PROJECT REPORT

Groundwater Modelling Report – Rosehill Service Facility

Sydney Metro West – Western Tunnelling Package

Document Details

Document Title	**Groundwater Modelling Report – Rosehill Service Facility**
Project Name	**Sydney Metro West – Western Tunnelling Package**
Client	**Sydney Metro**
GA Project No.	**00013/13065**
Document Reference No.	**SMWSTWTP-GLO-RSH-SF500-EN-RPT-000001**
Principal Contractor	**Gamuda Australia**
ABN	**27 632 738 768**
Project Address	**N/A**

TABLE OF CONTENTS

1 DOCUMENT CONTROL

The current document version number and date of revision are shown in the document footer. All changes made to the Management Plan during its implementation on a live project are to be recorded in the amendment tables below.

1.1 Revision History

Revision	Date	Description of changes	Prepared by	Approved by
A	28/06/22	First Issue	GH	RS
B	05/07/22	Updated references, minor text clarifications, additional attachments to support discussions.	GH	RS

2 INTRODUCTION

2.1 Project Description

The scope of the work being undertaken under the Sydney Metro West Western Tunnelling Package works (WTP) (the Project) includes but is not limited to, the following:

- Rosehill Services Facility, including shaft excavation, permanent lining and lateral support

Refer to Figure 1 for the location of the WTP project.

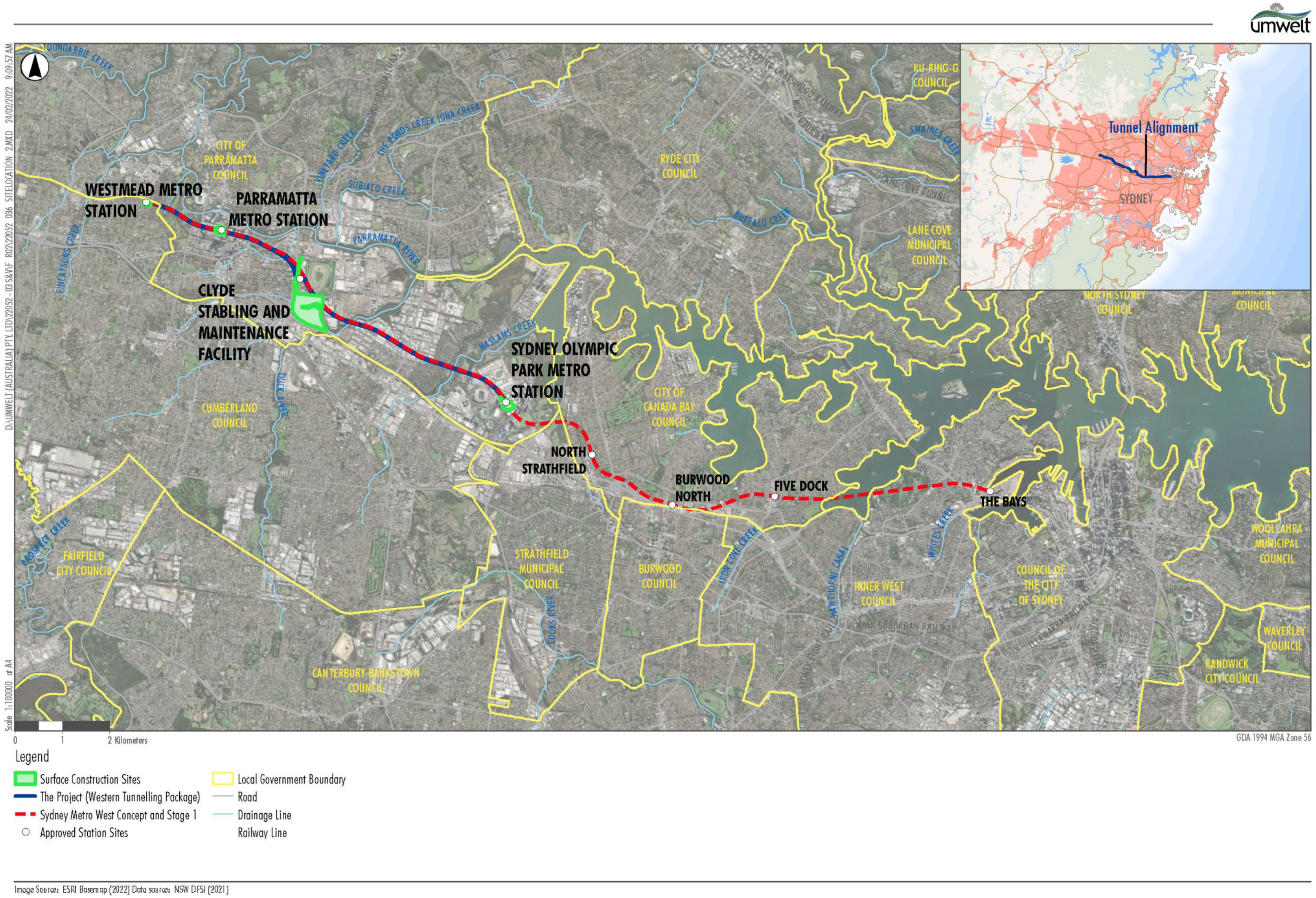


Figure 1: WTP Project Location

2.2 Context

Sydney Metro West – Westmead to The Bays Concept and Stage 1 was subject to environmental impact assessment under the NSW *Environmental Planning and Assessment Act 1979* (EP&A Act). It was also declared a Critical State Significant Infrastructure (CSSI) by the Minister for Planning & Public Spaces (the Minister).

An Environmental Impact Statement (EIS) has been prepared under Division 5.2 of the EP&A Act and in accordance with Part 3 of Schedule 2 of the Environmental Planning and Assessment Regulation 2000. Following exhibition of the EIS, an Amendment Report and Submissions Report were also prepared. After an assessment was carried out, the Minister determined that the Sydney Metro West – Stage 1 would be approved subject to conditions.

The planning approval (Infrastructure Approval SSI 10038) and related environmental assessment documents are located at: [https://www.planningportal.nsw.gov.au/major-projects/project/25631.](https://www.planningportal.nsw.gov.au/major-projects/project/25631)

Sydney Metro West – Westmead to The Bays Concept and Stage 1 received planning approval on 11 March 2021 (SSI 10038). The Project comprises the WTP, which is the western portion of Stage 1 of SSI 10038, from Sydney Olympic Park to Westmead.

The Project will be delivered by Gamuda Laing O'Rourke Consortium (GLC).

This Groundwater Modelling Report (Rosehill) has been developed to demonstrate how the Technical Report – Hydrogeological Design, Attachment 2, facilitates and complies with MCoA D122 for the Rosehill Construction Site. Refer Figure 2 for activities at the Rosehill site.

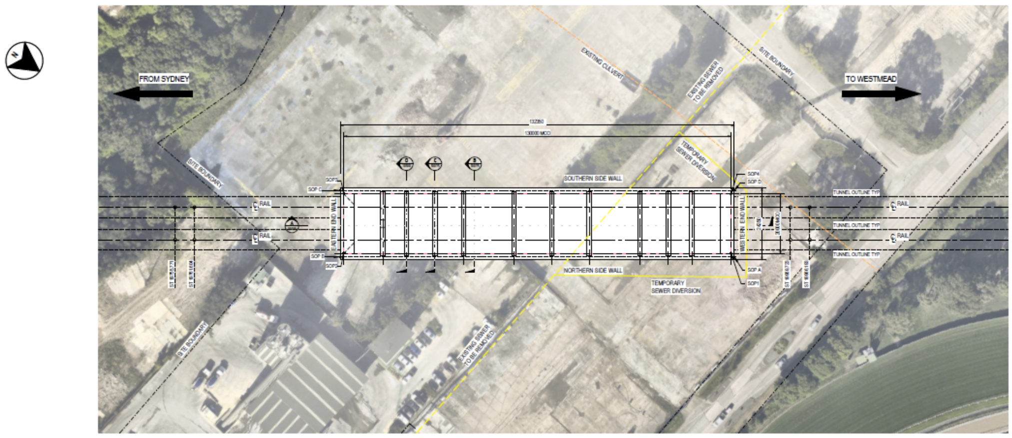


Figure 2: Rosehill General Arrangement Plan

2.3 Consultation Requirements

This document has been developed in consultation with Sydney Metro and the independent Environmental Representative.

2.4 Certification and Approval

This report has been prepared by GLC for the delivery of the WTP Project. It will be provided to the Planning Secretary for information before Bulk Excavation at the Rosehill Site.

3 ENVIRONMENTAL REQUIREMENTS

3.1 Compliance with MCoA D122

The table below outlines how this report addresses each of the requirements of MCoA D122.

Table 1: Compliance with MCoA D122

MCOA	REQUIREMENT	REFERENCE WHERE ADDRESSED
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 23/06/2022 indicates that there are no registered water supply groundwater bores within the zone of groundwater drawdown
D122	The Proponent must submit a revised Groundwater Modelling Report in association with Stage 1 of the CSSI to the Planning Secretary for information before bulk excavation at the relevant construction location.	Attachment 2, Sections 4, 5, 6, 7, 8
D122a	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 9.1
D122b	Predicted incidental groundwater take (dewatering) including cumulative project effects	Attachment 2, Section 9.2
D122c	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 9.3
D122d	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 9.4
D122e	Saltwater intrusion modelling analysis, from estuarine and saline groundwater in shale, into Tfhe Bays metro station site and other relevant metro station sites	Attachment 2, Section 9.5
D122f	A schematic of the conceptual hydrogeological model.	Attachment 2, Sections 3.6, 9.6

Table 2: Comparison of results against REMMs

REMM	IMPACT IDENTIFIED IN THE EIS	REQUIREMENT	ROSEHILL SITE
GW1	Loss of groundwater available to existing groundwater		A search of the Bureau of Meteorology (BOM) groundwater explorer on 23/06/2022 indicates that there are no registered water supply groundwater bores within the zone of groundwater drawdown (Section 8.2, Attachment 2).
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 those creeks or surface water bodies.	Due to the tidal nature of the river and creeks (a constant source of water) the impact on base flows (and reliant terrestrial ecosystems) is expected to be negligible
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	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 /	Not expected to be required as per the response in GW2

REMM	IMPACT IDENTIFIED IN THE EIS	REQUIREMENT	ROSEHILL SITE
	and Powells Creek Reserve. Requirements for baseline monitoring of hydrological attributes	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.	
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 monitoring would incorporate existing monitoring undertaken for the EIS and tender investigations within the new scope. Monitoring wells have been installed to facilitate baseline data crossover with the historical baseline data.
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	A detailed Geotechnical and Hydrogeological model have been developed and updated through the development of the design. No settlement impacts are predicted due to the construction of the Rosehill Service Facility. The

REMM	IMPACT IDENTIFIED IN THE EIS	REQUIREMENT	ROSEHILL SITE
			cumulative effects of the tunnel and box will be documented in the project wide Predicted Effects Report.
		–Assessment of the potential for damage to structures, services, basements and other sub-surface elements through settlement or strain	Settlement analysis has been undertaken and settlement effects and influence zones have been defined (Section 7.0, Attachment 1).
		–Predicted groundwater inflows, groundwater take and changes to groundwater levels including at nearby water supply works.	A search of the Bureau of Meteorology (BOM) groundwater explorer on 23/06/2022 indicates that there are no registered water supply groundwater bores within the zone of groundwater drawdown
		–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.	Groundwater effects have been incorporated, and settlement effects and influence zones have been defined for condition assessments. No buildings have been assessed with moderate or high settlement potential around Rosehill Service Facility
		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	A groundwater monitoring plan is in place with monitoring wells. An observational approach to groundwater monitoring would be adopted and updated as required in the Groundwater monitoring

REMM	IMPACT IDENTIFIED IN THE EIS	REQUIREMENT	ROSEHILL SITE
		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.	plan, based on the monitored groundwater levels. Trigger levels and response strategies are outlined in Attachment 3.
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.	Groundwater effects have been incorporated, and settlement effects and influence zones have been defined for condition assessments

ATTACHMENT 1

Technical Report – Geotechnical Design (without Appendices)

Technical Report – Geotechnical Design

Design Stage	Rosehill Service Facility – Stage 3
Author	Fiona Ma, Geotechnical Lead, Aurecon
Reviewed	Sharmeelee Subramaniam, Geotechnical Verifier, Aurecon
Date	7 July 2022

1.0 Purpose

This memorandum provides a summary of geotechnical data assumptions, interpretations, geotechnical models, geotechnical design analysis inputs and outputs for the Rosehill Service Facility – Stage 3 Design Resubmission #3 scheduled on 7 July 2022, which forms part of the early works for the Sydney Metro Western Tunnelling Package.

The information presented in this memo will be incorporated as part of the project wide Geotechnical Interpretive Report (SMWSTWTP-GLO-SWD-GE-RPT-010101) and Predicted Effects Report (SMWSTWP-GLO-SWD-SW000-GE-RPT-010104) to be submitted at a later stage by DJV.

A project wide Geotechnical Factual Report (SWMSDDS-GDS-SWD-GE-REP-000462.B.INF.B.01) to be submitted at a later stage by DJV.

2.0 Introduction

2.1 Purpose of this memo

This memo documents the development of project geotechnical interpretative models and geotechnical design analysis for the Rosehill Service Facility (the project) basement retention system. The development includes:

1. Review the supplied geotechnical data: to identify stratigraphy for the retention system design; to create geotechnical longitudinal and cross sections; to provide geotechnical design parameters
2. Undertake geotechnical analysis on:
   - resultant forces for structural design of retention system
   - slope stability for temporary excavation
   - end bearing capacity of diaphragm wall
3. Estimate ground settlement and potential impact to adjacent structures and services
4. Provide recommendations on:
   - additional geotechnical investigation to valid the geotechnical design parameters
   - instrumentation and monitoring of actual effects on existing ground
   - construction supervision

2.2 Available geotechnical investigation

Stratigraphy for the wall design has been developed mainly based on the geotechnical information provided by Sydney Metro for this Stage 3 Design Submission. No guarantee can be given as to the validity nature and continuity of the various subsurface features shown.

Additional geotechnical investigation has be undertaken after the Stage 3 submission in April to valid the geotechnical design assumptions. Detailed interpretation will be included in next design submission once laboratory test results become available.

2.2.1 Available boreholes

Seven (7) numbers of geotechnical boreholes including an inclined borehole and seventeen (17) numbers of environmental boreholes have been drilled at and in the vicinity of the Rosehill Service Facility. Groundwater monitoring wells are installed in selected boreholes.

Locations of the available geotechnical and environmental borehole related to the project are shown in Figure 1 in Attachment 1. Engineering logs and groundwater well construction records are included in Attachment 1a for easy reference.

Additional five (5) number of geotechnical boreholes have been drilled including an inclined borehole. Four (4) preliminary engineering logs are currently available and included in Attachment 1b. Groundwater monitoring wells are installed in selected boreholes.

2.2.2 Available laboratory tests

There is a limited number of laboratory tests on the alluvium soil samples taken from the existing site investigations as follow:

- Soil classification tests including moisture content, Atterberg Limits, linear shrinkage, particle size distribution at various depths and locations on selected soil samples.
- One oedometer test at SMW_ENV801 at the depth of 8.0-8.5 m
- One consolidated undrained triaxial compression test at SMW_ENV801 at the depth of 10.0-10.4 m
- One shear box test at SMW ENV801 at the depth of 11.5-11.9 m
- Soil durability tests

There are a number of laboratory tests on selected rock samples as follow:

- Point load test index (I_{s50}) on rock sample at 1m interval (on average)
- 21 numbers of uniaxial compressive strength (USC)
- 7 numbers of rock durability tests

Refer to Attachment 5a for the summary of laboratory results as extracted from the project Geotechnical Data Report (Golder and Douglas Partners, 2022) and selected laboratory certificates of the aforementioned tests.

Additional laboratory results will be included in next submission.

2.2.3 Field tests

There are few field tests undertaken to understand the permeability and fracture properties of the rock.

- 10 numbers of packer tests
- 3 numbers of geophysical tests at 3 boreholes

Refer to Attachment 5a for the summary of field tests as extracted from the project Geotechnical Data Report (Golder and Douglas Partners, 2022).

There are additional field tests to further valid the permeability and fracture properties of rock.

- 3 numbers of packer tests
- 3 numbers of geophysical tests at 3 boreholes

Refer to Attachment 5b for the summary of received additional field tests.

2.2.4 Cone Penetration Test

Three (3) cone penetration test (CPTu) have been completed as part of the additional geotechnical scope.

Refer to Attachment 5b for CPT records and preliminary CPT interpretations of the ground characterization.

3.0 Geotechnical model

Refer to Section 8.0 for the limitations associated with the interpretation of the ground model.

3.1 Regional and project geology

The proposed Rosehill Services Facility is located in a built-up area to the north of the proposed Clyde Maintenance and Service Facility (MSF) between Duck Creek and the Clyde Dive Structure. The existing ground surface is relatively level with a surface level of about RL 5.0 m to 6.0 m AHD.

With reference to the NSW Surface Geology mapping assessed using MinView, the site is underlain by Ashfield Shale (Twia) of the Wianamatta Group. This unit is the lowest formation of the Wianamatta Group and overlies the Mittagong Formation. This unit is predominated by black to dark-grey shale and laminate and forms part of the Permian Triassic Basins geology in the Sydney region. Quaternary alluvial terrace deposits (QP_at) are mapped in the site. These alluvial deposits are described as silt, clay, (fluvially-deposited) fine to medium grained quartz-lithic sand, polymictic gravel. The regional geology of the site is presented in Figure 2 in Attachment 2.

In accordance with the Soil Landscapes of the Sydney 1:100,000 sheet (2009), presented in Figure 3 in Attachment 2, the site is underlain by Disturbed Terrain (xx). This soil type describes areas which have been disturbed by human activity to a depth of at least 0.1m. The limitations of the soils are dependent on the nature of the fill material.

The Acid Sulphate Soils Risk Maps (2019) predicts the distribution of ASS, presented in Figure 4 in Attachment 2, the site is underlain by Disturbed Terrain (xx).

3.2 Structural geology feature

A potential 2 m wide dyke has been inferred from the SMW-GIR (Sydney Metro Authority, dated 28 April 2021) at the western end (Westmead) of the excavation box. However, from the available geotechnical borehole information, the dyke has not been intersected to date (including the additional inclined borehole). The projection of the dyke as contained in the GIR is indicatively shown in Figure 2 in Attachment 2.

3.3 Interpreted geotechnical model

Based on the available geotechnical investigation information, the anticipated ground conditions of the site are summarised as follow.

- The upper 1.0 m of the site is expected to be underlain by fill. The fill is underlain by alluvial soil units predominantly comprising layered clays of varying consistency.
- The upper alluvial layer is anticipated to comprise silty clay, in firm consistency (ALV-F) down to about RL 3.0 m AHD, underlain by silty clay, stiff in consistency (ALV-ST) down to about RL 1.0 m

AHD, then ALV-F down to RL -1.0 m AHD in the western side (Westmead). In the eastern side (Sydney), the upper alluvial layer is anticipated to comprise ALV-F down to about RL 3.0 m AHD, underlain by ALV-ST down to about RL 1.0 m AHD, then ALV-F down to RL -3.0 m AHD.

- The upper layers of ALV-F and ALV-ST described above, is underlain by a laterally continuous band of silty clay, soft to firm in consistency (ALV-S) down to about RL -6.0 m AHD. The alluvial sequence repeats with an approximately 5.0 m thick band of ALV-ST extending down to about RL -11.0 m AHD.
- Alluvial soils are underlain by a thin layer of residual soil or highly weathered siltstone. Siltstone unit is expected to increase in thickness from west to east within the Rosehill Services Facility. Slightly weathered to fresh (ST-II) is expected to be encountered at approximately RL -11.0 m to -12.0 m AHD extending to about RL -25.0 m AHD in the east and to about RL -22.0 m AHD in the west. Wedge failure can be expected in the lower section of the siltstone layer (ST-II (wedge)) with top of the layer at about RL -19.0 m AHD in the east (Sydney CBD) and about RL -18.0 m AHD in the west (Westmead).
- Slightly weathered to fresh sandstone (SS-II), pale grey and grey is expected to be encountered under the siltstone layer.
- Geophysical record of boreholes SMW_WTP_BH14, BH15 and BH16 show that there is a possibility of wedge failure mechanism towards the excavated site in the siltstone layer. The wedge zone is expected to be about 4.0 m to 5.7 m thick immediately above the sandstone layer as shown in the geotechnical long section. High permeability of this zone is expected as revealed from the available packet test results. Refer to Attachment 3 for the Stereonet Plots for N-E, N-W, S-E and S-W directions.

Refer to the geotechnical long section in Attachment 4 for further information.

3.4 Groundwater level

Groundwater level monitoring wells were installed, mainly targeting the alluvium layer, at the geotechnical and environmental BHs within the Rosehill Service Facility. The groundwater level has been recorded varying between RL 3.0 m and RL 4.2 m AHD. No long term groundwater level monitoring data has been received.

Five (5) piezometers have been installed within siltstone and sandstone in ENV283, RSF_BH01 and RSF_BH02. The water table level within siltstone measured from ENV283 was reported to be at the depth of about 2.8 m bgl (RL +2.9 m AHD) on 17 Nov 2021. The water table level within sandstone measured from RSF_BH02 was reported to be at the depth of 2.0 m bgl (RL +3.15 m AHD) surface on 05 May 2022.

Refer to the geotechnical long section in Attachment 4 for the water table levels measured at each borehole location during drillings or after wells developed.

A design groundwater level at RL 3.6 m AHD and RL 4.0 m AHD have been adopted in the geotechnical design for temporary and long-term cases, respectively. The long-term design groundwater level has considered the potential raise of groundwater level within the design life period.

Refer to the Technical Report - Hydrogeology Report for further information on values of hydraulic conductivity, potential short-term groundwater drawdown during construction based on excavation sequence, potential long-term groundwater drawdown contour and seepage flow estimates for the drained base condition.

4.0 Geotechnical design parameters

Soil and rock parameters have been based on available existing geotechnical investigations, field test results and laboratory testing where possible as well as some inputs from the Geotechnical Interpretive Report (GIR) prepared for the overall Sydney Metro Western Tunnel Package (GHD and SMEC, 2021).

Soil geotechnical parameters have been developed based on the available SPT output results as well as hand pocket penetrometers carried out on selected soil samples during the site investigations. The appreciation of the ground conditions is dependent on the limited laboratory tests data provided to date.

Soil Characterization Plots as listed below are presented in Attachment 5c.

- Corrected SPT-N $(N1)_{60}$ versus Reduced Level: Estimated $(N1)_{60}$ blow using Liao and Whitman Method (1986) to provide understanding of soil consistency.
- Undrained Shear Strength (Su) versus Reduced Level: Estimated Su based on the hand penetrometer value to provide indication of change of Su with level.
- Casagrande Plasticity Chart: to characterise the soil classification group of fine materials

For the rock units, rock mass classification schema developed by Pells *et al.* (1998) and (2019) has been used. Geotechnical parameters for rock units have been developed based on the available point load test index (I_{s50}), uniaxial compressive strength (USC) results carried out on selected rock samples during the site investigations, as well as the defect information provided in the borehole log or field tests.

Rock Characterization Plots as listed below are presented in Attachment 5c.

- I_{s50} axial verse I_{s50} diametral: indicates the range of the test results and to understand the rock strength anisotropy of each rock material type (siltstone and sandstone)
- USC verse I_{s50} axial: indicates the range of UCS test results and to understand the correlation factor of each rock material type against axial point load test index.

