	4.15 - 7	Clayey Sand	1.3 to 2.1	Jul 2018 to Sep 2019
SMW_BH007_w	20.800	6.49	15 - 22.4	Sandstone	-0.7 to -0.25	
SMW_BH043_w	20.410	12.78	6.5-12.5	Siltstone	6.9 to 7.8	May to Sep 2020
SMW_BH057_s	20.800	3.84	1 - 5.3	Sand	1.4 to 1.6	Nov 2019 to Sep 2020
SMW_BH057_w	20.800	3.84	23.3 - 26.3	Sandstone	1.6 to 2.15	
SMW_ENV009	20.760	4.28	2.7 - 7.3	Clayey Sand	2.0 to 2.4	
SMW_ENV010	20.278	4.28	3.2 - 6.6	Sandy Clay	1.8 to 2.4	
SMW_ENV011	20.780	3.81	3.0-7.0	Clayey Sand	2.4	Installation record
SMW_BH111_v	20.800	9.74	25.65^	Sandstone	1.5 to 1.75	Nov 2019 to Jan 2020

Hydraulic Parameters

The SMW-GIR interprets several faults and a dyke to be present, coincident with the tunnels transition from the Ashfield Shale through the Mittagong Formation and into the Hawkesbury Sandstone. Water pressure (packer) test results from boreholes nearby that may have intersected the dyke or structural features show hydraulic conductivity test results of up to 0.2 m/day in the Hawkesbury Sandstone. Table 5 presents a summary of the hydraulic conductivity results from water pressure (packer) tests in nearby boreholes.

Table 5: Summary of hydraulic conductivity test results at Clyde Dive

Bore Name	Approximate Chainage (km)	Test Interval (m bgl)	Lugeon Value (uL)	Hydraulic Conductivity (m/day)	Formation
3103-112	20.570	5.0-11.0	0.7	0.006	Ashfield Shale
		10.5-17.0	3.9	0.03	Ashfield Shale
		16.5-23.0	4.7	0.04	Ashfield Shale
		22.5-29.1	0.3	0.002	Mittagong Formation
		28.5-35.2	0.2	0.001	Hawkesbury Sandstone
3103-111	20.680	5.0-11.0	5.0	0.04	Ashfield Shale
		10.5-17.1	0.1	0.0009	Ashfield Shale
		16.5-23.0	0.2	0.001	Ashfield Shale
		22.5-29.1	0.3	0.002	Ashfield Shale

Bore Name	Approximate Chainage (km)	Test Interval (m bgl)	Lugeon Value (uL)	Hydraulic Conductivity (m/day)	Formation
		28.5-35.1	23.0	0.19	Hawkesbury Sandstone
		32.5-41.2	17.0	0.14	Hawkesbury Sandstone
		34.5-41.2	24.0	0.20	Hawkesbury Sandstone
SMW_BH111	20.800	15.885-22.0	0.3	0.002	Mittagong Formation
		21.8-29.0	0.6	0.005	Mittagong Formation
		28.8-34.0	2.0	0.017	Hawkesbury Sandstone
		33.8-39.98	0.8	0.006	Hawkesbury Sandstone
SMW_ADD_BH01A	20.660	20.15 - 26.15	1	0.00864	Ashfield Shale
		35.25 - 41.25	0.3	0.002592	Hawkesbury Sandstone
SMW_ADD_BH02	20.600	21.15 - 27.15	<0.1	0.000864	Hawkesbury Sandstone
		36.2 - 42.2	1	0.00864	Hawkesbury Sandstone

2.6.7 Spur Junction

The spur junction is a series of road header mined tunnels, decreasing in diameter towards the west, connecting the main alignment running tunnels to the Clyde MSF. The spur junction is within the Hawkesbury Sandstone and may be mined in advance of the TBM, or as break outs after TBM progression.

The groundwater level at bore SMW_BH707_w, screened in Hawkesbury Sandstone, ranges from 2.4 mAHD to 2.6 mAHD. At bore SMW_BH045_v, the Vibrating Wire Piezometer (VWP) sensor is in the Hawkesbury Sandstone and the groundwater level is around 2.38 mAHD.

Hydraulic conductivity test results at bore SMW_BH708, which is an inclined (-52°) hole, are presented in Table 6.

Table 6: Summary of hydraulic conductivity test results Running Tunnels Rosehill to Spur Junction

Bore Name	Approximate Chainage (km)	Test Interval (m bgl)	Lugeon Value (uL)	Hydraulic Conductivity (m/day)	Formation
SMW_BH708	20.175	24.3-30.3	20	0.17	Hawkesbury Sandstone
		27.3-30.3	50	0.43	Hawkesbury Sandstone
		29.3-36.28	0.1	0.0009	Hawkesbury Sandstone
		35.28-42.3	1.0	0.008	Hawkesbury Sandstone
		41.0-51.0	3.0	0.02	Hawkesbury Sandstone

2.6.8 Running Tunnels – Spur Junction to Parramatta

The running tunnels are situated within the Hawkesbury Sandstone and pass through the Parramatta Dyke around Ch22.310 km and inferred fault structures before the Station Box. There is one existing borehole along this location, bore 3103-109, with one packer test result for the test interval 34.50 m to 38.86 m at 0.03 m/day (3.7 lugeons) and one new project bore SMW_WTP_BH11 with a packer test result of 0.009 m/day in the Hawkesbury Sandstone.

A shallow groundwater table ranging from 2.5 mAHD to 3.9 mAHD is inferred from a nearby contaminated land investigation (5-7 Charles St Site Contamination Investigation Report (Sullivan Environmental Sciences, 2015), specifically from borehole GW4; and from Project borehole SMW_BH707_w.

2.6.9 Parramatta Station

Geological Summary

Parramatta Station is located in an area of thick alluvial sediments associated with the Parramatta River. The topography in the vicinity the Station Box is generally flat with existing ground levels about 10 m AHD. The geometry / composition of the alluvial sediments below the site remains somewhat uncertain, especially for the extent of the 'Parramatta Sands'.

Alluvial soils extend to depths of about 12 m to 15 m below ground level, with the rock-soil interface rising slightly in the eastern portion of the Station Box. The GIR interprets multiple structures and an unnamed dyke intersect the Station Box at Ch22.415, while the Parramatta Dyke is inferred to be located to the east of the Station Box excavation. Its presence outside of the excavation, however, does not mean it may not still influence the Station Box's groundwater conditions.

Groundwater Levels

Table 7 presents a summary of groundwater levels and standpipe piezometer details of alluvial and bedrock standpipe piezometers at Parramatta. The groundwater table in the alluvial sediments ranges from 3.3 mAHD to 4.5 mAHD (three standpipe piezometer observations) and indicates there is a gradient to the north towards the Parramatta River. The water level of the Parramatta River is approximately 0.2 mAHD but may fluctuate up to 1.5 m (higher) in response to weather conditions. The water level of the river above the Marsden St weir is a few metres higher (overflow occurs at 4.3 mAHD). The City of Parramatta provides the current day river water level at https://www.cityofparramatta.nsw.gov.au/environment/floodsmart-parramatta/check-your-river-and-rain-gauge-levels (date accessed 21 March 2022). A chart is currently unavailable for the Riverside Theatre site. The groundwater table in the alluvial sediments is variable and fluctuates in response to rainfall.

In the bedrock, the groundwater potentiometric surface ranges from –2.45 mAHD to 3.0 mAHD (three standpipe piezometer observations). The potentiometric surface is several metres lower than the alluvial sediments suggesting a downward gradient from the sediments and/or under-drainage effects. Construction within the Parramatta CBD includes many multi-story complexes with deep basements. Some of these basements may have dewatering systems in place.

The hydrograph provided for bores SMW_BH004_w and SMW_BH048_w located to the north of the Station Box shows that, following a significant rainfall event in February 2020, the groundwater level initially rose by almost a metre in the standpipe piezometer but then declined over a few weeks to be below the pre-rainfall level by half a metre. This supports the theory that local basement dewatering is occurring. In the shallow bores (i.e.SMW_BH004_s) a small initial response was observed in the alluvium for this event. Monitoring data has not been provided, nor publicly available for the period including the significant rainfall and flooding event of February 2022.

Table 7: Summary of groundwater levels at Parramatta Station

Bore Name	Approximate Chainage (km)	Ground Level (mAHD)	Screen Interval (m bgl)	Lithology Screened	Standing Water Level Range (mAHD)	Groundwater Level Data Period
SMW_BH002_w	22.370	8.99	26.5 - 32.4	sandstone	-11.50 to -3.90	Aug 2018 to Sep 2019
SMW_BH003_s	22.530	10.67	8.4 - 11.0	clayey sand	3.39 to 4.05	
SMW_BH003_w	22.530	10.67	13.0 - 18.0	clay and siltstone	2.80 to 3.80	
SMW_BH004_s	22.430	8.68	5.7 - 11.5	sand	7.50 to 7.90	
SMW_BH004_w	22.430	8.68	20.6 - 23.6	sandstone	1.40 to 3.00	
SMW_BH048_s	22.500	6.95	4.0 - 7.5	sand	2.25 to 2.75	Dec 2019 to Jun 2020
SMW_BH048_w	22.500	6.95	19.6 - 22.6	sandstone	-2.45 to – 1.85	
SMW_BH049_s	22.610	8.99	1.6 - 6.0	silty clay	Not Accessed	No results
SMW_BH049_w	22.610	8.99	16.9 - 22.1	sandstone	Not Accessed	

Hydraulic Parameters

Table 8 presents a summary of the hydraulic conductivity results at Parramatta Station from slug tests from selected standpipe pizometers documented the Douglas Partners (2016) Parramatta Square Geotechnical Investigation, and project water pressure (packer) tests in the Ashfield Shale and Hawkesbury Sandstone.

Table 8: Summary of hydraulic conductivity test results at Parramatta Station

Bore Name	Approximate Chainage (km)	Test Interval (m bgl)	Lugeon Value (uL)	Hydraulic Conductivity (m/day)	Formation
SMW_BH003	22.530	16.9-19.5	0.2	0.001	Ashfield Shale
		19.5-27	<0.1	0.0009	Hawkesbury Sandstone
		26.8-34	0.1	0.0009	Hawkesbury Sandstone
SMW_BH048	22.500	23.86-27.16	12.0	0.1	Hawkesbury Sandstone
		26.94-35.13	3.0	0.02	Hawkesbury Sandstone
SMW_BH049	22.610	14.0-18.14	<0.1	0.0009	Hawkesbury Sandstone
		17.5-24.11	14.0	0.12	Hawkesbury Sandstone

Bore Name	Approximate Chainage (km)	Test Interval (m bgl)	Lugeon Value (uL)	Hydraulic Conductivit y (m/day)	Formation
		18.2-28.78	<0.1	0.0009	Hawkesbury Sandstone
		27.5-34.16	0.2	0.001	Hawkesbury Sandstone
SWM_BH704	22.440	19.0-25.25	<0.1	0.0009	Hawkesbury Sandstone
		24.0-30.23	0.2	0.001	Hawkesbury Sandstone
SMW_BH705	22.4440	19.0-25.0	<0.1	0.0009	Hawkesbury Sandstone
		24.0-30.01	1.0	0.008	Hawkesbury Sandstone
3103-106	22.4540	14.15-20.15	1.8	0.015	Ashfield Shale
		20.15-21.15	0.01	0.0001	Hawkesbury Sandstone
		25.65-30.19	0.01	0.0001	Hawkesbury Sandstone
SMW_WTP_ Site01_BH01	22.370	12.0-18.0	10	0.086	Hawkesbury Sandstone
		18.0-24.0	8	0.069	Hawkesbury Sandstone
		18.0-24.0	8	0.069	Hawkesbury Sandstone
		24.0-30.0	50	0.43	Hawkesbury Sandstone
		27.0 - 30	50	0.43	Hawkesbury Sandstone
SMW_WTP_ Site01_BH02	22.370	12.6-14.6	1	0.0008	Hawkesbury Sandstone
		14.6 - 16.6	9	0.07	Hawkesbury Sandstone
		16.6 -18.6	1	0.0008	Hawkesbury Sandstone
		18.6-20.6	2	0.017	Hawkesbury Sandstone
		20.0-23.0	<0.1	0.0008	Hawkesbury Sandstone
		23.0-26.0	6	0.05	Hawkesbury Sandstone
		26.8-30.0	50	0.43	Hawkesbury Sandstone
SMW_WTP_ Site01_BH03	22.370	12.0-14.0	8	0.069	Hawkesbury Sandstone
		14.5-16.5	20	0.17	Hawkesbury Sandstone
		16.0-18.5	10	0.08	Hawkesbury Sandstone
		18.5-20.5	2	0.017	Hawkesbury Sandstone
		20.0-23.0	<0.1	0.0008	Hawkesbury Sandstone
		25.0-30.0	30	0.25	Hawkesbury Sandstone
601	22.520	16.0-23.0	NA	0.06	Siltstone
610A	22.500	3.5-7.0	NA	0.01	Clay
613A	22.370	3.8-7	NA	0.003	Weathered Siltstone
615	22.360	16.0-23.0	NA	0.12	Siltstone and Sandstone
617	22.400	16.0-23.0	NA	0.06	Siltstone and Sandstone
402	22.400	4.0-12.0	NA	0.06	Alluvial and dolerite

The water pressure (packer) tests in borehole SWM_WTP_Site01 are in close proximity to each other and next to existing building basements and foundations. The high values, whilst viewed with some caution, do suggest there is more permeable ground at depth in this area, likely the result of proximity to geological structure(s) (i.e. faults and dykes). The drilling records of these new holes recorded losses of the drilling fluid the bedrock and the TS-GIR suggests potentially brecciated material inferred to be associated with a NNE trending dyke in nearby bore SMW_BH705. Several bores by Douglas Partners (2016) intersected this dyke, including bore 401, located 70 m to the south east, of SMW_BH705.

2.6.10 Running Tunnels – Parramatta to Westmead

Geological summary

The running tunnels are situated within the Hawkesbury Sandstone and pass through inferred faults at Ch23.110 km and Ch23.703 km. The displacement inferred on these structures is favourable for the running tunnel excavation, resulting in the running tunnels continuing in Hawkesbury Sandstone until Ch 23.700 km, much further than would have been the case without the faulting. The tunnels intersect the Mittagong Formation on the approach to the Westmead Crossover Cavern. Existing boreholes include:

- SMW_BH049
- SMW_BH703
- 3101-105
- SMW_BH012
- 3101-104
- SMW_BH008
- SMW_BH016 and
- SMW_BH701

Groundwater Levels

The groundwater table is interpreted to range from 4.1 mAHD in the alluvial sediments to 23.3 mAHD in the elevated part of the Ashfield Shale. Groundwater level in bore SMW_BH008, screened in siltstone, is 16.6 mAHD.

Hydraulic Parameters

Hydraulic conductivity test results for bore SWM_BH012 range from 0.1 to 0.4 m/day, as shown on Attachment 1, and suggest the structure at Ch23.110 km may have increased permeability and flow.

2.6.11 Westmead Station, Caverns and Nozzles

Geological Summary

The Westmead Station and Caverns will be located in faulted Ashfield Shale with the eastern Cavern base intersecting the Mittagong Formation. Overlying the Ashfield Shale is several metres of residual soil and fill. The WTP-GIR infers several near vertical faults in the Station Box and Caverns.

Groundwater Levels

The depth to the groundwater table is inferred to range from 25 mAHD to 30 mAHD at the Station Box at the top of the hill to around 16 mAHD at the crossover cavern, east of the station. The groundwater table is a subdued version of the topography. Table 9 presents a summary of groundwater levels and standpipe piezometer details of residual and bedrock standpipe piezometers at Westmead. The groundwater levels in the residual clay soil may be perched.

Table 9: Summary of groundwater levels at Westmead Station

Bore Name	Approximate Chainage (km)	Ground Level (mAHD)	Screen Interval (m bgl)	Lithology Screened	Standing Water Level Range (mAHD)	Groundwater Level Data Period
SMW_BH001_s	24.140	31.12	0.6 - 1.49	silty clay	29.6 to 30.5	Aug 2018 to Sep 2019
SMW_BH001_w	24.140	31.13	6.7 - 11.7	siltstone	25.5 to 28.51	
SMW_BH008_w	23.870	21.28	13.0 – 18.0	siltstone	15.60 to 16.60	
SMW_BH701_w	24.100	29.38	4.0 - 9.0	siltstone	24.85 to 25.05	Apr to Jun 2021
SMW_ENV297_w	24.090	30.91	2.0 - 8.5	clay	Not Recorded	
SMW_ENV299_w	24.080	30.00	2.3 - 6.5	clay	Not Recorded	
SMW_ENV300_s	24.070	30.10	0.8 - 2.0	clay	28.64 to 28.95	Development Records Dec 2021
SMW_ENV300_w	24.070	3.06	2.3 - 5.5	clay	Not Recorded	
SMW_ENV301_s	24.100	30.63	0.8 - 2.0	clay	30.48	Development Record Oct 2021
SMW_ENV301_w	24.100	30.59	2.3 - 5.5	clay	-2.17 to -1.35	Development Record Oct 2021
SMW_ENV294_w	24.070	29.41	2.3 - 6.5	gravel and clay	24.79 to 27.78	Development Record Oct 2021
SMW_ENV295_w	24.090	29.76	2.3 - 6.5	clay	27.50 to 28.39	Development Records Dec 2021
SMW_WTP_BH01A_w	24.090	30.95	2.5 - 7.1	clay and siltstone	24.85	Installation Record
SMW_WTP_BH02_w	24.320	35.76	13.0 - 20.5	siltstone	32.50 to 33.20	Feb to Mar 2022
SMW_WTP_BH03A_w	23.980	26.37	13.1 - 22.0	siltstone	24.36 to 24.67	Dec 2021 to Mar 2022

Bore Name	Approximate Chainage (km)	Ground Level (mAHD)	Screen Interval (m bgl)	Lithology Screened	Standing Water Level Range (mAHD)	Groundwater Level Data Period
SMW_WTP_ BH31A_w	24.200	36.55	3.8 - 8.5	siltstone	Not Recorded	
SMW_WTP_ BH32A_w	24.150	32.09	3.5 - 10.1	siltstone	Not Recorded	

No readings are available from the VWP in SMW_BH013 except for the installation record of 32 mAHD for the shallow sensor (installed at 25.65 mAHD) and 14 mAHD for the deep sensor (installed at 56.6 mAHD). The water level presented on borehole logs for standpipe piezometer installations represents the combination of groundwater and drilling fluid and is not considered to represent the true groundwater level. An exception to this is SWM_WTP_BH01A_w which was drilled without water.

Hydraulic Parameters

Table 10 presents a summary of the results from water pressure (packer) tests in the Ashfield Shale and Hawkesbury Sandstone.

Table 10: Summary of hydraulic conductivity test results at Westmead

Bore Name	Approximate Chainage (km)	Test Interval (m bgl)	Lugeon Value (uL)	Hydraulic Conductivity (m/day)	Formation
SMW_BH005	24.200	18.5-24.5	0.2	0.001	Ashfield Shale
		27.5-33.27	0.3	0.002	Ashfield Shale
SMW_BH006	24.210	10.0-14.86	3.0	0.02	Ashfield Shale
		14.3-200.8	1.0	0.008	Ashfield Shale
		20.4-28.4	0.1	0.0009	Ashfield Shale
		28.0-34.0	0.2	0.001	Ashfield Shale
SMW_BH008	23.870	5.5-12.11	3.0	0.02	Ashfield Shale
		11.9-18.11	4.0	0.03	Ashfield Shale
		17.75-24.2	0.4	0.003	Ashfield Shale
SMW_BH013	24.181	30.55-36.55	0.2	0.001	Ashfield Shale
		36.05-42.55	<0.1	0.0009	Ashfield Shale
		42.05-48.55	<0.1	0.0009	Mittagong Formation
		48.05-54.58	0.2	0.001	Mittagong Formation
		54.05-60.55	8.0	0.06	Hawkesbury Sandstone
		60.05-66.55	5.0	0.04	Hawkesbury Sandstone
		66.05-70.15	10.0	0.08	Hawkesbury Sandstone
SMW_BH016	23.920	6.65-12.2	0.1	0.0009	Ashfield Shale
		12.0-18.2	0.2	0.001	Ashfield Shale

Bore Name	Approximate Chainage (km)	Test Interval (m bgl)	Lugeon Value (uL)	Hydraulic Conductivity (m/day)	Formation
		18.0-24.2	0.2	0.001	Ashfield Shale
SMW_BH700	24.250	23.7-30.2	<0.1	0.0009	Ashfield Shale
		29.3-36.3	<0.1	0.0009	Ashfield Shale
		35.3-42.3	<0.1	0.0009	Ashfield Shale
		42.3-48.5	<0.1	0.0009	Ashfield Shale
		47.45-54.45	<0.1	0.0009	Ashfield Shale
SMW_BH701	24.090	17.0-24.0	0.2	0.001	Ashfield Shale
		23.0-28.19	0.2	0.001	Hawkesbury Sandstone
SMW_WTP_ BH01	24.090	12.46-18.46	10	0.086	Ashfield Shale
		22.77-28.77	0.1	0.001	Ashfield Shale
		49.25-55.25	0.3	0.003	Ashfield Shale
SMW_WTP_ BH02	24.320	9.31-5.38	1.00	0.009	Ashfield Shale
		22.00-28.08	0.80	0.007	Ashfield Shale
		34.00-40.20	<0.1	0.001	Ashfield Shale
SMW_WTP_ BH31	24.210	6.00-12.00	4	0.035	Ashfield Shale
		12.14-18.14	0.4	0.003	Ashfield Shale
		21.10-27.10	0.4	0.003	Ashfield Shale
		31.06-36.06	1	0.009	Ashfield Shale
		39.09-45.09	2	0.017	Ashfield Shale
		49.05-55.05	1	0.009	Hawkesbury Sandstone
SMW_WTP_ BH32	24.150	12.60-18.60	0.3	0.003	Ashfield Shale
		35.40-41.65	0.3	0.003	Hawkesbury Sandstone
		52.30-58.32	0.4	0.003	Hawkesbury Sandstone
SMW_WTP_ BH33	24.160	5.50-9.30	0.30	0.003	Ashfield Shale
		9.30-15.30	2.00	0.017	Ashfield Shale
		15-20.12	3.00	0.026	Ashfield Shale

2.6.12 Westmead Stub Tunnels

The stub tunnels at Westmead are to be road header mined in Ashfield Shale. There two new boreholes completed in this area post-tender, SMW_WTP_BH02 and SMW_BH700, which indicate packer test results of 0.006 m/day and 0.008 m/day for the former and 0.0009 m/day for the latter in Ashfield Shale. Groundwater table is interpreted to range from 32.5 m to 33.2 mAHD in SMW_WTP_BH02_w however, it is noted the monitoring data at this bore covers only a period of few weeks.

2.7 Sydney regional tunnel inflows

Measurements of water inflow have been made in several tunnels excavated in Hawkesbury Sandstone in the Sydney region and are summarised in Table 11 from Hewitt (2012) and the New M5 Hydrogeological Report (WestConnex, 2017).

Table 11: Summary of reported inflows to tunnels in Sydney

Tunnel Name	Type	Length (km)	Inflow L/s/km	Comments
Northside Storage	Water	20	0.9	6 m diameter Higher inflow below Middle Harbour at 8 L/s, required grouting
Epping to Chatswood	Rail	13	0.9	7.2 m diameter twin tunnels Higher inflows of 3 L/s
M5 East	Road	3.9	0.8 to 0.9	8 m diameter twin tunnels
Eastern Distributor	Road	1.7	1	12 m diameter double deck
Cross City Tunnel	Road	2.1	<3	8 m diameter twin tunnel
Lane Cove	Road	3.6	0.5	9 m diameter twin tunnel 1.7 L/s/km between Dec 2001 and mid 2004
MetroGrid*	Electrical	3.5	0.8	2 m diameter
EA/City East Cable Tunnels^	Cable	3.5	1.0	3.5 m diameter
M8 (formerly New M5^)	Road	9	1.0	12 m twin tunnels

*Source Jacobs (2016), ^ Yim et al (2021)

Inflows at localised features from published information include:

- Inflows in excess of 0.08 L/s flowing from bedding associated with a dyke behaving as an aquitard in the M5 East Tunnel (Golder, 2019);
- 3 L/sec in the Epping to Chatswood Railway (Yim et al 2021); and
- 4 L/s in the MetroGrid Cable tunnel presumed associated with dykes (Hewitt, 2012).