Based on the available limited discrete information, geotechnical design parameters have been estimated in accordance with published correlations with soil consistency, rock defects and past experience based on typical unit properties observed in boreholes.

Table 4.1 Geotechnical design parameters for soils

Unit ID	Description	Unit Weight (kN/m^3)	C' (kPa)	Ø'	S_u (kPa)	E_{50}^{ref} (MN/m^2)	E_{oed}^{ref} (MN/m^2)	E_{ur}^{ref} (MN/m^2)	υ	e_0	OCR
ALV-S	Alluvium - Soft to Firm	18	2	26	20	5	5	15	0.3	0.85	1.0
ALV-F	Alluvium - Firm	19	5	26	40	16	16	48	0.3	0.85	1.6
ALV-ST	Alluvium - Stiff	19.5	5	28	80	32	32	96	0.3	0.85	2.5

Table 4.2 Geotechnical design parameters for rocks

Unit ID	Description	Unit Weight (kN/m3)	C' (kPa)	Ø'	E' (MN/m2)	υ	Mass Tensile Strength (kPa)
ST-II (wedge)	Siltstone (Class II, slightly weathered to fresh, medium to high strength)	24	300	32	1000	0.25	85
ST-II		24	450	36	2000	0.2	85
SS-II	Sandstone (Class II, slightly weathered to fresh, high strength)	24	800	38	2500	0.2	190

4.1 In-situ stress field

Locked-in high horizontal stresses at magnitudes beyond the corresponding overburden pressure are known within the Sydney Basin. The origin of these in-situ stresses is likely to be the result of regional tectonic forces together with topographical influences, such as valleys and paleochannels, and major discontinuities (faults and dykes).

Various historical estimations of the stress field have been presented in the SMW-GIR (Sydney Metro Authority, dated 28 April 2021). For the purpose of designing the retaining diaphragm wall for this project at this stage, the locked-in horizontal stresses with the following correlations have been used due to the lack of site specific field test at this design stage.

- For Sandstone (Class II):

  σ_H = 1.0 MPa + 3.5 σ_v

  $\sigma_H / \sigma_h = 1.5$

- For Siltstone (Class II and wedge)

  σ_H = 0.75 MPa + 2 σ_v

  $\sigma_H / \sigma_h = 1.5$

Based on the orientation of the Rosehill Services Facility, major horizontal principle stress is considered in the long wall design and minor horizontal principle stress is considered in the head wall design.

Measurement of rock locked-in horizontal stress as per the additional geotechnical investigations outlined in Attachment 5b, indicates that rock horizontal stress is:

- For Sandstone (Class II):

  σ_H = 1.89 MPa and σ_h = 1.39 MPa at the depth of 33.5 m

  σ_H = 1.88 MPa and σ_h = 1.22 MPa at the depth of 31.25 m

The above measurement is less than the values adopted in this Stage 3 design. Thus, localised loading due to the wedge zone is not governing as the adopted higher lateral rock pressure is more critical.

4.2 Excavatability

Preliminary excavatability assessment is presented below based on available strength data (i.e. axial point load index) for the materials anticipated across the project for the material down to RL -23.0 m AHD which is slightly below the proposed bulk excavation level. This assessment is based on the method of Pettifer and Fookes (1994). The fracture spacing for each material included on the plot has been based on available borehole information.

The upper 15.0 – 16.0 m below existing ground level in fill and alluvium units are expected to be excavated using conventional earthmoving equipment. Rock excavation is expected to range from easy to extremely hard ripping with use of rock breakers in higher strength zones.

When selecting plant for excavation in rock units, consideration should also be given to the joint spacing in rock. This will increase excavation difficulty towards the upper end of each zone shown in the excavatability plot as shown in Figure 4-1 below.

The effects of vibration from rock cutting and rock breaking may have a significant impact on adjacent structures; appropriate construction methodologies should be used to limit noise and vibrations within design criteria.

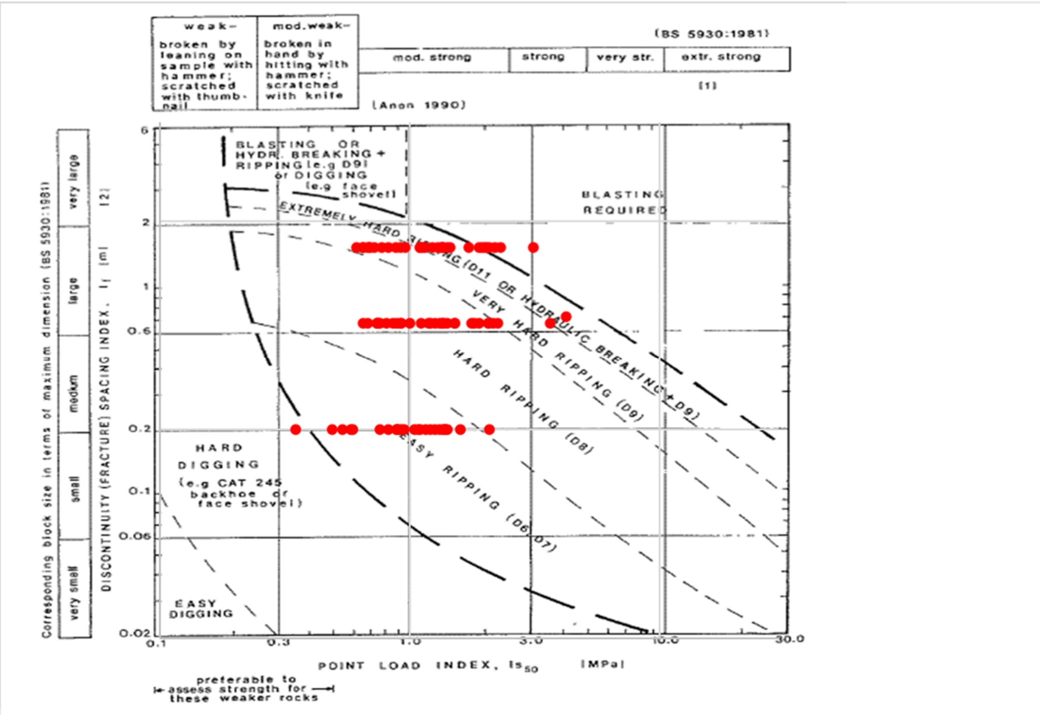


Figure 4-1: Excavatability of expected materials (Pettifer and Fookes, 1994)

4.3 Earthquake design

Project Particular Specification requires the Rosehill Service Facility to be designed against AS5100, AS1170 and ASA Specification SPC 301.

There is no active fault in the vicinity of the proposed Rosehill Service Facility. The site has a low seismic hazard.

Table 4.3 Indicative site class and hazard design factor at Rosehill Service Facility

Location	Description	Indicative Site Class (AS1170.4)	Hazard Design Factor (Z)
Rosehill Service Facility	Site with predominantly firm to stiff alluvium of no more than 25 m thick	C_e	0.08

The effect of seismic loading on the diaphragm wall structure has been considered in the structural model. This is not a critical design case.

4.4 End bearing capacity

The ultimate axial load (kN per 1 meter square) on the diaphragm wall with loads as specified in the Particular Specification drawings has been calculated using the tributary area concept. The calculated loads and adopted bearing capacity are tabulated in Table 4.4 and Table 4.5.

Table 4.4 Ultimate load on diaphragm wall

Load Types	Calculated Axial Load (kN per 1 meter square)
Total self weights (diaphragm wall, waler, strut)	1414
Load Case A (future infill one way slab and live load)	48
Load Case B (future column reactions and live load)	2187
Total load	3649

Table 4.5 End bearing capacity check

Founding Rock Class	Ultimate End Bearing Capacity (kPa)	Geotechnical Strength Reduction Factor	Design Ultimate End Bearing Capacity (kPa)
SS-II, Sandstone Class II	100,000	0.4	40,000

The applied ultimate axial pressure are less than the design ultimate bearing capacity of sandstone Class II on which the diaphragm wall is founded on.

5.0 Design analysis of the retention system

The Rosehill Services Facility consists of four (4) levels of basements and the construction involves excavation up to 27.8 m below existing ground level. The proposed retention structure is a 1.0 m thick diaphragm wall with minimum rock socket satisfying the deeper of the following criteria:

- 1000 mm below final excavation level.
- 800 mm below bottom of potential rock wedge zone layer and in sandstone Class II.
- 1200 mm below top of sandstone layer with at least 800 mm in sandstone Class II.

The proposed diaphragm wall is supported by four (4) levels of permanent precast struts with reinforced concrete (RC) topping and RC waler. The construction of the retention system is carried out in a 'top down' manner (construction of struts with topping and waler while carrying out excavation in stages).

For the design of retention system, a two-dimensional finite element software, PLAXIS 2D was adopted to model the structures and excavation sequences. The analysis includes temporary and permanent conditions to identify structural load and deformation. The software is equipped with soil-structure features to deal with various aspect of complex geotechnical structures and construction stages using robust and theoretically sound computational procedures.

5.1 Data assumptions and analysis input

A total of five (5) sections were analysed with PLAXIS software based on representation ground condition, excavation depth and permanent structure arrangement. The locations of these sections are shown below in Figure 5-1:

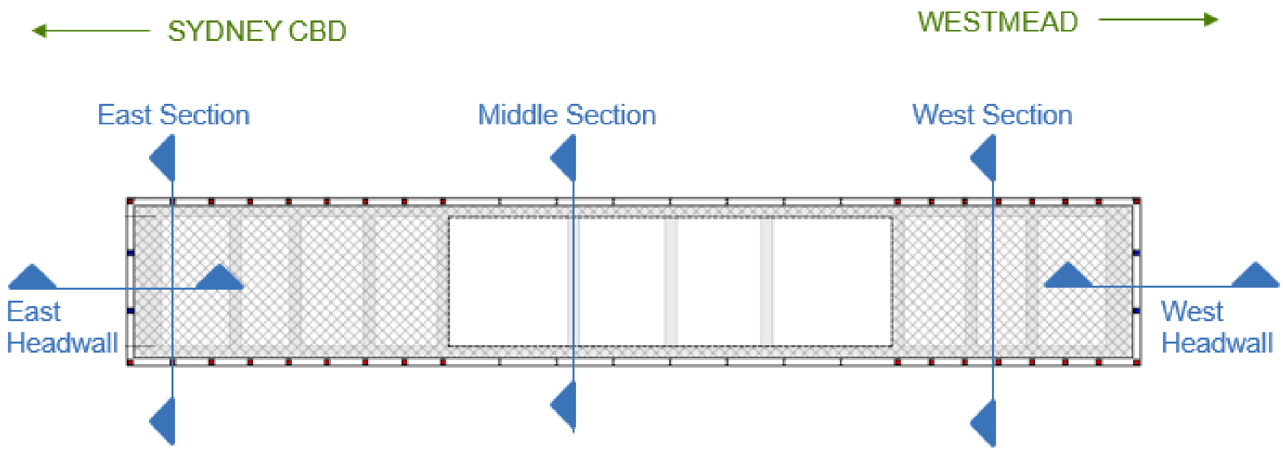


Figure 5-1 Cross-sections for PLAXIS analyses

For short sections (east, middle and west), a full cross section consisting of diaphragm wall on both sides are modelled. As the struts are designed to support dead and superimposed loads, struts are modelled as plate elements in the PLAXIS analysis to obtain bending moment and support reaction.

For headwall sections in the longitudinal direction (east and west headwalls), the longitudinal waler beams and base slab are modelled as fixed-end anchors with 1.0m spacing to support the headwall. The PLAXIS model has included floor loads transferred to the walers on the eastern and western headwalls.

The soil is modelled as Hardening Soil with undrained parameters (HS Undrained B) during the temporary stages. Once the excavation has reached the final level and casted the base slab, the soil model has been switched to Hardening Soil with drained parameters (HS Drained). Siltstone and Sandstone are modelled as Mohr-Coulomb material with drained parameters.

A nominal surcharge of 20 kPa has been considered in the analysis model to take into consideration construction loading.

A steady state groundwater flow has been considered in the analysis model to take into consideration of the future potential groundwater drawdown during operation. Refer to Technical Memo – Hydrogeology Report for the discussion of effect between transient state and steady state. From Slide2 output, steady state is considered as the critical case to be adopted in the PLAXIS analysis.

A typical connectivity plot for both short and long sections are shown in Figure 5-2 and Figure 5-3.

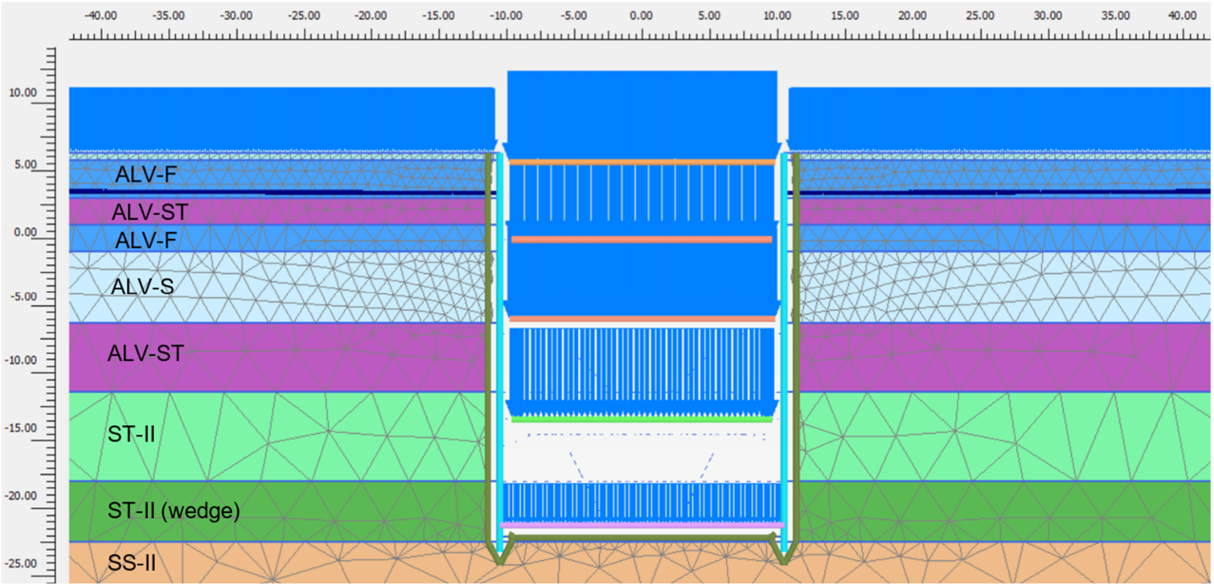


Figure 5-2 Typical connectivity plot for short sections

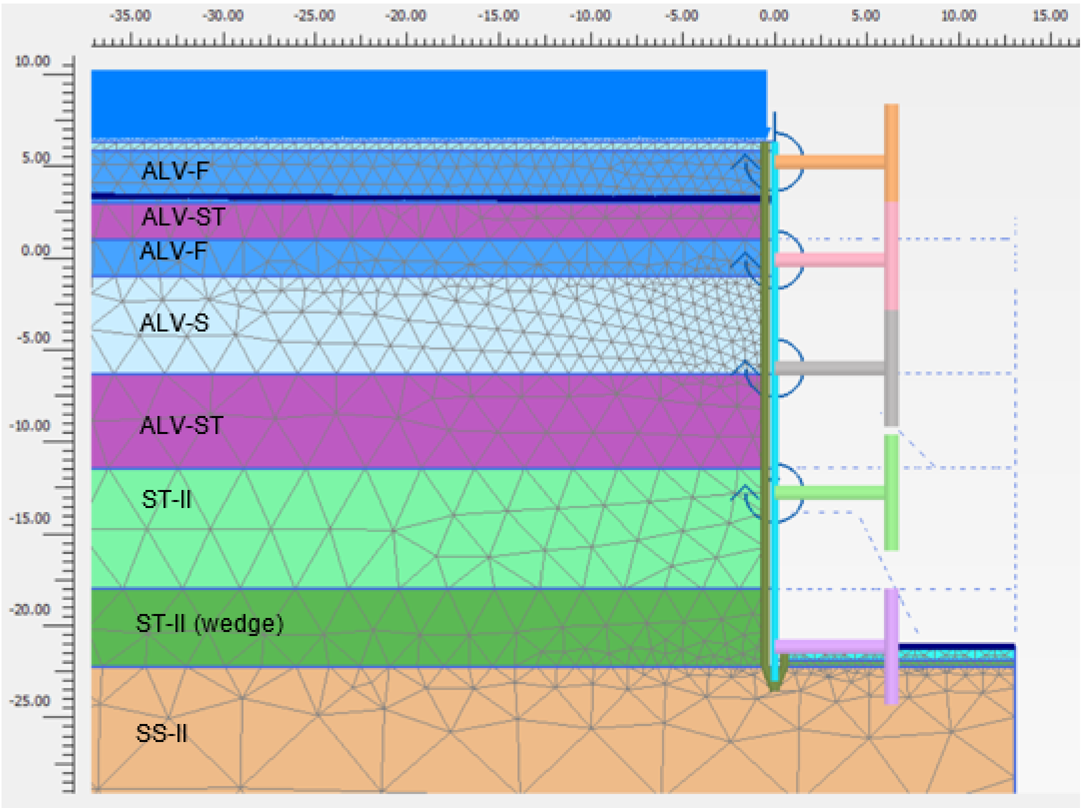


Figure 5-3 Typical connectivity plot for headwall sections

5.1.1 Structural properties

The structural elements considered in the PLAXIS model includes the followings:

- 1000mm thick diaphragm wall
- Precast struts with reinforced concrete topping
- Base slab

The properties for structural elements adopted in PLAXIS analysis for various sections are tabulated in Table 5.1 to Table 5.4.

Table 5.1 Structural input for East and West Sections

Component	Reduced Level (RL m AHD)	Structure Details	PLAXIS Element	Effective Spacing (m)	Axial Stiffness, EA (kN/m)	Bending Stiffness, EI (kNm^2/m)
Diaphragm Wall	-	1000mm Thick	Plate	-	3.48 X 10^7	2.90 X 10^6
G Strut	5.63	2 nos. 900mm (D) X 710mm (W) Precast Strut + 450mm Topping	Plate	9.63	6.12 X 10^6	1.04 X 10^6
B1 Strut	-0.10	2 nos. 900mm (D) X 710mm (W) Precast Strut + 900mm Topping	Plate	9.63	8.32 X 10^6	2.35 X 10^6
B2 Strut	-6.00	2 nos. 900mm (D) X 710mm (W) Precast Strut + 900mm Topping	Plate	9.63	8.32 X 10^6	2.35 X 10^6
B3 Strut	-13.40	2 nos. 900mm (D) X 710mm (W) Precast Strut + 1 no. 900mm X 600mm Precast Strut + 1100mm Topping	Plate	14.43	8.92 X 10^6	3.05 X 10^6
Base Slab	-21.97 (East) / -21.25 (West)	750mm Thick RC Slab	Plate	-	2.46 X 10^7	1.15 X 10^6

Table 5.2 Structural input for Middle Section

Component	Reduced Level (RL m AHD)	Structure Details	PLAXIS Element	Effective Spacing (m)	Axial Stiffness, EA (kN/m)	Bending Stiffness, EI (kNm^2/m)
Diaphragm Wall	-	1000mm Thick	Plate	-	3.48 X 10^7	2.90 X 10^6
G Strut	5.63	2 nos. 900mm (D) X 710mm (W) Precast Strut + 450mm Topping	Plate	14.80	3.98 X 10^6	6.76 X 10^5
B1 Strut	-0.10	2 nos. 900mm (D) X 710mm (W) Precast Strut + 900mm Topping	Plate	14.80	5.42 X 10^6	1.53 X 10^6
B2 Strut	-6.00	2 nos. 900mm (D) X 710mm (W) Precast Strut + 900mm Topping	Plate	14.80	5.42 X 10^6	1.53 X 10^6
B3 Strut	-13.40	2 nos. 900mm (D) X 710mm (W) Precast Strut + 1 no. 900mm X 600mm Precast Strut + 1100mm Topping	Plate	14.80	8.70 X 10^6	2.97 X 10^6
Base Slab	-21.7	600mm Thick RC Slab	Plate	-	1.97 X 10^7	5.90 X 10^5

Table 5.3 Structural input for East Headwall Section

Component	Reduced Level (RL m AHD)	Structure Details	PLAXIS Element	Effective Spacing (m)	Axial Stiffness, EA (kN/m)	Bending Stiffness, EI (kNm^2/m)
Diaphragm Wall	-	1000mm Thick	Plate	-	3.48×10^7	2.90×10^6
Capping Beam	5.225	2150mm (D) X 2000mm (W)	Fixed End Anchor	22.0	1.28×10^7	-
B1 Waler Beam	-0.10	1800mm (D) X 2000mm (W)	Fixed End Anchor	18.0	1.39×10^7	-
B2 Waler Beam	-6.00	1800mm (D) X 2000mm (W)	Fixed End Anchor	18.0	1.46×10^7	-
B3 Waler Beam	-13.40	2000mm (D) X 2500mm (W)	Fixed End Anchor	17.5	1.99×10^7	-
Base Slab	-21.97	750mm Thick RC Slab	Fixed End Anchor	-	2.46×10^7	-

Table 5.4 Structural input for West Headwall Section

Component	Reduced Level (RL m AHD)	Structure Details	PLAXIS Element	Effective Spacing (m)	Axial Stiffness, EA (kN/m)	Bending Stiffness, EI (kNm^2/m)
Diaphragm Wall	-	1000mm Thick	Plate	-	3.48×10^7	2.90×10^6
Capping Beam	5.225	2150mm (D) X 2000mm (W)	Fixed End Anchor	22.0	1.28×10^7	-
B1 Waler Beam	-0.10	1800mm (D) X 2000mm (W)	Fixed End Anchor	18.0	1.39×10^7	-
B2 Waler Beam	-6.00	1800mm (D) X 2000mm (W)	Fixed End Anchor	18.0	1.46×10^7	-
B3 Waler Beam	-12.77	1940mm (D) X 2500mm (W)	Fixed End Anchor	17.5	1.93×10^7	-
Base Slab	-21.13	750mm Thick RC Slab	Fixed End Anchor	-	2.46×10^7	-

5.1.2 Ground model

Table 5.5 provides a summary of the simplified stratigraphy at the eastern, middle and western sections that have been used in the PLAXIS models. The geotechnical design parameters derived from site specific geotechnical investigation are presented in Table 4.1 and

Table 4.2.

Table 5.5 Summary of simplified stratigraphy

Material	Reduced Level (m AHD)		
	Eastern Section (Sydney CBD)	Middle Section	Western Section (Westmead)
ALV – F	+5.2 to +3	+5.2 to +3	+5.8 to +3
ALV-ST	+3.0 to +1.0	+3.0 to +1.0	+3.0 to +1.0
ALV – F	+1.0 to -3.0	+1.0 to -3.0	+1.0 to -1.0

Material	Reduced Level (m AHD)		
	Eastern Section (Sydney CBD)	Middle Section	Western Section (Westmead)
ALV – S	-3.0 to -6.0	-3.0 to -6.0	-1.0 to -6.3
ALV – ST	-6.0 to -11.2	-6.0 to -11.2	-6.3 to -11.4
SLT II	-11.2 to -24.7	-11.2 to -23.7	-11.4 to -18.0
SS II	-24.7 and below	-23.7 and below	-22.2 and below

5.1.3 Construction sequence

The construction sequence adopted in PLAXIS analysis for both short sections and headwalls are summarised in Table 5.6 and Table 5.7.

Table 5.6 Construction Sequence for Short Section (East, Middle and West Sections)

Stage	Description
0	Existing Ground Level RL+5.2m AHD (East - Sydney CBD) / RL+5.8m AHD (West - Westmead) & Groundwater Table at RL+3.6m AHD. Install 1m thick diaphragm wall to RL+4.15m AHD. Apply surcharge 20 kPa on existing ground level and cast capping beam to RL+6.3 AHD. 70% EI and 100% EA to be applied for diaphragm wall. 100%EI and 100%EA to be applied for struts and base slab.
1	Excavate flat to RL+2.5m AHD.
2	Install G Strut at RL+5.63m AHD (2 Nos 710 x 900, 450 mm Topping). Excavate flat to RL-1.2m.
3	Install B1 Strut at RL-0.1m AHD (2 Nos 710 x 900, 900mm Topping). Excavate to RL-7.1m AHD, 4.0m width passive berm and 1(Vertical):1(Horizontal) slope down to RL-11.4m AHD.
4	Install B2 Strut at RL-6.0m AHD (2 Nos 710 x 900, 900mm Topping). Excavate to RL-14.5m AHD, 4.5m width passive berm and 2(Vertical):1(Horizontal) slope down to RL-20.4m AHD.
5	Install B3 Strut at RL-13.4m AHD (2 Nos 710 x 900 + 1 No. 600 x 900, 1100mm Topping). Excavate flat to base at RL-22.745m AHD (East) / RL-22.4m AHD (Mid) / RL-22.03m AHD (West).
6	Lay Drainage Layer and Cast Drained Base Slab to RL-21.59m AHD (East) / RL-21.40m AHD (Middle) / RL-20.88m AHD (West). The Drained Base Slab thickness is 0.6m (Middle) and 0.75m (East and West).
7	Change Undrained to Drained Soil Properties.
8	Backfill EGL to RL+6.6m AHD (20 kPa is applied at new ground level). (Note 1)
9	Activate Floor Slab Loading with UDL of 28 kPa (G to B3) & 13 kPa (Base Slab).
10	Reduce Concrete Modulus, E to 50% (diaphragm wall, Strut & Slab).
11	Apply Groundwater at RL+4.0 m AHD.
12	Carry out steady state groundwater flow.
13	Carry out consolidation analysis to 90% degree of construction.

Note 1: Backfill EGL to RL +6.3m AHD for Middle and West Sections for this submission. Analysis will be updated in next design submission. Based on output results, there are not the critical sections.

Table 5.7 Construction Sequence for Headwall Section (East and West Sections)

Stage	Description
0	Existing Ground Level RL+5.2m AHD (East - Sydney CBD) / RL+5.8m AHD (West - Westmead) & Groundwater Table RL+3.6m AHD. Install 1m thick Diaphragm wall to RL+4.80m AHD. Apply surcharge 20 kPa and cast capping beam to RL+6.3m AHD. 70% EI and 100% EA to be applied for Diaphragm wall. 100% EI and 100% EA to be applied for struts and base slab.
1	Excavate flat to RL+2.5m AHD.
2	Install Capping Beam at RL+5.225m AHD. Excavate flat to RL-1.2m AHD.
3	Install B1 Waler at RL-0.1m AHD. Excavate to RL -7.1m AHD, 5.5m passive berm and 1:1 slope down to RL-11.4m AHD.
4	Install B2 Waler at RL -6.0m AHD. Excavate to RL -14.5m AHD, 5.5m passive berm and 2:1 slope down to RL -20.4m AHD.
5	Install B3 Waler at RL-13.4m AHD (East) / RL-12.77m AHD (West). Excavate flat to base at RL-22.745 m AHD (East) / RL-21.91m AHD (West).
6	Lay Drainage Layer and Cast Drained Base Slab to RL-21.59m AHD (East) / RL-20.756m AHD (West). The Drained Base Slab thickness is 0.75m.
7	Change Undrained to Drained Properties.
8	Backfill EGL to RL 6.6 m AHD (20 kPa is applied at new ground level).
9	Activate Floor Slab loading with UDL of 28 kPa (G to B3) & 13 kPa (Base Slab).
10	Reduce Concrete Modulus, E to 50% (Diaphragm wall, Strut & Slab).
11	Apply Groundwater at RL=4.0m AHD.
12	Carry out steady state groundwater flow.
13	Carry out consolidation analysis to 90% degree of construction.

Summary of input parameters in PLAXIS model of the East Section is attached in Attachment 6.

5.2 Analysis output

Based on the analysis detailed in Section 5.1, the summary results and output envelopes are attached as Attachment 6.

5.3 Sensitivity Check for Flood Condition

Additional sensitivity analysis has been carried out to assess the effect of flood to the retaining system at East Section. The analysis has been carried out with the following conditions:

- Highest flood level is at RL+6.667m (reference: Appendix D6).
- No live load is applied at ground level.
- Flood occurred after completion on permanent structure.

Based on the PLAXIS result, the wall deflection, bending moment and strut force are not critical compared to design adopted. A summary comparing the results is attached in Attachment 6.

5.4 Analysis for Crane Load Check

During construction, heavy crane will be deployed for heavy lifting. The furnished crane base dimension, bearing pressure and temporary platform base are as follows:

- Bearing pressure from crane base: 260 kPa
- Track width of crane: 1100mm
- Distance of track from wall: 2.0m
- A 150mm thick hardstand is considered to distribute the crane pressure.

Since the crane will be deployed during construction stage, the analysis will consider this loading until final stage of excavation. This additional model has been undertaken at East Section. Based on the PLAXIS result, the impact of crane loading is minimum with slightly increase of:

- 3% (12 kPa) increase in G Strut axial force for level G Strut.
- 5% (16 kNm) increase in G Strut bending moment for level G Strut.

The redundant capacity of level G Strut is adequate to cater for this additional load. A summary of the result is attached in Attachment 6.

6.0 Design analysis of the temporary excavation

6.1 Data assumptions and analysis input

The undrained analysis of the temporary excavation was modelled into the Geostudio 2021 SLOPE/W software using limit equilibrium Morgenstern-Price method. The model was split into a long-section and a short-section of the temporary excavation. The soil profile and parameters were based on the Western End (Westmead) with thicker layer of soft alluvium. The summary of the data assumptions and analysis inputs are similar to those listed in Table 5.5.

TfNSW (RMS) PS331 of recent highway projects have been used in assessing the stability of unsupported slope. A maximum Geotechnical Strength Reduction Factor (fg) of 0.83, equivalent of minimum factor of safety (FOS) of 1.20 may be deemed to be satisfactory for intermediate construction stages. The minimum acceptance criteria includes the consideration of the nature, extent and duration of the immediate stages, the consequence of failure, details and extent of risk management with contingency plan, degree of emergency and other factors.

A surcharge load of 20 kPa was imposed at the top of the berm for every long-section analysis to represent the weight of the machineries during construction. The short section has considered the self-weight of the concrete waler/ concrete strut resting on berm and temporary walkaway attached to precast strut. The self-weight of the concrete struts at every level will be distributed differently on top of bench. The groundwater level was assumed to be 0.5 m below ground level at every stage of excavation. Table 6.1 shows the calculation for the short-section surcharge loads at basement B2 and B3 strut levels.

Table 6.1 Summary of Estimated Surcharge Load for Each Stage.

Stage	Waler Dimension (W x D)	Waler Bearing Pressure (kPa)	Weight of Strut + Temporary Walkaway (kN)	Strut Weight Distribution Area (W x B)	Strut Bearing Pressure (kPa)
Stage 5 Basement B2	2.0m × 1.8m	45	400 + 176 (without concrete topping)	1.0m x 2.5m	115
Stage 7 Basement B3	2.5m × 2.0m	63	1,388 + 176 (with concrete topping)	2.0m x 2.06m	190

6.2 Analysis output

The geometry of benches defined in the sequence of excavation works has been adopted for stability analysis. A total of five (5) sections have been analysed. The excavation stages are summarised in Table 6.2 for the longitudinal-section and Table 6.3 for the short-section. The factor of safety for each section achieved the minimum requirement of 1.2 for temporary slope during construction stages. Refer to Attachment 7 for slope stability analysis outputs and drawing SMWSTWTP-GLO-RSH-SF500-RS-DRG-010110 and 010111 for options and stage numbers.

Table 6.2 Summary of Analysis Result for Longitudinal Section

Stages	Top of Berm (m AHD)	Bottom of Berm (m AHD)	Slope Gradient	Surcharge Load (kPa)	Factor of Safety
Stage 5 Excavate to B2	+5.8	-1.15	1.0(V):2.0(H)	20 kPa with 5m surcharge exclusion zone from crest at RL+5.8m	Global: 1.38
Stage 7 Excavate to B3	-1.15	-7.0	1.0(V):2.0(H)	20 kPa with 5m surcharge exclusion zone from crest at RL-1.15m	Global: > 3 Local: > 1.75
	-7.0	-11.4	1.0(V):1.5(H)		
Stage 9 Excavate to B3	-7.1	-11.4	1.0(V):1.5(H)	20	Global: > 5 Local: > 5
	-11.4	-20.4	2.0(V):1.0(H)		
Stage 11 Excavate to Base	-14.5	-22.6	2.0(V):1.0(H)	20	Global: > 5

Table 6.3 Summary of Analysis Result for Short Section

Stages	Top of Berm (mAHD)	Bottom of Berm (mAHD)	Slope Gradient (V:H)	Factor of Safety
Stage 5 Basement B2	-7.1	-11.4	1.0(V):1.0(H)	Global: 2.92 Local: 3.14
Stage 7 Basement B3	-14.5	-20.4	2.0V):1.0(H)	Global: > 5 Local: > 5

7.0 Settlement due to proposed excavation and groundwater drawdown

7.1 Data assumptions and analysis input

As discussed in Section 5.1, a total of five (5) sections were analysed using PLAXIS. For the purpose of predicting the settlement at each section, the combined effect of the following assumptions have been included:

- Wall lateral movement due to the construction
- Applied 20 kPa vertical surcharge (35 m wide – perpendicular to the wall)
- Extra surcharge due to future backfill to RL +6.3 m AHD (35 m wide – perpendicular to the wall)
- Increased in effective stress due to water drawdown

- Horizontal and vertical permeabilities adopted based on the “Baseline K” scenario – Refer to the Technical Memo - Hydrogeology Report

Since the excavation depth is about 28.0 m measured from the existing ground level, the assumption of 35 m wide vertical load and backfill is considered reasonable as it extends beyond the theoretical 1V:1H influence zone which is the area of interest for the effect of the excavation and construction activities. In addition, the project boundary on the north western side of the Rosehill Service Facility (where the proposed Sydney Water utility will be located) is expected to be approximately 35 m away from the service facility.

It should be noted that localised soft to firm thin and localised layer of alluviums can be encountered in this site but they are not expected to impact the total settlement. The CPT tests result on alluvium indicates a minimum undrained shear strength of 50 kPa corresponding to the transition from firm to stiff layer with a minimum OCR value of 2.5. Consolidation settlement will be re-evaluated upon receipt of laboratory tests results and potential impact assessment will be updated if there is a variation to the current interpretation.

In addition, the predicted settlements discussed in Section 7.2 are based on future backfill to RL +6.3 m AHD, during the project design development, the future backfill has been increased to RL +6.6 m AHD. The settlement will be re-evaluated in next design submission but the effect due to this minor change is considered to be minimal.

7.2 Analysis output

The estimated settlement of the existing ground surface at the final stage of the analysis (i.e. after Stage 13) as listed in Section 5.1.3 are shown in Figure 7-1 to Figure 7-3.

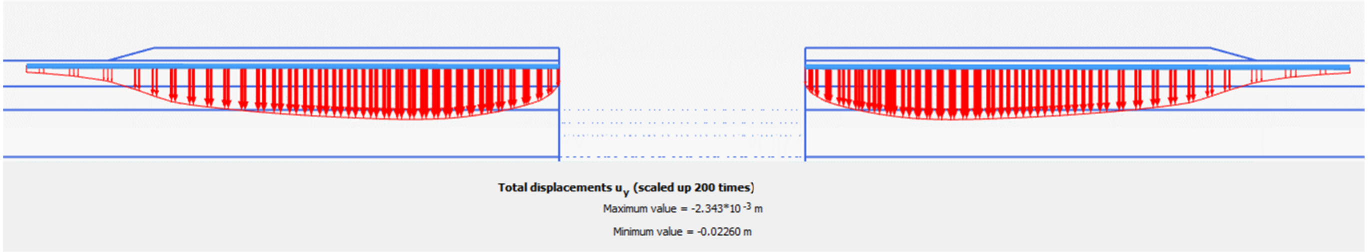


Figure 7-1 Settlement in the final stage for East Section

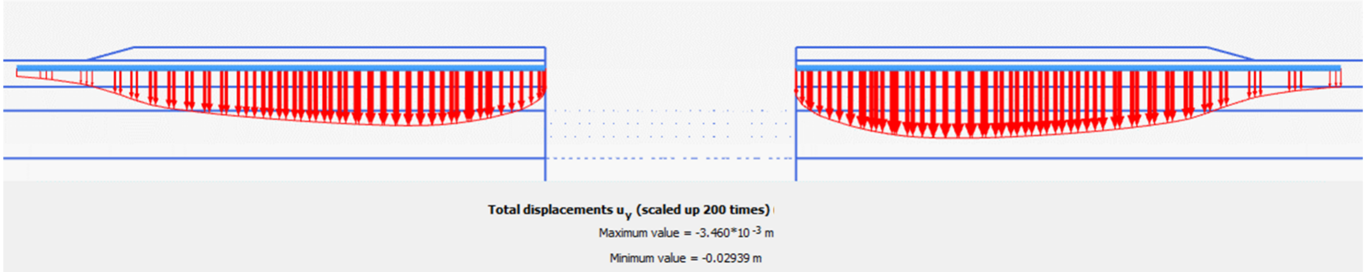


Figure 7-2 Settlement in the final stage for Middle Section

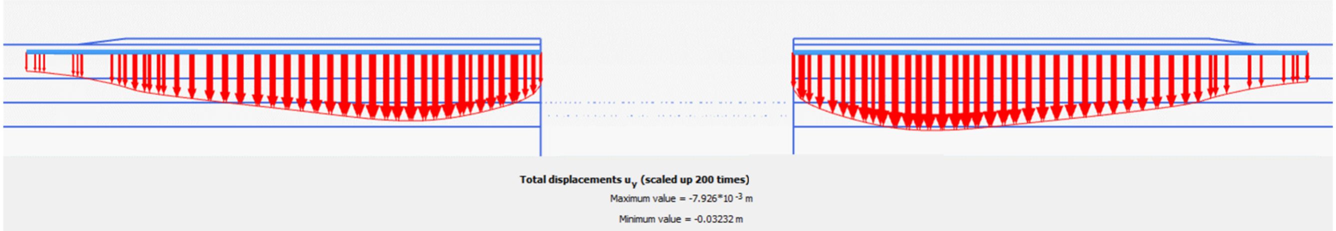


Figure 7-3 Settlement in the final stage for West Section

The predicted settlement contours surrounding the service facility is shown in drawing SMWSTWTP-GLO-RSH-SF500-RS-DRG-010116. The settlement contours have been prepared based on a sensitivity check for each section using the constant head boundaries described in the Hydrogeological Technical Report in Appendix D2. The groundwater drawdown profiles from Slide2 model has been extracted and imported into PLAXIS model in the final stage based on which the settlement contour in drawing SMWSTWTP-GLO-RSH-SF500-RS-DRG-010116 has been prepared.

The contours are prepared manually with engineering adjustment to interpolate the estimated settlement between the sections as presented in Section 7.2. Reduction of ground movements around corners of excavations due to the increased stiffness of retaining wall at the corner of an excavation has been considered (Fuentes and Devriendt, 2010).

As discussed in Section 7.1, the predicted settlement at each section is due to the combined effect of wall lateral movement, applied surcharge, extra surcharge due to future backfill and increased effective stress due to groundwater drawdown.

7.3 Impact assessment

Based on the information provided on Design Utilities as part of the Sydney Metro West – Western Tunnelling package, there is a temporary sewer diversion required to enable decommissioning of sewer prior to excavation (Turnbull, 2021a). The location is as shown in Figure 7-4. The temporary sewer line will be a polyethylene (PE) temporary sewer diversion around the Rosehill Service Facility with the acceptance tensile and compression strain of 1:2500 microstrain (Turnbull, 2021b).

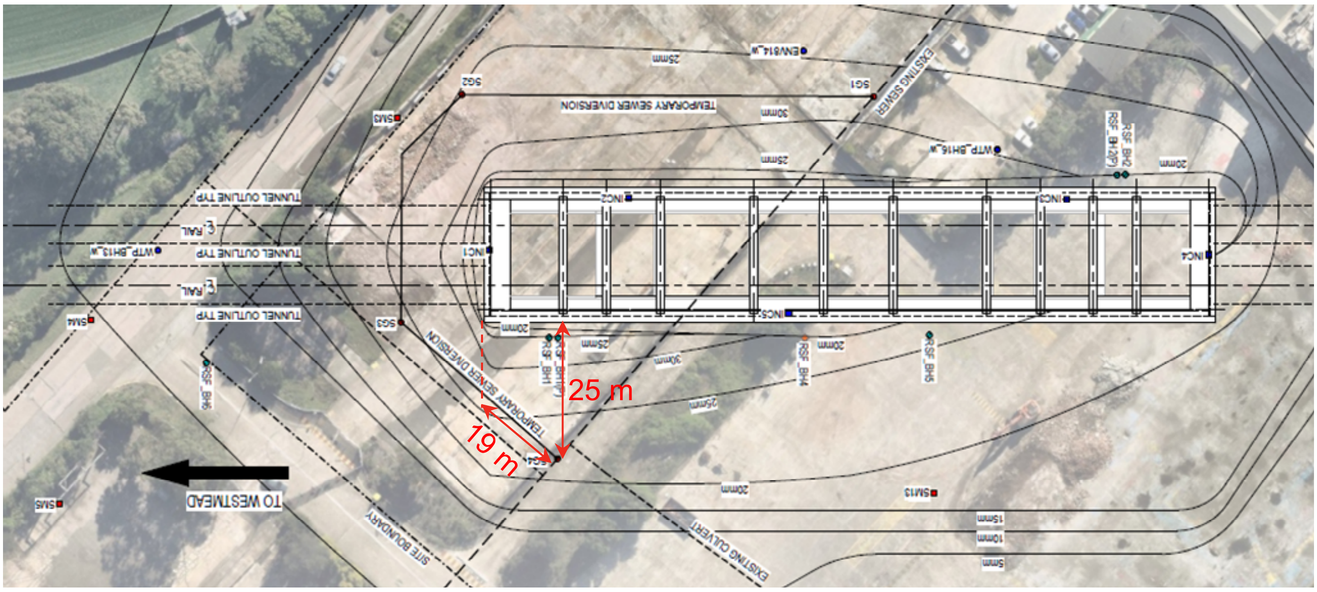


Figure 7-4 Temporary sewer diversion location

The temporary sewer pipe has not been modelled in PLAXIS explicitly, so it is assumed that soil lateral movement at the proposed location of the temporary sewer pipe is the same as the pipe. This is a conservative assumption as pipe movement is expected to be less than soil due to the soil-pipe interface.

The most critical section is where the highest differential displacement/movement occurs over the shortest length of the pipe. The critical section is expected to be in the western side of Rosehill Service Facility where differential settlement of about 5 mm occurs in the existing sewer over the length of about 19 m and the pipe is semi-perpendicular to the excavation face resulting in higher tension strain due to differential horizontal movement (pipe is parallel to the excavation face at most of its length). Analytical calculations have been done as follow based on the horizontal and vertical movement at both ends of the pipe as shown in Figure 7-5. As shown, the strain is expected to be less than 1:2500 microstrain.

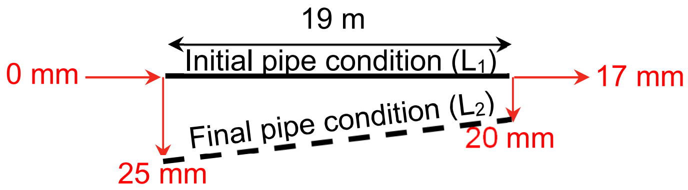


Figure 7-5 Expected horizontal and vertical movements at both ends of the pipe at the most critical section

$$L_1 = 19{,}000mm$$

$$L_2 = \sqrt{(19{,}000 + 17)^2 + (25 - 20)^2} = 19{,}017mm$$

$$\Delta L = L_2 - L_1 = 17mm$$

$$\varepsilon = \frac{\Delta L}{L_1} = 8.9 \times 10^{-4}\mathrm{s}$$

$$\varepsilon = 894\mu\mathrm{s} < 2500\mu\mathrm{s}$$

The above closed-form solution is reasonably accurate, although conservative in that it assumes soil movement is the same as pipe movement. Due to the interface/interaction between the pipe and soil as well as pipe connections at different locations, pipe movement is expected to be less than soil movement. It should be note that there is limitation in the accuracy of the prediction in the sewer strain due to pipe location accuracy, different groundwater level due to seasonal variation, local ground variation around the pipe, accuracy limitation of groundwater drawdown which based on analytical modelling in 2D flow analysis, and accuracy limitation of the monitoring survey measurement.

As advised by our interface Contractor Turnbull Engineering, there is a gas main along the south side of Unwin Street which is expected to be decommissioned and relocated as part of the early works in July 2022. At this stage it is assumed that the relocation finishes before excavation commences, that notwithstanding we have nominated surface monitoring locations along Unwin Street which would detect adverse settlements in the event the decommissioning is delayed.

There is a heritage façade wall along Unwin Street to the west of the site which needs to be protected as a condition of the Ministers Conditions of Approval (MCOA). The wall is located outside the influence zone of the Rosehill Service Facility excavation. Based on the estimated ground settlement, it is expected the impact on the wall from the Rosehill Service Facility excavation is minimal, to ensure the MCOA are adhered to, monitoring of the Façade has been proposed as part of the monitoring regime project wise. The cumulative effects of the tunnel and box will be documented in the project wide Predicted Effects Report.

8.0 Limitations

The following limitations applied to the geotechnical interpretation and design:

1. The proposed geotechnical models are based on the geotechnical and environmental boreholes carried out within the proposed Rosehill Service Facility. No CPTs or laboratory tests on the strength, stiffness and compressibility of the alluvium material have been conducted. Additionally, no pressuremeter test data (in-situ locked-in stress) is available for the Rosehill Service Facility. The variation in the underground profile and material properties (strength, deformation and permeability) remains a risk as insufficient data was available at the time of geotechnical interpretation for preparation of the ground model.

2. Based on the provided SMW-GIR, a potential dyke is inferred at the western end (Westmead) of the excavation which could result in increased groundwater inflow. The dyke was not intersected in available investigations to date but is reported to be approximately 2m in width.
3. While the borehole logs mostly indicate cohesive alluviums, there is a risk associated with the existence of granular alluvial layers since these materials have higher permeability which can cause more water drawdown, hence more settlement and flow rate.
4. While the packer test results on sandstone indicates a permeability of about 0.002 m/day (i.e. 2.3e-8 m/s), there is a risk associated with possible higher permeability of sandstone.