3 INFLOW AND DRAWDOWN ASSESSMENT

3.1 Design Criteria

Volume 4B (Particular Specification) Sydney Metro West Western Tunnelling Package Schedule C1 (Version 6 Sydney Metro, 2022b) provides the design criteria for the assessment on inflow and drawdown. A summary of the most relevant criteria is provided below.

It is noted the assessment does not estimate flow through the concrete lining of tunnels, caverns, adits and secant pile / diaphragm walls along the alignment against specifications. These features are assumed to be “tanked” at handover, with inflows limited to waterproofing ingress criteria. This assessment does, however, assess where there is the potential for areas or features or zones along the alignment of higher potential inflows (Attachment 1, Figure 1) to guide construction management. This assessment does not model for discrete features which may contribute to higher inflow. Waterproofing design is covered within the applicable design package returnable schedule.

3.1.1 Inflow

Watertightness (Volume 4B, Section 4.1.5)

There are criteria specified for watertightness, which relate to the seepage of groundwater through finished internal walls of infrastructure that is undrained at handover. This relates specifically to the design of the walls and is outside the scope of this assessment. This assessment does, however, highlight areas of potentially higher inflows, which can be used by the wall designers to understand if the surrounding geology will require treatment (such as grouting or installation of strip drains and membranes) so that the walls can meet the watertightness criteria.

Groundwater Control (Volume 4B, Section 4.1.7)

The tunnelling contractor must comply with the following for the drainage of assets:

- Running tunnels – undrained
- Cross-passages – undrained
- Cross-passages with sump – undrained
- Nozzle enlargements – undrained
- Cross-over caverns – undrained
- Station caverns– undrained
- Station excavations – drained
- Shaft excavations – drained
- Clyde Junction – undrained
- Portal structure – drained
- Clyde Dive Structure – drained
- Parramatta Station Excavation above the soil retention system toe level – undrained
- Parramatta Station Excavation below the soil retention system toe level – drained
- Rosehill Excavation – drained
- Rosehill Structure – undrained

As the spur tunnel access shaft was not detailed in the particular specification, it has been assumed to be drained up until lining of the spur tunnel at which time it will be backfilled such that it will be undrained.

We understand that the above drainage criteria relates to the condition of the infrastructure at ‘handover’ to Sydney Metro for subsequent construction on internal station features. Handover is expected to approximate a period of two years (at Westmead and Parramatta) after commencement of the construction works as indicated in the final tender program (dated 16 Feb 2022). The handover timeframes differ slightly for Clyde (2.6 years) and are detailed in Section 7.4.4 of the technical memorandum (SMWSTWTP-GLO-TJ550-GE-MEM-010101 Rev A.1).

Groundwater Seepage (Volume 4B, Section 4.1.8)

The groundwater seepage within each station excavation must not exceed:

- 15,000 Litres in any 24 hour period, measured over any square with an area of 10 m^2, at any and all locations within the sides and bases of the excavations; and
- The volumes identified below in any 24-hour period:
  - Westmead Station Excavation – 100,000 litres
  - Parramatta Station Excavation – 134,000 litres
  - Further, the groundwater seepage through the drained base slab of the Rosehill Structure must not exceed 45,000 litres in any 24-hour period. It is noted that the Rosehill Service Facility groundwater assessment is being managed by Aurecon for GLC and is not part of the scope of this assessment.
  - There is also a requirement to ensure groundwater seepage through the Clyde Dive structure does not exceed 5.0 ml per hour per m^2 of wall and base surfaces. As the Clyde Dive will be permanently drained and the permanent structure will be handed over to Sydney Metro by GLC, this criteria relates to the design of the permanent structure which is outside the scope of this investigation. The predicted total inflows for this assessment does, inform the design of the permanent infrastructure for this specification.

Additionally, there are criteria that relate to seepage through tanked infrastructure, which are an extension of the watertightness criteria outlined above. These criteria relate to the design of the internal wall linings of tanked infrastructure and are not considered to be applicable to this assessment. However, as noted above this assessment does highlight areas of potential higher inflows, which can be used by the wall designers to understand if the surrounding geology will require treatment (such as grouting, strip drains and membranes) so that the walls can meet the seepage criteria.

3.1.2 Drawdown

With regard to drawdown Section 4.1.5.1 Watertightness – General (h) states that the tunnelling contractor must design the Project Works to limit the effect on the groundwater management regime during construction, maintenance and operation, such that there is minimal adverse effect on the built environment.

For this assessment this was interpreted to represent the drawdown created at handover of drained and undrained infrastructure (approximately 2 to 3 years). Drawdowns under steady state conditions (100 years) for undrained infrastructure have not been assessed.

With regard to the interpretation of the results, the results have been compared against the conditions of approval relating to groundwater and the mitigation and management measures for groundwater included in the SMW Amendment Report (SMW, 2020c). The approval conditions and mitigation/management measures are presented in Table 12 and Table 13.

In regard to D122 the groundwater modelling completed and summarised herein and described in the attached technical memorandums is considered to be suitable to meet the groundwater modelling reporting requirements.

Table 12: Conditions of approval for groundwater

CoA	COA Description	Report reference
D121	"Make-good" provisions for groundwater users must be provided in the event of a material decline in water supply levels, quality or quantity from registered existing bores associated with groundwater changes from construction.	Attachment 2, Section 3.3.2
D122	The Proponent must submit a revised Groundwater Modelling Report in association with Stage 1 of the Critical State Significant Infrastructure (CSSI) to the Planning Secretary for information before bulk excavation at the relevant construction location. The Groundwater Modelling Report must include:	This report documents the groundwater modelling works and scope to meet this requirement.
	(a) For each construction site where excavation will be undertaken, cumulative (additive) impacts from nearby developments, parallel transport projects and nearby excavation associated with the CSSI.	Attachment 2, Section 3.3.2
	(b) Predicted incidental groundwater take (dewatering) including cumulative project effects.	Attachment 2, Section 3.3.2.
	(c) Potential impacts for all latter stages of the CSSI or detail and demonstrate why these later stages of the CSSI will not have lasting impacts to the groundwater system, ongoing groundwater incidental take and groundwater level drawdown effects.	Attachment 2, Section 3.3.2
	(d) Actions required after Stage 1 to minimise the risk of inflows (including in the event latter stages of the CSSI are delayed or do not progress) and a strategy for accounting for any water taken beyond the life of the operation of the CSSI.	Attachment 2, Section 3.3.2
	(e) Saltwater intrusion modelling analysis, from estuarine and saline groundwater in shale, into The Bays metro station site and other relevant metro station sties.	Attachment 2, Section 3.3.2 and Table 12.
	(f) A schematic of the conceptual hydrogeological model.	Attachment 2, Section 3.3.2

Table 13: Summary of potential groundwater impacts and management measures proposed in the EIS Amendment Report (SMW 2020c)

Reference	Impact	Mitigation measure	Location note 1	Applicability to WTP	Report reference
GW1	Loss of groundwater available to existing groundwater	Site inspection would be carried out on private domestic supply bore GW305646 to confirm the current viability of that bore. If found to be viable and predicted to be significantly impacted, make good measures would be implemented if a loss of yield were to occur.	BNS	Not applicable	Not applicable
GW2	Potential reduced baseflow to Toongabbie Creek, Domain Creek, A'Becketts Creek, Duck Creek, Haslams Creek, Powells Creek and the Mason Park wetlands, Bicentennial Park wetlands, Brickpit and Powells Creek Reserve. Requirements for baseline monitoring of hydrological attributes	A review of additional geotechnical and hydrogeology data would be undertaken to confirm the geological and groundwater conditions and determine, based on these local conditions, whether predicted groundwater drawdown from Stage 1 is likely to occur in the vicinity of these creeks. Where the additional data review shows local conditions and predicted groundwater drawdown are likely to cause surface water/groundwater interaction, then additional site investigations (in accordance with GW3) would be undertaken for those creeks or surface water bodies.	WMS, CSMF, SOPMS NSMS	Applicable	Attachment 2, Section 3.3.2
GW3	Potential reduced baseflow to Toongabbie Creek, Domain Creek, A'Becketts Creek, Duck Creek, Haslams Creek, Powells Creek and the Mason Park	Additional site investigations would be carried out at creeks or surface water bodies where the additional data review in GW2 shows there is a likely surface water / groundwater interaction. This would involve baseline monitoring of creek flows (streamflow gauging) prior to construction, and baseflow streamflow analysis to confirm the existing groundwater baseflow contribution to streamflow for each creek. Where a significant reduction in baseflow is predicted due to Stage 1, design responses would be	WMS, CSMF, SOPMS NSMS	Applicable	Attachment 2, Section 3.3.2

Reference	Impact	Mitigation measure	Location note 1	Applicability to WTP	Report reference
	wetlands, Bicentennial Park wetlands, Brickpit and Powells Creek Reserve. Requirements for baseline monitoring of hydrological attributes	implemented at station and shaft excavations to reduce potential baseflow loss.			
GW4	Requirements for baseline monitoring of hydrological attributes migration of contaminants in groundwater and reduction in beneficial uses of aquifers	Monitoring of groundwater levels and quality of the site area would occur before, during and after construction. This would also include monitoring of potential contaminants of concern. Groundwater level data would be regularly reviewed during and after construction by a qualified hydrogeologist. Groundwater monitoring data would be provided to the NSW Environment Protection Authority and Department of Planning, Industry and Environment and the Natural Resources Access Regulator for information.	WMS, PMS, CSMF, SSF, SOPMS NSMS, BNS, FDS, TBS	Applicable	Attachment 2, Section 3.3.2
GW5	Ground movement and settlement	A detailed geotechnical and hydrogeological model for Stage 1 would be developed and progressively updated during design and construction. The detailed geotechnical and hydrogeological model would include: – Assessment of the potential for damage to structures, services, basements and other sub-surface elements through settlement or strain – Predicted groundwater inflows, groundwater take and changes to groundwater levels including at nearby water supply works. – Where building damage risk is rated as moderate or higher (as per the CIRIA 1996 risk-based criteria), a structural assessment of the affected buildings/structures would be	Where required	Applicable	Attachment 2, Section 3.3.1.

Reference	Impact	Mitigation measure	Location note 1	Applicability to WTP	Report reference
		carried out and specific measures implemented to address the risk of damage. – Where a significant exceedance of target changes to groundwater levels are predicted at surrounding land uses and nearby water supply works, an appropriate groundwater monitoring program would be developed and implemented. The program would aim to confirm no adverse impacts on groundwater levels or to appropriately manage any impacts. Monitoring at any specific location would be subject to the status of the water supply work and agreement with the landowner.			
GW6	Ground movement and settlement	Condition surveys of buildings and structures in the vicinity of the tunnel and excavations would be carried out prior to the commencement of excavation at each site.	Where required	Applicable	Attachment 2, Section 3.3.2

Note 1 WMS: Westmead metro station; PMS: Parramatta metro station; CSMF: Clyde stabling and maintenance facility; SSF: Silverwater services facility; SOPMS: Sydney Olympic Park metro station; NSMS: North Strathfield metro station; BNS: Burwood North Station; FDS: Five Dock Station; TBS: The Bays Station; Metro rail tunnels: Metro rail tunnels not related to other sites (eg tunnel boring machine works); PSR: Power supply routes

While not presented as part of the conditions of approval or as mitigation measures the EIS groundwater impact assessment (Jacobs 2020) noted the following additional works were required to characterise the potential for impacts further:

- Additional site investigations at the Clyde Maintenance and Stabling Facility to assess the potential for exposure of acid sulphate soils and contamination migration risks on existing and future human and ecological receptors in and surrounding the station infrastructure.
- Additional site investigations at Parramatta Station to assess the potential for exposure of acid sulphate soils and contamination migrations risks on human and ecological receptors in and surrounding the station infrastructure.

These investigations and the associated potential for impacts are associated with the contamination discipline and are outside the scope of this investigation, although the simulated groundwater drawdowns within the HIR could be used to target those investigations to appropriate areas of potential impact.

It is also noted that impacts relating to Rosehill Service facility are being dealt with by Aurecon on behalf of GLC and are not discussed further here.

3.2 Assessment Methods

Three assessment different methods were adopted for assessing inflows to the WTP subsurface infrastructure. These are listed below:

- Numerical groundwater modelling for stations, nozzles, caverns, access shaft, spur tunnels, spur junction, portal and dive structure
- Analytical analysis for running tunnels and cross passages; and
- Qualitative review for the water conveyancing structures and the retention basin at Clyde Maintenance and Stabling Facility

A summary of the methods are provided in the following sections.

3.2.1 Assessment method for stations, nozzles, caverns, junction, access shafts dive and portal.

Three separate 3D numerical models were developed for assessing the groundwater inflows (and associated drawdown) to subsurface infrastructure. This included models at the following locations:

- Clyde (the portal, Clyde Dive, the spur tunnel access shaft, the spur tunnels and Clyde Junction)
- Westmead (station, caverns and nozzles)
- Parramatta (station and nozzles).

The assessment methods adopted for the listed infrastructure was broadly similar and is summarised below. Further detail on the assessment method adopted for the stations and the Clyde infrastructure is provided in Section 7.0 of the technical memorandums that have been developed for key infrastructure at Westmead (SMWSTWTP-GLO-WMD-SN650-GE-MEM-010102 Rev A), Parramatta (SMWSTWTP-GLO-PTA-SN600-GE-MEM-010102 Rev A) and Clyde (SMWSTWTP-GLO-TJ550-GE-MEM-010101 Rev A.1).

This assessment does not model discrete features which may contribute to higher inflow. Waterproofing design is covered within the applicable design package returnable schedule. Such features will need to be observed or prepared for during construction with appropriate mitigation measures.

3.2.1.1 Modelling objective

The modelling objective was to assess groundwater inflows and groundwater drawdown during construction and at handover (approximately 2 to 3 years).

3.2.1.2 Modelling approach

A 3D numerical modelling approach was adopted to allow a more quantifiable assessment of inflows, drawdown and potential hydrogeological effects compared to the analytical assessments adopted for the TS-GIR.

The 3D groundwater models were developed using the MODFLOW groundwater modelling code in the Groundwater Vistas platform (Version 7.11 build 15) or using the GMS (Version 10.6) modelling platform.

The modelling includes a staged approach with initial simplistic 3D modelling followed by increased refinement at each stage of the design process (3 stages). This initial phase (Stage 1) of groundwater modelling focused on the development of simplistic 'numerical calculator' models (Class 1 numerical models as defined in the Australian modelling guidance (Barnet et al, 2012))

To manage uncertainty in the Stage 1 approach, a realistic range of input parameters (using a high flow, likely flow and low flow inputs) was adopted for the assessment. This results in the development of low flow, likely flow and high flow models at each location.

The input parameters used in the models are outlined in the remainder of this section. Model calibration and input parameter refinement may be undertaken in later stages of design if needed to further characterise inflows and effects.

Model design

Broadly the models were designed to incorporate the relevant infrastructure within a model domain greater than 2.5 km by 2.5 km with the cell size ranging from 2 m at the infrastructure of concern up to 80 m at the periphery of the model domain. The model domains were rotated so the cells aligned with the infrastructure being assessed. This allowed more refinement in the representation of the infrastructure being modelled.

Each model generally comprised of seven layers representing key hydrogeological units (the shallow unconsolidated sediments, the Ashfield Shale and the Hawkesbury Sandstone (including the Mittagong Formation) and the depths to the base of key infrastructure.

The models did not include dykes and faults as the hydraulic characteristics of these features where represented in the range of bulk formation hydraulic conductivities assessed in the modelling.

The model boundary conditions included constant head boundaries at the model domain extent/periphery with groundwater elevations representative of maximum regional aquifer groundwater elevations at the modelled infrastructure. A small amount of rainfall recharge was then applied to the model to simulate recharge to groundwater from rainfall across the model domain. Recharge rates were generally less than 0.5 % (including evapotranspiration effects). The selected recharge rate was low compared to groundwater models developed for infrastructure projects in Sydney due to the uncalibrated approach adopted whereas other more detailed models, such as for the Western Harbour Tunnel project, are calibrated. While the low recharge was expected to overstate drawdown and understate inflow, the change was expected to be small relative to the ranges simulated for the low flow, likely flow and high flow models. On this basis the modelling was considered to be suitable for the intended purpose (inflow estimation and drawdown) and to provide suitable certainty in the estimation of drawdown and inflow.

Surface water features were not included in the modelling, however, intersection of the simulated drawdown with a surface water feature associated with infrastructure dewatering was considered to represent a potential for interaction to occur and therefore would be a basis for further assessment, if required. The absence of surface water features was partly responsible for low recharge rates in the model as there was no discharge to surface water features within the model domain.

Drains cells were used to represent the metro infrastructure being modelled with elevations in the model being set to the invert levels of the relevant infrastructure. The drain cell conductance was set a value that was non-limiting to drain cell inflow.

Initial base case steady state models were run to provide starting heads that could be used for transient runs. The transient runs were then developed to simulate inflows to infrastructure approximately halfway through construction (approximately 1 year for Westmead and Parramatta and 1.5 years for Clyde) and at the end of construction (approximately 2 years for Westmead and Parramatta to 2.6 years for Clyde) to handover, after which time, the temporary works are no longer the responsibility of GLC.

3.2.1.3 Input parameters

A summary of the dimensions and aquifer input parameters used for the modelling are presented in Table 14 to Table 16.

The assumptions and limitations adopted for the assessment are presented in the technical memorandums (particularly within Section 4.0) that have been developed for key infrastructure at Westmead (SMWSTWTP-GLO-WMD-SN650-GE-MEM-010102 Rev A), Parramatta (SMWSTWTP-GLO-PTA-SN600-GE-MEM-010102 Rev A) and Clyde (SMWSTWTP-GLO-TJ550-GE-MEM-010101 Rev A.1).

No allowance for climate change has been included in the interpretation of groundwater elevations due to the time at handover of the project being less than three years.

Table 14: Geometry of infrastructure and associated model layer

Location	Geometry and levels	Value	Model layer
Westmead (station excavation, nozzles, and caverns)	Width of station box	24 m	1 to 3
	Length of station box	150 m	1 to 3
	Width of cross-over cavern	20 m	3/4
	Length of cross-over cavern	235 m	3/4
	Width of nozzles (including western cavern)	25 m	2/3
	Length of nozzles (including western cavern)	40 m	2/3
	Station box and nozzle invert elevation	RL 0 m AHD	2/3
	Cross-over cavern invert elevation	RL -6 m AHD	4
Parramatta (station box and nozzles)	Width of station box	26 m	1 to 4
	Length of station box	192 m	1 to 4
	Excavation level	RL -18.6 m AHD	4
	Toe of D-wall at headwall	RL -20 m AHD	4
	Width of nozzles (combined)	26 m	4
	Length of nozzles	14 m	4
Clyde (portal, dive, spur tunnels, access shaft and junction)	Access shaft diameter (m)	30 m	1 to 4
	Portal and Clyde Dive – length x width	210 m x 16 m	1 to 2
	Spur Tunnels (x 2) length x width	350 m x 13 m	2 to 5
	Spur Junctions (x 2) length x width	146 m x 13 m	5
	Access shaft invert	-12 m AHD	1 to 4
	Portal and Clyde Dive invert	6 m AHD to – 5 m AHD	1 to 2
	Spur Tunnel invert	-5 m AHD to -24 m AHD	2 to 5
	Spur Junction invert	-29.2 m AHD	5

Table 15: Aquifer Parameters – Storage

Geology	Specific Storage (Ss 1/m)			Specific Yield (Sy)		
	High	Likely	Low	High	Likely	Low
Unconsolidated residual and alluvium	-	-	-	0.1	0.1	0.1
Ashfield Shale	1x10-05	5x10-06	1x10-06	0.05	0.01	0.005
Mittagong Formation	1x10-05	5x10-06	1x10-06	0.05	0.01	0.005
Hawkesbury Sandstone	1x10-05	5x10-06	1x10-06	0.05	0.01	0.005

3.2.1.4 Adopted input parameters

Dimensions

The dimensions adopted for station infrastructure are presented in Table 14 and are based on the dimensions presented in the GIR and on the WTP5.3B alignment. These may differ slightly to the infrastructure presented on the long section (Attachment 1) which is developed on the WTP4.3 alignment.

Groundwater intersection depth

The groundwater elevations recorded within all aquifers intersected at and around each station (see Section 2.6), and the inferred water table from Attachment 1, were reviewed in the adoption of model groundwater elevations. The groundwater elevation adopted was applied at the periphery of the model domains as constant head boundaries, which were subsequently applied to all layers of the models. The groundwater elevations adopted are provided in the technical memorandums that have been developed for key infrastructure at Westmead (SMWSTWTP-GLO-WMD-SN650-GE-MEM-010102 Rev A), Parramatta (SMWSTWTP-GLO-PTA-SN600-GE-MEM-010102 Rev A) and Clyde (SMWSTWTP-GLO-TJ550-GE-MEM-010101 Rev A.1). For the Westmead model lower groundwater elevations for Ashfield Shales (assuming perched groundwater in residual soils) were adopted to account for the expected fall in groundwater elevations with topography away from the station.

Hydraulic conductivity

Hydraulic conductivity values have been estimated from the available packer test and slug test data presented in Section 2.6. The hydraulic conductivities adopted are provided Table 16. For each model the assessment has focused on the packer testing data available at that location being assessed. Generally, the 90th percentile value has been used as the upper case, the arithmetic mean as the most likely case and the 20th percentile as the lower case. This has been cross-checked against adopted values for groundwater modelling works across the Sydney region and upscaled or downscaled based on professional experience. Where data was scarce, statistics for the entire alignment data set have been adopted.

The 95th percentile hydraulic conductivity adopted for the high flow model at Parramatta to represent bulk formation hydraulic conductivities is skewed by localised packer testing results at SMW_WTPSite01_BH01, SMW_WTPSite01_BH02 and SMW_WTPSite01_BH03 located at the eastern end of the station box at 25 Smith Street. This may result in an overstatement of the bulk formation permeability, which is used in the model, and which result in inflow estimates that have a higher probability of being beyond a reasonable worst-case condition.

The representativeness of the data from these new test locations is uncertain, given the proximity of the bores to each other and that the water pressure testing appears to have been completed when all locations were open and drilled to their target depths. This may have resulted in packer testing results being influenced by the neighbouring borehole from fractures that may have been interconnected by the holes allowing water in one bore to transmit to the other, producing very high permeability test results. It is noted the results are within the same order of magnitude as other permeability test results from the Parramatta Square investigations. Therefore, there is higher uncertainty in the high flow model outputs for Parramatta.