9.0 Recommendations

9.1 Additional geotechnical investigation

The geotechnical site investigations carried out to date are shown in Figure 1 in Attachment 1. As discussed above in Section 8.0, there is design limitations/ risks associated with either insufficient geotechnical investigations or from discrepancies among the geotechnical investigation results at the Rosehill Service Facility, some of which are listed below:

5. Very limited laboratory tests with regards to the assessment of the strength or stiffness of the alluvium have been carried out.
6. There are discrepancies between the Standard Penetration Tests (SPT) and Hand Pocket Penetrometer (HP) values carried out on the cohesive alluvium.
7. Only discrete data from the BHs are available through SPT and HP.
8. No Cone Penetrometer Tests (CPT) have been carried out to allow continuous characterisation of the variation of geoengineering parameters of the ground profile.
9. Nearly all the monitoring wells have all been installed in the alluvium.
10. Only one monitoring wells have been installed in the rock beneath the alluvium to target the groundwater level in the rock.
11. The presence of the Dyke is currently inferred only and has not been proven through geotechnical investigations.
12. No field validation of the orientation and magnitude of the locked-in stress.

A list of additional geotechnical investigations has been undertaken to cover the data gap and to valid the design assumptions. The proposed locations of the additional geotechnical investigations shown in Figure 6 in Attachment 8 and a summary table of schedule is provided in Attachment 8.

The main purpose of these additional tests is to have a better understanding of the design parameters for the alluvium at the Rosehill Service Facility, the groundwater level, piezometer head and conductivity in the rock, as well as the presence of the dyke.

Only preliminary borehole logs and field tests records are available for this submission. Laboratory test results and detail interpretation will be included in next design submission.

9.2 Instrumentation and monitoring

This section of the technical memo identifies and describes the procedures involved in the implementation of geotechnical instrumentation and monitoring system. Monitoring is undertaken to measure the performance and stability of the retaining system during excavation as well as to check the surrounding ground at each stage of construction.

The monitoring systems are divided into 2 main categories; to monitor the impact on adjacent structures/utilities and for design verification of excavation work. The proposed instruments and primary function of the monitoring are tabulated in Table 9.1. The summary and descriptions with the estimated total number of instruments to be installed is provided in the subsequent sections. Refer to drawings SMWSTWTP-GLO-RSH-SF500-RS-DRG-010115, 010116, 010117 and 010118 for details.

Table 9.1 Instrumentation Plan

Monitoring Category	Instruments	Location	Function
Impact on Adjacent structures/Utilities	• Optical Prism • Utility Settlement Gauge • Surface Settlement Marker	Façade all, sewer, road and underground utilities within the influence zone of the excavation area.	To monitor and determine the impact of excavation work to the surrounding area, utilities, and nearby buildings.
Design Verification	• Inclinometer • Vibrating Wire Strain Gauge • Standpipe Piezometer • Vibrating Wire Piezometer	Within the diaphragm wall and on the associated struts as well as within the influence zone adjacent to the diaphragm wall.	To verify and justify the design assumptions made for excavation during designs stage.

A general description of the proposed instruments are as below:

a) Surface Settlement Marker

The surface settlement marker is used to monitor the surface movement surrounding the excavation area. Generally, it is installed on the ground with protection cap. Thirteen (13) nos. of surface settlement markers will be installed below existing concrete slab and on the road premix along Unwin Street to monitor ground settlement.

b) Wall Inclinometer

The inclinometer is used to monitor lateral deflection of diaphragm wall. It is installed inside the diaphragm wall with base socketed into rock. Five (5) nos. of inclinometers shall be installed, three (3) on the longitudinal walls and two (2) at headwalls.

c) Standpipe Piezometer

Standpipe Piezometer is used to monitor the ground water levels. Eleven (11) nos. of existing standpipe piezometers and five (5) nos. of proposed additional standpipe piezometers will be used for the monitoring work and groundwater sampling.

d) Vibrating Wire Piezometer

The vibrating wire piezometer provides the water pressure measurement at three specific depths within the soil profile. It is used as part of a system for early detection of change in water pressure during excavation in Alluvium, Siltstone and Sandstone. The instrument shall be installed in WTP_BH18 (existing borehole) and RSF-BH4 (additional ground investigation point) for the monitoring work.

e) Vibrating Wire Stain Gauges

The vibrating wire strain gauges are used to monitor the lateral forces in concrete struts. Eight (8) pairs of strain gauge are attached to the concrete strut surface of level ground floor, B1, B2 and B3 struts.

f) Utilities Settlement Gauge

The utilities settlement gauge is used to measure the localised settlement or heave of the underground utilities. Four (4) nos. of utilities settlement gauges which consist of round steel pipes will be installed on top of the temporary sewer pipe for the monitoring work.

g) Optical Prism

Precise survey using optical prism has been proposed to monitor the façade wall fronting Unwin Street. A pair of prism will be mounted on this structure at 1.5m vertical interval to monitor the settlement and verticality during excavation works. Four (4) locations have been proposed along the façade wall. If there is new crack occurred during the excavation works, crack meter shall be installed to monitor the condition of the crack.

9.2.1 Monitoring Frequency and Proposed Location

The monitoring frequency shall be varied in accordance with the responses of instruments used and construction activities. The recommended monitoring frequency for various type of instruments are as per Table 9.2 below:

Table 9.2 Instrumentation Monitoring Frequency and Installation Location

Instruments	Installation Location	Monitoring Frequency		
		Prior to Excavation	During Excavation	After the completion of Base Slab
Surface Settlement Marker	Around the excavation zone and along Unwin Street	Weekly	Daily	Weekly
Wall Inclinometer	Within the Diaphragm wall and extended 2m below toe	Weekly	Daily	Weekly
Standpipe Piezometer	Around the excavation zone	Real Time	Real Time	Real Time
Vibrating Wire Piezometer	WTP_BH18 and RSF-BH4	Real Time	Real Time	Real Time
Vibrating Wire Strain Gauge	Concrete strut	NA	Real Time	Real Time
Utilities Settlement Gauge	Temporary Sewer	Weekly	Daily	Weekly
Optical Prism	Façade Wall along Unwin Street	Weekly	Daily	Weekly

Prior to commencement of monitoring work, at least three (3) consistent readings shall be taken continuously to establish the base reading. Closer monitoring frequency may require if the readings show inconsistent trend. Monitoring program shall be terminated after three (3) months of stable readings following the completion of construction works and handed over to permanent works contractor.

9.2.2 Instrumentation Review Level

Trigger levels and suggestive actions will need to be established prior to construction. There will be three (3) review levels to be set and known as Alert level, Action level and Alarm level. With reference to Technical Direction – Excavation Adjacent to Transport for NSW Infrastructure GTD 2020/001 Version No. 01 - 2 July 2020, the trigger value for each review level has been pre-set at 70%, 80% and 100% of predicted value.

For each design value a 'trigger' level is typically assigned, which if reached during the excavation or construction, should initiate immediate action. Suggested trigger levels and response strategies (suggestive action) are provided in the in the Table 9.3. Suggested trigger values are included in drawing, SMWSTWTP-GLO-RSH-SF500-RS-DRG-010118.

Table 9.3 Trigger Levels

Trigger Level	Trigger Value (% of the Predicted Value)	Procedures /Requirement
Alert	70 - 80	1) Notify the designer immediately with description of related works in the vicinity of the instrument. 2) Instrument Reading confirmation on reliability i.e. not related to instrument and human errors or abnormalities, and to rectify the causes of the erroneous readings, if any. 3) Conduct a visual inspection of the affected building, structure, and related works. 4) Initiate relevant trigger action plan. 5) Monitoring frequency and applicability of trigger levels to be reviewed.
Action	> 80	1) Notify the designer immediately with description of related works in the vicinity of the instrument. 2) Review frequency and applicability of trigger levels. if necessary, install additional instruments or increase the frequency of monitoring. 3) Initiate relevant trigger action plan including reporting procedure. 4) Discussion held between the relevant parties (i.e. designer, contractor and other stakeholders) to establish the next step forward and appropriate response to the alert.
Alarm	> 100	1) Suspend all concerned works within the agreed zone of danger. 2) Notify the designer immediately with description of related works in the vicinity of the instrument. 3) Conduct a joint inspection of the affected building / structure and related works with the designer. 4) Initiate relevant trigger action plan including reporting procedure and contingency plan. 5) Review monitoring frequency and applicability of trigger levels. 6) Recommence the affected works upon demonstrating to the designer that it is safe to do so and upon agreement with all relevant parties.

9.3 Construction supervision

Details of construction supervisions, witness and hold points have been included in the project diaphragm wall specification. It is recommended that a competent geotechnical engineer shall inspect the construction activities to:

- Assess the founding materials at the socket of the Diaphragm wall during trench excavation
- Assess excavated material during bulk excavation inside the Rosehill Service Facility for any signs of deviation from material types that were originally assumed in the design stage
- Inspect temporary cut batter slopes stability inside the Rosehill Service Facility

10.0 References

- Chapman, G. A., Murphy, C. L., Tille P. J., Atkinson, G. & Morse, R. J., 2009. Soil Landscapes of the Sydney 1:100 000 Sheet. Sydney: Soil Conservation Service of NSW.
- Colquhoun G.P., Hughes K.S., Deyssing L., Ballard J.C., Phillips G., Troesdon A.L., Folkes C.B. & Fitzherbert J.A. 2021. New South Wales Seamless Geology dataset, version 2.1 [Digital Dataset]. Geological Survey of New South Wales, NSW Department of Planning and Environment, Maitland.
- Fuentes, R., Devriendt, M., 2010; Ground Movements around Corners of Excavations: Empirical Calculation Method. Journal of Geotechnical and Geoenvironmental Engineering, ASCE.
- GHD and SMEC, 2021; Sydney Metro West – Western Tunnelling Package, Geotechnical Interpretive Report, dated 15 October 2021.
- Golder and Douglas Partners, 2022; Sydney Metro West. Geotechnical Data Report. 20446669-001-R-GDR-RevB, dated 10 February 2022.
- Herbert C., 1983, Sydney 1:100 000 Geological Sheet 9130, First Edition. Geological Survey of New South Wales, Sydney.
- Liao, Samson S. C., Whitman, Robert V., 1986. Overburden Correction Factors for SPT In Sand. Journal of Geotechnical Engineering, Vol. 112, No. 3.
- Office of Environment & Heritage NSW (OEH), 2019. Acid Sulphate Soil Elevations. Accessed via https://www.environment.nsw.gov.au/eSpade2Webapp#. Accessed 03 February 2022.
- Pells, P. J., Mostyn, G., & Walker, B. F. (1998). Foundations on Sandstone and Shale in the Sydney Region. *Australian Geomechanics, 33*(3).
- Pells, P. J., Mostyn, G., Bertuzzi, R., & Wong, P. K. (2019). Classification of Sandstone and Shales in the Sydney Region: A Forty Year Review. *Australian Geomechanics, 54*(2), 29-55.
- Pettifer, G. S., & Fookes, P. G. (1994). A revision of the graphical method for assessing the excavatability of rock. Quarterly Journal of Engineering Geology, 27(2), pp 145-164.
- Sydney Metro Authority, 2021; Sydney Metro West – Scoping & Definition Design Services, Geotechnical Interpretive Report – Westmead to The Bays ECI Design, Document Number: SMW_10-CCM-RW-ZZ-RP-GE-000001. Rev 7, dated 28 April 2021
- Turnbull, 2021a; Sydney Metro West – Western Tunnelling Package Tender. Tender Advice Note. TAN -UT2-Design Utilities. TAN No. SMSMW215-GALC-SWDSW000-UT-TAN-000002 Rev 2, dated 25 August 2021.
- Turnbull, 2021b; Sydney Metro West – Western Tunnelling Package Tender. Tender Advice Note. TAN -UT3-Predicted Effects. TAN No. SMSMW215-GALC-SWDSW000-UT-TAN-000003 Rev 2, dated 28 June 2021.

ATTACHMENT 2

Technical Report – Hydrogeological Design

Technical Report – Hydrogeological Design

Design Stage	Rosehill Service Facility - Stage 3
Author	Graham Hawkes (Stage 1/2), Kourosh Todeshkejoei (Stage 3), Aurecon
Reviewed	Hugo Acosta Martinez (Stage 1/2), Graham Hawkes (Stage 3), Aurecon
Date	7 July 2022

1.0 Purpose

This report presents the hydrogeological conditions and predicted groundwater impacts for the short term (construction staging) and long term operation of the Rosehill Service Facility – Stage 3 design resubmission which forms part of the early works for the Sydney Metro West Western Tunnelling Package.

2.0 Introduction

2.1 The purpose of this report

This report documents the followings:

- A seepage analysis has been conducted to estimate the total groundwater inflow for the option of a fully drained base.
- Impacts have been considered for the long-term operational phase.
- Groundwater drawdown has also been predicted in the long term and the implications assessed by applying two-dimensional analytical modelling.
- Similarly, groundwater drawdown has also been predicted in the short term and the implications assessed by applying two-dimensional analytical modelling. The outcome of this prediction has been used to determine the temporary excavation sequence.

The report refers to the Geotechnical Interpretative Report (GIR) and Durability Report presented as Appendix D1 and D4 respectively.

2.2 Project construction

This report relates to the construction of a full depth diaphragm wall (D-Wall).

The construction scheduling and timing of the D-Wall construction is currently under development and will be confirmed in next design stage based on the confirmation of alluvium permeability. It is understood that during the early construction, the eastern (Sydney CBD) longitudinal D-wall and eastern headwall are to be constructed first and socketed into the sandstone. The western (Westmead) D-wall and headwall of the excavation will remain open during construction while excavation of half eastern portion (Sydney CBD) undergoing until when the D-Walls will be completed.

During construction it is understood that there are no inflow criteria to be met other than what is deemed acceptable from a temporary water management perspective by the contractor. In contrast the project specifications for the long term groundwater inflow is less than 45,000 litres in any 24-hour period in order to be compliant with clause ref 4.1.8 (j) (under departures acceptance under RFC 8708).

2.3 Objectives

This report has been prepared to address the requirements of Condition D122 of Sydney Metro West – Concept and Stage 1 Conditions of Approval (Dept of Planning, 2021). To satisfy Condition D122 a revised groundwater modelling report in association with Stage 1 of the Critical State Significant Infrastructure (CSSI) is required to be submitted to the Planning Secretary for information before bulk excavation of the Rosehill Service Facility.

The requirements to be included in the report and where these requirements have been addressed in this report are presented in Table 2.1.

Table 2.1 Conditions addressed in this report

Condition	Where this is addressed in the report
(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.	Section 9.1
(b) Predicted incidental groundwater take (dewatering) including cumulative effects.	Section 9.2
(c) Potential impacts for all later 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.	Section 9.3
(d) Actions required after Stage 1 to minimise the risk of inflows (including in the event latter stages of the CSSI are developed or do not progress) and a strategy for accounting for any water taken beyond the life of the operation of the CSSI.	Section 9.4
(e) Saltwater intrusion modelling analysis, from estuarine and saline groundwater in shale, into the Bays metro station site and other relevant metro station sites; and	Section 9.5
(f) A schematic of the conceptual hydrogeological model	Section 3.6, 9.6

2.4 Construction sequencing

The construction staging is as follows:

- Stage 0: Install the eastern half (Sydney side) of the D-Wall.
- Stage 1: Excavate the eastern half (Sydney side) of the Rosehill box to RL +2.5 m AHD with 1V:2H batter slope.
- Stage 2: Excavate the eastern side (Sydney side) of the Rosehill box to RL +0.5 m AHD with 1V:2H batter slope.
- Stage 3: Install the cut-off wall at about 5 m behind the crest of the batter slope assuming 500 mm embedment into siltstone.
- Stage 4: Excavate the eastern side (Sydney side) of the Rosehill box to RL -1.15 m AHD with 1V:2H batter slope.
- Stage 5: Complete the D-Wall all around the Rosehill Service Facility.
- Stage 6: Remove the cut-off wall and excavate the whole are within the D-Wall to RL -1.15 m AHD.
- Stage 7 to Stage 12: Continue the excavation as per the details set out in Drawing SMWSTWTP-GLO-RSH-SF500-RS-DRG 010111.

Refer to the drawing "SMWSTWTP-GLO-RSH-SF500-RS-DRG-010110 & 010111" for the details of the geometry during construction staging.

3.0 Groundwater conceptual model review

The hydrogeological conceptual model has been derived from the available information. Parameters required for the groundwater modelling are presented and discussed.

3.1 Geological setting

The Rosehill Service Facility is underlain by fill and alluvium and the sub-horizontal sediments of the Triassic aged Sydney Basin sediments. As described in the geotechnical design report the geology at the site at depths (m AHD) is as follows:

- RL +6 to RL +5: fill
- RL +5 to RL -3 : firm to stiff alluvial soil
- RL -3 to RL -6: soft to firm alluvial soil
- RL -6 to TL -11: stiff alluvial soil
- RL -11 to RL -12.: weathered siltstone
- RL -12.0 to RL -25 (east) and RL-22 (west): slightly weathered to fresh siltstone
- Hawkesbury Sandstone underneath siltstone

An inferred dyke may be present in the western end of the Rosehill Service Facility but was not intersected by available borehole drilling as part of the site investigation.

3.2 Hydrogeology

Groundwater is present in the alluvial soil and lower hydrostratigraphic units. The natural groundwater level is approximately 1.5 m below ground level. Groundwater within the alluvial clay and Hawkesbury Sandstone is typically fresh to brackish whereas the groundwater within the siltstone is generally of poorer quality with a higher salinity.

The Rosehill Service facility is bounded by Duck Creek approximately 60 m to the south and the Parramatta River approximately one kilometre to the north. Consequently, to the north there is a groundwater divide between the Rosehill Service facility and the Parramatta River resulting in a relatively flat watertable. Groundwater at the site is currently be discharging into Duck Creek via horizontal flow.

The Hydrogeological Interpretative Report (HIR) (GHD and SMEC, 2021a) has calculated groundwater inflows from the siltstone and sandstone based on previous design assumptions and structural system. These inflows may not be relevant to the current stage of design.

3.3 Hydraulic conductivity

The hydraulic conductivity data provided is based on works conducted by SMEC and GHD as presented in the HIR (SMEC and GHD, 2021a) and is presented in Table 3-1 along with values of hydraulic conductivity values from WestConnex groundwater modelling as discussed below.

Table 3-1 Hydraulic conductivity values

Material	Kh (Most likely) (m/day)	Kh (WestConnex modelling*) (m/day)
Fill	0.08	Not provided
Alluvium	0.08	1
Siltstone	0.016	Included in sandstone
Sandstone	0.01	0.002 – 0.13

Note: * M4-M5 WestConnex (HydroSimulations, 2017)

Hydraulic conductivity values compiled in the HIR (SMEC and GHD, 2021a) are based on available site specific packer testing and slug test data. The hydraulic conductivity data is presented as follows:

- Upper case - 90th percentile value
- Most likely – average result
- Lower case – 20th percentile value

In comparison to modelling that was undertaken for the WestConnex road tunnels (HydroSimulations, 2017) within the Sydney Basin, the adopted parameters summarised in Table 3-1 appear reasonable. While the WestConnex M4-M5 Link project is some 20 km from the Rosehill Service Facility, the intersected Sydney Basin geology is similar. The alluvium and fill are site specific so development of hydraulic conductivity parameters relies on the available field data. The adopted value of 0.08 m/day is quite low for alluvium and fill however this may be due to poor connectivity within the fill and alluvium which may be due to a high clay content. The adopted hydraulic conductivity values for the siltstone and sandstone are within the values applied in the WestConnex modelling.

It is noted that additional packer tests were conducted after the HIR and GIR were completed and consequently not included in the summary statistics in these reports. The results collected in between July and September 2021 have been collated and are presented in Table 3-2.

Table 3-2 Summary of additional packer test results

Test Bore	Lithology	Test interval (m depth)	Lu	m/sec	m/day
BH13	Siltstone	17.5 - 23.5	10	0.000001	0.0864
	Siltstone/ Sandstone	23 - 29	0.1	1E-08	0.000864
BH14	Siltstone	18.15 - 24.15	9	9E-07	0.07776
	Sandstone	29.15 - 35.15	0.3	3E-08	0.002592
BH15	Siltstone	22 - 28	20	0.000002	0.1728
BH16	Siltstone	18.12 - 24.12	10	0.000001	0.0864
	Sandstone	29.26 - 35.26	0.3	3E-08	0.002592
BH17	Siltstone	24 - 30.3	6	6E-07	0.05184
	Siltstone/ Sandstone	31 - 37.45	0.8	8E-08	0.006912

The packer test results have been converted from lugeons (Lu) to hydraulic conductivity by the conversion of 1 Lu = 10^{-7} m/sec. Hydraulic conductivity is expressed in the units of m/day and m/sec. Thus, the averages of these results are:

- Siltstone 0.1 m/day
- Sandstone 0.002 m/day
- Sandstone + Siltstone 0.044 m/day

These packer test results are within the range for the WestConnex modelling. The results also show that the siltstone is considerably more permeable than the sandstone by two orders of magnitude.

3.4 Constant head boundary

A constant head boundary has been adopted at 150 m and 60 m from the western and eastern side of the Rosehill Service Facility, respectively to simulate recharge from Duck Creek. This approach is considered reasonable since the groundwater gradients are expected to flow from Duck Creek towards the Rosehill Service Facility once the excavation is constructed.

3.5 Kv/Kh ratio

A Kv/Kh ratio of 0.2 for the siltstone and sandstone has been considered as baseline value. This ratio is similar to other modelling undertaken elsewhere across the Sydney basin. The HIR states that in the Ashfield Shale Kv is likely to be an order of magnitude lower than Kh. Works undertaken by others as part of the WestConnex tunnel investigations indicates the Kv:Kh ratio within the Hawkesbury Sandstone can vary by up to two orders of magnitude (Hawkes, 2017). Modelling undertaken for the M4-M5 Link indicated that the Kv:Kh ratio varied between 0.5 to 0.001 (HydroSimulations, 2017). Similarly in groundwater modelling conducted for the F6 tunnels the Kv/Kh ratio also varied between 0.5 to 0.001 (RPS, 2018).

Thus, the Kv/Kh ratio of 0.2 adopted is within the accepted range.

3.6 Hydrogeological conceptual model

A schematic diagram of the hydrogeological conceptual model is shown as Figure 3.1. The conceptual model shows the Rosehill Service Facility excavated through the fill, alluvium, siltstone and Hawkesbury Sandstone with the D-Wall generally socketed 1.2 m into the Hawkesbury Sandstone. The water table level has been recorded varying between RL 3.0 m and RL 4.2m AHD. Recharge is via lateral groundwater movement and inflow from Duck Creek to the east. In the modelling recharge from Duck Creek has been simulated by the adoption of a constant head boundary (refer to Figure 3.1). Recharge via rainfall infiltration is considered minimal and has not been included as a model input (Sections 10.2 and 12.3). This is due to rainfall recharge to the alluvium being prevented from entering the excavation due to cut-off walls and the average daily recharge of 0.1 mm/day is considered negligible compared to the drawdown will be in the order of one metre. During construction, the inflow to the excavation will be through the toe of the D-Wall and the excavation base from the sandstone. The migration of saline groundwater from Duck Creek into the excavation due to reversed groundwater gradients, has been considered because saline water ingress to the drained basement would impact the durability of building materials (Sections 10.1 and 11.3). Analytical calculations (Sections 11.1 and 11.3) demonstrate that over the 120 year design life saline groundwater from Duck Creek is likely to enter the drained basement. Migration of potentially contaminated groundwater derived from nearby industrial estates entering the excavation has also been considered. Contaminated land investigations conducted at the site identified contaminants marginally exceeding the adopted guidelines including heavy metals chromium, cadmium, copper, nickel and zinc, PAHs and ammonia. The concentrations exceeding the guidelines may be at background levels or representative of remnant industrial activity. The risk of contaminated groundwater entering the excavation from surrounding former industrial sites is considered low due to natural attenuation, dilution from rainfall recharge and dilution as contaminated groundwater is mixed with non contaminated groundwater within the drawdown footprint. The risks of intersecting groundwater contamination are discussed further in Section 12.2.