Table 16: Aquifer parameters – hydraulic conductivity

Location	Materials	High flow K (m/day)	Likely flow K (m/day)	Low flow K (m/day)	Kv/kh	Basis of values
Westmead (station excavation, nozzles, and caverns)	Unconsolidated alluvium and residual soils	0.1	0.1	0.1	0.1	Regional (SMW, 2020b)
	Ashfield Shale	0.026	0.01	0.0009	0.1	Local Data: 20th%, mean and 90th %
	Mittagong Formation and Hawkesbury Sandstone	0.072	0.026	0.0022	0.1	Local Data: 20th%, mean and 90th
Parramatta (station box and nozzles)	Unconsolidated alluvium and residual soils	0.1	0.1	0.1	0.1	Regional (SMW, 2020b)
	Ashfield Shale	0.26	0.076	0.00044	0.1	Local Data: 20th%, 90th % and 95th %
	Mittagong Formation and Hawkesbury Sandstone	0.43	0.22	0.001	0.1	Local Data: 20th%, 90th % and 95th %
Clyde (portal, dive, spur tunnels, access shaft and junction)	Unconsolidated alluvium and residual soils	2.0	0.1	0.1	0.1	Regional (SMW, 2020b)
	Ashfield Shale	0.035	0.016	0.004	0.1	Local Data: 20th%, mean or 90th%, 95th%
	Mittagong Formation and	0.1	0.034	0.001	0.1	Local Data: 20th%, mean or 90th%, 95th%

Location	Materials	High flow K (m/day)	Likely flow K (m/day)	Low flow K (m/day)	Kv/kh	Basis of values
	Hawkesbury Sandstone					

Time

Two times were adopted for assessing groundwater inflows to station box infrastructure:

- Maximum expected inflows after 1 year (Westmead and Parramatta) to 1.5 years (Clyde) at excavation completion, to inform construction water management and
- Handover inflows which are required to meet the Volume 4B inflow specifications (2 years at Westmead and Parramatta to 2.6 years at Clyde).

The times adopted to represent 'excavation' completion and 'handover' were based on the information presented in the final tender schedule (Schedule A23 – GALC, 2022).

3.2.1.5 Modelling assumptions

The assumption and limitations associated with modelling approach outlined above are provided in the preceding text and within the attached technical memorandums, particularly Section 4.

3.2.2 Assessment method for the tunnel boring machine and cross-passages

Goodman et al (1965) adjusted as per Heuer (2005) has been used to assess inflows to the running tunnels and cross-passages. The Goodman equation is as follows.

$$Q = 2 \,.\, K \frac{\Delta h}{\ln(\frac{2 \,.\, \Delta h}{r})}$$

where:

- Q = tunnel inflow (m3/s)
- K = hydraulic conductivity (m/s)
- Dh = distance between the centre of the tunnel and the groundwater table (m)
- r = tunnel radius (m)

Heuer (2005) noted that the Goodman et al (1965) equation generally overstated tunnel inflows by a factor of eight and therefore proposed that the Goodman equation results be divided by eight.

3.2.2.1 Adopted Input Parameters

Dimensions

The running tunnel assumes an 8 m external diameter (i.e. 4 m radius) of the working face of the TBM with 17 m of open tunnel (not segmentally lined) behind the working face. For the cross passages, to facilitate ease of analytical modelling, a cylinder is conceptualised to run between (and perpendicular to) the metro tubes, with the same diameter as the running tunnel and a length of 10 m. This is considered to be conservative with regard to open area of the designed cross-passages.

Groundwater Intersection Depth

The shallowest groundwater elevations recorded adjacent to the alignment (see Section 2.6) were propagated onto the alignment (using the change in surface elevations) to estimate a groundwater table elevation. Groundwater elevations were then interpolated between the available groundwater data along the alignment using professional judgement of the nature of groundwater elevation changes with topography. The data was subsequently digitised for inclusion in the tunnel and cross-passage inflow equation. The interpreted groundwater elevations used in the model are presented in Attachment 1.

The invert elevation of the relevant section of the TBM or cross-passage was then subtracted from the interpreted groundwater elevation to obtain a saturated aquifer thickness above the tunnel invert and cross-passage to give the aquifer head used in the equations.

Hydraulic Conductivity

The hydraulic conductivity values for the tunnel boring machine and cross-passage inflow have been established using the arithmetic mean of packer testing data along the alignment, which is considered a reasonable 'likely case' with 0.05 m/day for Hawkesbury Sandstone and 0.02 m/day for Ashfield Shale. The adopted hydraulic conductivities/permeabilities have then been assessed relative to the potential presence of structural features and the result upscaled accordingly based on packer test results in areas along the alignment where tunnel scale structures are present. These locations are highlighted in Attachment 1. The estimated inflows at these locations may be subject to change in position and inflow rates if further data resolves their presence/absence or the location of the structural intersection more precisely. It should be noted that the simulated tunnel and cross-passage inflows adopt 'likely case' bulk formation properties, which do not account for localised high inflows associated with rock structures that may be intersected.

Time

The TBM was considered to migrate at a rate of 20 metres per day, with the lined of tunnel running 17 m behind the open TBM face. It was assumed that cross-passages, which are road header excavated, would take two months per cross-passage to complete. This period of construction was considered to be of little relevance to the results, however, because the model assumes the cross passages excavation are wished in place (instantaneously excavated). This is considered to represent a maximum inflow conceptual condition.

Other Construction Considerations

It was assumed that the cross-passages were installed after the TBM has passed and that the inflows associated with each piece of infrastructure are mutually exclusive. If they are completed together addition of the simulated inflows at each given location would be a reasonable approach, but would tend to overstate the total inflow. Cross-passage inflows will gradually increase to those simulated until the excavation is completely open. They will then subside gradually with the time the cross-passage is open.

It was also assumed that each metro tube was installed separately to the other tube and that the inflows associated with each piece of infrastructure are mutually exclusive. If they are completed together, addition of the simulated inflows at each given location would be a reasonable approach, but would tend to overstate the total inflow.

Due to the short-term construction periods for the cross-passages and the migrating nature of the TBM, the expected drawdowns associated with tunnelling and cross-passage installation would be small and as such drawdown associated with these items of infrastructure were considered unlikely to generate ground settlement or environmental impacts. This was similar to the approach adopted for the Environmental Impact Statement (EIS) (Jacobs, 2020).

3.2.3 Assessment Method for the MSF Water Conveyancing Structures.

A qualitative review has been adopted for the water conveyancing structures at the MSF including the water retention basin, which has included comparison of the available groundwater elevations against the estimated base of the proposed infrastructure.

Observed groundwater elevations above the base of the retention basin (estimated to be a minimum of 2 m AHD) were considered to represent a potential groundwater inflow issue for detailed design and construction.

Based on the tidal nature of A'Becketts Creek and Duck Creek the re-alignment works to install culverts are expected to result in excavations extending below the surrounding groundwater water table. Consideration of which construction methods will enable management of this is required.

Preliminary construction options are subsequently proposed in Section 4 to manage the groundwater inflow.

3.3 Assessment findings

The results of the analytical modelling completed are presented in the following sections.

3.3.1 Inflows

3.3.1.1 Station, Cavern, Nozzle, Portal, Dive, Spur Tunnel and Junction infrastructure

The estimated inflows to the Westmead and Parramatta infrastructure is presented in Table 17, while the estimated inflows to the Clyde infrastructure are presented Table 18.

Table 17: Estimated inflows in m^3/day for Westmead and Parramatta

Location	Infrastructure	Scenario	End Year 1 (m^3/day)	End Year 2 (m^3/day)
Westmead	Station box	high flow	123	49
		likely flow	94	46.6
		low flow	9.5	5.8
	Eastern cross-over cavern	high flow	-	135
		likely flow	-	104
		low flow	-	8.9
	Nozzles and western Cavern	High flow	-	12.5
		Likely flow	-	11
		Low flow	-	0.8
Parramatta	Station box	high flow	275	241
		likely flow	142	126
		low flow	0.7	0.7
	Nozzles	high flow	-	170
		likely flow	-	90
		low flow	-	0.5

Notes:

Crossover cavern and nozzle inflows represent inflow immediately prior to sealing the cavern. It assumes the cavern is entirely open after one year of construction. A progressive excavation and lining process would reduce inflow broadly proportionate to the length of open area.
Red text represents values that exceed the inflow specification which is 100 m^3/day for Westmead station box and 134 m^3/day for Parramatta station box..
Data is not available for the nozzles and caverns at Year 1 as construction is modelled not to have commenced.
The inflows for the nozzles and caverns are not subject to inflow criteria as they will be undrained at handover.

Table 18: Estimated inflows in m^3/day for Clyde

Infrastructure	Scenario	0.25 Years (m^3/day)	1.25 Years (m^3/day)	2.6 Years (m^3/day)
Portal and Clyde Dive	high flow	-	52	39
	likely flow	-	42	32
	low flow	-	8	6
Clyde access shaft	high flow	45	24	30
	likely flow	18	12	13
	low flow	1.0	0.7	0.7
Spur tunnels	high flow	0	456	0
	likely flow	0	238	0
	low flow	0	11	0.1
Clyde Junction	high flow	0	49	231
	likely flow	0	29	136
	low flow	0	1.3	6.7

Notes:
- inflows have not been provided where the 'wished in place' design of the model has resulted in an inaccurate estimation of inflow
Zero values represent pre-construction conditions.

A summary of the results is provided below.

Westmead

The range in estimated inflow between the low flow, likely flow and high flow cases is broadly an order of magnitude, which reflects the uncertainty in the currently available data and modelling approach, however, it is likely that the inflows will be within this range.

It is expected that these ranges would tighten with model design improvements and model calibration if required for subsequent stages of modelling.

The high flow modelling results should be treated with caution as they adopt parameters that assume very permeable ground conditions are more frequent than currently characterised.

The mined cross-over cavern does not have any associated inflow criteria limits.

Station Box

Inflows to Westmead Station box at the end of year 1 construction could be higher than the inflow criteria of 100 m^3/day, when considering high flow scenarios, however, with targeted grouting of localised seepage to meet the localised seepage criteria it is expected that inflows would likely be lower than the criteria.

At handover the inflows are expected to be below the inflow criteria (49 m^3/day in the high flow scenario).

The reduction in inflow over time is due to gradual storage loss in the surrounding media over the construction period. As the station box is 'wished into place' the groundwater inflows during excavation would gradually increase to those simulated at year 1 when the station excavation reaches depth.

The flow into the station box at the end of construction will be subdued by dewatering associated with cross-over cavern and nozzle enlargements. It is expected that inflows to the station box could increase after the completion of construction but are not expected to exceed the inflow criteria (as simulated up to approximately 100 years after construction).

Estimated inflows at Year 1 are higher than estimated for the TS-HIR. The higher inflow simulated for this assessment is expected to be due the difference between the previous and current assessment in the timing of construction of the nozzles, caverns and stations and the relative influence they have on inflows 1 year into construction and at handover.

There is the potential for localised inflows on the excavation walls and floor to exceed the localised seepage criteria (i.e. 15,000 Litres in any 24-hour period, measured over any square with an area of 10 m^2) and these areas may require localised treatment measures such as targeted grouting.

Nozzle Enlargements and Western Cavern

The nozzle enlargements and cavern excavation are expected to have inflows ranging between 0.8 m^3/day and 12.5 m^3/day. The estimated flows assume the nozzles excavation is completely open before lining occurs. A progressive excavation and lining process would reduce inflow broadly proportionate to the length of open area.

A revised construction process (excavation and lining) could be investigated in subsequent revisions of groundwater modelling.

Eastern Cross-over Cavern

The eastern cross-over cavern is expected to have inflows ranging between 8.9 m^3/day and 135 m^3/day. The estimated flows assume the cavern is completely open before lining occurs. A progressive excavation and lining process would reduce inflow broadly proportionate to the length of open area.

A revised construction process (excavation and lining) could be investigated in subsequent revisions of the modelling.

Parramatta

The range in estimated inflow between the low flow, likely flow and high flow cases is several orders of magnitude, which reflects the uncertainty in the currently available data and the uncertainty in the simple conceptualisation adopted for this modelling. It is expected that this range will tighten with model design improvements and model calibration in following modelling stages.

Station Box

Inflows to Parramatta Station box one year into construction are expected to be higher than the inflow criteria of 134 m^3/day, however at handover they are likely to be below the inflow criteria (126 m^3/day in likely flow scenario). Due to the high degree of faulting, there is potential that inflows will be notably higher than the inflow criteria as indicated by the high flow modelling result (241 m^3/day). The reduction in inflow overtime is due to gradual storage loss in the surrounding media over the construction period.

The predicted high case flows are higher than those estimated in the TS-HIR due to the increased depth of the station box, new packer test data used for this assessment that has intersected more permeable features and the incorporation of vertical groundwater flow for this assessment, which is potentially a significant groundwater flow pathway at this location. The potential for high inflows will need to be considered in construction water drainage/management requirements. As noted in 3.2.1.4, there is the potential for the high flow case to be overstating a reasonable worst case condition.

The modelled inflows are sensitive to the vertical hydraulic conductivities (kh/kv) adopted for the modelling. A value of 0.1 is considered to be a representative value in the absence of site specific testing. The GIR has assessed that there is significant vertical faulting in this area, which has produced vertical displacement and which persist to the base of the residual and alluvial material. Therefore, there is the potential, due to the vertical continuation of faulting, that kh/kv could be higher than 0.1. Based on the modelling results, there is a reasonable potential that the base of the excavation would require treatment to meet the inflow specification.

Nozzle Enlargements

The nozzle enlargements are expected to have inflows ranging between 0.5 m^3/day and 170 m^3/day. The estimated flows assume the nozzles excavation is completely open before lining occurs. A progressive excavation and lining process would reduce inflow broadly proportionate to the length of open area.

Permeability of the rock mass at the nozzles reflects the anomalously high packer test results which may be influenced by their proximity to faults and dykes. There are no specification inflow criteria for the nozzles. It should be noted that the TS-HIR including inflow from the nozzles within the station box excavation assessment.

The predicted inflows at handover will need to be considered in the construction water drainage/management requirements.

A revised construction process (excavation and lining) could be investigated in subsequent revisions of the modelling.

Clyde

The range in estimated inflow between the low flow, likely flow and high flow cases is broadly one to two orders of magnitude, which reflects the uncertainty in the currently available data and modelling approach, however, there is high confidence that the inflows will be within this range.

It is expected that this range would tighten with model design improvements and model calibration if required for following stages of modelling.

Clyde Dive and the Portal

Combined flows for the Portal and Clyde Dive are expected to be between 8 m^3/day and 52 m^3/day approximately 1 year into construction. The inflow is expected to fall over time to between 6 m^3/day and 39 m^3/day at the completion of construction (at handover).

The estimated inflows are higher than assessed in the TS-HIR, which is expected to be primarily due to the relocation of the portal and dive approximately 100 m to the south where it will intersect alluvials as it descends to connect with the spur tunnels.

There is the potential for localised inflows on the walls and base of the Clyde Dive to exceed the localised seepage criteria (i.e. 5.0 ml/m^2/hr). These areas may require localised treatment measures such as targeted grouting, although it is expected that the permanent portal and dive drainage system (such as strip drains) would be designed to prevent seepage through the walls and base in excess of the criteria from occurring.

Clyde Access Shaft

Flows into the access shaft are expected to be less than 45 m^3/day over the lifetime of the project.

Spur Tunnels

Flows into the spur tunnels have the potential to be up to 456 m^3/day but are likely to be in the order of 238 m^3/day approximately 1 year into construction. The inflows would gradually fall to zero as the spur tunnels are lined before handover. It is noted that the model assumes that the tunnel lining proceeds six months behind the excavation of the spur tunnels. This means the model assumes that approximately 800 m of tunnel is open and unlined at any one time during the construction process. It is noted that this is more than what was assumed to be open in the TS-HIR, which was 14 m. As such it is understandable that the simulated inflows are significantly higher. Regardless, the simulated inflow rates even under likely case conditions are approximately 3.4 L/s/km which is at the high end of observed flows into tunnels in Sydney presented in Section 2.7, although the flows presented in Section 2.7 could be understated by already treated (e.g. grouted) operational conditions.

Clyde Junction

Inflows into Clyde Junction once construction commences are expected to be subdued due to dewatering for the construction of the spur tunnels. Once the spur tunnels are lined the inflows to the Junction excavation are expected to increase to be between 7 m^3/day and 231 m^3/day immediately prior to lining of the excavation. It is noted that these inflows assume the excavation is completed in its entirety before lining begins. The flows are interpreted to be less than what was simulated for the TS-HIR, although the TS-HIR inflows sit within the range of the current estimates. It should be noted that the TS-HIR analytical assessment assumed the Clyde Junction excavation would be open for two years.

3.3.1.2 Running Tunnels

Tunnel Boring Machine (TBM)

The estimated inflows at the tunnel boring machine face are presented on the hydrogeological long section in Attachment 1. The inflows rely on bulk formation hydraulic conductivities. Localised high hydraulic conductivity rock features may be encountered that result in higher incidental inflow.

The results are for a single tube progression only and assume a 17 m open section of tunnel behind the working face of the TBM (with an 8 m external diameter). If the tubes are bored side by side the result should be doubled. If one tube is leading the estimated inflows for the other may be reduced.

The results are summarised below:

- Mean inflow rates approximate 9 m^3/day for open (unlined) 17 m assumed section of tunnel prior to placement of the permanent lining, as the TBM progresses.
- The highest and lowest rates estimated are 50 m^3/day and 2 m^3/day respectively.
- Locations of higher estimated inflows are listed below:
  - Chainage 32.170 to 32.070 km (where deep tunnel excavation intersects fault) where flows are estimated to approximate 29 m^3/day
  - Chainage 22.670 to 22.580 km (the western exit from Parramatta Station which may be affected by subvertical connection to overlying alluvial sediments and crowning in low cover rock with sub-horizontal shearing) where flows are estimated to approximate 12 m^3/day
  - Chainage 22.370 to 22.300 km (the eastern exit from Parramatta Station where faulting and dyke intrusion are anticipated) where flows are estimated to approximate 50 m^3/day
  - Chainage 20.740 to 20.510 km (where deep tunnel excavation intersects three faults and a dyke) where flows are estimated to approximate up to 18 m^3/day.

Attachment 1 indicates that these locations are broadly associated with inferred structural and geomorphological intersections. The estimated inflows at these locations may be subject to change in position and inflow rates if further data resolves their presence/absence or locates the structural intersection more precisely. It should be noted that the simulated tunnel and cross-passage inflows are based on 'likely case' bulk formation properties and do not account for localised high inflows associated with rock defects that may be intersected.

The estimated mean inflows approximate 5 to 6 L/s/km, which are approximately two to three times higher than those normally be simulated in groundwater models (< 2 L/s/km) for tunnels across Sydney, suggesting the hydraulic conductivities adopted for this assessment are at the high end.

Provision should be made for probing and grouting ahead of the TBM where inflows are estimated to exceed the capacity of construction operation equipment and where potential rock defects have been inferred (as presented in Attachment 1 and summarised below).

Cross-Passages

The estimated inflows to the 34 cross passages (XP) are presented in Attachment 1. It should be noted that the location of the cross passages for the WTP 5.3B alignment have not been provided and it is assumed they will be similar to the previous locations with a slight adjustment to XP72 to the eastern side of the RSF. As for the TBM, inflows rely on bulk formation hydraulic conductivities properties. Localised high hydraulic conductivity rock features may be encountered that result in higher incidental inflow. The interpreted inflows are, however, considered to be a reasonable upper-end estimate.

Cross passages are assumed to have an open length of 8 m (along the tunnel wall) that is 10 m wide (between tubes). The results are summarised below:

- Mean inflow rates approximate 4 m^3/day
- The highest and lowest rates are 8 m^3/day and 1 m^3/day respectively
- Locations of higher estimated inflows are listed below:
  - XP75 (local inflexion point in tunnel alignment) where flows are estimated to approximate 8 m^3/day
  - XP74 (local inflexion point in tunnel alignment) where flows are estimated to approximate 6 m^3/day
- Provision should be made for probing and grouting ahead of the TBM where inflows are estimated to exceed the capacity of construction operation equipment and where potential rock defects have been inferred (as presented in Attachment 1 and summarised below).

Westmead Stub Tunnels

As for the TBM the road header turnback estimated inflows rely on bulk formation hydraulic conductivity properties. The estimated inflow is presented in Attachment 1. The result is for a single heading progression and assumes 4 m length of the cavern segment open behind the working face of the road header. If the stub tunnels are excavated side by side the result should be doubled. If one heading is leading the estimated inflows for the other may be reduced. Inflow is estimated at 3 m^3/day for the 90 m length.

Uncertainty in the Estimation of Inflows

The Project hydrogeological risk has been assessed to the nearest 100 m of chainage, as shown on Attachment 1 as a red (high), orange (moderate) or green (low) bar and is summarised below:

- Ch15.160 to Ch15.520 km – low
- Ch15.520 to Ch17.300 km – moderate, due to the tunnel passing along the contact between shale and the underlying Hawkesbury Sandstone and hydrogeological features at Haslams Creek (Ch16.770 to Ch16.080)
- Ch17.300 to Ch18.280 km – low

- Ch18.280 to Ch20.180 km – moderate due to tunnel passing along the contact between shale and the underlying Hawkesbury Sandstone and hydrogeological features including the inferred dyke in near the Rose Hill Service Facility location
- Ch20.180 to Ch20.480 km – low
- Ch20.480 to Ch20.0740 km – high, due to hydrogeological features (dyke and faults) and interactions with existing infrastructure elements
- Ch20.740 to Ch22.280 km – low
- Ch22.280 to Ch22.400 km – high, due to hydrogeological features (including Parramatta Dyke) and deep alluvial material
- Ch22.400 to Ch22.620 km – moderate, due to hydrogeological features and deep alluvial material
- Ch22.620 to Ch23.640 km – low
- Ch23.640 to Ch23.920 km – moderate, due to passing of the tunnel from the shale to the underlying Hawkesbury Sandstone and hydrogeological features (including Domain Creek)
- Ch23.92 to 24.350 km – low

Attachment 1 indicates that the locations highlighted as orange and red sections are broadly associated with inferred structural and geomorphological intersections (e.g. faults, dykes and transition between lithological units) and zones where there is limited investigation information. The estimated inflows at these locations may be subject to change in position and inflow rates if further data resolves their presence/absence or locates structural intersection more precisely.

TBM tunnelling is proposed from Rosehill to the Sydney Olympic Park Station initially. The TBMs will then be relocated to Rosehill and tunnelling to proceed westwards to Westmead. Tunnelling towards Parramatta is more likely to experience conditions of delamination, opening of fractures and therefore a greater potential for higher initial inflows.

Attachment 1 Figure 1 indicates that these locations are broadly associated with inferred structural and geomorphological intersections. The estimated inflows at these locations may be subject to change in position and inflow rates, if further data resolves their presence/absence or locates the structural intersection more precisely. The position of faulting is commonly inferred from limited information and is considered indicative in many instances.

3.3.1.3 Clyde Maintenance and Stabling Facility Water Conveyancing Structures

The available groundwater information indicates that the retention basin is likely to intersect groundwater located predominantly within clay sediments by more than to 2 m (as indicated by groundwater elevations in SMW_ENV083, which were greater than 4 m AHD during sampling on 24/01/2020 and 13/02/2020). This will result in groundwater flowing into the excavation during construction.

3.3.2 Groundwater Drawdown

The drawdown can be used to inform the settlement assessment, the contamination assessment (including acid sulphate soils) and to assess consistency with the impact assessment completed for the EIS. As noted in section 3.2.1, the simulated drawdown may be overstated by the nature of the modelling design adopted for the Stage 1 assessment. This means the high flow case result for all models is unlikely to occur. Further modelling refinement for future stages of design would explore this further.

Drawdown associated with the TBM works and cross-passages are expected to be highly localised and of short duration and as such are not expected to generate impacts. On this basis they have not been assessed for drawdown.

A comparison of key WTP infrastructure associated with the Stage 1 temporary works against the conditions of approval are provided in Table 2.1 of Attachment 2. A comparison of key WTP infrastructure associated with the Stage 1 temporary works against the SMW Amendment Report are provided in Table 2.2 of Attachment 2.

A summary of the key finding from the comparison is included in the discussion for each location below.

3.3.2.1 Westmead

The estimated range in drawdown at handover (i.e. 2 years into construction) at Westmead are presented in Figure 7 and Figure 8. A comparison of the simulated drawdown against the conditions of approval and mitigation measures for groundwater is provided in Attachment 2.

The simulated drawdown results are summarised below:

- The limit of drawdown influence, considered by the 0.5 m contour, is expected to approximate 500 m but could extend up to 1,070 m from the edge of Westmead Station box.
- The simulated drawdown is generally less than that simulated for the EIS, noting the influence of Parramatta River seepage in the EIS prevented migration of the zone of drawdown further to the north.
- A search of the Bureau of Meteorology (BOM) groundwater explorer (date accessed 19 March 2022) indicates that, while there is a registered water supply bore (GW108373) within the potential radius of drawdown, there are currently no registered groundwater bores within the 2 m drawdown contour presented in the figures. A 2 m drawdown is defined in the NSW aquifer interference policy (DPI-Water, 2012) as representative of a more than minimal effect.
- The simulated zone of drawdown associated with the temporary works is expected to intersect Domain Creek indicating that further assessment is warranted in accordance with mitigation measure GW2. The surface water features in Domain Creek are elevated by weirs and earth dams with spill ways, which suggest water levels in the creek may be perched above and preferentially recharging the shallow groundwater system in areas potentially impacted by station dewatering. Additional monitoring is proposed in accordance with mitigation measure GW2 to assess the interaction further. While the zone of drawdown is simulated to intersect Toongabbie Creek and Parramatta River the duration and magnitude are expected to be insufficient to create adverse impacts to flows and reliant terrestrial ecosystem health beyond what would be experienced under normal climatic conditions.
- Surface water impacts are expected to be temporary and minor relative to the follow-on works which are expected to be assessed by Sydney Metro and follow-on contractors.
- A search of the NSW major projects planning portal, indicates that there are a number of new developments at Westmead Hospital to the north and in the surrounding area. These developments may include basements that require dewatering and which were not assessed in the EIS. While there is likely to be cumulative drawdown effects they are expected to be subdued by a reduction in seepage as the drawdown cone associated with the station intersects each basement. Cumulative impacts associated with the temporary works are expected to be of a short-term nature and within the bounds of background fluctuation. Further, the effects will be minor relative to follow on works, which are expected to be assessed by Sydney Metro and follow-on contractors.

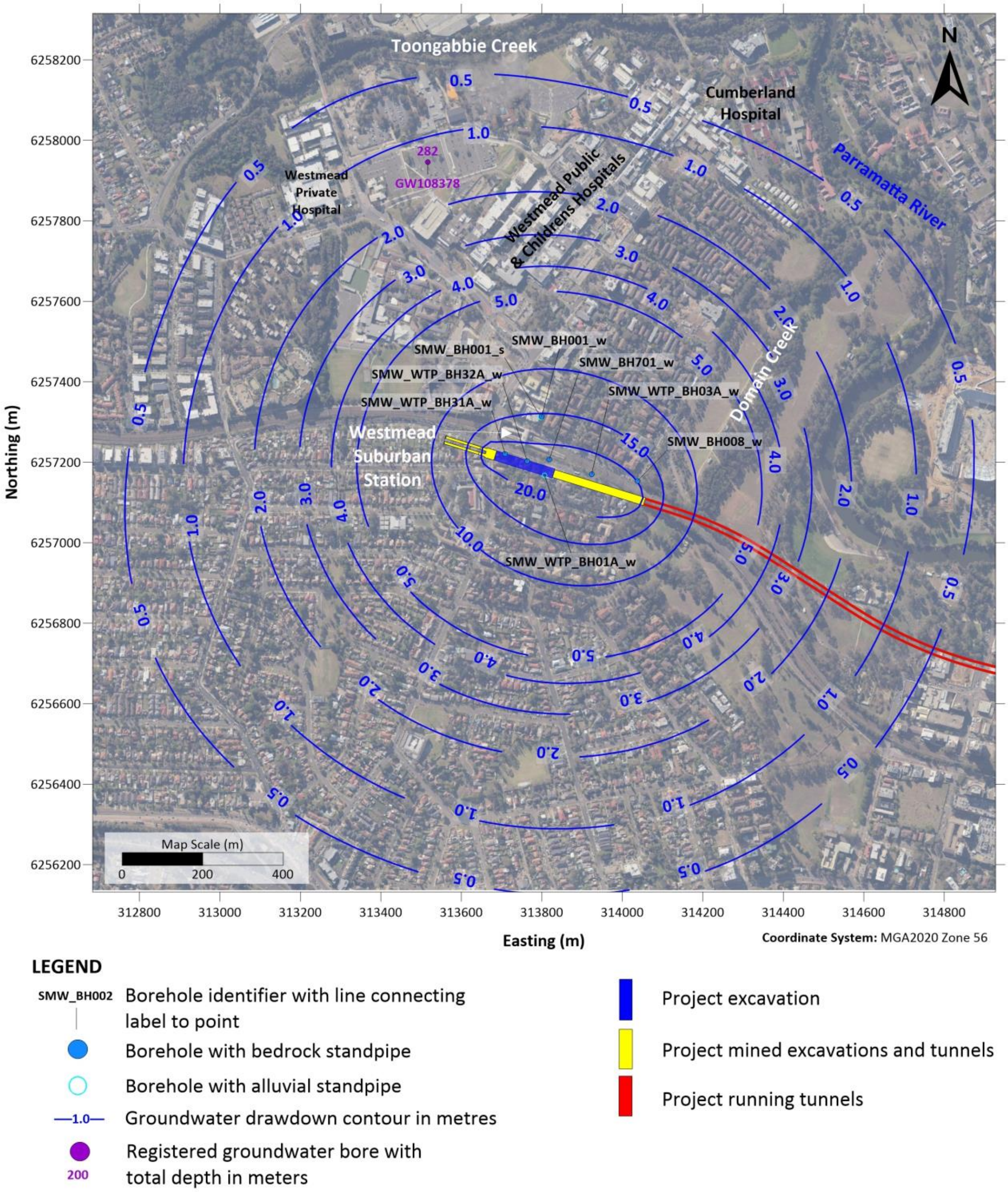


Figure 7: Drawdown (m) in the Ashfield Shale (assumed water table aquifer) for the high flow case model (at handover – 2 years after commencement of construction) at Westmead

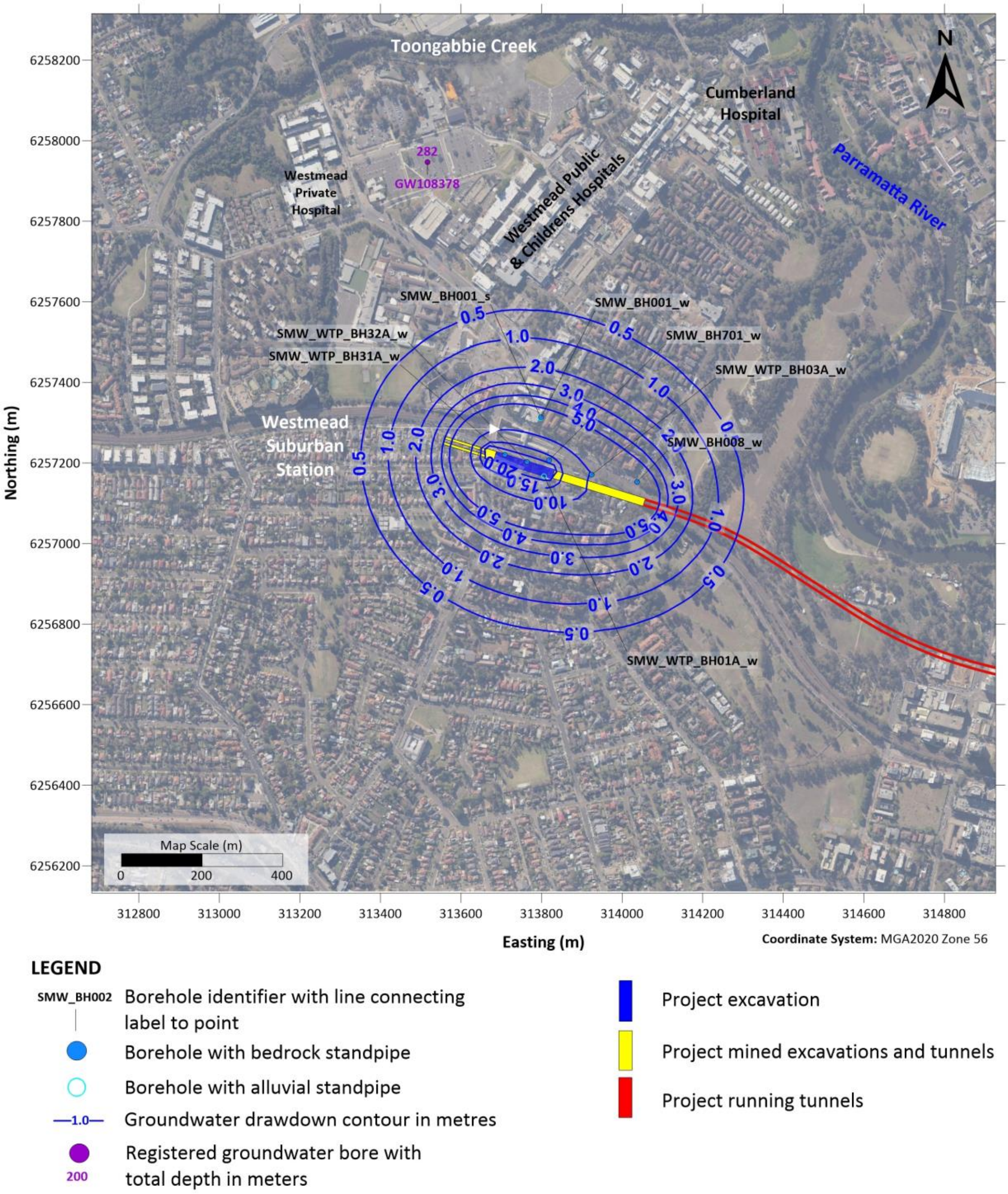


Figure 8: Drawdown (m) in the Ashfield Shale (assumed water table aquifer) for the likely flow case model (at handover – 2 years after commencement of construction) at Westmead

3.3.2.2 Parramatta

Only the drawdown associated with inflow criteria meeting the specification have been presented herein as alternative higher inflows scenarios are currently not considered to be within specification. The high flow case with treatment is shown on Figure 9 and Figure 10 and the likely case without treatment is shown on Figure 11.

It should be noted that the drawdown estimation does not account for existing drawdown effects associated with dewatering of surrounding basements which will reduce the drawdown effects simulated by the model. These existing dewatering effects could potentially result in a reduced or negligible impact of the station box temporary drainage on shallow groundwater levels in the Parramatta Sands. There is insufficient information currently available to confidently assess how much the reduction in impacts over those presented could be. Further site investigations including pumping tests designed and completed in accordance with relevant standards (e.g. AS 2368) would inform this data gap.

Figure 9 and Figure 10 present the results from the high flow model (with grout treatment to reduce inflows to specification) at one year and two years into construction respectively. The nozzles excavation area has also been grouted in this model as the effects of dewatering of the nozzles on drawdown are simulated to be large if there is no treatment. Other measures such as construction techniques that minimise open area could also be investigated as a solution for minimising inflow and drawdown associated with nozzle construction.

The modelling suggests that groundwater drawdown in the unconsolidated material will approximate 1.7 m at the edge of the station box after 1 year, with the 1 m contour extending up to 130 m from the edge of the excavation and the 0.5 m contour extending up to 170 m from the edge of the excavation. At the completion of temporary works (2 years) the drawdown at the box approximates 3.0 m with the 1 m contour extending up to 240 m from the edge of the excavation and the 0.5 m contour extending up to 430 m from the edge of the excavation (and intersecting with Parramatta River).

Figure 11 presents the results from the high flow model (with no grout treatment). The inflows rates are similar to the high flow model with treatment and the extent of drawdown is similar, however, the drawdown contours closest to station box are greater than the treated case. This is due to the lower hydraulic conductivities adopted relative to the high flow model.

It is expected that the extent of the drawdown will be less than due to:

- The existing basement dewatering in the CBD may have existing drawdown effects, which are not incorporated within this model and the associated estimate of drawdown..
- Recharge from the Parramatta River, which is not incorporated into this model setup, which would reduce the extent of the drawdown towards the river.
- The higher hydraulic conductivities, adopted for the model due to high permeability data localised around 25 Smith Street, potentially overestimating the inflows and associated drawdown.

Should the be unacceptable ground effects, inflows or environmental impacts there may be the need to refine the model setup for further assessment.

After completion of the temporary works and tanking of the box for the permanent works this drawdown would recover to pre-construction conditions.

Incidental grouting or the emplacement of a grouted zone at the base of the excavation to manage high inflows is expected to be likely even under likely flow case conditions, which would result in a further reduction in the drawdown simulated.

A comparison of the simulated drawdown against the conditions of approval and mitigation measures for groundwater is provided in Attachment 2. The findings are summarised below:

- The simulated drawdown is generally less than that simulated for the EIS, noting the influence of Parramatta River seepage in the EIS prevented migration further to the north.
- A search of the Bureau of Meteorology (BOM) groundwater explorer on 19/03/2022 indicates that there are currently no registered groundwater bores within the 2 m drawdown contour presented in the figures. A 2 m drawdown is defined in the NSW aquifer interference policy (DPI-Water, 2012) as representative of a more than minimal effect.
- The EIS indicated that further work was required to assess impacts to areas with potential acid sulphate soils and from contaminated sites. These investigations are being completed by the GLC contamination discipline. The interpreted drawdown provided herein will support that assessment.
- A search of the NSW major projects planning portal, indicates that there are a number of new developments that may include basements that require dewatering and which were not assessed in the EIS. While there is likely to be cumulative drawdown effects they are expected to be subdued by a reduction in seepage as the drawdown cone associated with the station intersects each basement. The potential for and magnitude of impacts would be greater during the follow-on works, which are expected to be assessed by Sydney Metro and follow-on contractors. It is noted however that after follow on works are complete (tanking of the station) any long term contribution to cumulative impacts associated with the project are expected to subside.

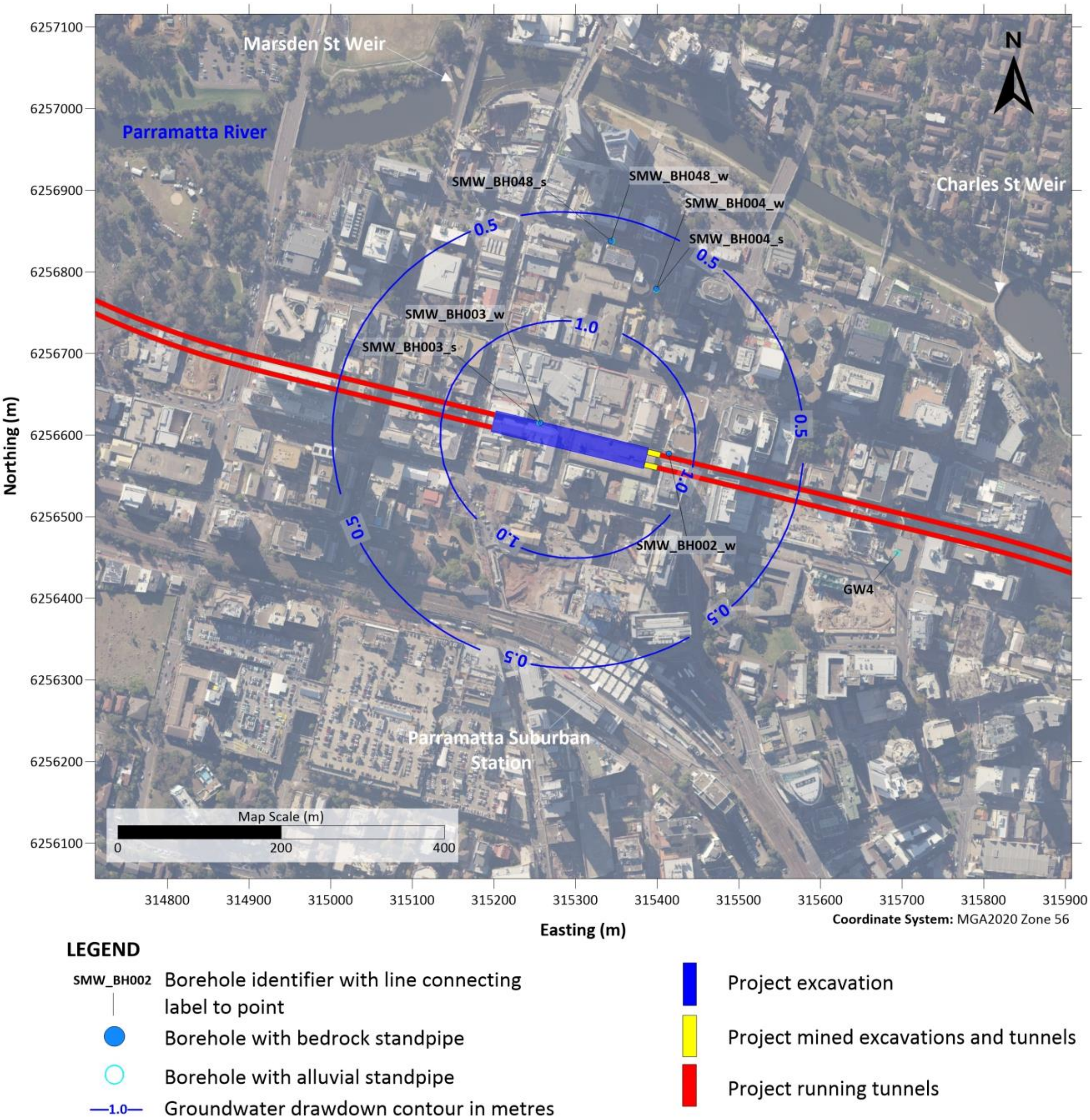


Figure 9: Drawdown (m) in the unconsolidated aquifer (assumed water table aquifer) for the high flow case model with treatment at the base and around the nozzles (1 year into construction) at Parramatta

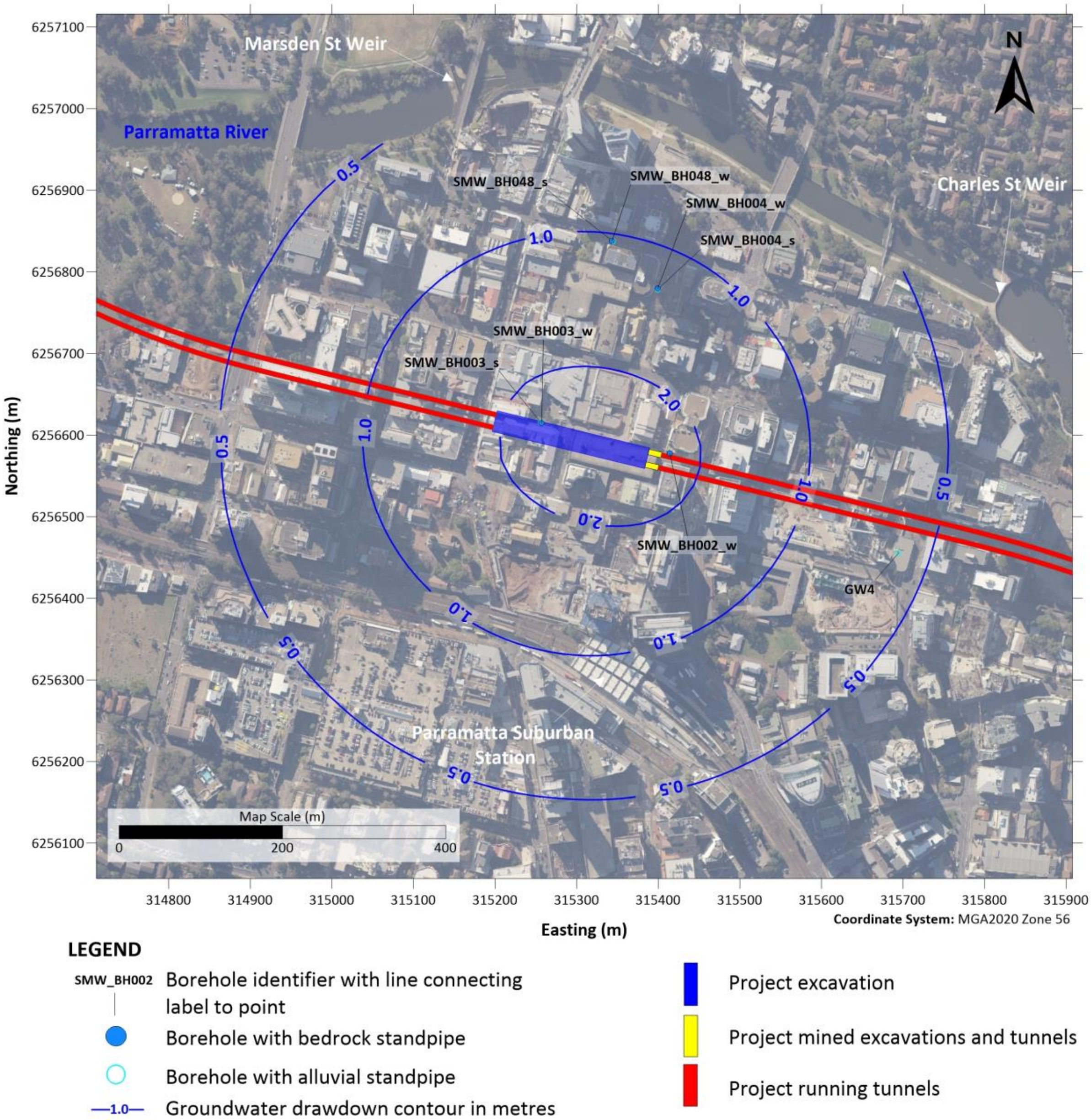


Figure 10: Drawdown (m) in the unconsolidated aquifer (assumed water table aquifer) for the high flow case model with treatment at the base and around the nozzles (at handover - 2 years) at Parramatta

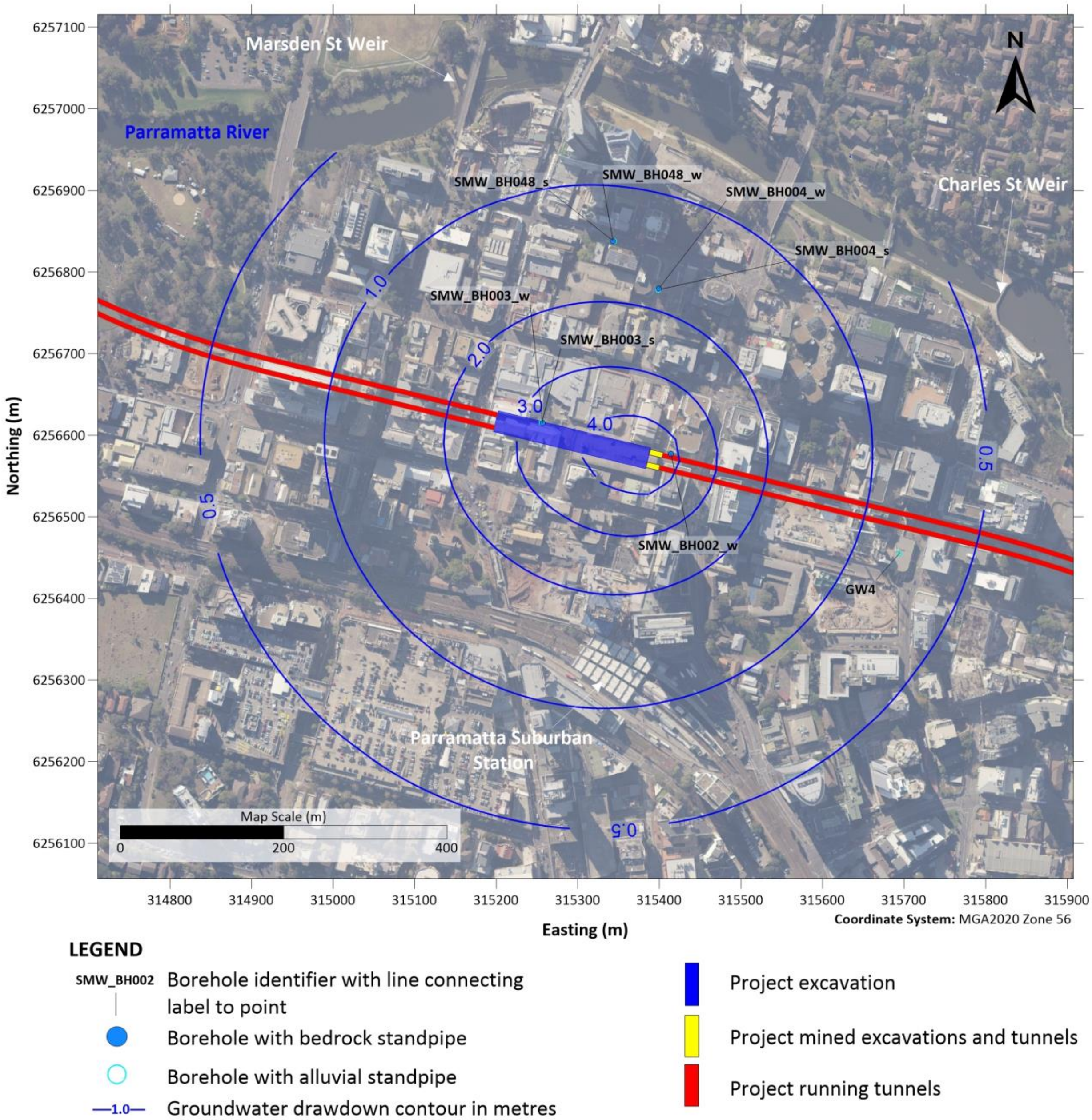


Figure 11: Drawdown (m) in the unconsolidated aquifer (assumed water table aquifer) for the likely flow case model (no treatment) that is grouted at the base and around the nozzles (at handover - 2 years) at Parramatta

3.3.2.3 Clyde

The estimated range in drawdown at handover (i.e. approximately 2.6 years into construction), are presented in Figure 12 to Figure 14[1] for the low flow and high flow cases respectively. The results suggest the zone of drawdown influence (estimated to be the 0.5 m contour) will extend between 250 m and 750 m from the edge of the Clyde infrastructure at the end of construction. It is expected that the zone of drawdown would gradually contract after completion of construction to be more focused around the permanently drained infrastructure of the portal and the Clyde Dive.