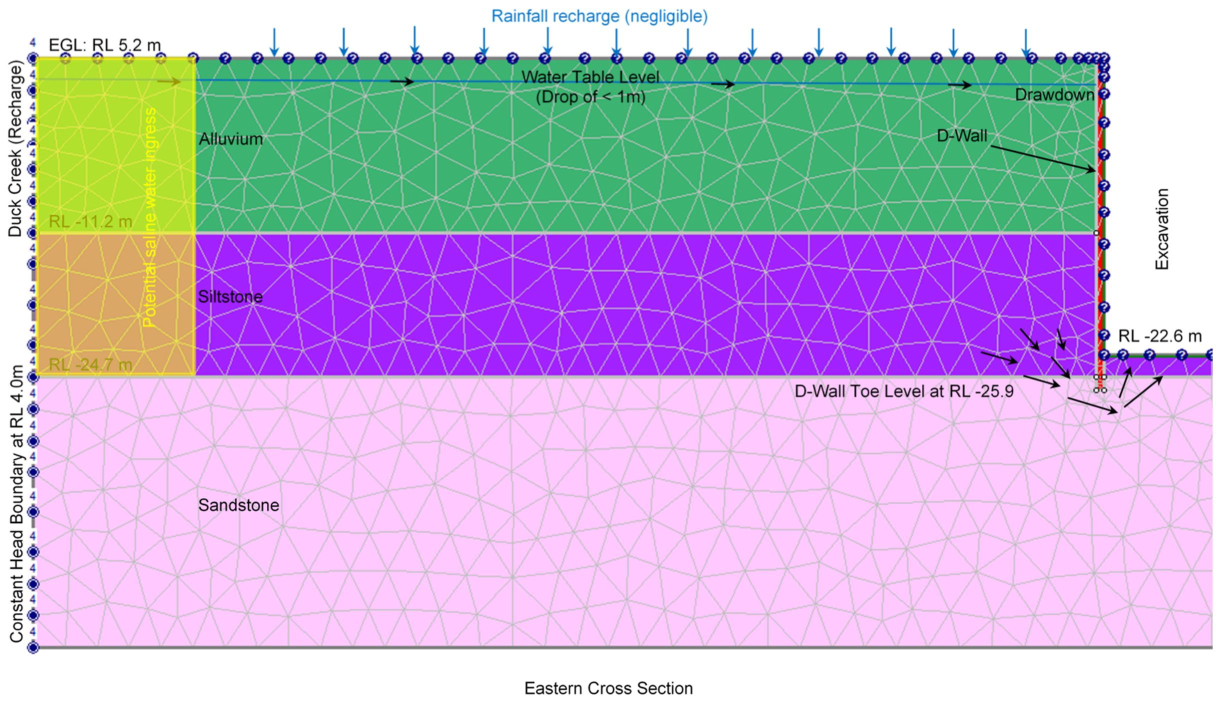


Figure 3-1 Schematic Diagram of the Conceptual Hydrogeological Model

4.0 Seepage analysis using SLIDE2

Two-dimensional modelling platforms such as SLIDE2 are considered an acceptable and common approach to estimate groundwater inflows into excavations. In many cases the extra work and costs involved in developing a three-dimensional model such as MODFLOW to calculate inflows is not required. This approach has been used for more complex tunnelling projects such as calculating inflows to the proposed M6 tunnel (Coffey, 2021) by the D&C contractor and is in accordance with the Australian groundwater modelling guidelines (Barnett et al, 2012).

Often 3D models are constructed for the whole alignment to predict groundwater impacts along the alignment and as such these models are considered regional models. Consequently, these models do not have sufficient detail in specific areas of interest such as the Rosehill Service Facility. It is therefore considered that the construction of multiple 2D sections across the area of interest is more appropriate in this instance than developing a 3D model.

4.1 Assumptions and methodology

The following categories of assumptions have been adopted in the analysis:

1. Geometry and levels
2. Ground profiles
3. Hydrogeology and hydraulic aquifer parameters
4. Conceptualisation

Model geometry and levels are presented in Table 4-1.

Table 4-1 Geometry and levels

Geometry and levels	Value
Width of box	20 m
Length of box	130 m
Excavation level (Western end)	RL -21.75 m AHD
Excavation level (Eastern end)	RL -22.6 m AHD
Toe of D-wall at headwall	RL -26.0 (east) & RL -23.0 m (west)
Toe of D-wall at longitudinal direction	1.2 m embedded into sandstone

Simplified ground profiles have been assumed and showed in Table 4-2.

Table 4-2 Ground profiles

Western End	RL (m AHD)
Top of ground level	+5.8
Top of alluvium	+5.8
Top of siltstone II	-11.4
Top of sandstone II	-22.2
Design groundwater level	+5.1
Eastern End	**RL (m AHD)**
Top of ground level	+5.2
Top of alluvium	+5.2
Top of siltstone II	-11.2
Top of sandstone II	-24.7
Design groundwater level	+4.0

Aquifer parameters are presented in Table 4-3.

Table 4-3 Aquifer Parameters

Materials	Unit	Baseline Kh (m/day)	Baseline Kv/kh	Wc sat	Mv (1/kPa)	E (kPa)
Soft Alluvium	ALL-S	0.08	0.2	0.46	2.00e-4	5e3
firm Alluvium	ALL-F	0.08	0.2	0.46	6.25e-5	16e3
Stiff Alluvium	ALL-St	0.08	0.2	0.46	3.10e-5	32e3
Siltstone II	Slt-II	0.1	0.2	0.075	5.00e-7	2e6
Sandstone II	SS-II	0.002	0.2	0.185	4.00e-7	2.5e6
D-Wall	-	8.6e-26	1.0	0.001	3.00e-8	32.8e6
Cut-off Wall	-	8.6e-26	1.0	0.001	5.00e-9	200e6

5.0 Operational (long term permanent)

Modelling conceptualisation is as follows:

1. Recharging boundaries (Represented by a constant head boundary in the models) are as follows:
   a. Western end:

i. Distance between the Duck Creek and the Western end is set at 180 m.
ii. The constant head boundary at the Western end on the opposite side of the Duck Creek is set at 260 m (2 times the length of the Rosehill Service facility) away from the D-Wall.

b. Eastern end:
  i. Distance between the Duck Creek and the Eastern end is set at 60 m.
  ii. The constant head boundary at the Eastern end on the opposite side of the Duck Creek is set at 100 m from the D-Wall.

5.1 Model scenarios

Two modelling scenarios have been analysed along the eastern and western cross sections based on baseline permeabilities with the updated rock permeabilities using the new packer tests results. These scenarios are presented in Table 5-1.

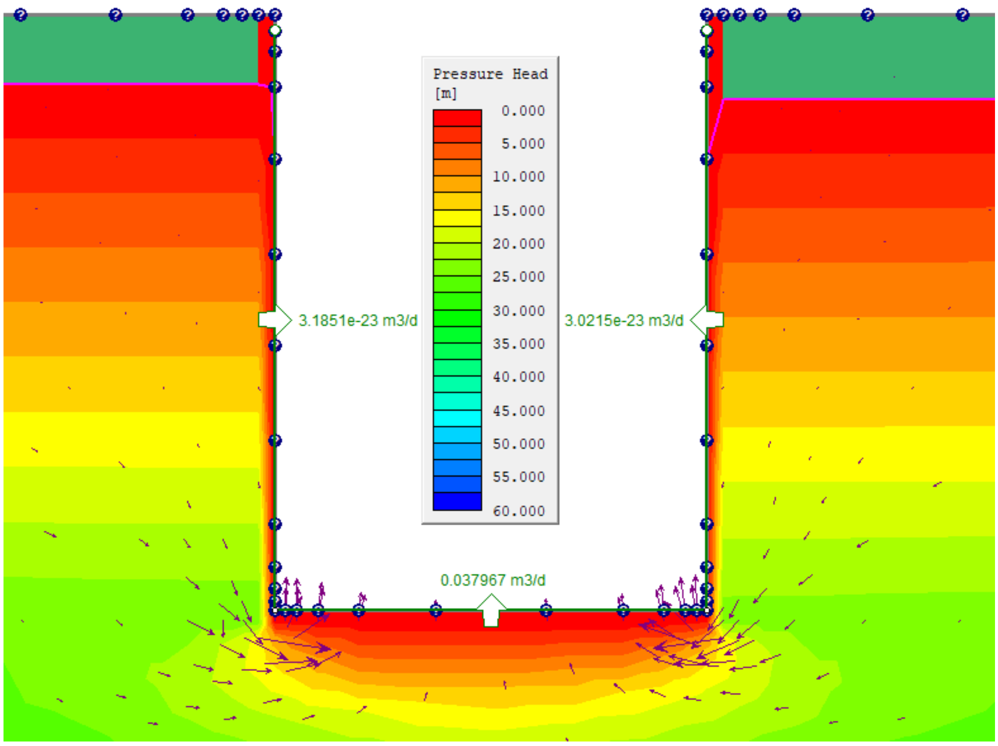
Table 5-1 Model scenarios

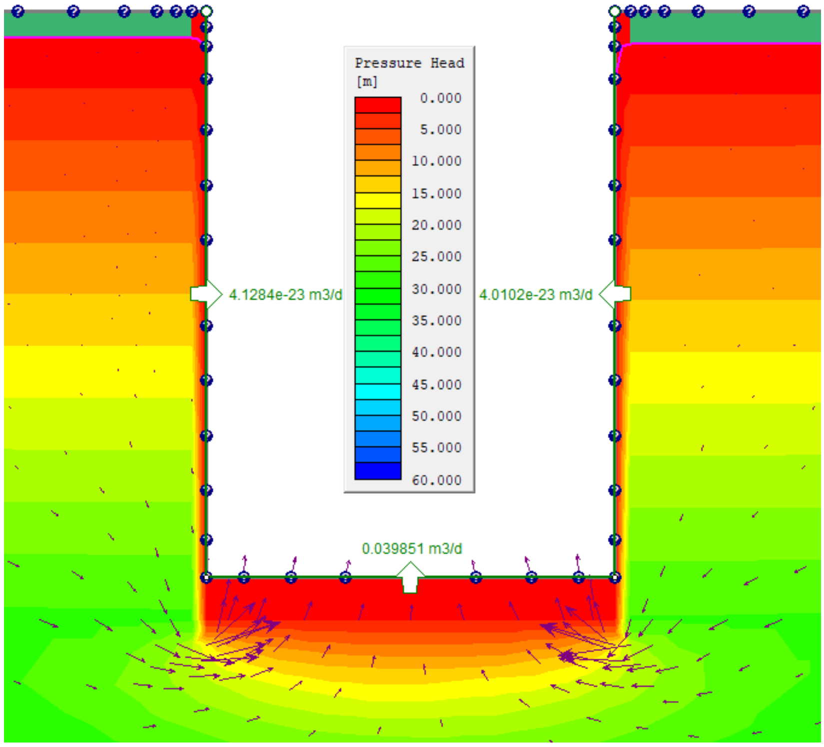
Cross Section (Western end)	Parameters
Case 1: Permanent case	Baseline
Cross Section (Eastern end)	Parameters
Case 2: Permanent case	Baseline

5.2 Estimated groundwater inflow

Graphical outputs of SLIDE2 results for Case 1 and Case 2 are provided in Table 5-2 below.

Table 5-2 Graphical outputs

Case	**Graphical output**
Case 1	

<table>
<tr><th>Case</th><th>Graphical output</th></tr>
<tr><td>Case 2</td><td>
</td></tr>
</table>

Estimated permanent inflows for the whole box are provided in Table 5-3.

Table 5-3 Estimated permanent inflow (whole box)

Case	Inflow (Litres/day)
Most critical of Cases 1/2: Baseline	40 L/day/m * (130m length + 20m width) = 6,000 L/day

5.3 Estimated water drawdown

The analyses discussed in Section 5.2 are for the steady state flow conditions. For Cases 1/2 which are based on the baseline permeabilities, water drawdown is expected to be between 0.2 m and 2.1 m at the diaphragm wall. Hence, water drawdown during transient conditions are also less than 2.1 m water drawdown at the steady state condition is the most critical.

6.0 During construction (temporary dewatering)

For the details of the geometry of the batter slopes during construction refer to Drawing “SMWSTWTP-GLO-RSH-SF500-RS-DRG-010110 & 010111”.

The longitudinal and cross section of the Rosehill Service have been modelled in SLIDE2 with the dimensions shown in Figure 6-1. As shown in Figure 6-1, it has been assumed in the longitudinal section that, the eastern D-Wall is 50 m and the western D-wall is about 100 m away from the Duck Creek (constant head at 3.6 m AHD).

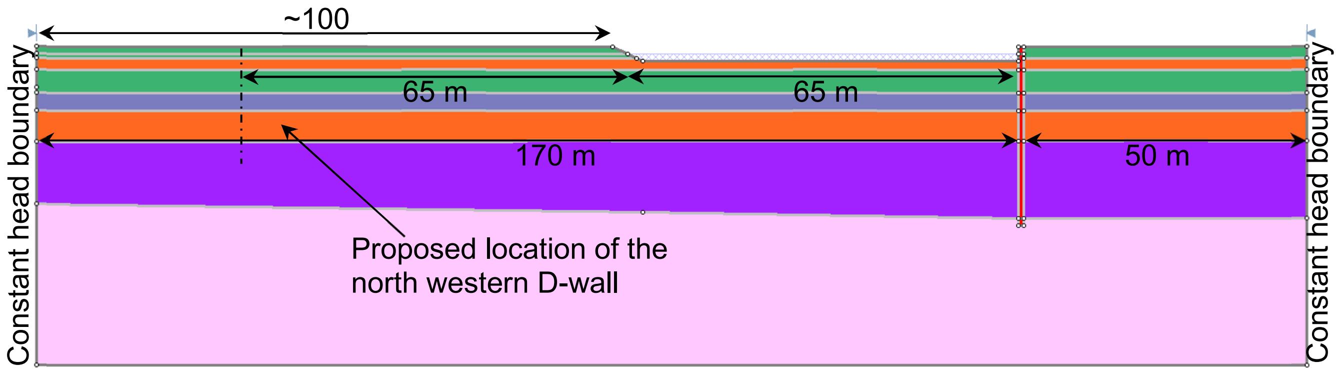


Figure 6-1 Overall model (a) longitudinal section

6.1 First drop to RL +0.5 m AHD

6.1.1 Analysis approach/steps (first drop to RL +0.5 m AHD):

- Stage 1: Steady state flow conditions representing current conditions with water table at RL +3.6 m AHD.
- Stage 2: Excavate the right half side (Sydney side) of the Rosehill box to RL +0.5 m AHD in one day with the 1V:2H batter slope (refer to the drawing "SMWSTWTP-GLO-RSH-SF500-RS-DRG-010110 & 010111" for the details of the geometry). Assess water flow rate immediately following excavation.
- Stage 3: Assess water flow rate at 5 days following excavation.
- Stage 4: Assess water flow rate at 10 days following excavation.
- Stage 5: Assess water flow rate at 30 days following excavation.
- Stage 6: Assess water flow rate at 100 days following excavation.

Figure 6-2 to Figure 6-4 show the model and flow rate results at selected days following excavation.

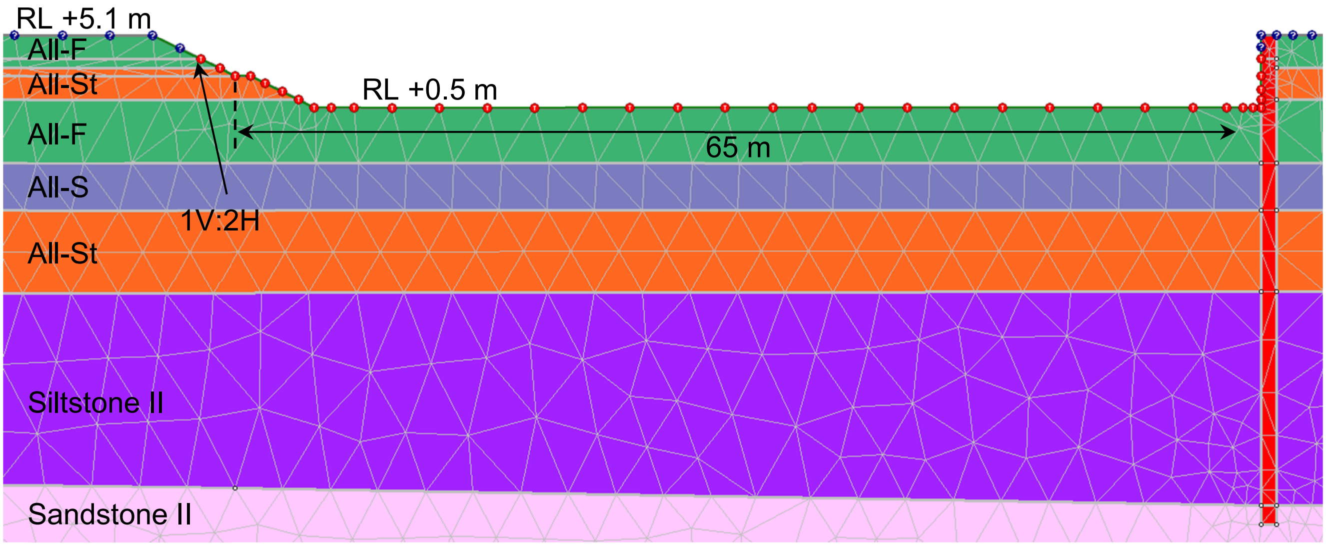


Figure 6-2 SLIDE2 model discussed in Section 6.1 (Stage 1)

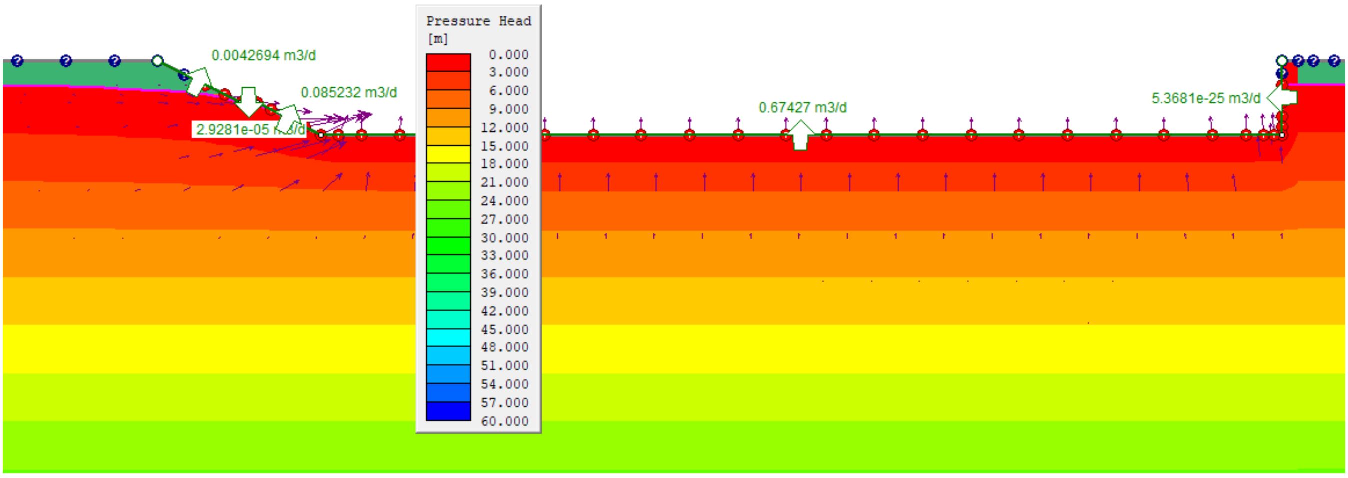


Figure 6-3 Estimated groundwater inflow immediately after excavation (Stage 2)

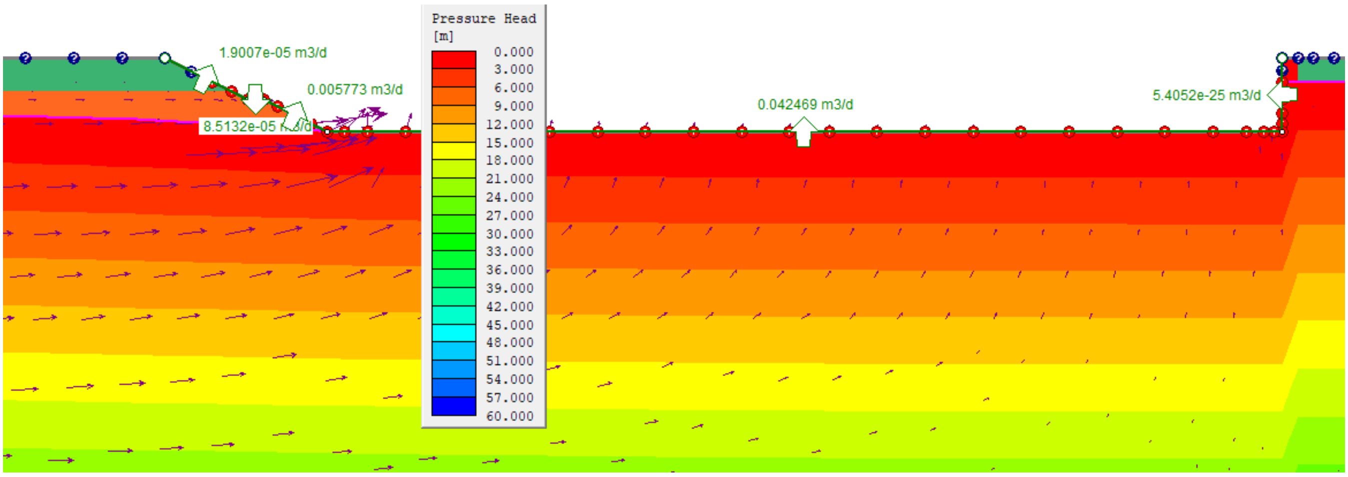


Figure 6-4 Estimated groundwater inflow 100 days after excavation (Stage 6)

According to Section 5.0 and 8.2, the final long term drawdown is calculated to be less than one metre based on the baseline permeability values. Hence, to eliminate the impact of the water drawdown during construction (i.e. eliminate settlement and impact on structure durability due to potential oxidation of acid sulfate soils), it is proposed to limit the lowering of the water table level at the location of the western D-Wall to 1.0 m only. Table 6-1 shows the water table level at the proposed location of the north-western D-Wall at the selected days following excavation.

Table 6-1 Water table level at the proposed location of the north-western D-Wall (i.e. 65 m away from the toe of the batter slope)

Days Following Excavation	RL (m AHD)	Water Table Decline (mm)
Immediately after excavation (Stage 2)	3.6	0
100 days after excavation (Stage 6)	2.7	900

6.1.2 Discussion and Conclusion

A water level decline of about 1.0 m is expected at the proposed location of the western D-Wall if the batter slope is excavated down to RL +0.5 m. Hence, any further excavation below RL +0.5 m AHD is recommended to be carried out following the installation of the cut-off wall. The following section describes the model and flow rate based on the aforementioned conclusion with the inclusion of the cut-off wall.

6.2 Second Drop to RL -1.0 m AHD

6.2.1 Analysis approach/steps (second drop to RL -1.0 m AHD):

- Stage 1: Steady state flow conditions with water table level at far boundary to be at RL 3.6 m and within the excavation to be at RL +0.5 m representing the previous stage as discussed in Section 6.1. The cut off wall is installed at about 5 m behind the crest of the batter slope as shown below in Figure 6-5 assuming 500 mm embedment into siltstone.
- Stage 2: Excavate the right half side (Sydney side) of the Rosehill box to RL -1.0 m AHD in one day with the 1V:2H batter slope (refer to the drawing "SMWSTWTP-GLO-RSH-SF500-RS-DRG-010110 & 010111" for the details of the geometry). Assess water flow rate immediately following excavation.
- Stage 3: Assess water flow rate at 5 days following excavation.
- Stage 4: Assess water flow rate at 10 days following excavation.
- Stage 5: Assess water flow rate at 30 days following excavation.
- Stage 5: Assess water flow rate at 100 days following excavation.