Figure 15 presents the likely case groundwater drawdown associated with the permanently drained infrastructure at the Portal and the Clyde Dive 100 years after the commencement of construction. The cumulative effects of the Portal, Clyde Dive and Rosehill Station are not currently included in the figure. Due to the depth of Rosehill Service Facility relative to the depth of the Portal and Clyde Dive it is expected that it will dominate drawdown in the area to the east and south east of the Portal and Clyde Dive.

The zone of drawdown influence is simulated to intersect with the Parramatta River during temporary construction works and during long term operation, which has potential to initiate migration of brackish water into the aquifer systems.

A comparison of the simulated drawdown against the conditions of approval and mitigation measures for groundwater is provided in Attachment 2. The results are summarised below:

- The simulated drawdown is generally not comparable to the Tender EIS due to difference in the infrastructure assessed. The Tender EIS assessed Rosehill Service Facility and the Portal and Clyde Dive only (and noting the Tender EIS location of the infrastructure differs to the current design), whereas this assessment does not include Rosehill Service Facility but does include the spur tunnels and Clyde Junction. As a result of this, the drawdown simulated for this assessment is generally less than the Tender EIS in and around the portal and to the south east of the Portal, but greater to the north due to effects from the spur tunnels and Clyde Junction.
- A search of the Bureau of Meteorology (BOM) groundwater explorer on 07/04/2022 indicates that there is potentially one registered water supply groundwater bore (GW024667) that appears within the 2 m drawdown contour. A 2 m drawdown is defined in the NSW aquifer interference policy (DPI-Water, 2012) as representative of a more than minimal effect. The bore is reported to be hand dug in 1966 for domestic purposes and is unlikely to exist, however, an assessment of the location and status of this bore may be required to meet the conditions of approval (Condition D121)
- A search of the NSW major projects planning portal, indicates that there are a number of new developments that may include basements that require dewatering and which were not assessed in the EIS. While there is likely to be cumulative drawdown effects they are expected to be subdued by a reduction in seepage as the drawdown cone associated with the station intersects each basement. The temporary works will be of a short-term nature and due to the

[1] The figures present the water table elevations within the shale rather than the overlying alluvial. This was because: the saturated zone within the alluvium was relatively thin (approximately 2 m in the model), and may be perched or transient; the saturated zone was simulated to be dewatered in locations near to the Clyde Infrastructure; and the understanding of the saturated thickness of the alluvium within the zone of drawdown (away from the alignment) was less well resolved away from the alignment. As such the zone of drawdown in the underlying Ashfield Shale was presented. This may result in an overstatement of the drawdown in the alluvium near to the Station Box. The extent of the modelled drawdown contours (for drawdown contours < 2 m) were simulated to be similar between the Ashfield Shale and the overlying alluvial aquifer.

tidal nature of the surrounding surface water (and the associated terrestrial GDE's) they are not expected to be adversely impacted.

- Long term impacts associated with permanent dewatering of the Portal and Clyde Dive are not expected to adversely impact surface water flows and associated groundwater dependent ecosystems because they are primarily tidal. The long-term impact associated with the permanent works is anticipated to be a reduction in base flow to A'Becketts Creek and Duck Creek. Long term the drawdown of the water table and fluctuations of the water table, due to recharge, are likely to interact with acid sulfate soils. This is anticipated to result in lower pH waters reporting to the portal structure. The implications of this for the design are being assessed by the GLC durability and contamination disciplines.
- Cumulative impacts associated with other developments (including Rosehill Service Facility) is still being estimated, but are anticipated to relate to settlement effects, which is being assessed by the GLC settlement discipline.
- The EIS indicated that further work was required to assess impacts to areas with potential acid sulphate soils and from contaminated sites. These investigations are being completed by the GLC contamination discipline. The interpreted drawdown provided herein will support that assessment.

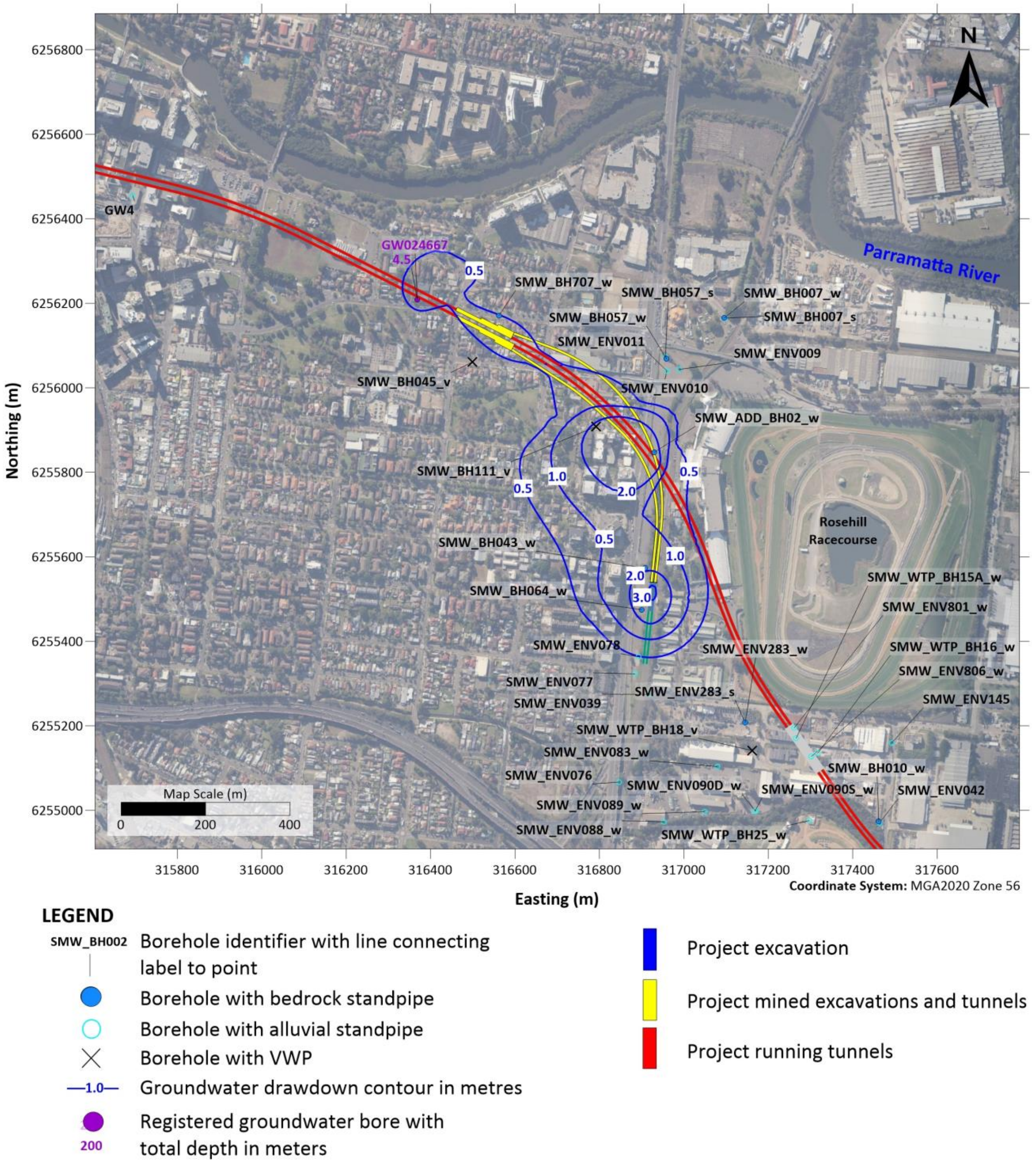


Figure 12: Drawdown in the Ashfield Shale (assumed water table aquifer) for the low case model (at approximate time of handover – 2.6 years at Clyde

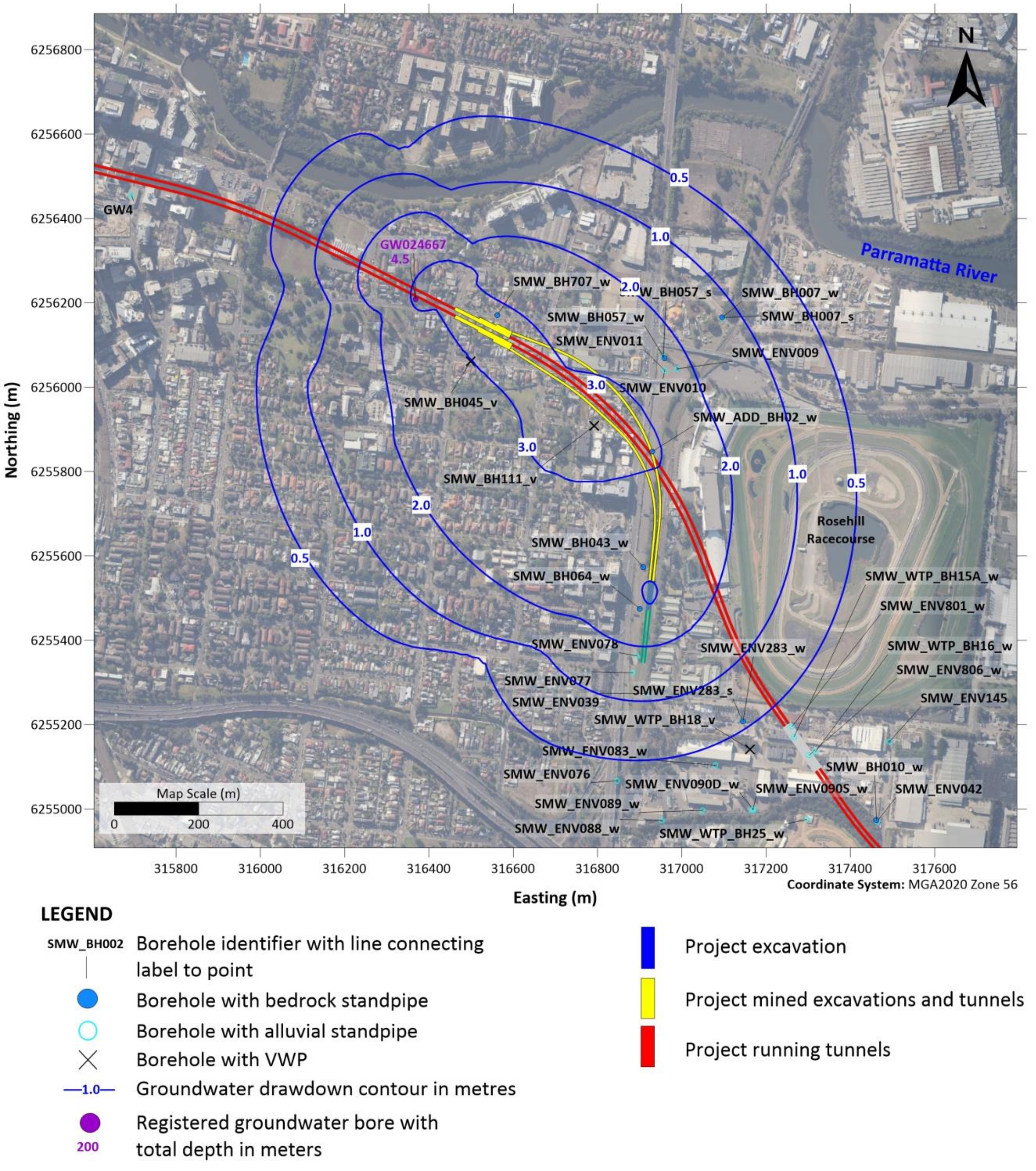


Figure 13: Drawdown in the Ashfield Shale (assumed water table aquifer) for the likely case model (at approximate time of handover – 2.6 years) at Clyde

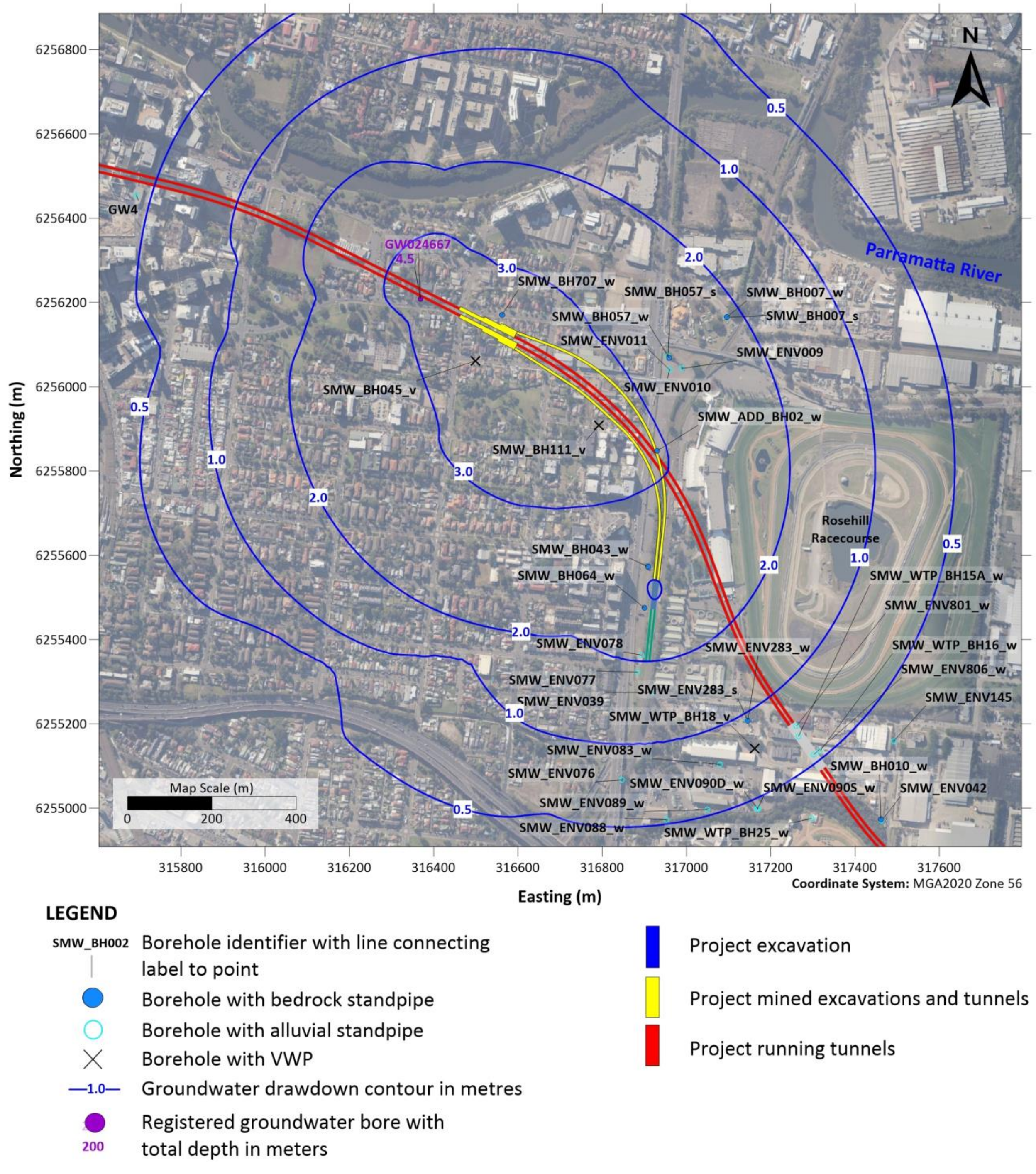


Figure 14: Drawdown in the Ashfield Shale (assumed water table aquifer) for the high case model (at approximate time of handover – 2.6 years) at Clyde

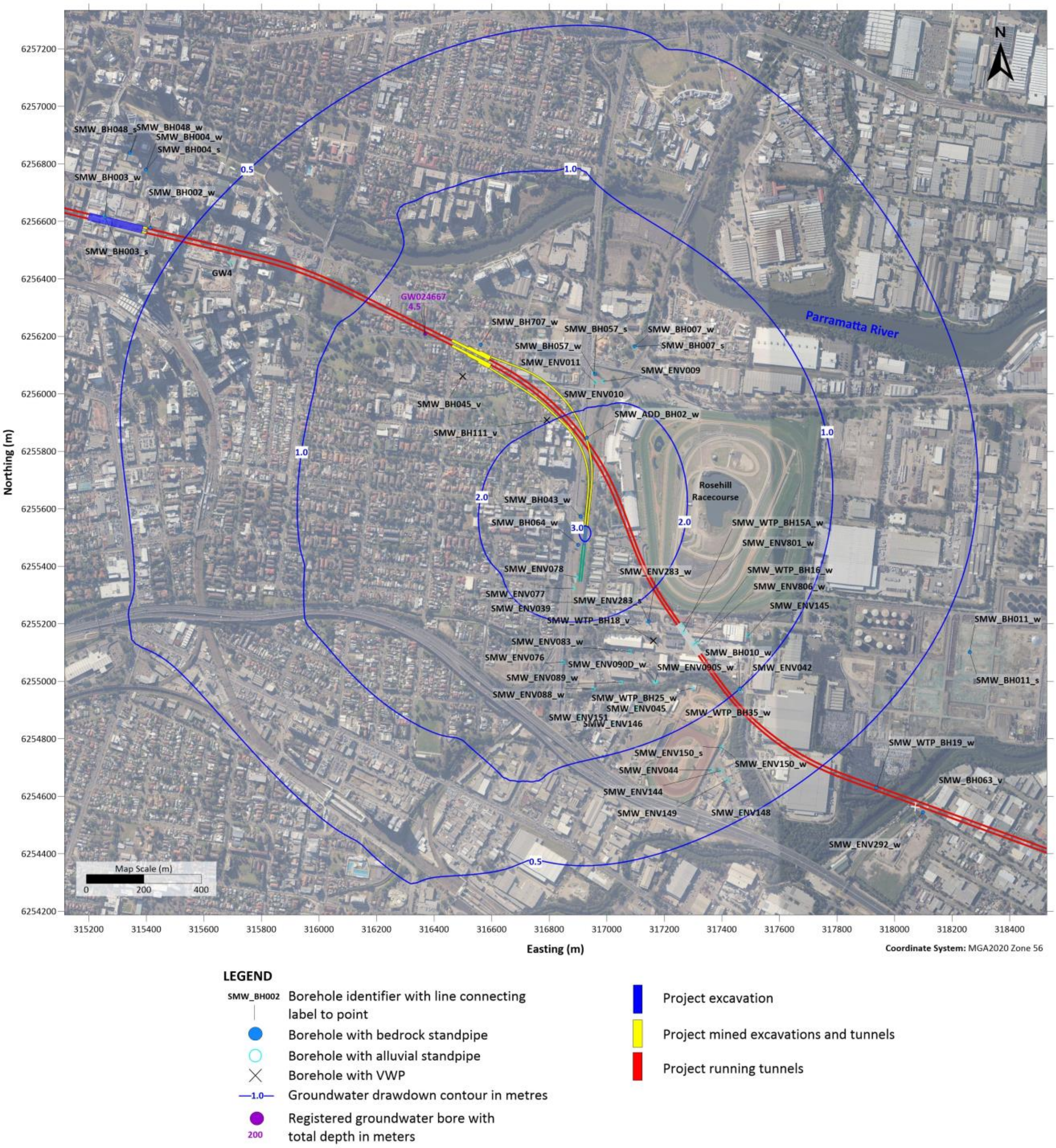


Figure 15: Drawdown in the Ashfield Shale (assumed water table aquifer) for the likely case model (100 years after commencement of construction) at Clyde

4 MITIGATION AND TREATMENT

4.1 Management of Groundwater Inflows

Where inflows fall within the specification for station infrastructure, such as is likely at Westmead Station, suitable pumps and treatment infrastructure are required to manage the expected range in inflows presented in Table 17 and Table 18. For clarity, the specification (Sydney Metro 2022a) Section 4.2.2 (n) requires the provision of two sumps for groundwater and stormwater collection at the base of each station excavation. One sump is expected to be required at each end of the station and each of the sumps will collect both groundwater and surface water.

As highlighted in Section 3.0 and in Table 17 and Table 18, the key areas where inflows are expected to require further mitigation and treatment to reduce inflows to acceptable rates include Parramatta Station with some potential requirements for mitigation and treatment at Westmead. There are also a number of locations along the alignment of higher uncertainty due to data paucity and potentially higher permeability rock features. These features may affect the mitigation methods that will need to be adopted for the TBM and cross passages.

The treatment requirements for these features to manage inflows are discussed below.

4.1.1 Westmead

There is expected to be low potential for inflows to exceed the inflow criteria (100 m^3/day) at the end of year 1 when construction reaches the required excavation depth. Inflows are not expected to exceed criteria at handover or thereafter.

Targeted grouting of localised seepage as excavation progresses, to meet the localised seepage criteria is likely to result in the inflow and local seepage criteria being successfully met.

Observation of inflows and for ground conditions conducive to higher flows as excavation progresses would support a target grouting program. This could form part of a pre-grouting program that identifies and manages zones of potential higher seepage before they represent an inflow issue.

4.1.2 Parramatta

Station box

Based on the modelling, there is a reasonable potential that the base of the excavation will require treatment to meet the inflow specification. It is noted however, that the hydrogeological conceptualisation used for interpretation of inflows and drawdown may be overstated by the high proportion of packer testing within a very localised zone at 25 Smith Street (SMW_WTP_Site01_BH01, SMW_WTP_Site01_BH02 and SMW_WTP_Site01_BH03), which had comparably high results and uncertainty around the validity of testing result in these areas (see sections 3.2.1.4 and 3.3.1.1).

In the event of adverse inflows being encountered treatment (e.g. grouting) of highly permeable geological structures may be required, otherwise a relaxation of the inflow criteria may be required.

If practical it would be beneficial to partially excavate the station box before grouting to understand ground conditions (faulting) in the sandstone. This will facilitate targeted grouting of fractured zones, although care would need to be taken to make sure grouting occurs before flows prevent grout from setting (by washing out). This may be informed by observing flow rates from the fracture zones at the base of the excavation as it progresses and using this information to initiate grouting at depths below the excavation base.

A five metre thick grouted zone below the base of the excavation and across its entirety that interconnects with the D-Wall was simulated using the high flow conditions to understand the potential requirements needed to confidently reduce flows to within the specification (134 m^3/day). It was found that if a grout permeability of two lugeons was achieved the inflow criteria could be met. While the simulated inflow for the likely case scenario is at or just above the inflow specification, based on the interpreted hydrogeological conditions, it is likely that localised zones of higher flows will be experienced on the base of the excavation (exceeding the localised seepage criteria of 15,000 Litres in any 24 hour period, measured over any square with an area of 10 m^2) that could require treatment. Grouting, albeit targeted would also mitigate this. This represents a preliminary assessment of an option that could potentially reduce total inflows to the station box, to below specification. With uncertainty in the modelling parameters adopted there is a reasonable possibility that is will not be required. As such, this type of treatment should be considered in the event that adverse ground and inflow conditions are encountered during excavation.

Parramatta Station Nozzle

There is a potential for high inflows in this area during construction, with consequential risk to the stability of the nozzle excavation, ability to apply shotcrete, and induce settlement. In particular, in the instance of a sandy layer being present within the overlying alluvial soils, very high inflows could occur if breached. Permeation grouting of clay soils from the station box is not likely to be effective due to absence of penetration, though may be considered if sandy soils are encountered.

4.1.3 Clyde Dive

The design of the permanent drainage infrastructure should consider the estimated inflows for the Clyde Dive to ensure the design meets the localised inflow criteria (5.0 ml/m^2/hr).

4.1.4 Running tunnels and cross-passages

There are a number of zones of hydrogeological uncertainty identified along the alignment including:

- Ch20.600 km due to an unnamed dyke.
- Ch22.320 km due to the Parramatta Dyke.
- Ch16.100 km to Ch16.600 km due to rock mass underlying a potential alluvial aquifer of potentially higher permeability.
- Ch21.600 km to Ch21.800 km due to the higher permeability rock mass underlying an alluvial aquifer also of higher permeability.
- There are also zones of reduced certainty associated with dykes and fractures that will also present a tunnelling and cross-passage excavation inflow risk.

To manage the potential for high inflows at these locations and other zones of uncertainty along the tunnel alignment the following recommendations are provided:

- Further characterisation of hydraulic conditions in each of the above areas with additional site investigations.
- Subject to the above, consideration should be given to the adoption of tunnelling techniques that limit the ingress of water from the surrounding rock features, where inflow rates pose a risk that could result in the TBM becoming inoperable. This should be considered in the selection and specification of the TBM type. Probing and grouting ahead of the TBM in key risk areas is another option, however, such is not amenable to TBM progress and construction timelines.

- Where possible observations made during TBM tunnelling of high inflows should be provided to the cross-passages construction team to inform pre-construction grouting requirements of cross-passages and associated construction methods to reduce construction inflows. This mapping may also assist in repositioning some cross-passages to more favourable locations. Probing of cross passages prior to excavation is recommended in areas of high inflow uncertainty.

4.1.5 MSF – Water conveyancing structures

Retention Basin

Groundwater is expected to be intersected by the retention basin. While inflows are expected to be low due to the presence of predominantly low permeability clays they will need to be managed accordingly. Given the location of the retention basin in an industrial area there is likely to be groundwater contamination management issues associated with the groundwater seepage. It is noted that management of groundwater seepage quality is outside the scope of this assessment.

The final design of the retention basin walls and base will need to consider the intersection of groundwater.

Estimation of inflows will support the design and construction planning and will be completed for following stages of works. Ongoing monitoring and hydraulic testing (slugs only) at SMW_ENV083, SMW_ENV282_w, SWM_ENV284w and SMW_WTP_BH18_w would support this assessment.

Water conveyance structures on A'Becketts and Duck Creeks

Groundwater will be intersected during the construction of the water conveyance structures on A'Becketts Creek and Duck Creek. The potential for seepage may vary depending on the intersection of fill comprising of clays, silts or sands or natural clay sediments. The construction process adopted will provide the primary means of reducing inflows. After diversion of surface water flows from the excavation, options that could be considered to manage groundwater seepage include:

- The adoption of wet construction techniques.
- Methods that reduce inflows such as impermeable/low permeability walls (such as piles) or the use of small excavation areas.
- Dewatering systems such as effective sump/well abstractions systems within the excavation or spear dewatering systems outside the perimeter of the excavation.

Given the location of the retention basin in an industrial area there is likely to be groundwater contamination management issues associated with the groundwater seepage. It is noted that management of groundwater seepage quality is outside the scope of this assessment.

The design of the structures will need to consider the intersection of groundwater. Ongoing monitoring of groundwater elevations in monitoring wells in this area would support the design decisions.

4.2 Ongoing monitoring and long-term management plans

Environmental monitoring should be undertaken in accordance with CoA C17 to confirm that the design and management strategies are the same or less than the predicted effects, that the effects are acceptable and that environmental impacts are minimal.

The proposed monitoring requirements at each location, to monitor for potential effects estimated in this HIR, are presented in Table 19, and to assist with responding to actual effects relative to the conditions approval. The proposed new locations will be incorporated into the groundwater monitoring program (GLC, 2022a).. A combined review of tender and detailed design investigations with the environmental monitoring program is required, to consolidate monitoring locations and ensure that the overall construction monitoring program is meeting the requirements CoA C17. Additional testing and equipment may be required to be installed at some locations. It is noted that the proposed monitoring could form part of the overall monitoring program required to meet CoA C17.

Table 19: Proposed monitoring for the assessment of predicted affects and environment impacts

Areas	Actions
Westmead	• At least four additional long-term groundwater monitoring bores are recommended for installation north and south of the new station box location and cross over cavern. These should be positioned within the footprint of the two-metre drawdown contour for the purpose of monitoring groundwater levels during excavation and the assessment of predicted versus actual drawdown and hydrogeological effects. The new locations should be installed as soon as possible to facilitate collection of baseline monitoring data within Ashfield Shale and Mittagong Formation and include standpipe piezometers and/or VWP's with pressure transducers with electronic data logging capabilities set at minimum hourly interval. • An additional monitoring well should be installed in the vicinity of Domain Creek to monitor groundwater drawdown at this potentially sensitive receptor. This should be coupled with water elevation monitoring in Domain Creek in understand the hydraulic relationship between surface water and groundwater. The monitoring program should include the new locations with existing locations of SMW_BH013_v, SMW_BH001_s and w, SMW_BH008_s and w, SMW_ BH03A_w, SMW_WTP_BH31A_w and SWM_WTP_BH32A_w. It is recommended the monitoring locations of the groundwater program be connected to a telemetry system for near real-time data. Groundwater level information during construction should be collected at not less than a monthly frequency and reviewed by a hydrogeologist every three months. • Observations of inflows during construction should be undertaken to support a targeted pre-grouting program to maintain inflows to within the specified localised and station box inflow criteria, to characterise contributions from surface water and groundwater in the excavation and to meet CoA C17 (e) and (j). Assessment of relative inputs from surface water and groundwater would be supported by the installation of a site-specific rain gauge.
Parramatta	• Ongoing monitoring at the existing and tender/detailed design locations will be required in accordance with the groundwater monitoring program (GLC, 2022a). The new locations should be installed as soon as possible to facilitate collection of baseline monitoring data on which predicted effects and environmental impacts can be assessed. • To manage the risk associated with dewatering inflow criteria exceedances and settlement of the Parramatta Sands a minimum 5 day aquifer pumping test is recommended however depending on the response a longer test duration may be required. This should be completed in the Hawkesbury Sandstone after installation of the D-Wall with monitoring of the overlying bedrock aquifers and the Parramatta Sands to characterise vertical hydraulic

	conductivity. The pumping test wells should be located inside the perimeter of the box with the monitoring wells located outside. The primary aim of the test will be to assess the effectiveness of the D-Wall at minimising the effect of dewatering on groundwater elevations in the overlying alluvial aquifer and inform ground treatment requirements. • Inflows into the excavation during the construction works should be monitored as excavation progresses to inform targeted pre-grouting requirements at the base of the excavation, to characterise contributions from surface water and groundwater in the excavation and to meet CoA C17 (e) and (j). Assessment of relative inputs from surface water and groundwater would be supported by the installation of a site-specific rain gauge.
Clyde	A survey of the registered groundwater supply bore GW024667 should be undertaken, with subsequent monitoring if it is currently being used. At least three additional groundwater monitoring bores (nested, long-term) are recommended for installation to the west and east of the Clyde Spur tunnels and access shaft. These should be positioned within the footprint of the two-metre drawdown contour for the purpose of monitoring groundwater levels during excavation and the assessment of predicted versus actual drawdown and hydrogeological effects. The new locations should be installed as soon as possible to facilitate collection of baseline monitoring data and within the unconsolidated sediments (where saturated), Ashfield Shale and Hawkesbury Sandstone and include standpipe piezometers and/or VWP's with pressure transducers with electronic data logging capabilities set at a minimum hourly interval. The new locations will need to be considered in the context of tender/detailed design site investigations and environmental monitoring location as detailed in the groundwater monitoring program (GLC, 2022a). Where possible, the monitoring program should include the new locations that are to be installed at the Clyde Access shaft and existing locations of SMW_BH045_v, SMW_BH111_v, SMW_BH057_s, SMW_BH057_w, SMW_ADD_BH02_w, SMWBH043_W, SMW_BH064_w, SMW_ENV_078, SMW_ENV077 and SMW_ENV039. It is recommended the monitoring locations of the groundwater program be connected to a telemetry system for near real-time data. Groundwater level information during construction should be collected at not less than a monthly frequency and reviewed by a hydrogeologist every three months. • Inflows into the excavation during the construction works should be monitored as excavation progresses to inform targeted pre-grouting requirements at the base of the excavation, to characterise contributions from surface water and groundwater in the excavation and to meet CoA C17 (e) and (j).. Assessment of relative inputs from surface water and groundwater would be supported by the installation of a site-specific rain gauge.
MSF Water conveyancing structures	• Ongoing groundwater elevations monitoring (using data loggers) and hydraulic testing (slug testing only) at SMW_ENV083, SMW_ENV282_w, SWM_ENV284w and SMW_WTP_BH18_w is required to inform the design of the retention basin. • Ongoing groundwater elevation monitoring and slug testing in at least two monitoring wells in the vicinity of the water conveyancing works on

A'Becketts Creek and Duck Creek to inform construction dewatering requirements and design.

5 CONCLUSIONS

Table 20 provides a summary of the key hydrogeological features of uncertainty or high inflows and the associated management solutions.

Table 20: Schedule of key hydrogeological features and management solutions

Location	Feature	Design & Construction Aspects	Treatment
Clyde Dive	Potential faulting and dyke	High groundwater flows currently simulated	Provision should be made for probing ahead of the road header, where there is potential for inflows from rock defects, to evaluate appropriate mitigation measures such as groundwater management and mitigation.
Ch22.30 km	Parramatta Dyke and unnamed dyke in Station Box	High ground water inflow to the TBM	Provision should be made for probing, where there is potential for increased inflows from rock defects to be encountered, to evaluate mitigation measures.
Parramatta Nozzle	Deep soil profile	Potential for high inflows, excavation stability and ability to apply shotcrete	A combination of design and construction mitigation measures are recommended to be applied, including close spaced canopy tubes, provision for probe investigation of both the soil and rock profile, and grouting.
Parramatta Station	Potential faulting and dyke within Shale and Sandstone combined with saturated fill and alluvial soils	High groundwater flows currently simulated	Provision for targeted grouting within the excavation to address inflows, in the event that adverse ground conditions and inflows are encountered at locally fractured areas and/or provision for management of inflows.
Chainage 23.110 km	Potential faulting	High groundwater flows currently simulated	Provision should be made for probing, where there is potential for increased inflows from rock defects to be encountered, to evaluate mitigation measures.

With the implementation of the solutions outlined in Table 20 at the locations of key hydrogeological concern in the table it is expected that construction program and design risks associated with high inflows and hydrogeological data uncertainty can be effectively managed for the Project.

Additional works proposed in the table will be included within the schedule of geotechnical works proposed to support construction design.

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ATTACHMENTS

Attachment 1 – Hydrogeological model

It should be noted that Table 1-1 shows for points of information from this HIR and the hydrogeological long section but does not include all the points of information which comprise the packer test plots, registered groundwater bores or bore data from historical reports not provided in the AGS files. The borehole ID at some locations refers to nested standpipes for which the deeper geotechnical hole has been converted a standpipe or VWP and a secondary hole has been drilled for the shallow standpipe. The standpipe constructed in the geotechnical hole has been provided as a separate borehole ID (generally as _w) in the AGS files transmitted, whilst the shallow borehole contains only basic geological details inferred from the deeper geotechnical hole. For example, SMW_BH001 is also SMW_BH001_w and SMW_BH001_s is a separate shallow hole drilled nearby and not geotechnically logged. Where this has occurred, it is outlined in comments field of Table 1-1.

Table1-1: Summary of boreholes contained in the HIR and Hydrogeological Model

Borehole ID	Easting (MGA 2020 zone 56)	Northing (MGA 2020 zone 56)	Ground Level (mAHD)	Project Chainage (km)	Hole Orientation	Groundwater Level Record	Standpipe Installed	VWP Installed	Packer Testing	Slug Testing	Hydrograph	Comments
3103-104	314669.479	6256590.419	14.000	23.06	-90				Y			
3103-105	315011.479	6256506.419	10.000	22.74	-90				Y			
3103-106	315307.479	6256449.419	10.000	22.44	-90				Y			
3103-107	315408.479	6256407.419	13.000	22.33	-90				Y			
3103-109	315794.479	6256288.418	8.000	21.92	-90				Y			
3103-111	316764.479	6255738.418	16.000	20.68	-90				Y			
3103-112	316908.479	6255726.418	16.000	20.57	-90				Y			
3103-113	317271.479	6255520.418	7.000	20.23	-90				Y			
3103-114	317455.479	6255229.418	5.000	19.84	-90				Y			
3103-118	318600.478	6254254.418	6.000	18.27	-90				Y			

Borehole ID	Easting (MGA 2020 zone 56)	Northing (MGA 2020 zone 56)	Ground Level (mAHD)	Project Chainage (km)	Hole Orientation	Groundwater Level Record	Standpipe Installed	VWP Installed	Packer Testing	Slug Testing	Hydrograph	Comments
3103-119	319042.478	6253958.418	4	17.71	-90				Y			
3103-121	319397.478	6253835.418	12.000	17.35	-90				Y			
3103-122	320212.478	6253741.418	3.000	16.57	-90				Y			
3103-124	320715.478	6253599.418	9.000	16.05	-90				Y			
402	315361.9	6256494.7	10.390	22.4	-90	Y	Y			Y		
601	315218.2	6256401.1	10.990	22.52	-90	Y	Y			Y		
604	315203.7	6256455.5	10.760	22.54	-90	Y	Y			Y		
606	315234.2	6256450.3	10.520	22.51	-90	Y	Y			Y		
610	315235	6256398.4	10.990	22.5	-90	Y	Y			Y		
611	315237.3	6256426.4	11.170	22.5	-90	Y	Y			Y		
613	315376.6	6256416	11.300	22.37	-90	Y	Y			Y		
615	315371.2	6256387.1	12.280	22.36	-90	Y	Y			Y		
617	315337.8	6256402.9	11.560	22.4	-90	Y	Y			Y		
622	315357	6256356.3	13.120	22.37	-90	Y	Y			Y		
623	315320.6	6256385.1	12.100	22.41	-90	Y	Y			Y		
624	315289.8	6256400.1	10.680	22.45	-90	Y	Y			Y		
601A	315217.6	6256402	11.000	22.52	-90	Y	Y			Y		Tender document report R1671-8PS by Douglas Partners for Parramatta Square in 2016
604A	315203.3	6256454.8	10.810	22.54	-90	Y	Y			Y		
606A	315262.4	6256439.7	10.020	22.48	-90	Y	Y			Y		
610A	315233.6	6256398.9	11.010	22.5	-90	Y	Y			Y		

Borehole ID	Easting (MGA 2020 zone 56)	Northing (MGA 2020 zone 56)	Ground Level (mAHD)	Project Chainage (km)	Hole Orientation	Groundwater Level Record	Standpipe Installed	VWP Installed	Packer Testing	Slug Testing	Hydrograph	Comments
611A	315236.4	6256425.8	11.121	22.5	-90	Y	Y			Y		
613A	315376.1	6256416.4	11.300	22.37	-90	Y	Y			Y		
615A	315371.2	6256387.1	12.270	22.36	-90	Y	Y			Y		
617A	315338.3	6256401.3	11.520	22.4	-90	Y	Y			Y		
622A	315357	6256355	13.120	22.37	-90	Y	Y			Y		
623A	315321.7	6256384.9	12.100	22.41	-90	Y	Y			Y		
624A	315289.6	6256399.1	10.770	22.45	-90	Y	Y			Y		
BH02	313746.976	6257246.598	29.564	24.17	-90				Y			
SMW_ADD_BH01	316929.900	6255847.600	10.600	20.66	-90							
SMW_ADD_BH01 A	316927.800	6255847.700	10.460	20.66	-90				Y			
SMW_ADD_BH02	316919.400	6255766.500	13.560	20.6	-90	N	Y		Y			Also called SMW_ADD_BH02_w
SMW_BH001	313798.979	6257313.419	31.130	24.14	-90	Y	Y		Y		Y	Also called SMW_BH001_w
SMW_BH001_s	313797.779	6257311.919	31.120	24.15	-90	Y	Y				Y	
SMW_BH002	315414.179	6256577.419	8.990	22.37	-90	Y	Y		Y			Also called SMW_BH002_w
SMW_BH003	315256.979	6256615.019	10.670	22.53	-90	Y	Y		Y		Y	Also called SMW_BH003_w
SMW_BH003_s	315255.979	6256615.319	10.670	22.53	-90	Y	Y				Y	

Borehole ID	Easting (MGA 2020 zone 56)	Northing (MGA 2020 zone 56)	Ground Level (mAHD)	Project Chainage (km)	Hole Orientation	Groundwater Level Record	Standpipe Installed	VWP Installed	Packer Testing	Slug Testing	Hydrograph	Comments
SMW_BH004	315399.179	6256779.519	8.680	22.43	-90	Y	Y		Y		Y	Also called SMW_BH004_w
SMW_BH004_s	315398.979	6256778.319	8.720	22.43	-90	Y	Y				Y	
SMW_BH005	313717.179	6257231.019	35.630	24.2	-90				Y			
SMW_BH006	313724.679	6257307.219	34.980	24.21	-90				Y			
SMW_BH007	317095.879	6256165.318	6.490	20.77	-90	Y	Y		Y		Y	Also called SMW_BH007_w
SMW_BH007_s	317096.279	6256164.918	6.490	20.77	-90	Y	Y				Y	
SMW_BH008	314037.479	6257153.619	21.280	23.87	-90	Y	Y		Y			Also called SMW_BH008_w
SMW_BH010	317461.678	6254973.518	4.350	19.63	-90	Y	Y		Y			Also called SMW_BH010_w
SMW_BH011	318261.179	6255102.018	3.850	18.88	-90	Y	Y		Y		Y	Also called SMW_BH011_w
SMW_BH011_s	318261.279	6255102.818	3.850	18.88	-90	Y	Y				Y	
SMW_BH012	314829.679	6257048.419	7.730	23.11	-90				Y			
SMW_BH013	313708.479	6257125.919	39.110	24.18	-60	Y			Y			Also called SMW_BH013_v
SMW_BH015	321381.478	6252898.317	22.940	15.17	-90	Y	Y				Y	Also called SMW_BH015_w
SMW_BH015_s	321381.078	6252899.017	22.920	15.17	-90	Y	Y				Y	
SMW_BH016	313975.479	6257134.419	23.880	23.92	-90				Y			

Borehole ID	Easting (MGA 2020 zone 56)	Northing (MGA 2020 zone 56)	Ground Level (mAHD)	Project Chainage (km)	Hole Orientation	Groundwater Level Record	Standpipe Installed	VWP Installed	Packer Testing	Slug Testing	Hydrograph	Comments
SMW_BH022	318603.078	6254882.618	2.380	18.48	-61			Y			Y	Also called SMW_BH022_v
SMW_BH022_s	318601.978	6254883.218	2.390	18.48	-90	Y					Y	
SMW_BH026	314644.779	6257620.419	15.000	23.57	-90				Y			
SMW_BH030	320330.078	6254018.118	3.830	16.55	-90	Y						Also called SMW_BH030_v
SMW_BH031	320501.578	6253883.018	4.690	16.34	-90	Y						Also called SMW_BH031_v
SMW_BH043	316908.279	6255573.218	12.780	20.41	-90	Y	Y		Y		Y	Also called SMW_BH043_w
SMW_BH045	316499.079	6256061.218	4.540	21.15	-90	Y		Y	Y			Also called SMW_BH045_v
SMW_BH048	315343.279	6256837.819	6.950	22.5	-90	Y	Y				Y	Also called SMW_BH048_w
SMW_BH048_s	315344.079	6256837.619	6.960	22.5	-90	Y	Y				Y	
SMW_BH049	315207.679	6256760.319	8.990	22.61	-90	N	Y		Y			Also called SMW_BH049_w
SMW_BH049_s	315207.679	6256760.319	8.990	22.61	-90	N	Y					
SMW_BH057	316958.779	6256068.418	3.840	20.8	-90	Y	Y		Y		Y	Also called SMW_BH057_w
SMW_BH057_s	316956.179	6256072.418	3.840	20.8	-90	Y	Y				Y	
SMW_BH060	318454.678	6254506.818	5.290	18.49	-90				Y			

Borehole ID	Easting (MGA 2020 zone 56)	Northing (MGA 2020 zone 56)	Ground Level (mAHD)	Project Chainage (km)	Hole Orientation	Groundwater Level Record	Standpipe Installed	VWP Installed	Packer Testing	Slug Testing	Hydrograph	Comments
SMW_BH063	318071.978	6254562.618	5.050	18.87	-50	Y		Y	Y			Also called SMW_BH063_v
SMW_BH064	316900.279	6255474.818	9.500	20.32	-90	Y	Y				Y	Also called SMW_BH064_w
SMW_BH071	321034.178	6253355.118	13.500	15.66	-90				Y			
SMW_BH072	316887.079	6255335.618	6.120	20.2	-90							
SMW_BH111	316791.879	6255908.318	9.740	20.8	-90	Y		Y	Y		Y	Also called SMW_BH111_v
SMW_BH115	318839.178	6254308.418	5.540	18.06	-90	Y		Y	Y		Y	Also called SMW_BH115_v
SMW_BH120	321260.178	6253046.018	17.380	15.27	-90	Y	Y		Y		Y	Also called SMW_BH120_w
SMW_BH121	320533.578	6253587.118	4.510	16.22	-90	Y	Y		Y		Y	Also called SMW_BH121_w
SMW_BH122	317399.778	6254769.018	1.940	19.53	-90							
SMW_BH123	317103.878	6254908.418	4.630	19.78	-90							
SMW_BH124	317466.478	6254546.318	3.610	19.37	-90							
SMW_BH600	316847.078	6255067.618	5.150	20.04	-90							
SMW_BH700	313633.979	6257251.919	38.250	24.28	-51				Y			
SMW_BH701	313818.379	6257207.219	29.380	24.09	-90	Y	Y		Y		Y	Also called SMW_BH701_w
SMW_BH702	316915.679	6255642.418	15.130	20.48	-90							