Figure 6-5 to Figure 6-7 show the model and flow rate results at certain days following excavation.

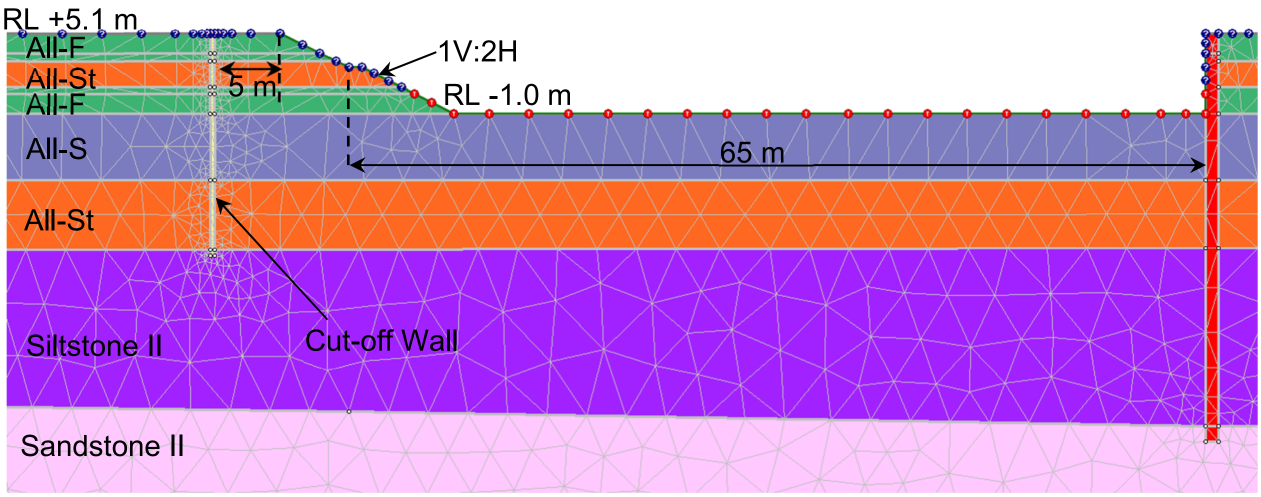


Figure 6-5 SLIDE2 model discussed in Section 6.2 (Stage 1)

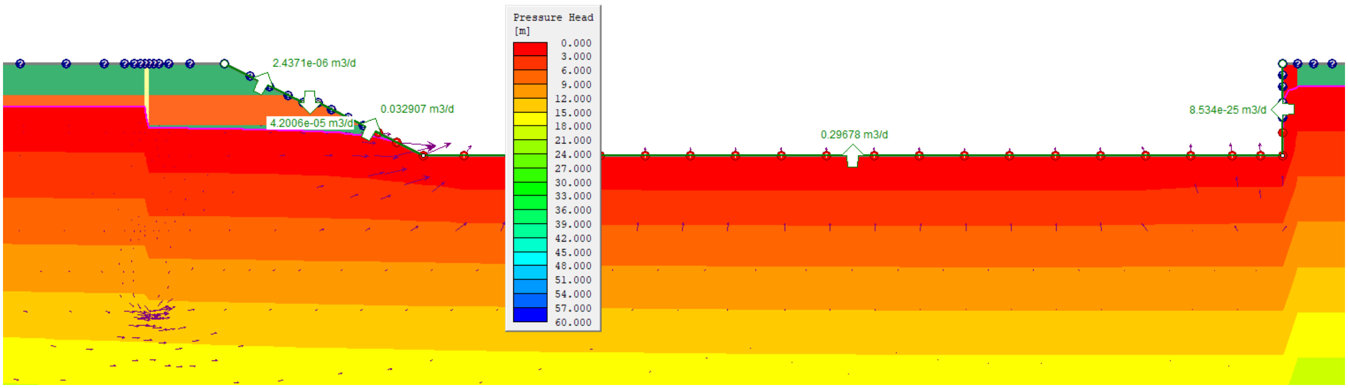


Figure 6-6 Estimated groundwater inflow immediately after excavation (Stage 2)

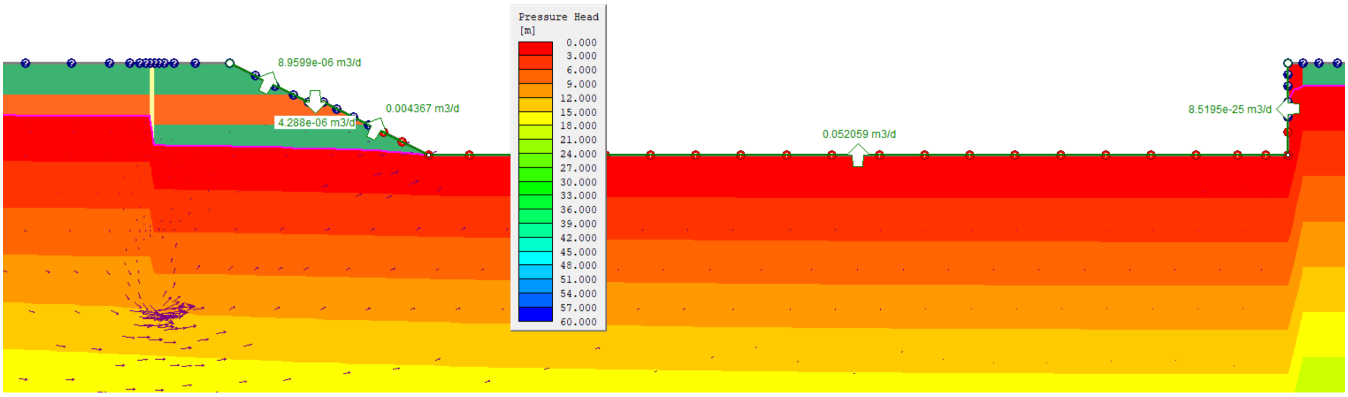


Figure 6-7 Estimated groundwater inflow 100 days after excavation (Stage 6)

Table 6-2 shows the water table elevation at the proposed location of the north-western D-Wall at selected days following excavation.

Table 6-2 Water table elevation at the proposed location of the north-western D-Wall (i.e. 65 m away from the toe of the batter slope)

Days Following Excavation	RL (m AHD)	Water Table Decline (mm)
Immediately after excavation (Stage 2)	2.9	700
100 days after excavation (Stage 6)	2.6	1000

6.2.2 Discussion and Conclusion

A water level decline of about 1.0 m is expected at the proposed location of the western D-Wall if the batter slope is excavated down to RL -1.0 m with the proposed cut-off wall as discussed in Section 6.1.2.

7.0 Summary of predicted inflows

The total permanent (operational) inflow from the base for the baseline value HIR parameters is predicted to be 6,000 L/day (0.07 L/sec).

8.0 Drawdown analysis

Groundwater drawdown modelling is required to calculate the drawdown during construction and long term operational phase.

The drawdown calculations conducted in this assessment are considered conservative as the beneficial effects of rainfall recharge have been ignored. In addition, it is expected that there would be recharge from the Rosehill Racecourse from sprinklers and irrigation. For the purposes of this investigation the 2D modelling approach is considered sufficient. The drawdown calculations have been used to calculate settlement which is discussed in the GIR (Appendix D1).

8.1 Drawdown during construction

The aforementioned construction staging (Section 2.4) has been developed based on the assumption of the acceptable total water drawdown of less than 1 m behind the proposed location of the north western D-Wall. This criterion is based on the acceptable water table drop one metre during the operation (permanent stage) discussed in Section 8.2.

8.2 Drawdown during operation

Long term groundwater drawdown for both scenarios using varying hydraulic parameters has been calculated based on the baseline permeabilities that has been updated as per the new packer tests results.

Predicted groundwater drawdown as calculated by the PLAXIS and SLIDE2 models are presented in Figure 8.1. Note the figure has a 0.5 m contour interval. The contours represent steady state conditions after construction and during the operations phase. The maximum drawdown at the D-Wall ranges between 0.2 m and 2.1 m. The contours are based on three cross sections through the excavated structure with engineering judgement applied to complete the contours. As expected the contours are elongated on the north western side of the structure where the influence of recharge from Duck Creek is diminished.

A search of the Bureau of Meteorology (BOM) groundwater explorer database on 23 June 2022 indicates there are no registered water supply bores within the zone of drawdown. Thus there will be no impacts to any registered water supply bores due to the project drawdown.

Proposed groundwater monitoring measures are discussed in Sections 9.4, 11.4 and 12.1.

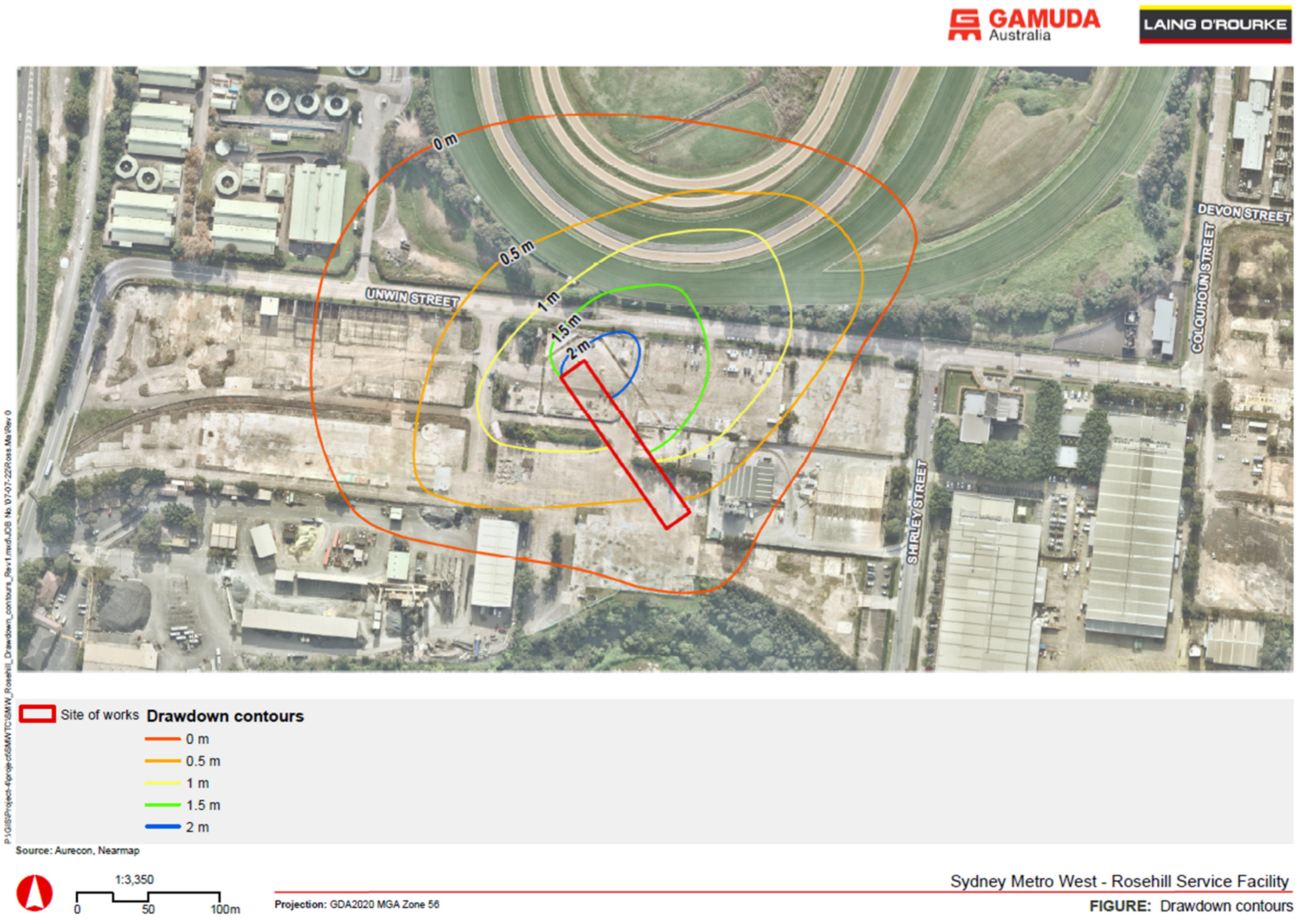


Figure 8.1 Predicted long term drawdown

9.0 Discussion of Requirements for Condition D122

9.1 (a) Cumulative impacts

Condition D122a states: *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.*

Cumulative impacts relating to groundwater ingress and groundwater drawdown for the construction and operation of the Rosehill Service Facility are not considered applicable for the following reasons:

- There are no registered groundwater users within one kilometre radius of the excavation that extract groundwater and could contribute to a cumulative impact during construction. Similarly there is no major subsurface infrastructure that impacts groundwater flow such as deep building basements or drained tunnels. There is a heritage façade wall along Unwin Street to the est of the site which requires protection as a condition of the Ministers Conditions of Approval (MCOA). Potential impacts relating to settlement to this feature are discussed in the GIR (Appendix D1). Cumulative impacts due the construction of the Clyde Dive and Parramatta Station are to be assessed in the site wide HIR being prepared by GHD. Long term during the operations phase groundwater ingress into the excavation is estimated to be between 0.07 L/sec long term drawdown is predicted to be between 0.2 and 1.2 m next to the D-Wall based on the baseline permeabilities that has been updated as per the new packer tests results. The Sydney Metro Tunnels are to be constructed after the completion of the shaft. Expected cumulative impacts due to the construction of the Sydney Metro Tunnels and the interaction with the shaft will be assessed during the Sydney Metro Tunnels groundwater impact assessment.

9.2 (b) Dewatering take

Condition D122a states: *Predicted incidental groundwater take (dewatering) including cumulative effects.*

During the construction of the Rosehill Service Facility shaft structure temporary dewatering will be required to maintain dry working conditions. The dewatering 'take' will be dependant upon a number of factors including the scheduling of the excavation, construction of the D-Walls and how long the excavation will be open during construction. Thus, the groundwater 'take' cannot be calculated until the D-Wall construction sequencing is further developed.

During construction it is expected that there will be no cumulative effects to consider since there is no other infrastructure or bores that impact the local hydrogeological system that would contribute to cumulative impacts. Similarly, the construction of the Rosehill Service Facility shaft structure willParramatta or Clyde Dive structures since the construction program will be completed before the tunnel construction commences. Cumulative impacts between the Rosehill Service Facility shaft structure and the Sydney Metro Tunnel construction will be addressed in the site wide groundwater impact assessment for the construction of the Sydney Metro Tunnels.

9.3 (c) Potential long term impacts

Condition D122a states: *Potential impacts for all later 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.*

Long term groundwater impacts due to the Rosehill Service Facility shaft structure are groundwater ingress to the excavation from lateral flow beneath the D-Wall and through the drained base slab. This will result in on-going groundwater inflows and associated groundwater drawdown. Groundwater modelling has predicted total permanent inflow from the base for the baseline value and conservative

HIR parameters is predicted to be 6,000 L/day (0.07 L/sec). Associated long term drawdown is predicted to be up to 2.1 m based on the baseline permeabilities that has been updated as per the new packer tests results. The Aquifer Interference Policy allows two metres of drawdown before any mitigation measures are required to be implemented. It is unlikely that any mitigation measures will be required immediately adjacent to the D-Wall.

Cumulative impacts for latter stages of the CSSI will be addressed in the groundwater impact assessment for the Sydney Metro Tunnels.

Potential long term impacts are discussed further Section 12.1.

9.4 (d) Minimising water inflows

Condition D122a states: *Actions required after Stage 1 to minimise the risk of inflows (including in the event latter stages of the CSSI are developed or do not progress) and a strategy for accounting for any water taken beyond the life of the operation of the CSSI.*

During construction groundwater inflow will be collected, treated and discharged with flows being recorded with a flow meter.

Later stages of the CSSI (following the completion of the Rosehill Service Facility shaft structure) would include the construction of the Sydney Metro tunnels. Groundwater ingress to the tunnels will be managed by the construction contractors in accordance with the recommendations of the Sydney Metro Groundwater Impact Assessment.

To minimise long term groundwater inflows the Rosehill Service Facility shaft structure is to be constructed with its perimeter walls socketed into the sandstone. As such there will be limited groundwater ingress through the embedded walls below the D-Wall and through the drained base of the excavation. Long term discharge flows are expected to be monitored with flow meters in accordance with the Operational Environmental Management Plan (OEMP). Following construction of the Rosehill Service Facility shaft structure there are likely to be groundwater impacts associated with the construction and long term operations of the Sydney Metro Tunnels. These impacts will be assessed, and management and mitigation measures to minimise groundwater impacts recommended in the Metro Tunnel Groundwater Impact Assessment.

Beyond the life of the CSSI infrastructure the structures are likely to be infilled or repurposed in which case the ongoing groundwater take or groundwater inflows will be either halted or managed by the repurposed infrastructure. It is expected that in a repurposed scenario groundwater inflows to the excavations would be managed in a similar way that is proposed for the Sydney Metro project.

9.5 (e) Saltwater Intrusion

Condition D122a states: *Saltwater intrusion modelling analysis, from estuarine and saline groundwater in shale, into the Bays metro station site and other relevant metro station sites.*

This condition is not relevant as there are no estuarine waters that would inflow into the Rosehill Service Facility during temporary construction dewatering. There will be ingress of some saline groundwater from the Ashfield Shale and Mittagong Formation during temporary dewatering but this water is to be collected and treated prior to discharge.

Potential impacts due to saltwater intrusion are discussed further in Sections, 10.1, 11.1 and 11.3.

9.6 (f) Schematic Hydrogeological Conceptual Model

Condition D122a states: *A schematic of the conceptual hydrogeological model.*

The schematic diagram of the hydrogeological conceptual model is presented as Figure 3-1 and described in Section 3.6.

10.0 EDS Design Review Comments March 2022

Additional technical information is provided to address hydrogeological related questions provided in the March 2022 EDS Design Review Comments – Rosehill Stage 2 Design, prepared by Transport for NSW.

10.1 Potential saline water intrusion

Question: *Duck Creek is connected to the Parramatta River and is a saline estuary in close proximity to the proposed excavation. Boreholes near to the creek are also saline. Please explain what steps will be taken to demonstrate, by modelling, what the potential risk of saline ingress will be during the 10 year construction stage and what durability measure will be considered.*

Answer: Groundwater flow from Duck Creek to the excavation has been estimated in the underlying siltstone and sandstone since any groundwater entering the excavation would be beneath the toe of the D-Wall. Groundwater from the alluvium would not enter the excavation due to the installation of cut-off walls, however the groundwater flow velocity would be similar for the alluvium and the fractured bedrock as the hydraulic conductivity values are similar.

The risk of potential inflow of saline water from entering the excavation can be estimated by calculating the time taken for groundwater to flow 60 m through the siltstone and sandstone from Duck Creek to the excavation. Groundwater velocity can be calculated in a fractured rock aquifer assuming an effective porosity (ϕ) of between 5% and 10% within a homogeneous aquifer. The watertable elevation varies from 3.6m AHD at Duck Creek to 2.55 mAHD calculated drawdown at the D-Wall over a distance of 60 metres. The hydraulic conductivity is estimated at 0.016 m/day for a siltstone/sandstone. The linear (or true) velocity is calculated by the following equation:

$$V=Ki/\phi$$

Where V=velocity (m/day)

K= hydraulic conductivity (m/day)

i = hydraulic gradient (m/m)

ϕ = effective porosity (%)

Drawing from the groundwater modelling already undertaken to calculate groundwater drawdown the following aquifer parameters were adopted:

- Distance between the eastern end of the excavation and Duck Creek = 60m
- Elevation of Duck Creek = 3.6m AHD (constant head boundary)
- Groundwater elevation drawdown at the D-Wall – 2.55 mAHD (calculated from modelling)
- Hydraulic gradient = (3.6-2.55) /60 = 0.0175
- Hydraulic conductivity of sandstone/siltstone = 0.016
- Effective porosity of sandstone (fractured rock 5%)

Thus V = (0.016 x 0.0175 / 0.05) x 365 = 2.0 m/year. [ϕ = 5%]

V= = (0.016 x 0.0175 / 0.10) x 365 = 1.0 m/year. [ϕ = 10%]

The groundwater velocity is calculated at between 1 and 2 m/year and is considered conservative due to the conservative parameters used. Groundwater flow at the excavation would be further restricted by the diaphragm walls. The calculations demonstrate that if the excavation is dewatered for one year groundwater from Duck Creek will travel between one and two metres towards the excavation. Should the excavation be open and dewatered for 10 years groundwater would migrate between 10 and 20 metres but would not migrate the 60 metres to the excavation.

Groundwater modelling is not considered necessary to assess groundwater travel times since the models would use the type of analytical equations as used in this report.

A sensitivity analysis for the saltwater intrusion calculations is presented in Section 11.1 and saltwater intrusion over the 120 year project life is discussed further in Section 11.3.

10.2 Omission of rainfall recharge in groundwater modelling

Question: *The hydrological modelling excludes rainfall recharge to groundwater, please explain why this is omitted. Taking into account the cumulative amount over the construction period and the life of the project and the areal extent of the drawdown cone, is it correct to assume the amount to be negligible and would this omission not affect estimates of drawdown and settlement?*

Answer: Recharge to the alluvium has not been considered since groundwater from the alluvium will not enter the excavation due to the installation of cut-off walls. Instead recharge to the underlying bedrock is considered as the siltstone will be dewatered throughout the project construction.

Annual rainfall at BOM station Number 66124 located at North Parramatta is 968.3 mm. In groundwater modelling groundwater recharge is typically applied to the groundwater model by applying a percentage of annual rainfall. The percentage is variable and depends on the lithology outcropping and whether the area is paved or unpaved. Groundwater modelling in the Sydney Basin typically adopts 4 to 5% for the Hawkesbury Sandstone. Given the bedrock at the site is predominately a siltstone 4% recharge is considered reasonable which equates to 33.7 mm per year or 0.1 mm/day. Considering the dewatering will lower the watertable temporarily in the order of one metre a mean increase of 0.1 mm/day due to rainfall recharge is considered negligible. Similarly, the cumulative addition of 37.4 mm over a year is considered negligible. Consequently, there would be no impacts on estimates of settlement since the impacts of drawdown due to the omission of rainfall recharge are negligible.

The absence of water gain from rainfall recharge would be in part balanced by losses due to evaporation and evapotranspiration.

Rainfall recharge is discussed further in Section 12.3.

11.0 DS Design Review Comments June 2022

Additional technical information is provided to address hydrogeological related questions provided in the June 2022 EDS Design Review Comments – Rosehill Stage 3 Design, prepared by Transport for NSW.

11.1 Saline water intrusion – sensitivity analysis

Question: *A sensitivity study may give a different conclusion. Duck Creek is a saline estuary, a condition of approval is that ingress from saline estuaries are to be modelled.*

Answer:

The saltwater intrusion calculations have been extended from the previous calculations presented in the design report to cover the temporary construction and design life of 120 years. There will be on-going drawdown due to the permanent drained structure.

The initial saltwater intrusion calculations are described in Section 10.1, Appendix D2 of the Design Report. It should be noted that groundwater modelling also includes the application of analytical equations as has been done in this saline water intrusion investigation. Two and three dimensional groundwater modelling platforms apply the same analytical equations within the model code as has been used in this investigation.