Borehole ID	Easting (MGA 2020 zone 56)	Northing (MGA 2020 zone 56)	Ground Level (mAHD)	Project Chainage (km)	Hole Orientation	Groundwater Level Record	Standpipe Installed	VWP Installed	Packer Testing	Slug Testing	Hydrograph	Comments
SMW_BH702a	316895.279	6255400.218	7.130	20.25	-90							
SMW_BH703	315050.379	6256616.419	8.850	22.73	-90				Y			
SMW_BH704	315324.479	6256610.319	10.310	22.46	-90				Y			
SMW_BH705	315335.779	6256561.319	10.140	22.44	-90				Y			
SMW_BH707	316562.079	6256170.918	4.570	21.15	-90	Y	Y				Y	Also called SMW_BH707_w
SMW_BH708	316547.979	6256169.818	4.760	21.16	-52				Y			
SMW_BH709	319319.978	6254062.718	5.440	17.52	-90	Y	Y		Y		Y	Also called SMW_BH709_w
SMW_BH709_s	319320.078	6254063.418	5.440	17.52	-90	Y	Y				Y	
SMW_ENV009	316988.4786	6256043.418	4.280	20.76	-90	Y	Y				Y	
SMW_ENV010	316959.4786	6256040.418	4.280	20.78	-90	Y	Y				Y	
SMW_ENV011	316959.4786	6256040.418	3.810	20.78	-90	Y	Y					
SMW_ENV039	316919.4	6255274.5	6.410	20.15	-90	Y	Y			Y		
SMW_ENV042	317462.9	6254970.2	4.430	19.62	-90	Y	Y			Y	Y	
SMW_ENV044	317364.2	6254691	3.510	19.5	-90	Y	Y			Y		
SMW_ENV045	317102.8	6254905.7	4.620	19.78	-90	Y	Y			Y	Y	
SMW_ENV076	316846.8	6255066.9	5.300	20.04	-90	Y	Y			Y		
SMW_ENV077	316884	6255323	6.030	20.2	-90	Y	Y			Y		
SMW_ENV078	316893	6255363	6.380	20.22	-90	Y	Y			Y		

Borehole ID	Easting (MGA 2020 zone 56)	Northing (MGA 2020 zone 56)	Ground Level (mAHD)	Project Chainage (km)	Hole Orientation	Groundwater Level Record	Standpipe Installed	VWP Installed	Packer Testing	Slug Testing	Hydrograph	Comments
SMW_BH709	319319.978	6254062.718	5.440	17.52	-90	Y	Y		Y		Y	Also called SMW_BH709_w
SMW_BH709_s	319320.078	6254063.418	5.440	17.52	-90	Y	Y				Y	
SMW_ENV009	316988.4786	6256043.418	4.280	20.76	-90	Y	Y				Y	
SMW_ENV010	316959.4786	6256040.418	4.280	20.78	-90	Y	Y				Y	
SMW_ENV011	316959.4786	6256040.418	3.810	20.78	-90	Y	Y					
SMW_ENV039	316919.4	6255274.5	6.410	20.15	-90	Y	Y			Y		
SMW_ENV042	317462.9	6254970.2	4.430	19.62	-90	Y	Y			Y	Y	
SMW_ENV044	317364.2	6254691	3.510	19.5	-90	Y	Y			Y		
SMW_ENV045	317102.8	6254905.7	4.620	19.78	-90	Y	Y			Y	Y	
SMW_ENV076	316846.8	6255066.9	5.300	20.04	-90	Y	Y			Y		
SMW_ENV077	316884	6255323	6.030	20.2	-90	Y	Y			Y		
SMW_ENV078	316893	6255363	6.380	20.22	-90	Y	Y			Y		
SMW_ENV083	317079.278	6255103.518	5.03	19.96	-90	Y	Y					Also called SMW_ENV083_w
SMW_ENV088	316952.578	6254972.918	4.85	19.92	-90	Y	Y					Also called SMW_ENV088_w
SMW_ENV089	317049.478	6254996.718	4.96	19.88	-90	Y	Y					Also called SMW_ENV089_w
SMW_ENV090	317170.078	6254998.118	4.58	19.82	-90	Y	Y					Also called SMW_ENV090D_w

Borehole ID	Easting (MGA 2020 zone 56)	Northing (MGA 2020 zone 56)	Ground Level (mAHD)	Project Chainage (km)	Hole Orientation	Groundwater Level Record	Standpipe Installed	VWP Installed	Packer Testing	Slug Testing	Hydrograph	Comments
SMW_ENV090S_ w	317168.278	6254997.918	4.57	19.82	-90	Y	Y					Also called SMW_ENV090S_w
SMW_ENV144	317395.4	6254688.7	3.520	19.48	-90	Y	Y			Y	Y	
SMW_ENV145	317491.8	6255159	4.740	19.77	-90	Y	Y			Y	Y	
SMW_ENV146	317021.5	6254853.1	4.280	19.78	-90	Y	Y			Y		
SMW_ENV148	317466.9	6254543.7	3.470	19.37	-90	Y	Y			Y		
SMW_ENV149	317418.7	6254656.7	3.400	19.45	-90	Y	Y			Y		
SMW_ENV150_S	317399.2	6254768.6	1.950	19.53	-90	Y	Y				Y	
SMW_ENV150_w	317399.2	6254768.6	1.950	19.53	-90	Y	Y			Y		
SMW_ENV151	316902.6	6254875	3.960	19.87	-90	Y	Y			Y		
SMW_ENV200	317057.200	6254941.900	4.520	19.84	-90	Y	Y					Also called SMW_ENV200_w
SMW_ENV201	317080.700	6254966.200	4.110	19.84	-90	Y	Y					Also called SMW_ENV201_w
SMW_ENV202	317100.000	6254962.100	4.300	19.83	-90	Y	Y					Also called SMW_ENV202_w
SMW_ENV204	317350.800	6254641.900	4.090	19.48	-90	Y	Y					Also called SMW_ENV204_w
SMW_ENV206	317359.500	6254662.200	4.010	19.49	-90	Y	Y					Also called SMW_ENV206_w
SMW_ENV207	317378.500	6254697.600	3.740	19.5	-90	Y	Y					Also called SMW_ENV207_w

Borehole ID	Easting (MGA 2020 zone 56)	Northing (MGA 2020 zone 56)	Ground Level (mAHD)	Project Chainage (km)	Hole Orientation	Groundwater Level Record	Standpipe Installed	VWP Installed	Packer Testing	Slug Testing	Hydrograph	Comments
SMW_ENV208	317391.700	6254678.200	3.860	19.48	-90	Y	Y					Also called SMW_ENV208_w
SMW_ENV209	317392.500	6254650.900	3.930	19.46	-90	Y	Y					Also called SMW_ENV209_w
SMW_ENV210	317385.100	6254637.400	3.990	19.46	-90	Y	Y					Also called SMW_ENV210_w
SMW_ENV218	316862.200	6254930.500	4.370	19.94	-90	Y	Y					Also called SMW_ENV218_w
SMW_ENV219	316912.700	6254927.100	4.780	19.91	-90	Y	Y					Also called SMW_ENV219_w
SMW_ENV220	316961.700	6254943.400	4.640	19.89	-90	Y	Y					Also called SMW_ENV220_w
SMW_ENV221	316999.100	6254954.400	4.460	19.88	-90	Y	Y					Also called SMW_ENV221_w
SMW_ENV222	317258.400	6254921.000	5.720	19.71	-90	Y	Y					Also called SMW_ENV222_w
SMW_ENV222_s	317258.400	6254921.000	5.720	19.71	-90	Y	Y					
SMW_ENV223	317337.400	6254898.700	4.610	19.65	-90	Y	Y					Also called SMW_ENV223_w
SMW_ENV223_s	317337.500	6254898.400	4.600	19.65	-90	Y	Y					
SMW_ENV226	317273.300	6254986.800	5.770	19.75	-90	Y	Y					Also called SMW_ENV226_w
SMW_ENV226_s	317273.400	6254987.400	5.730	19.75	-90	Y	Y					

Borehole ID	Easting (MGA 2020 zone 56)	Northing (MGA 2020 zone 56)	Ground Level (mAHD)	Project Chainage (km)	Hole Orientation	Groundwater Level Record	Standpipe Installed	VWP Installed	Packer Testing	Slug Testing	Hydrograph	Comments
SMW_ENV227	317274.578	6254965.418	5.710	19.73	-90	Y	Y					Also called SMW_ENV227_w
SMW_ENV229	317322.778	6254966.418	5.850	19.7	-90	Y	Y					Also called SMW_ENV229_w
SMW_ENV231	317371.400	6254951.300	6.450	19.67	-90	Y	Y					Also called SMW_ENV231_w
SMW_ENV232A	317335.400	6254974.900	5.930	19.7	-90	Y	Y					Also called SMW_ENV232A_w
SMW_ENV234	317419.200	6254762.900	2.160	19.51	-90	Y	Y					Also called SMW_ENV234_w
SMW_ENV238	317433.100	6254575.900	3.930	19.4	-90	N	Y					Also called SMW_ENV238_w
SMW_ENV241	317398.500	6254719.100	2.860	19.5	-90	Y	Y					Also called SMW_ENV241_w
SMW_ENV242	317380.000	6254694.200	3.620	19.5	-90	Y	Y					Also called SMW_ENV242_w
SMW_ENV243	317384.900	6254664.500	3.820	19.48	-90	Y	Y					Also called SMW_ENV243_w
SMW_ENV244	317361.900	6254560.500	4.190	19.44	-90	Y	Y					Also called SMW_ENV244_w
SMW_ENV247	317216.278	6254646.818	4.370	19.56	-90	Y	Y					Also called SMW_ENV247_w

Borehole ID	Easting (MGA 2020 zone 56)	Northing (MGA 2020 zone 56)	Ground Level (mAHD)	Project Chainage (km)	Hole Orientation	Groundwater Level Record	Standpipe Installed	VWP Installed	Packer Testing	Slug Testing	Hydrograph	Comments
SMW_ENV250	317055.300	6254713.800	4.860	19.67	-90	Y	Y					Also called SMW_ENV250_w
SMW_ENV258	317307.100	6254764.700	3.860	19.58	-90	Y	Y					Also called SMW_ENV258_w
SMW_ENV262	317280.000	6254871.800	4.350	19.66	-90	Y	Y					Also called SMW_ENV262_w
SMW_ENV263	317331.600	6254848.100	4.610	19.62	-90	Y	Y					Also called SMW_ENV263_w
SMW_ENV264	317253.700	6254935.600	5.680	19.72	-90	Y	Y					Also called SMW_ENV264_w
SMW_ENV264_s	317252.800	6254934.100	5.820	19.72	-90	Y	Y					
SMW_ENV266	317327.578	6254922.818	4.600	19.67	-90	Y	Y					Also called SMW_ENV266_w
SMW_ENV269	317311.200	6254869.300	4.730	19.64	-90	Y	Y					Also called SMW_ENV269_w
SMW_ENV271	317236.000	6254852.500	5.880	19.67	-90	Y	Y					Also called SMW_ENV271_w
SMW_ENV272	316980.800	6254846.500	4.280	19.8	-90	Y	Y					Also called SMW_ENV272_w
SMW_ENV275	316926.200	6254790.200	5.000	19.79	-90	Y	Y					Also called SMW_ENV275_w
SMW_ENV276	316932.200	6254850.400	4.530	19.83	-90	Y	Y					Also called SMW_ENV276_w

Borehole ID	Easting (MGA 2020 zone 56)	Northing (MGA 2020 zone 56)	Ground Level (mAHD)	Project Chainage (km)	Hole Orientation	Groundwater Level Record	Standpipe Installed	VWP Installed	Packer Testing	Slug Testing	Hydrograph	Comments
SMW_ENV279	317009.200	6254899.200	4.750	19.83	-90	Y	Y					Also called SMW_ENV279_w
SMW_ENV280	316956.400	6254897.600	4.440	19.86	-90	Y	Y					Also called SMW_ENV280_w
SMW_ENV282	317055.700	6255217.100	5.140	20.05	-90	Y	Y					Also called SMW_ENV282_w
SMW_ENV283	317145.200	6255207.800	5.730	20	-90	Y	Y					Also called SMW_ENV283_w
SMW_ENV283_s	317144.700	6255207.900	5.730	20	-90	Y	Y					
SMW_ENV284	317142.400	6255142.500	5.020	19.95	-90	Y	Y					Also called SMW_ENV284_w
SMW_ENV287	317337.100	6254912.700	4.600	19.66	-90	Y	Y					Also called SMW_ENV287_w
SMW_ENV287_s	317338.000	6254912.100	4.590	19.66	-90	Y	Y					
SMW_ENV292	318098.300	6254544.100	6.870	18.84	-90	Y	Y					Also called SMW_ENV292_w
SMW_ENV293	316920.700	6255138.200	5.470	20.06	-90	Y	Y					Also called SMW_ENV293_w
SMW_ENV294	313832.900	6257182.200	29.410	24.07	-90	Y	Y					Also called SMW_ENV294_w
SMW_ENV295	313820.300	6257185.800	29.760	24.09	-90	Y	Y					Also called SMW_ENV295_w

Borehole ID	Easting (MGA 2020 zone 56)	Northing (MGA 2020 zone 56)	Ground Level (mAHD)	Project Chainage (km)	Hole Orientation	Groundwater Level Record	Standpipe Installed	VWP Installed	Packer Testing	Slug Testing	Hydrograph	Comments
SMW_ENV297	313813.500	6257167.000	30.910	24.09	-90	N	Y					Also called SMW_ENV297_w
SMW_ENV299	313820.800	6257180.200	30.000	24.08	-90	Y	Y					Also called SMW_ENV299_w
SMW_ENV300	313829.800	6257171.400	30.060	24.07	-90	N	Y					Also called SMW_ENV300_w
SMW_ENV300_s	313829.700	6257170.900	30.100	24.07	-90	Y	Y					
SMW_ENV301	313801.200	6257177.300	30.590	24.1	-90	Y	Y					Also called SMW_ENV301_w
SMW_ENV301_s	313801.000	6257176.800	30.630	24.1	-90	Y	Y					
SMW_ENV712	321334.878	6252998.717	19.720	15.19	-90	Y	Y				Y	Also called SMW_ENV712_w
SMW_ENV712_s	321334.178	6252998.017	19.710	15.19	-90	Y	Y				Y	
SMW_ENV715B	321763.778	6252490.417	14.620	15.17	-90	Y	Y				Y	Also called SMW_ENV715B_w
SMW_ENV801	317261.000	6255194.900	5.830	19.93	-90	Y	Y					Also called SMW_ENV801_w
SMW_ENV801_s	317262.400	6255194.700	5.860	19.93	-90	N	Y					
SMW_ENV806	317302.000	6255127.400	5.300	19.85	-90	Y	Y					Also called SMW_ENV806_w
SMW_ENV808	317280.400	6255155.600	4.920	19.88	-90	Y	Y					Also called SMW_ENV808_w

Borehole ID	Easting (MGA 2020 zone 56)	Northing (MGA 2020 zone 56)	Ground Level (mAHD)	Project Chainage (km)	Hole Orientation	Groundwater Level Record	Standpipe Installed	VWP Installed	Packer Testing	Slug Testing	Hydrograph	Comments
SMW_ENV809	317306.500	6255134.800	5.210	19.85	-90	Y	Y					Also called SMW_ENV809_w
SMW_ENV811	317302.500	6255102.400	5.060	19.83	-90	Y	Y					Also called SMW_ENV811_w
SMW_ENV812	317336.900	6255189.300	5.520	19.88	-90	Y	Y					Also called SMW_ENV812_w
SMW_ENV813	317362.000	6255210.300	5.500	19.88	-90	Y	Y					Also called SMW_ENV813_w
SMW_ENV814	317311.400	6255174.100	5.470	19.88	-90	Y	Y					Also called SMW_ENV814_w
SMW_WTP_BH01	313807.300	6257167.600	30.800	24.09	-43				Y			
SMW_WTP_BH01 A	313806.400	6257167.200	30.950	24.09	-90	Y	Y					Also called SMW_WTP_BH01A_w
SMW_WTP_BH02	313599.200	6257266.700	35.760	24.32	-90	Y	Y		Y			Also called SMW_WTP_BH02_w
SMW_WTP_BH03	313922.900	6257171.000	26.490	23.98	-52				Y			
SMW_WTP_BH03 A	313923.900	6257170.800	26.370	23.98	-90	Y	Y					Also called SMW_WTP_BH03A_w
SMW_WTP_BH11	316270.700	6256285.500	6.400	21.46	-90				Y			
SMW_WTP_BH13	317214.300	6255249.100	5.390	20	-90	Y	Y		Y			Also called SMW_WTP_BH13_w
SMW_WTP_BH14	317252.100	6255195.900	5.760	19.93	-90	Y	Y		Y			Also called SMW_WTP_BH14_w

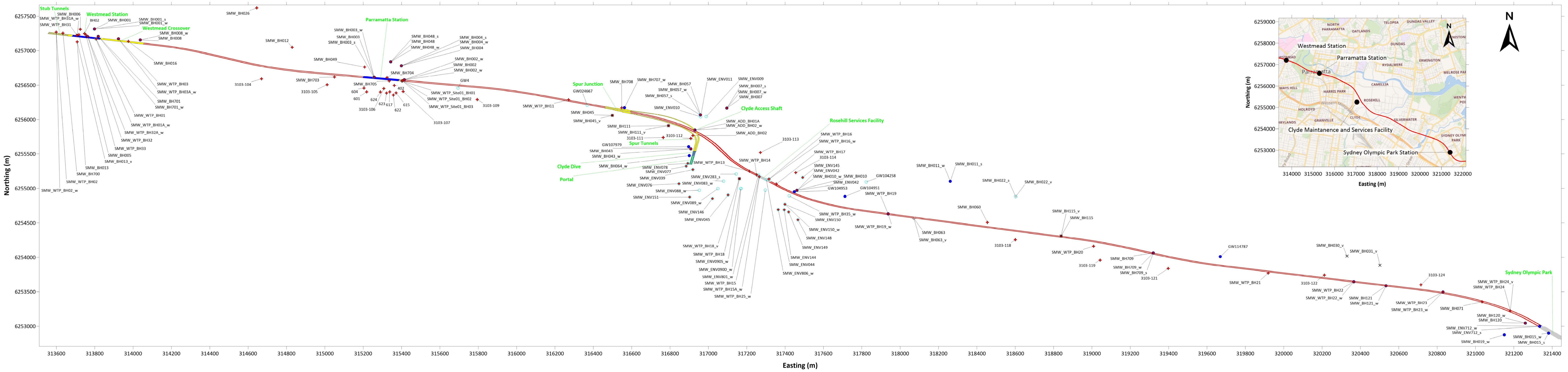
Borehole ID	Easting (MGA 2020 zone 56)	Northing (MGA 2020 zone 56)	Ground Level (mAHD)	Project Chainage (km)	Hole Orientation	Groundwater Level Record	Standpipe Installed	VWP Installed	Packer Testing	Slug Testing	Hydrograph	Comments
SMW_WTP_BH15	317265.500	6255170.200	5.540	19.9	-63				Y			
SMW_WTP_BH15A	317265.900	6255170.600	5.540	19.9	-90	Y	Y					Also called SMW_WTP_BH15A_w
SMW_WTP_BH16	317315.900	6255135.800	5.190	19.85	-90	Y	Y		Y			Also called SMW_WTP_BH16_w
SMW_WTP_BH17	317354.100	6255063.200	5.100	19.77	-60	Y	Y		Y			Also called SMW_WTP_BH17_w
SMW_WTP_BH18	317161.700	6255141.400	5.090	19.94	-90	N		Y	Y			Also called SMW_WTP_BH18_v
SMW_WTP_BH18_w	317164.400	6255141.200	5.070	19.94	-90	Y	Y					
SMW_WTP_BH19	317937.200	6254629.400	5.620	19.02	-90	Y	Y		Y			Also called SMW_WTP_BH19_w
SMW_WTP_BH20	319008.400	6254160.100	3.320	17.85	-90				Y			
SMW_WTP_BH21	319919.100	6253770.500	12.020	16.86	-90				Y			
SMW_WTP_BH22	320365.400	6253645.900	3.040	16.39	-90	Y	Y		Y			Also called SMW_WTP_BH22_w
SMW_WTP_BH23	320830.500	6253497.000	10.260	15.91	-90	Y	Y		Y			Also called SMW_WTP_BH23_w
SMW_WTP_BH24	321181.500	6253222.800	15.690	15.46	-90	N			Y			Also called SMW_WTP_BH24_v
SMW_WTP_BH25	317297.000	6254975.600	5.560	19.73	-90	Y	Y					Also called SMW_WTP_BH25_w

Borehole ID	Easting (MGA 2020 zone 56)	Northing (MGA 2020 zone 56)	Ground Level (mAHD)	Project Chainage (km)	Hole Orientation	Groundwater Level Record	Standpipe Installed	VWP Installed	Packer Testing	Slug Testing	Hydrograph	Comments
SMW_WTP_BH25_s	317296.600	6254976.200	5.560	19.73	-90	Y	Y					
SMW_WTP_BH26	317173.778	6254833.718	5.010	19.69	-90	Y	Y					Also called SMW_WTP_BH26_w
SMW_WTP_BH27	317112.600	6254822.200	5.020	19.71	-90	Y	Y					Also called SMW_WTP_BH27_w
SMW_WTP_BH29	316920.200	6254923.000	4.520	19.9	-90	Y	Y					Also called SMW_WTP_BH29_w
SMW_WTP_BH30	317250.178	6254701.518	4.270	19.57	-90	Y	Y					Also called SMW_WTP_BH30_w
SMW_WTP_BH30_s	317250.778	6254701.218	4.240	19.57	-90	Y	Y					
SMW_WTP_BH31	313706.400	6257221.100	36.580	24.21	-45				Y			
SMW_WTP_BH31A	313708.700	6257220.600	36.550	24.2	-90	Y	Y					Also called SMW_WTP_BH31A_w
SMW_WTP_BH32	313763.700	6257203.100	32.080	24.15	-43				Y			
SMW_WTP_BH32A	313763.400	6257203.200	32.090	24.15	-90	Y	Y					Also called SMW_WTP_BH32A_w
SMW_WTP_BH33	313754.000	6257229.600	32.810	24.16	-90				Y			
SMW_WTP_BH35	317421.600	6254891.800	5.810	19.59	-90	Y	Y					Also called SMW_WTP_BH35_w
SMW_WTP_BH38	317314.800	6254574.100	4.130	19.47	-90	N	Y					Also called SMW_WTP_BH38_w

Borehole ID	Easting (MGA 2020 zone 56)	Northing (MGA 2020 zone 56)	Ground Level (mAHD)	Project Chainage (km)	Hole Orientation	Groundwater Level Record	Standpipe Installed	VWP Installed	Packer Testing	Slug Testing	Hydrograph	Comments
SMW_WTP_BH40A	317148.600	6254633.100	10.640	19.58	-90	Y	Y					Also called SMW_WTP_BH40A_w
SMW_WTP_BH41	317031.400	6254715.800	5.070	19.68	-90	Y	Y					Also called SMW_WTP_BH41_w
SMW_WTP_Site01_BH01	315402.600	6256558.100	9.070	22.37	-90				Y			Also called EDS_Site01_BH01
SMW_WTP_Site01_BH02	315403.400	6256561.900	9.120	22.37	-90				Y			Also called EDS_Site01_BH02
SMW_WTP_Site01_BH03	315404.000	6256564.200	9.110	22.37	-90				Y			Also called EDS_Site01_BH03
SMW_WTP_BH41	317031.400	6254715.800	5.070	19.68	-90	Y	Y					Also called SMW_WTP_BH41_w

PLAN VIEW LEGEND

- BH02 Borehole identifier with line connecting label to point
- Borehole with permeability data
- Borehole with bedrock water level
- Borehole with alluvial water level
- Borehole with VWP
- XP50 Cross passage identifier
- Project excavation
- Project mined excavations and tunnels
- Project running tunnels
- Project features outside of this scope

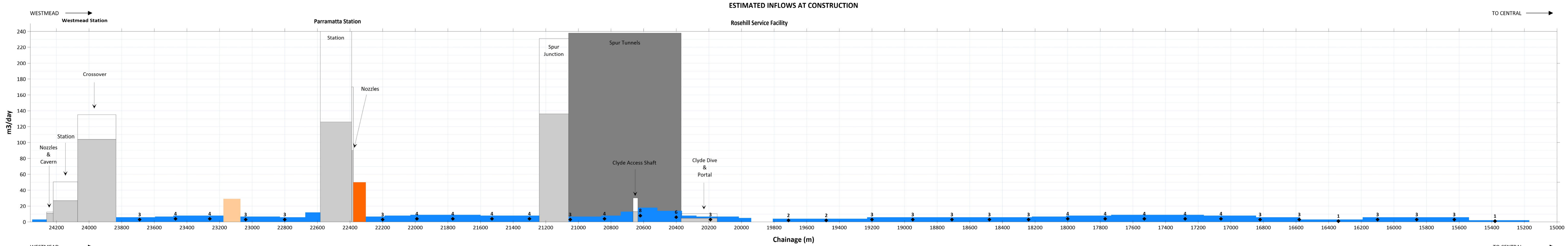


ESTIMATED INFLOWS AT CONSTRUCTION LEGEND

- 10 Cross passage and inflow estimate
- High flow / Likely flow — Station, cavern or shaft inflows at handover
- Spur Tunnels at 1.25 years

TBM estimated inflows at construction for a single tunnel

- Estimated inflow > 60 m³/day
- Estimated inflow 40 to 60 m³/day
- Estimated inflow 20 to 40 m³/day
- Estimated inflow 0 to 20 m³/day



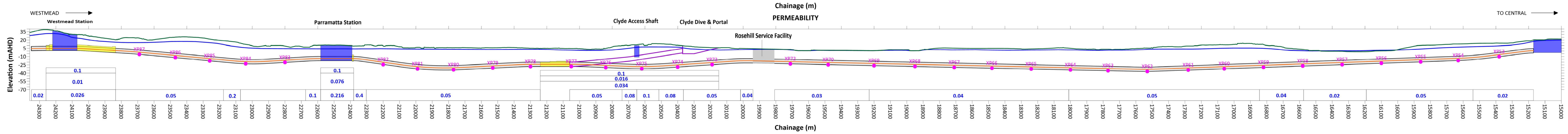
PERMEABILITY AND UNCERTAINTY LEGEND

- Ground surface
- Water head above the tunnel
- Tunnel centerline
- XP80 Cross passage invert level
- 0.05 Tunnel section interpreted hydraulic permeability (m/day)

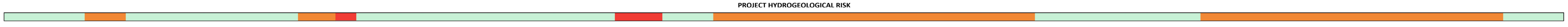
Interpreted risk and or uncertainty level

- low
- medium
- high

NOTE:
Uncertainty and risks are coloured to represent areas of data gaps, geological uncertainty/ features and or high estimated inflows



PROJECT HYDROGEOLOGICAL RISK

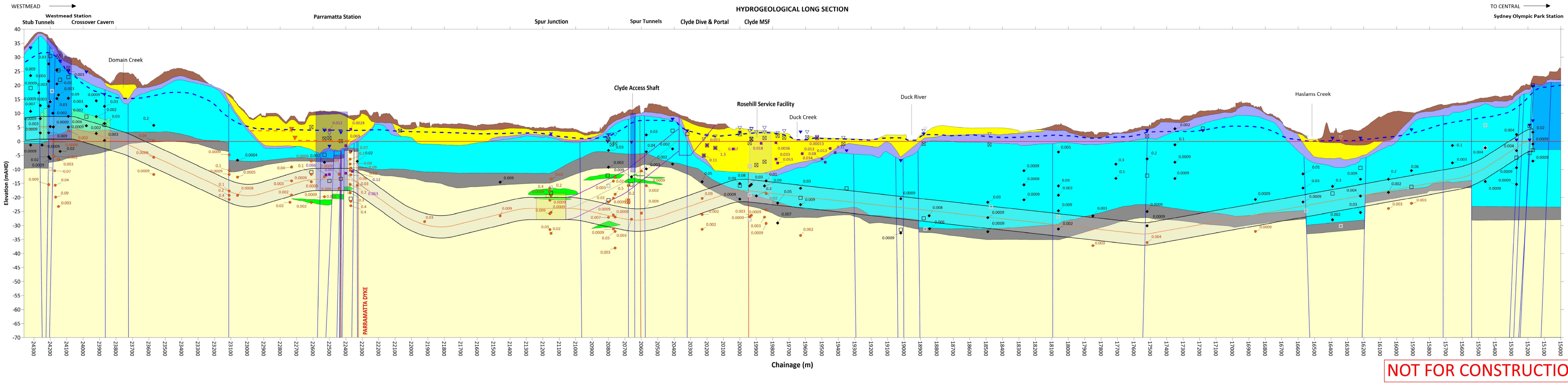


HYDROGEOLOGICAL LONG SECTION LEGEND

Geological Units

- Fill
- Alluvial
- Residual
- Ashfield Shale
- Mittagong Formation
- Siltstone / Laminite
- Hawkesbury Sandstone
- Inferred Fault or Joint
- Inferred Dyke
- Interpreted Parramatta River level
- 0 mAHD
- Alluvial screen center
- Bedrock screen center
- VWP sensor
- Interpreted groundwater table
- Bedrock groundwater level
- Alluvial groundwater level
- VWP groundwater level
- 1.3 Slug test location and interpreted permeability in m/day
- 0.2 Hawkesbury sandstone packer test mid point and interpreted permeability in m/day
- 0.01 Ashfield shale and mittagong formation packer test mid point and interpreted permeability in m/day

Note: Location of tests are projects relative to alignment chainage, offset is not shown.



NOT FOR CONSTRUCTION

SMEC
Member of the Surbana Jurong Group

GHD

revision	description	drawn	approved	date
	Rev0	CJ		28/04/2022

Geodetic Parameters
Coordinate System: MGA2020 Zone56
Project Chainage: meters

Scale: NOT TO SCALE

LOOKING NORTH

drawn	CJ
approved	HBC
date	28/04/2022
scale	Not to Scale
original size	A0

Gamuda Australia Laing O'Rourke Consortium

project: Sydney Metro West Western Tunnelling Package

title: HYDROGEOLOGICAL INTERPRETATION INFLOW ESTIMATES AND DATA GAPS AND RISKS

project no:

figure: Attachment 1 - Figure 1

Attachment 2 – Comparison of results against the conditions of approval and proposed mitigation and management measures

Comparison of key WTP infrastructure associated with the Stage 1 temporary works against the conditions of approval are provided below. Cells highlighted in blue require further consideration by GLC or Sydney Metro as part of this discipline. Cells highlighted in yellow required further consideration by other GLC disciplines.

Table A2-1 Assessment against conditions of approval

CoA	COA Description	Westmead	Parramatta	Clyde
D121	"Make-good" provisions for groundwater users must be provided in the event of a material decline in water supply levels, quality or quantity from registered existing bores associated with groundwater changes from construction.	A search of the Bureau of Meteorology (BOM) groundwater explorer on 19/03/2022 indicates that there are currently no registered groundwater bores within the 2 m drawdown contour presented in Section 3.3.2. A 2 m drawdown is defined in the NSW aquifer interference policy (DPI-Water, 2012) as representative of a more than minimal effect.	A search of the Bureau of Meteorology (BOM) groundwater explorer on 19/03/2022 indicates that there is currently no registered groundwater supply within the 2 m drawdown contour presented in Section 3.3.2. A 2 m drawdown is defined in the NSW aquifer interference policy (DPI-Water, 2012) as representative of a more than minimal effect.	A search of the Bureau of Meteorology (BOM) groundwater explorer on 07/04/2022 indicates that there is potentially one registered water supply groundwater bore (GW024667) that appears to be within the 2 m drawdown contour for the Clyde infrastructure. A 2 m drawdown is defined in the NSW aquifer interference policy (DPI-Water, 2012) as representative of a more than minimal effect. The bore is reported to be hand dug in 1966 for domestic purposes and is unlikely to exist, however, an assessment of the location and status of this bore may be required.
D122	The Proponent must submit a revised Groundwater Modelling Report in association with Stage 1 of the Critical State Significant Infrastructure (CSSI) to the Planning	This report includes revised groundwater modelling results for the Stage 1 works.		

CoA	COA Description	Westmead	Parramatta	Clyde
	Secretary for information before bulk excavation at the relevant construction location. The Groundwater Modelling Report must include:			
	(a) For each construction site where excavation will be undertaken, cumulative (additive) impacts from nearby developments, parallel transport projects and nearby excavation associated with the CSSI.	Other linear infrastructure and interference from other areas of the Sydney Metro West project were assessed in the EIS. A search of the NSW major projects planning portal, indicates that there are a number of new developments at Westmead Hospital to the north and in the surrounding area. These developments may include basements that require dewatering and which were not assessed in the EIS. While there is likely to be cumulative drawdown effects they are expected to be subdued by a reduction in seepage as the drawdown cone associated with the station intersects each basement. With regard to cumulative effects, the temporary works will be of a short-term nature and cumulative effects are expected to be within the bounds of background	Other linear infrastructure and interference from other areas of the Sydney Metro West project were assessed in the EIS. A search of the NSW major projects planning portal, indicates that there are a number of new developments in the Parramatta CBD area that are within the expected zone of drawdown influence. These developments may include basements that may require dewatering and which were not assessed in the EIS. Among others, this includes the Powerhouse Museum, Westfield Shopping Centre and Western Sydney University Innovation Hub. While there is likely to be cumulative drawdown effects they are expected to be subdued by a reduction in seepage as the drawdown cone associated with the station intersects each basement. The potential for and magnitude of impacts would be greater during the	Temporary effects of linear infrastructure and interference from other areas of the Sydney Metro West project were assessed in the EIS. The long term cumulative impacts associated with the Portal, Clyde Dive and Rosehill Service Facility are still being considered. A search of the NSW major projects planning portal, indicates that there are several new developments in the Clyde area, although these appear to be outside the zone of drawdown influence of temporary works and any excavations are expected to be shallow and not intersect groundwater. There may be overlap on a temporary basis with developments occurring in the Parramatta CBD. With regard to cumulative effects, the temporary works will be of a short-term nature and within an area with existing tidal fluctuation. As such

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		fluctuation. Further, the effects will be minor relative to the follow on works, which are expected to be assessed by Sydney Metro and follow-on contractors.	follow-on works, which are expected to be assessed by Sydney Metro and follow-on contractors. It is noted however that after follow on works are complete (tanking of the station) any long term contribution to cumulative impacts associated with the project are expected to subside.	changes to groundwater elevations and base flows beyond normal fluctuations are expected to be minimal. The long term effects of the Portal and Clyde Dive will not overlap with temporary effects associated with temporary works in other locations (such as at Parramatta station) however, there will be cumulative impacts with Rosehill Service Facility and potentially basements in Parramatta CBD which are still being considered. The cumulative effects are expected to be primarily associated with a reduction in seepage, which will reduce the cumulative drawdown impacts.
	(b) Predicted incidental groundwater take (dewatering) including cumulative project effects.	This assessment provides estimates of incidental groundwater take as part of establishing inflows to the station box and caverns (see Section 3.3.1). Cumulative effects are discussed in item (a) above. The Stage 1 EIS indicated that more detailed assessment was required to assess the impact of station drawdown on surface water baseflows. Impacts to baseflow are discussed in Table A2.2 below.	This assessment provides estimates of incidental groundwater take as part of establishing inflows to the station box and nozzles (see Section 3.3.1). Cumulative effects are discussed in item (a) above. The EIS indicated that there is disturbed terrain within the zone of drawdown influence and potential acid sulphate soils within sediments beneath and flanking the Parramatta River. There were also potential contaminated sites present. It was recommended in the EIS that investigations be completed to further characterise	This assessment provides estimates of incidental groundwater take as part of establishing inflows to the Clyde infrastructure (see Section 3.3.1). Cumulative effects are discussed in item (a) above. The EIS noted additional investigations were required as part of stage 1 works to characterise baseflow impacts, groundwater contamination migration and exposure of acid sulphate soils. Contamination migration and acid sulfate soils exposure are being considered by the GLC contamination discipline. Impacts to

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			the impacts of contamination and acid sulphate soils. Contamination migration and acid sulfate soils exposure are being considered by GLC contamination consultant. Impacts to baseflow are discussed in Table A2.2 below.	baseflow are discussed in Table A2.2 below.
	(c) Potential impacts for all latter stages of the CSSI or detail and demonstrate why these later stages of the CSSI will not have lasting impacts to the groundwater system, ongoing groundwater incidental take and groundwater level drawdown effects.	The design and construction for all latter stages of the project are to be undertaken by subsequent follow on contractors on behalf of Sydney Metro, who are expected to be responsible for assessing the associated impacts of these stages. Our assessment focuses on the temporary works component of the project (Stage 1) and the associated quantification of inflows, drawdown and predicted effects.		The Stage 1 temporary works infrastructure at Clyde will be permanent and either drained or undrained. The permanently drained infrastructure will include the portal and the Clyde Dive. All other infrastructure will have temporary impacts that will subside after installation. The long term impacts associated with the Portal and Clyde Dive are not expected to result in adverse impacts to the surrounding environment as potentially sensitive receptors do not occur within the zone of long term drawdown, other than a potential water supply well where make good provisions may apply (see the response to CoA 121 above). Existing potential contamination and low pH groundwater from acid sulfate soils within the zone of drawdown would be captured and treated by the WTP drainage systems and treated prior to disposal. Durability if this

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				infrastructure is being considered by the GLC durability discipline.
	(d) Actions required after Stage 1 to minimise the risk of inflows (including in the event latter stages of the CSSI are delayed or do not progress) and a strategy for accounting for any water taken beyond the life of the operation of the CSSI.	The design and construction for all latter stages of the project are to be undertaken by subsequent follow on contractors on behalf of Sydney Metro, who are expected to be responsible for assessing the associated impacts of these stages. Our assessment focuses on the temporary works component of the project (Stage 1) and the associated quantification of inflows, drawdown and predicted effects		The Stage 1 temporary works infrastructure at Clyde will be permanent and either drained or undrained. The permanently undrained infrastructure will include the portal and the Clyde Dive. All other infrastructure will have temporary impacts that will subside after installation. Consideration of long term impacts associated with the Portal and Clyde Dive infrastructure may be required as part of following stages of assessment. Our assessment of temporary impacts at the handover of stage 1 works is provided in Section 3.3.1.
	(e) Saltwater intrusion modelling analysis, from estuarine and saline groundwater in shale, into The Bays metro station site and other relevant metro station sties.	Due to the location of this station saltwater intrusion and saline groundwater in shale appears not to be an approval risk. Long term impacts, while likely to be small, will be the responsibility of Sydney Metro and follow on contractors.	Due to the location of this station saltwater intrusion and saline groundwater in shale appears not to be an approval risk. Long term impacts, while likely to be small, will be the responsibility of Sydney Metro and follow on contractors.	There is potential for saline water intrusion from Duck Creek and A'Becketts Creek on a long term (post Stage 1 temporary works) basis although given the distance of the Portal and Dive from these surface water features the dilution of saline water migrating into the box is expected to be significant. Saline water migration from Parramatta River may occur during construction and operation. The beneficial use potential of the groundwater resource in this area is

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				expected to be limited due to the industrial nature of historical land use and given water supplies are generally reticulated.
	(f) A schematic of the conceptual hydrogeological model.	See Attachment 1	See Attachment 1	See Attachment 1

Comparison of key WTP infrastructure associated with the Stage 1 temporary works against the EIS management and mitigation measures are provided below. Cells highlighted in blue require further consideration by GLC or Sydney Metro as part of this discipline. Cells highlighted in yellow required further consideration by other GLC disciplines.

Table A2.2 Assessment against EIS mitigation measures

Reference	Impact identified in EIS	Mitigation measure in EIS	Proposed actions for Westmead	Proposed actions for Parramatta	Proposed actions for Clyde
GW1	Loss of groundwater available to existing groundwater (supply bore) users.	Site inspection would be carried out on private domestic supply bore GW305646 to confirm the current viability of that bore. If found to be viable and predicted to be significantly impacted, make good measures would be implemented if a loss of yield were to occur.	Not applicable – See Table A2.1 CoA 121 for additional information.	Not applicable – See Table A2.1 CoA 121 for additional information	Not applicable – See Table A2.1 CoA 121 for additional information
GW2	Potential reduced baseflow to Toongabbie Creek, Domain Creek, A'Becketts Creek, Duck Creek, Haslams Creek, Powells Creek and the Mason Park wetlands, Bicentennial Park wetlands, Brickpit and Powells Creek Reserve. Requirements for baseline	A review of additional geotechnical and hydrogeology data would be undertaken to confirm the geological and groundwater conditions and determine, based on these local conditions, whether predicted groundwater drawdown from Stage 1 is likely to occur in the vicinity of these creeks. Where the additional data review shows local conditions and predicted groundwater drawdown are likely to cause surface water/groundwater interaction, then additional site investigations (in accordance with GW3) would be undertaken for	The zone of drawdown associated with Stage 1 temporary works is simulated to intersect Domain Creek (and reliant terrestrial ecosystems). Domain Creek appears to be dominated by man-made weirs and artificial lakes that suggest that water levels may be artificially inflated relative to natural conditions and may therefore recharge groundwater. Further monitoring is proposed (GLC, 2022a) to characterise groundwater elevations in alluvium near to the creek and relate them to surface water levels in the creek with	While the zone of drawdown is simulated to intersect Parramatta River the duration and magnitude are expected to be insufficient to create adverse impacts to flows and potentially reliant terrestrial ecosystem health beyond what would be experienced under normal climatic conditions. The management of longer term more persistent impacts during follow on	The zone of drawdown associated with temporary and permanent works potentially intersects Duck Creek and A'Becketts Creek. Due to the tidal nature of the river and creeks (a constant source of water) the impact on groundwater elevations and base flows is expected to be minimal but should be considered further by an ecologist. As such

Reference	Impact identified in EIS	Mitigation measure in EIS	Proposed actions for Westmead	Proposed actions for Parramatta	Proposed actions for Clyde
	monitoring of hydrological attributes	those creeks or surface water bodies.	subsequent quantification of flows outlined in GW3 if required. While the zone of drawdown is simulated to intersect Toongabbie Creek and Parramatta River the duration and magnitude are expected to be insufficient to create adverse impacts to flows and reliant terrestrial ecosystem health beyond what would be experienced under normal climatic conditions. The management of longer term more persistent impacts would be the responsibility of Sydney Metro and follow-on contractors.	works would be the responsibility of Sydney Metro and follow on contractors.	additional investigations to characterise impacts relating to loss of baseflow are not warranted. The same applies to Parramatta River where there is a temporary and permanent intersection of the zone of groundwater drawdown with the river.
GW3	Potential reduced baseflow to Toongabbie Creek, Domain Creek, A'Becketts Creek, Duck Creek, Haslams Creek, Powells Creek and the Mason Park wetlands, Bicentennial Park wetlands, Brickpit and Powells Creek Reserve. Requirements for baseline	Additional site investigations would be carried out at creeks or surface water bodies where the additional data review in GW2 shows there is a likely surface water / groundwater interaction. This would involve baseline monitoring of creek flows (streamflow gauging) prior to construction, and baseflow streamflow analysis to confirm the existing groundwater baseflow contribution to streamflow for each creek. Where a significant reduction in baseflow is predicted due to Stage 1, design responses would be implemented at station and shaft	Flow monitoring would be initiated if the investigations outlined in GW2 above indicate the need for further characterisation.	Not expected to be required as per the response in GW2	Not expected to be required as per the response in GW2

Reference	Impact identified in EIS	Mitigation measure in EIS	Proposed actions for Westmead	Proposed actions for Parramatta	Proposed actions for Clyde
	monitoring of hydrological attributes	excavations to reduce potential baseflow loss.			
GW4	Requirements for baseline monitoring of hydrological attributes. Migration of contaminants in groundwater and reduction in beneficial uses of aquifers	Monitoring of groundwater levels and quality of the site area would occur before, during and after construction. This would also include monitoring of potential contaminants of concern. Groundwater level data would be regularly reviewed during and after construction by a qualified hydrogeologist. Groundwater monitoring data would be provided to the NSW Environment Protection Authority and Department of Planning, Industry and Environment and the Natural Resources Access Regulator for information.	Baseline groundwater monitoring program (GLC, 2022a) will incorporate existing monitoring undertaken for the EIS and additional monitoring locations installed to inform design (tender and detailed design). A collective review of all existing and proposed environmental and geotechnical monitoring locations will be undertaken (GLC, 2022a) to assess whether the existing network is suitable to characterise baseline conditions at key ecological features and address the conditions of approval for groundwater monitoring, CoA C17.		
GW5	Ground movement and settlement	A detailed geotechnical and hydrogeological model for Stage 1 would be developed and progressively updated during design and construction. The detailed geotechnical and hydrogeological model would include	The HIR includes modelling of groundwater drawdown and inflows associated with key WTP infrastructure, which has been considered in the assessment of ground movement and settlement. Ground settlement predictions are provided in the ground settlement report (GHD and SMEC, 2022d) A site investigation program to install groundwater monitoring infrastructure has been developed to monitor and manage ground movement and settlement and will be incorporated into the groundwater monitoring program (GLC, 2022a) when completed.		

Reference	Impact identified in EIS	Mitigation measure in EIS	Proposed actions for Westmead	Proposed actions for Parramatta	Proposed actions for Clyde
		– Assessment of the potential for damage to structures, services, basements and other sub-surface elements through settlement or strain – Predicted groundwater inflows, groundwater take and changes to groundwater levels including at nearby water supply works. – Where building damage risk is rated as moderate or higher (as per the CIRIA 1996 risk-based criteria), a structural assessment of the affected buildings/structures would be carried out and specific measures implemented to address the risk of damage. Where a significant exceedance of target changes to groundwater levels are predicted at surrounding land uses and nearby water supply works, an appropriate groundwater monitoring program would be developed and implemented. The program would aim to confirm no adverse impacts on groundwater levels or to appropriately manage any impacts. Monitoring at any specific location would be subject to			

Reference	Impact identified in EIS	Mitigation measure in EIS	Proposed actions for Westmead	Proposed actions for Parramatta	Proposed actions for Clyde
		the status of the water supply work and agreement with the landowner.			
GW6	Ground movement and settlement	Condition surveys of buildings and structures in the vicinity of the tunnel and excavations would be carried out prior to the commencement of excavation at each site.	The HIR includes modelling of groundwater drawdown and inflows associated with key WTP infrastructure, which has been considered in the assessment of ground movement and settlement. Ground settlement predictions are provided in the ground settlement report (GHD and SMEC, 2022d) A site investigation program to install groundwater monitoring infrastructure has been developed to monitor and manage ground movement and settlement and will be incorporated into the groundwater monitoring program (GLC, 2022a) when completed.		