Groundwater flow has been calculated by application of the following analytical equation:

$$V = Ki/\phi$$

Where V=velocity (m/day)

K= hydraulic conductivity (m/day)

i = hydraulic gradient (m/m)

ϕ = effective porosity (%)

A sensitivity analysis has been undertaken by varying the hydraulic conductivity and effective porosity. Groundwater from the alluvium would not enter the excavation due to the installation of cut-off walls and has not been considered further. The hydraulic gradient is considered a constant based on the difference in head in Duck Creek (assumed at ground level) and the base of the excavation over a distance of 60 metres. The hydraulic gradient differs for construction and long term as the drawdown during construction is greater. Drawdown during construction was modelled as 1.05 m and a maximum of 0.60 m during the operations phase.

The adopted values used in the calculations are:

- Effective porosity 5%;
- Hydraulic conductivity siltstone 0.016 m/day
- Hydraulic conductivity sandstone 0.01 m/day

For the sensitivity analyses the effective porosity has been varied between 2%, 5% and 10%. The hydraulic conductivity has been varied for siltstone and sandstone by an order of magnitude.

11.1.1 Construction Sensitivity Analysis

During construction the hydraulic gradient between Duck Creek and watertable at the excavation is 0.0175. Construction is expected to take 8 months, so temporary dewatering will occur for 8 months. The results of the sensitivity analysis are presented in Table 3.

Table 3 Construction sensitivity analysis for distance travelled

Lithology	K	i	Effective porosity	V	Distance travelled (m)
Siltstone	m/day		%	m/year	8 months
	0.0016	0.0175	0.02	0.51	0.3
	0.0016	0.0175	0.05	0.20	0.14
	0.0016	0.0175	0.1	0.10	0.07
	0.016	0.0175	0.02	5.11	3.4
	0.016	0.0175	0.05	2.04	1.4
	0.016	0.0175	0.1	1.02	0.7
	0.16	0.0175	0.02	51.1	34.1
	0.16	0.0175	0.05	20.4	13.6
	0.16	0.0175	0.1	10.2	6.8
sandstone	0.001	0.0175	0.02	0.32	0.21
	0.001	0.0175	0.05	0.13	0.09
	0.001	0.0175	0.1	0.06	0.04
	0.01	0.0175	0.02	3.19	2.13
	0.01	0.0175	0.05	1.28	0.85
	0.01	0.0175	0.1	0.64	0.43
	0.1	0.0175	0.02	31.9	21.3
	0.1	0.0175	0.05	12.8	8.5
	0.1	0.0175	0.1	6.4	4.3

In Table 3 the adopted values of hydraulic conductivity and effective porosity for siltstone and sandstone are highlighted yellow. Based on the parameters used the distance travelled for a particle of water from Duck Creek has been calculated for 8 months.

The calculations demonstrate that if the excavation is dewatered for 8 months groundwater from Duck Creek will travel between one and two metres towards the excavation for the optimal parameters (highlighted yellow). In each scenario water from Duck Creek will not enter the excavation via groundwater migration.

11.1.2 Long Term Sensitivity Analysis

Long term the hydraulic gradient between Duck Creek and the watertable above the drained excavation is 0.01. The distance travelled for water at the edge of Duck Creek has been calculated over the design like of 120 years. The results of the sensitivity analysis are presented in Table 4 and have been revised (June 2022) to remove unrealistic combinations of porosity and hydraulic conductivity.

Table 4 Long term sensitivity analysis for distance travelled

Lithology	K	i	Effective porosity	V	Distance travelled (m)
siltstone	0.0016	0.01	0.02	0.292	35.0
	0.0016	0.01	0.05	0.1168	14.0
	0.0016	0.01	0.1	0.0584	7.0
	0.016	0.01	0.05	1.168	140
	0.016	0.01	0.1	0.584	70
sandstone	0.001	0.01	0.02	0.02	2.4
	0.001	0.01	0.05	0.07	8.8
	0.001	0.01	0.1	0.04	4.4
	0.01	0.01	0.02	1.83	219
	0.01	0.01	0.05	0.73	88
	0.01	0.01	0.1	0.37	44

Highlighted red – particle flow exceeds 60 metres and will enter the drained structure

In Table 4 the adopted values of hydraulic conductivity and effective porosity for siltstone and sandstone are highlighted yellow. Based on the parameters used the distance travelled for a particle of water from Duck Creek has been calculated after 120 years. Where the distance travelled to the excavation is reached (greater than 60 m) the distance has been highlighted red. It can be seen that for the optimal parameters saline water from Duck Creek is likely to reach the Rosehill Service Facility within the 120 year design life.

11.2 Groundwater Sampling

Recent groundwater quality sampling (Epic Environmental, 2022) was conducted in three monitoring wells on 10 May 2022, located between the excavation and Duck Creek. Sampling details including the distance from Duck Creek and monitored electrical conductivity (µS/cm), sulfate and chloride concentrations are summarised in Table 5.

Table 5 Groundwater sampling

Bore	Distance from Duck Creek (m)	Screen Interval (mbgl)	Electrical Conductivity (µS/cm)	SO_4 mg/L
Env_811	100	1.9 – 7.9	33,000	1,600
BH15A	140	13.0 – 15.8	27,000	890
Env_801	180	2.9 – 7.3	20,000	1,100

Note: mbgl metres below ground level;

Analysis of the water quality data indicates that the natural groundwater is saline at the excavation. In addition there is a salinity gradient where the salinity becomes more saline with increasing proximity to Duck Creek. Sulfate and chloride parameters are major durability parameters and are elevated in the natural groundwater with sulfate concentrations ranging from 890 to 1,600 mg/L. Similarly chloride concentrations range from 6,600 to 12,000 mg/L.

These concentrations of durability parameters indicate the groundwater is naturally aggressive at the Rosehill Service Facility excavation. The migration of saline groundwater from Duck Creek is eventually likely to increase the salinity to around that of seawater within the siltstone and shale at the drained structure. It is recommended that the building materials for the Rosehill Service Facility are selected to be durable to saline water, sulfate and chloride irrespective of any saltwater migration to the excavation.

11.3 Saltwater intrusion conclusion

Groundwater modelling by application of analytical calculations has been undertaken to predict the saline intrusion from Duck Creek to the Rosehill Service Facility excavation during construction and long term over a design life of 120 years.

A sensitivity analysis has been conducted for the calculations during construction and operation phases varying the parameters of hydraulic conductivity and porosity. The calculations demonstrated that during the eight months of construction the temporary dewatering would not induce groundwater to travel 60 metres from Duck Creek into the excavation.

A sensitivity analysis for long term impacts due to the drained structure at the Rosehill Service Facility indicated that saline water from Duck Creek would travel 60 metres over the 120 year design life for the optimal parameters selected for the values of hydraulic conductivity and porosity adopted.

Given the elevated natural concentrations of the durability parameters electrical conductivity, sulfate and chloride the native groundwater is naturally aggressive. Migration of saline groundwater from Duck Creek is eventually likely to increase the salinity to around that of seawater within the siltstone and shale at the drained structure. It is recommended that the building materials for the Rosehill Service Facility selected are durable to aggressive saline water, sulfate and chloride irrespective of any saltwater migration to the excavation. A durability assessment undertaken for this project (Appendix D4) has used seawater salinity and chloride concentrations to account for the predicted saltwater intrusion.

11.4 Permanent groundwater inflow through the drained base slab

Question: *Please advise what steps will be taken to ensure that the groundwater inflow through the drained base slab does not exceed the 45,000 litres in any 24 hour period?*

Answer:

Conservative modelling has predicted that groundwater inflow through the drained basal slab will be 6,000 L/day which is significantly below the 45,000 litres per day criterion. The calculations are based on hydraulic conductivity parameters derived from previous site investigations. Additional geotechnical investigations including packer tests confirmed the hydraulic conductivity values adopted. Cut-off walls will prevent groundwater from entering the structure from the alluvium and upper weathered siltstone. Groundwater seeping into the structure through the basal slab will be collected and discharged. The volume of groundwater will be monitored with a flow meter to check that the inflows are below the 45,000 L/day criterion.

11.5 Ground Settlement

Question: *Provide evidence that the influence of groundwater drawdown during construction on the settlement is considered in the analysis.*

Answer:

A ground settlement analysis was conducted using Plaxis as the modelling platform and is presented in Appendix D1. To calculate ground settlement the model requires groundwater drawdown as an input parameter.

12.0 Stage 3 IC Comments June 2022

12.1 Impact on the watertable and implementation of monitoring and management measures.

Question 1: *The requirement SM-W-WTP-PS-1025, Particular Specification clause 4.1.7(b) "The Tunnelling Contractor must assess by modelling the impact on the groundwater table and specify control and monitoring measures to demonstrate compliance with the Acceptable Effects" is not detailed. The influence of the retaining structure acting as a cutoff of the natural groundwater flow system does not discuss e.g. lateral effects like potential accumulation in front of the impermeable walls and assumed difference in groundwater flow speeds due to the barrier build into it.*

Answer 1: Seepage modelling has been conducted using SLIDE2 to estimate construction and long groundwater inflows into the excavation (Section 4.0 and 5.0 Appendix D2). Predicted groundwater drawdown (long term and short term) has been modelled using SLIDE2 and PLAXIS modelling platforms which predicts the long term impact on the watertable as described in (Appendix D2, Section 8.0). The long term operational steady state groundwater drawdown is shown on Figure 8.1. The modelling shows that the drawdown adjacent to the excavation ranges between 0.2 m and 1.2 m. Groundwater modelling has predicted that groundwater inflow through the drained basement will be up to 19,650 L/day which does not exceed the design criterion of 45,000 litres in any 24 hour period.

Groundwater monitoring measures including monitoring of groundwater levels, groundwater ingress to the drained basement measurements and groundwater discharge quality would be outlined in a Groundwater Management Plan as part of a Construction Environment Management Plan (CEMP) during construction and an Operational Environment Management Plan (OEMP). Proposed groundwater level monitoring including trigger levels has been outlined in the GIR (Sections 9.2.1 and 9.2.2). Monitoring of groundwater flow to the drained basement has been addressed in Sections 9.4 and 11.4 of the HIR. Groundwater quality monitoring of groundwater entering the drained basement will be undertaken as part of the CEMP/OEMP to assess if groundwater treatment is required prior to discharge.

The D-Wall has been designed to be socketed two metres into the siltstone to prevent groundwater within the alluvium from entering the drained basement. The barrier will effectively create a barrier to groundwater flow potentially creating groundwater mounding behind the up hydraulic gradient part of the wall. Given that the groundwater velocity within the alluvium is relatively high (compared to the underlying siltstone) it is expected that the mounding will be minimal as groundwater will flow around the wall as new steady state conditions are established.

12.2 Impact of potential contaminated groundwater.

Question 2: raised in discussion on 21 June 2022. *It has been suggested that since the land-use in the vicinity of the Rosehill Service Facility has been industrial for many years there may be pockets of groundwater contamination that could be mobilised due to the dewatering and hydraulic gradient reversals.*

Answer 2: A contaminated land investigation was conducted at the Rosehill Service Facility based on 14 boreholes within and close to the excavation footprint. This investigation is presented as Appendix D3 of the design report and is based on soil and groundwater samples from 14 boreholes. Groundwater samples were submitted to the laboratory for an extensive analytical suite including:

- Heavy metals
- Total recoverable hydrocarbons (TRH)
- Benzene, toluene, ethylbenzene, xylene (BTEX)
- Polyaromatic Hydrocarbons (PAH)
- Organochlorine pesticides (OCPs)
- Organophosphate pesticides (OPPs
- Volatile Organic Compounds (VOCs / xVOCs)
- Per and poly-fluoroalkyl substance and perfluorooctanoic acid (PFAS / PFOA)
- Amino Aliphatics
- Anilines
- E-Nitrobenzenes
- Herbicides
- Phenolics
- Phthalates
- Water quality parameters (pH, redox, electrical conductivity, temperature)

The results were compared against the adopted ecological groundwater investigation levels specified in the ANZECC at 95% species protection level for freshwater. Contaminants marginally exceeding these guidelines were heavy metals chromium, cadmium, copper, nickel and zinc as well as PAHs and ammonia. The concentrations exceeding the guidelines may be at background levels or representative of remnant industrial activity.

Review of the NSW EPA register of contaminated sites revealed 11 sites within a 1.5 km radius of the Rosehill Service Facility. The site location, distance from the Rosehill Service Facility, current owner and groundwater contaminant(s) are summarised in Table 6.

Table 6 NSW Registered Contaminated sites within 1.5 km of the Rosehill Service Facility

Location	Site	Distance from site	Contaminants in Groundwater
Durham Street, Rosehill	Former Shell refinery	300m east	PFOS, Pb, Cr^{6+}, TPH, BTEX, PAH
12 Grand Ave, Camellia	Former Cr plating factory and Bitumen manufacturer	1 km north	Cr^{6+}, TPH

Location	Site	Distance from site	Contaminants in Groundwater
14 Grand Ave, Camellia	Concrete Plant	1 km north	Cr^{6+},
39 Grand Ave, Camellia	Former Cr plating factory Asciano	1 km north	Cr^{6+},
1 Grand Ave, Camellia	Former James Hardy factory	1 km north	Zn, phenol; PAH
41 Grand Ave, Camellia	Former Cr plating factory Sydney Water	1 km north	Cr^{6+},
37 Grand Ave, Camellia	Former Cr plating factory, Veolia	1 km north	Cr^{6+},
13 Grand Ave, Camellia	Former Cr plating factory, Wrigg	1 km north	Cr^{6+},
4 Grand Ave Rosehill	Former Cr plating factory,	1 km north	Cr^{6+},VCHs
5 Devon St Rosehill	Former Asbestos factory, James Hardy	1km east	No groundwater contamination
2 Richie St, Rosehill	Former Asbestos factory, James Hardy	1km west	No groundwater contamination

At nine of the above sites groundwater contamination was identified. Two registered sites were asbestos factories and did not contain any groundwater contamination. The industrial sites that contribute to groundwater contamination are the former Shell refinery and chrome plating operations (Chrome Chemicals Australia Pty Ltd) that took place to the north of the Rosehill Service Facility. Currently contamination from these sites discharge into the Parramatta River. Groundwater contamination detected at the nine sites includes hexavalent chrome (Cr6+), Pb, Zn, TPH, BTEX, PAH, phenol, PFOS and VCH (volatile chlorinated hydrocarbons).

Despite there being low levels of groundwater contamination detected at the site, contaminated groundwater could be drawn into the drained basement from the industrial sites due to reversed hydraulic gradients over the 120 year design life of the Rosehill Service Facility.

Since the basement is designed as a drained basement there are no barriers that can be put in place, other than the D-Wall to prevent groundwater ingress from the alluvium, siltstone and sandstone. Thus, in the event contaminated groundwater seepage enters the basement a management strategy will be put in place to manage the risk. In accordance with the OEMP captured groundwater would be routinely monitored for a suite of contaminants including the contaminants identified at surrounding industrial sites prior to discharge. Initially a visual and olfactory inspection could detect any changes in groundwater quality such as the intersection of ammonia or hydrocarbons. Laboratory analytical results would confirm the presence and concentrations of contaminants and if additional treatment was required prior to discharge.

The risk of groundwater contaminants within the basement seepage exceeding the adopted guideline threshold is considered low because:

- predicted basement seepage is less than half than the allowable criteria

- the concentration of any intersected groundwater contamination will decline along the flow path due to natural attenuation, dilution by rainfall recharge and dilution as the contaminated groundwater is mixed with non contaminated groundwater within the drawdown footprint.
- Two dimensional groundwater modelling shows long term groundwater drawdown contour will extend between 80 and 160 metres. It is noted that the drawdown footprint does not necessarily match with the capture zone.

12.3 Impacts of rainfall recharge .

Question 3: Impacts of rainfall recharge. raised in discussion on 21 June 2022. *The impact of groundwater recharge on groundwater inflows was requested to be explained further.*

Answer 3: The omission of rainfall recharge in groundwater modelling conducted in this assessment was outlined in Section 10.2. It has been suggested that the rainfall recharge that infiltrates into the groundwater drawdown footprint could be significant.

As outlined above the groundwater drawdown footprint extends between 80 and 160 m. Thus the drawdown footprint area covers between approximately 5,026 m^2 and 80,400 m^2 . As outlined in Section 10.2 the recharge for the Hawkesbury Sandstone would be expected to be around 5% or 48.4 mm of the annual rainfall. This equates to a total recharge of 973 m^3/year or 2.67 m^3/day for a radius of 80 m. For a radius of 160 m the rainfall recharge would be 3,890 m^3/year or 10.6 m^3/day.

The maximum allowable drained inflow into the basement is 45,000 L/day or 45 m^3/day and the modelled inflow is 6.0 m^3/day. Hence if the above rainfall recharge estimates are added to the maximum daily predicted ingress the groundwater inflows would increase from 6.0 m^3/day to 8.7 m^3/day for an 80 m drawdown radius. Similarly for a 160 m drawdown radius the groundwater ingress to the basin would increase from 6.0 m^3/day to 16.6 m^3/day.

There is no time series groundwater level data available for the site to correlate rainfall events with groundwater recharge. From experience elsewhere in the Sydney Basin groundwater levels do not rise significantly in the confined Hawkesbury Sandstone suggesting that groundwater level fluctuations due to large rainfall events will be subdued and spread out over time. Consequently, it is considered that a large rainfall event will not substantially increase groundwater ingress to the basement.

In summary the impact of rainfall recharge over the drawdown footprint has been assessed and shown to not increase the basement inflows beyond the design criteria of 45 m^3/day for a 160 m drawdown radius.

Ignoring the beneficial effects of rainfall recharge has resulted in the groundwater drawdown calculations being conservative.

13.0 Sydney Metro Comments – July 2022

Comments were raised by C Shultz on 23 June 2022, which were separate to the comments register and were discussed in a meeting with Sydney Metro, Gamuda and the Independent Certifier on 1 July 2022. Questions and general comments are addressed here and relevant sections have been updated elsewhere in this report.

13.1 Construction sequencing and dewatering options

General Comment: *The HIR did not adequately outline the construction sequencing of how the D-Wall was to be constructed in stages. Two dewatering options were considered and discussed in the HIR with drawdown for each calculated however it was not clear in the report why two options were presented.*

Answer: The construction of the D-Wall in stages and the use of sheet piling is discussed in a new Section 2.4 to make the construction sequencing clear.

Preparation of the HIR has been undertaken in stages and the calculations are based on hydrogeological parameters derived from field investigations. Initially two dewatering options were considered based on two sets of hydrogeological parameters. Subsequent to these calculations additional geotechnical investigations including packer tests were conducted confirming the hydrogeological parameters at the Rosehill Services Facility. The confirmation of these parameters made Option 2 obsolete however this option was retained in the HIR for completeness. In this version of the HIR Option 2 and the discussion around Option 2 has been removed to make the report more focused and remove any confusion.

It is noted that drawing "SMWSTWTP-GLO-RSH-SF500-RS-DRG-010110 & 010111" has been amended to remove Option 2.

13.2 Groundwater drawdown in comparison to other Metro Stations

Question 1: *The drawdown is stated at less than 1m and of very limited extent, By comparison the Site wide HIR shows much greater drawdown extent at Parramatta (also a D-Wall) and at Clyde Dive access shaft,*

Answer 1: Additional cross sections have been compiled in the Slide2 model to provide additional data points to better represent the drawdown contours. Figure 8.1 has been amended accordingly with the maximum drawdown of 2.1m adjacent to the D-Wall with drawdown reducing asymmetrically away from the D-Wall due in response to the differential recharge influence from Duck Creek.

Discussions with GHD who are preparing the site wide HIR indicate the site geological and hydrogeological site conditions at the Parramatta and Clyde sites are different to those at the Rosehill Services Facility. It is understood that the extent of alluvium is greater at Clyde and Parramatta. It has been commented that the structures all intersect Class ii sandstone in accordance with the Pells classification system (Pells et, al, 1998) which should provide some commonality between the calculations. It should be noted that the Pells classification system is a geotechnical classification system and within each class of sandstone there is a wide range of hydrogeological properties including hydraulic conductivity. GHD confirm that the sandstone hydraulic conductivity at the Clyde and Parramatta structures is higher that at Rosehill which would lead to different inflow and drawdown results.

It is understood that the hydrogeological interpretation at the Clyde and Parramatta sites are to be revised in the next technical memos for their respective Design Stage 2 submission, where tighter drawdowns are predicted compared to the Design Stage 1.

The greatest difference between construction of the Rosehill Service Facility and the structures at Parramatta and Clyde is the construction techniques. At Rosehill the construction dewatering is to be staged to minimise groundwater drawdown, with groundwater modelling used to manage groundwater drawdown. It is understood that at the other sites the excavation will be a bulk excavation with dewatering undertaken to dewater the whole site at once, which will result in greater groundwater inflows and a more extensive drawdown footprint.

It has been suggested that differences in groundwater modelling techniques may have contributed to differences in the extent of the drawdown footprints between Rosehill and the other sites. GHD used three dimensional (3D) modelling (MODFLOW) to calculate the groundwater impacts at Clyde and Parramatta where as two dimensional modelling (2D) using SLIDE2 was applied at Rosehill. Reference to the Australian Groundwater Modelling Guidelines (Barnett, 1998) states that 2D modelling is valid for the construction of models where the approaches are adequate to address the modelling objectives. Since the theory and analyses behind the modelling platforms are the same any differences in the predicted results due to different modelling applications are considered minor.

13.3 Clarification of Groundwater Contamination

Question 2: The contamination report says there is contamination, but the hydrogeology report says there is none in the groundwater. Can clarification or correction be provided?

Answer 2: Clarification is provided in Section 12.2. The phase no contamination present has been corrected.

13.4 Saltwater Intrusion Clarification

Question 3: Various sections of the hydrogeology report say there is no inflow of saline water or that it may occur in 10 years. The report also states that when pumping stops the saline water will flow away from the box. We note there is no pumping and water will always flow into the under base drainage system. Can wording be corrected.

Answer 3: Text in the report has been amended to indicate there will be no saline water intrusion during construction from Duck Creek but saltwater intrusion is likely during the 120 design life of the project. The phrase flow reversal has been corrected. The question of saltwater intrusion is addressed in earlier versions of the HIR in Sections 11.1 and 11.3 with a sensitivity analysis and discussion and clarification provided. The implication of saltwater intrusion is addressed in the Durability Report (Appendix D4).

13.5 Cumulative Drawdown Impacts

Question 4: There is no indication of how the various geohydrology impacts overlap / accumulate. Can a combined HIR be drafted which considers these combined effects? Is the risk of groundwater drawdown from the Clyde Dive a risk to the structure and durability of the Rosehill Service Facility? Are there any settlement impacts due to the cumulative drawdown?

Answer 4: It is understood in the current construction schedule the Rosehill Service Facility will be constructed first followed by the Clyde and Parramatta structures (unknown sequencing or perhaps both are to be constructed at the same time). Depending on the project scheduling the construction of the Rosehill Services Facility may be complete prior to the construction of the Clyde and Parramatta structures. In this case the cumulative impacts due to the construction of the Rosehill structure will be the on-going basement drainage.

Preliminary modelling undertaken by GHD indicates the predicted drawdown at the Rosehill Service Facility due to construction of the Clyde Dive will be in the order of 0.5 m. This worst case scenario of the combined cumulative drawdown due to the construction of the Clyde Dive and Rosehill Services Facility will be the total drawdown of 0.5m at Clyde and up to 1.2 m at Rosehill giving a maximum total predicted cumulative drawdown of 1.7 m, adjacent to the D-Wall.

It is noted that predicting a cumulative drawdown is not as simple as adding the two drawdown predictions together and, in all likelihood, the actual cumulative drawdown is expected to be less than this total. Cumulative drawdown impacts due to the overlap drawdown footprints are to be assessed in the site wide HIR being prepared by GHD. Calculated groundwater drawdown contours due to the Rosehill Service Facility have been provided to GHD for input into this assessment.

The maximum 1.7 m cumulative drawdown is unlikely to have an impact on the structure of the Rosehill Service Facility as the drawdown is within the range of natural groundwater level fluctuation and any settlement due to lower groundwater levels would have already taken place. Potential durability impacts on the Rosehill Service Facility are outlined in the saltwater intrusion discussion (Section 11.1, 11.3 and 13.4) where the building materials have been designed for elevated salinity and low pH. Generation of acidic groundwater due to drawing down the watertable and exposing potential acid sulfate soils is not considered a problem since acid sulphate soils will not be exposed.

A review of settlement due to cumulative groundwater drawdown will be undertaken in the site wide HIR and interim technical memo at the Clyde Dive subject to review of other cumulative impacts from other nearby site activities.

13.6 Groundwater drawdown contours - limitations

Question 5: The generated groundwater drawdown contours for the Rosehill Service Facility were symmetrical and did not account for the recharge influence of Duck Creek.

Answer 5: Figure 8.1 showing predicted groundwater drawdown has been amended. The revised contours are based on three cross sections through the excavated structure with engineering judgement applied to complete the contours. Recharge from Duck Creek is applied with a constant head boundary. As expected the contours are elongated on the north western side of the structure where the influence of recharge from Duck Creek is diminished.

13.7 Groundwater monitoring

Question 6: Can groundwater level trigger levels be amended during construction to reflect a staged monitoring plan to incorporate global effects if the drawdown exceeds one metre once excavation commences in other areas?

Answer 6: Groundwater trigger levels have been amended in the GIR based on the cumulative groundwater level predictions. It should be noted that groundwater level trigger levels are a guide only and if exceeded it is a trigger to review nearby construction activities to develop a response, to either alter construction activities or revise trigger levels. It is noted that the policy governing groundwater impacts in NSW "The Aquifer Interference Policy (NoW, 2012)" allows 2m of drawdown for an activity before any "make good" responses need to be considered.

14.0 Limitations

The following limitations of the analysis apply in conducting the seepage analysis.

1. An inferred dyke may be present in the western (Westmead) end of the Rosehill Service Facility but was not intersected during geotechnical drilling.
2. Flow rates have been calculated using the available aquifer parameters. These parameters are locally derived from packer tests and slug tests but could vary and hence the estimated inflows could be different to the actual in the field. The baseline values (as detailed by SMEC and GHD with amendment made based on latest available packer test results) have been applied in the modelling rather than the upper and lower values. It is noted that the structural geology is not well mapped and there could be water bearing structures intersected during construction of the shaft structure such as faults, dykes and joints.
3. The water tightness of the D-Wall is dependant upon the skill of the contractor and there could be leakages between panels or from the base where there is no grouting proposed.
4. The estimated inflows are based on theoretical calculations based on estimated aquifer parameters and the actual flow rate in the field may be different due to the variability in the geology intersected and hydraulic conductivity of the sandstone and siltstone units. Groundwater analysis cannot account for the full complexity of the hydrogeological conditions and as such contain some uncertainty.
5. Calculation prediction for groundwater inflow and drawdown during construction cannot be made until the construction sequencing of the D-Walls is further developed.

15.0 Recommendations

Groundwater monitoring wells are to be installed to monitor groundwater levels during construction and during operations as outlined in the GIR.

The drained base option is feasible and the most likely expected groundwater inflow during construction and for the permanent case is predicted by modelling to be less than the project limiting criteria.

The provision of contingency measures could be considered to mitigate against the standard risks associated with geotechnical and hydrogeological uncertainty. These measures may be further considered in subsequent design stages.

16.0 References

- Barnett B, Townley LR, Post V, Evans RE, Hunt RJ, Peeters L, Richardson S, Werner AD, Knapton A and Boronkay A (2012); Australian Groundwater Modelling Guidelines, Waterlines Report Series No 82, National Water Commission, Canberra, 191 pp. June.

- Coffey 2021; Hydrogeology and Groundwater Interpretative Report – Project Wide. M6 Motorway Stage 1, Package RP18. Prepared for CPB, Ghella and UGL, dated October.
- Department of Planning, 2021; Sydney Metro West – Concept and Stage 1 Conditions of Approval. Critical State Significant Infrastructure. Section 5.19 of the Environmental Planning & Assessment Act 1979. Application No SSI 10038, dated 11 March.
- Gamuda, 2021; Seepage Analysis – Sydney Metro: Rosehill Service Facility. Technical Memo, dated 26 November.
- GHD and SMEC, 2021a; Sydney Metro West. Tender Advice Note. Provision of groundwater inflow and drawdown data for tender design. TAN No DJV-046-GT3, dated 2 May 2021.
- GHD and SMEC, 2021b; Sydney Metro West. Tender Advice Note. Geotechnical Interpretative Report. TAN No DJV-045-GT2-Rev1, dated 21 May 2021.
- Hawkes G., 2017; Analysis of aquifer parameters in the Triassic Sydney Basin during the WestConnex hydrogeological investigations. Australasian Groundwater Conference, University of NSW, Kensington, Sydney, 11 – 13 July 2017.
- HydroSimulations, 2017; WestConnex M4-M5 Link. Groundwater Modelling Report. Prepared for AECOM Pty Ltd. Report No HS2017/01 AEC003, dated June
- NoW, 2012; NSW Aquifer Interference Policy. State of NSW, Department of Trade and Investment, Regional Infrastructure Services. NSW Office of Water.
- Pells P.J.N., Mostyn G and Walker B.F., 1998; Foundations on Sandstone and Shale in the Sydney Region. Australian Geomechanics, Vol 33 No 3, pp 17-29.
- RPS, 2018; F6 Extension Stage 1, Groundwater Modelling Report. Prepared for AECOM Pty Ltd. Report No EWP72727.001, Draft B, dated April.

ATTACHMENT 3

Drawing – Trigger Levels and Response Strategies

GENERAL NOTES

1. FOR GENERAL NOTES REFER TO SMWSTWTP-GLO-RSH-SF500-RS-DRG-010010 TO 010011.

GENERAL NOTES FOR INSTRUMENTATION AND MONITORING PLAN

1. ALL NECESSARY PRECAUTIONS SHALL BE TAKEN TO PROTECT THE INSTRUMENTS AND MAINTAIN THE INSTRUMENTS FROM DISTURBANCES AND IN GOOD WORKING CONDITION AFTER COMMISSIONING. FOR ALL INSTRUMENTS WHICH PROJECT THROUGH AND ABOVE THE GROUND, SPECIAL PRECAUTIONS SHALL BE TAKEN TO PROVIDE PROTECTION FROM VEHICLES AND PLANT INCLUDING SUBSTANTIAL AND READILY VISIBLE BARRIERS AT A DISTANCE OF 750MM AROUND EACH INSTRUMENT. HEAVY EQUIPMENT SHALL NOT APPROACH WITHIN 1.0M OF PROJECTING INSTRUMENTS. DAMAGED INSTRUMENTS SHALL BE REPLACED OR REPAIRED BY THE CONTRACTOR WITHIN SEVEN (7) DAYS.
2. ALL INSTRUMENTS SHALL BE LABELLED WITH THEIR REFERENCE NUMBER AT THE LOCATION WHERE READINGS OR MEASUREMENTS ARE TAKEN. ALL INSTRUMENTS SHALL BE LABELLED/TAGGED USING PLASTIC NAMEPLATES CONSISTENT FOR ALL SITES OF SUITABLE SIZE AS AGREED BY THE DESIGNER AND SHALL BE ALWAYS MAINTAINED. THE INSTRUMENT'S ID OR NAMING CONVENTION SHOULD BE CONSISTENT FOR ALL TYPES.
3. THE EXACT LOCATION OF INSTRUMENTS TO BE INSTALLED SHALL BE CONFIRMED ON SITE AND AGREED BY THE DESIGNER PRIOR TO INSTALLATION.
4. WHERE PROPOSED INSTRUMENTATIONS OBSTRUCTED BY CONSTRUCTION WORKS, PUBLIC TRAFFIC, OR OTHER OBSTRUCTION, AN ALTERNATIVE LOCATION SHALL BE AGREED WITH THE DESIGNER.
5. ALL NECESSARY PRECAUTIONARY MEASURES SHALL BE IMPLEMENTED DURING THE CONSTRUCTION WORKS TO PROTECT AND MINIMIZE SETTLEMENT OF GROUND, BUILDING, STRUCTURES, SLOPE, WALL, AND UNDERGROUND UTILITIES.
6. NON-DESTRUCTIVE DRILLING SHALL BE UNDERTAKEN IN THE VICINITY OF UNDERGROUND UTILITIES OR OTHER STRUCTURES PRIOR TO INSTALLATION OF INSTRUMENTS WHICH REQUIRE DRILLING. THE DIMENSIONS OF THE PIT SHALL NOT BE LESS THAN 0.25 SQUARE METER ON PLAN AND NOT MORE THAN 3.0 METERS IN DEPTH.
7. GENERALLY, THE BASELINE READINGS SHALL BE TAKEN AT LEAST ONE (1) MONTH ON THREE (3) CONSECUTIVE WEEKLY READING PRIOR TO CONSTRUCTION ACTIVITIES (WITHIN THE INSTRUMENT'S VICINITY) AND SHALL NOT BE ALLOWED TO COMMENCE ANY CONSTRUCTION WORK UNTIL ALL INSTRUMENTATION ARE IN PLACE AND BASELINE READINGS HAVE BEEN SUBMITTED AND ACCEPTED BY THE DESIGNER. THE BASELINE READINGS FOR INC AND VWSG SHALL BE TAKEN AS AVERAGE OF REAL TIME READINGS OF MIN ONE (1) WEEK PRIOR TO THE EXCAVATION WORK WITHIN THE INSTRUMENT'S VICINITY. FOR ANY ADDITIONAL/REPLACEMENT INSTRUMENTS, BASELINE SHALL BE ESTABLISHED BASED ON 3 CONSECUTIVE READINGS AS PER MONITORING FREQUENCY.
8. THE BASELINE READINGS OF ALL INSTRUMENTS SHALL BE AGREED WITH THE DESIGNER UPON CONSIDERING THE SEQUENCE OF WORKS.
9. FOR BASELINE, THREE (3) SETS OF READING SHALL BE TAKEN BEFORE MAJOR ACTIVITY TAKE PLACE AND SHALL BE AVERAGED, (EXCEPT FOR GROUNDWATER LEVELS). REPRESENTING THE CONDITION BEFORE EXCAVATION. IF THERE ARE SIGNIFICANT DIFFERENCES OR ANOMALIES IN THE READINGS, FURTHER READINGS SHALL BE TAKEN TO OBTAIN CONFIRMATION.
10. PRIOR TO COMMENCEMENT OF WORK, THE READOUT EQUIPMENT SHALL BE ENSURED TO BE IN GOOD WORKING CONDITION AND THE CALIBRATION OF EQUIPMENT IS STILL VALID.
11. INSTALLATION RECORD SHEET SHALL BE PREPARED FOR EACH INSTRUMENT INSTALLED. THE FORMAT OF THE MONITORING SHEET SHALL BE SUBMITTED TO THE DESIGNER FOR APPROVAL AT LEAST 28 DAYS BEFORE INSTALLATION COMMENCES.
12. DETAILS OF SETTING OUT CONTROL POINTS, BENCHMARKS AND TEMPORARY BENCHMARKS SHALL BE SUBMITTED TO THE DESIGNER FOR AGREEMENT PRIOR TO THE START OF SURVEY WORKS.
13. DILAPIDATION SURVEY SHALL BE CARRIED OUT ON EXISTING BUILDING/STRUCTURES PRIOR TO COMMENCEMENT OF WORKS.
14. ALL THE REPORTS OF DILAPIDATION SURVEY, BUILDING/STRUCTURE CONDITION SURVEY AND BASELINE READINGS/SURVEY SHALL BE APPROVED BY THE DESIGNER PRIOR TO COMMENCEMENT OF WORKS.
15. THE CONTRACTOR SHALL BE RESPONSIBLE TO OBTAIN PERMITS AND PERMISSION FOR THE INSTALLATION OF THE INSTRUMENTS WHERE THE INSTRUMENTS ARE TO BE INSTALLED OUTSIDE THE WORKS BOUNDARY.
16. CONSTRUCTION DRAWINGS SHALL BE PREPARED SHOWING THE EXACT INSTRUMENT LOCATIONS AND METHOD STATEMENTS FOR ALL THE PROPOSED INSTRUMENTS, INCLUDING INSTALLATION METHOD, SPECIFICATION OF INSTRUMENT, PRODUCT INFORMATION AND SAMPLES (IF NECESSARY), FOR APPROVAL BY THE DESIGNER. THE LOCATION REFERENCE PLANS FOR ALL 'AS-BUILT' INSTRUMENTATION, WHICH SHALL INCLUDE COORDINATE AND LEVEL (IF RELEVANT) INFORMATION FOR INDIVIDUAL INSTRUMENTS SHALL ALSO BE PROVIDED.
17. ALL MONITORING DATA SHALL BE CRITICALLY VALIDATED PRIOR TO SUBMISSION TO THE DESIGNER FOR APPROVAL.
18. CRITICAL INSTRUMENTATION SHALL BE CONNECTED TO DATA LOGGING EQUIPMENT AND THE REAL TIME DATA SHALL BE CONTINUOUSLY ACCESSIBLE ON COMPUTERS IN THE SITE OFFICE.
19. THE LEVEL OF A DEEP DATUM OR PERMANENT BENCHMARK SHALL BE ESTABLISHED BY STANDARD LEVELLING TECHNIQUES FROM AGREED BENCHMARKS IN THE VICINITY. LEVELLING SHALL BE CLOSED BACK TO THE TEMPORARY BENCHMARKS TO CHECK THE ACCURACY. THE LEVEL SHALL BE MEASURED THREE TIMES SOON AFTER INSTALLATION OF THE DATUM AND SHALL BE CHECKED EVERY 3 MONTHS.
20. ALL MONITORING INSTRUMENTS SHALL BE REMOVED AFTER DECOMMISSIONING. THE CONTRACTOR SHALL REINSTATE THE GROUND AND STRUCTURES AFTER DISMANTLING.

INSTRUMENTATION AND MONITORING FOR STRUCTURES/UTILITIES

1. SETTLEMENT MARKER (SM)

1.1 SM IS PROPOSED TO MONITOR GROUND SURFACE MOVEMENT.
1.2 THE GALVANIZED ROD SHALL BE DRIVEN INTO THE GROUND AT MINIMUM DEPTH OF 1.0M.
1.3 THE MONITORING SHALL BE CARRIED OUT USING MANUAL SURVEY.
1.4 TOP OF THE RODS LEVEL SHALL BE MEASURED TO +/- 1mm ACCURACY.
1.5 THE SETTLEMENT MARKER SHALL BE SECURED WITH CONCRETE BELOW THE GROUND LEVEL OR CONCRETE SLAB AS INDICATED IN THE DRAWING.

2. OPTICAL PRISM (OP)

2.1 OP IS PROPOSED TO MONITOR THE MOVEMENT OF THE AFFECTED STRUCTURES.
2.2 PRISM ARE MOUNTED IN PAIRS ON AFFECTED STRUCTURES WITH A TOTAL STATION SET UP AT A DESIGNATED LOCATION. VERTICAL DISTANCE BETWEEN EACH PAIR OF PRISMS SHALL BE AT LEAST 1.5M.
2.3 PRECAUTION SHALL BE MADE TO ENSURE THE PRISMS ARE FACING TOWARD THE TOTAL STATION POSITION WITHOUT ANY OBSTRUCTION IN BETWEEN.
2.4 REFRACTION OF LIGHT FROM THE SURROUNDING ENVIRONMENT AND PASSING OF VEHICLES SHALL BE AVOIDED WHENEVER POSSIBLE.
2.5 CRACK METER SHALL BE INSTALLED ON THE FORMER RTA DEPOT FAÇADE STRUCTURE IF NEW CRACKS OCCURRED DURING THE EXCAVATION WORKS.
2.6 OP SHALL BE FIXED ON THE STRUCTURE USING 3-DIMENSIONAL 'L' ADJUSTMENT BRACKET AND BOLT SYSTEM OR APPROVED ADHESIVE.

3. UTILITIES SETTLEMENT GAUGE (SG)

3.1 SG IS PROPOSED TO MONITOR VERTICAL MOVEMENT (SETTLEMENT AND HEAVE) OF BURIED UTILITIES.
3.2 CARE SHALL BE TAKEN DURING THE INSTALLATION IN ORDER NOT TO DAMAGE THE UTILITIES.
3.3 NECESSARY PRECAUTION SHALL BE TAKEN TO PROTECT THE INSTRUMENTS AND MAINTAIN THE INSTRUMENTS IN GOOD WORKING ORDER AFTER COMMISSIONING. TO PROTECT INSTRUMENTS PROTRUDING ABOVE THE GROUND, LOCKABLE STEELS PROTECTIVE COVER AND VISIBLE REFLECTIVE BARRIERS AT RADIUS OF 750MM AROUND THE INSTRUMENTS SHALL BE INSTALLED.
3.4 LEVEL SHALL BE TAKEN ON TOP OF THE 25MM DIAMETER ROD.

4. INCLINOMETER IN WALL (INC)

4.1 INC IS PROPOSED TO MEASURE LATERAL MOVEMENT OF DIAPHRAGM WALL.
4.2 BASE OF INCLINOMETER SHALL BE INSTALLED AT 2M BELOW TOE OF DIAPHRAGM WALL AND SOCKETED INTO ROCK.
4.3 SPECIAL CARE SHALL BE TAKEN TO ENSURE THE STEEL PIPE ATTACHED ON BACK FACE OF STEEL REINFORCEMENT CAGE IS NOT DAMAGED DURING THE DIAPHRAGM WALL CASTING.
4.4 A BOREHOLE OF NOMINAL 100MM DIAMETER SHALL BE DRILLED THROUGH STEEL ACCESS PIPE AND INTO THE ROCK UNDERLYING THE TOE OF THE WALL FOR SOCKETING.
4.5 THE FINAL INSTALLATION DEPTH SHALL BE CONFIRMED BY THE DESIGNER UPON REVIEW THE ROCK SAMPLES BELOW TOE OF DIAPHRAGM WALL.
4.6 THE INCLINOMETER ACCESS TUBE WITH NOMINAL DIAMETER OF 70MM SHALL BE INSTALLED IN 150MM DIAMETER STEEL PIPE PRE-INSTALLED INSIDE THE DIAPHRAGM WALL
4.7 THE BOTTOM OF TUBE SHOULD BE SEALED WITH PVC END CAP AND THE TOP IS COVERED WITH A REMOVABLE PROTECTION CAP, COMPLETELY SEALED BY SEALING TAPES.
4.8 THE ANNULAR SPACE BETWEEN STEEL PIPE AND INCLINOMETER ACCESS TUBING SHALL BE FILLED WITH CEMENT GROUT HAVING SIMILAR COMPRESSIVE STRENGTH WITH DIAPHRAGM WALL.
4.9 THE DATA IS OBTAINED BY MANUALLY RETRACTING THE PROBE IN ORDER TO GET A SERIES OF WALL MOVEMENT.
4.10 BEFORE PASSING THE TORPEDO DOWN THE ACCESS TUBE FOR THE MONITORING WORK, A DUMMY TORPEDO SHOULD BE LOWERED TO THE BASE OF THE TUBE AND PULLED UP TO CHECK FOR ANY OBSTRUCTIONS. THE INCLINOMETER TORPEDO SHALL THEN BE LOWERED TO THE BASE OF THE ACCESS TUBE AND THE READINGS ARE TAKEN FROM BOTTOM FOR EVERY 0.5M UNTIL THE TORPEDO REACHES THE TOP. THE READINGS SHALL BE STORED IN THE DATA LOGGER.
4.11 ALTERNATIVE OPTIONS FOR MONIOTING THE INCLINOMETERS CAN BE PROPOSED BY THE CONTRACTOR SUCH AS INPLACE INCLINOMETER SYSTEMS WITH SENSOR SPACING OF 1.5m.

5. VIBRATING WIRE STRAIN GAUGE (VWSG)

5.1 VWSG IS PROPOSED TO MEASURE STRAIN OR LOAD ON AN STRUT/STRUCTURE.
5.2 VIBRATING WIRE STRAIN GAUGE SHALL BE PROTECTED USING STEEL COVER OR ANY OTHER METHOD IN ORDER TO AVOID ANY DAMAGE TO THE INSTRUMENTS.
5.3 THE STEEL PROTECTIVE COVER SHALL BE FIXED WITH SOME DISTANCE FROM THE GAUGE SO THAT IT DOES NOT INTERFERE WITH THE PERFORMANCE OF THE GAUGE.
5.4 CABLES SHOULD BE CONCEALED OR ROUTED SO THAT THEY ARE UN-LIKELY TO BE DAMAGED DURING THE CONSTRUCTION.
5.5 THE CABLES SHALL BE TEMPERATURE RATED TO A MINIMUM RANGE OF 0°C TO 50°C.

6. VIBRATING WIRE PIEZOMETER (VWP)

6.1 VWP IS PROPOSED TO PROVIDE THE WATER PRESSURE MEASUREMENTS AT THREE SPECIFIC DEPTHS WITHIN THE SOIL PROFILE.
6.2 THE RELEVANT DOCUMENTS RELATING TO VWP SUCH AS THE CALIBRATION CERTIFICATE AND SPECIFICATION SHALL BE SUBMITTED TO THE DESIGNER FOR APPROVAL.
6.3 THE VWP SHOULD BE INSTALLED IN THE BOREHOLE WITH NOMINAL DIAMETER OF 75mm, CASED TO THE DEPTH WHERE THE STRATA ARE SUFFICIENTLY COMPETENT FOR THE HOLE TO REMAIN OPEN.
6.4 PRIOR TO THE INSTALLATION AT SITE, THE PIEZOMETER TIP WILL BE PRESSURE TESTED IN THE CONTAINER OF WATER WITH PRESSURE TO CHECK FOR POOR CONNECTIONS AND FUNCTIONALITY.
6.5 PRIOR TO INSTALLATION AT SITE, THE PIEZOMETER ASSEMBLY WILL BE LEFT IN THE WATER FILLED CONTAINER TO ALLOW THE TEMPERATURE AND PRESSURE WITHIN THE PIEZOMETER TO STABILIZE. A "ZERO PRESSURE" READING WILL THEN BE RECORDED.
6.6 IF WATER IS NOT FOUND IN THE BOREHOLE, WATER WILL BE ADDED TO AT LEAST THE INTENDED DEPTH OF THE SAND CELL AND BENTONITE PLUG TO COMPACT AND SATURATE THE SAND CELL AND TO SATURATE THE BENTONITE PELLETS. COARSE, CLEAN FILTER SAND TO THE SPECIFIED GRADING WILL BE POURED THROUGH THE WATER TO THE PROPOSED BASE OF THE PIEZOMETER TIP AND ALLOWED TO SETTLE.
6.7 COARSE, CLEAN FILTER SAND WILL BE POURED SLOWLY THROUGH THE WATER UNTIL THE SAND CELL FILLED WITH REQUIRED LENGTH. DEPTHS WILL BE TAKEN USING THE WEIGHTED MEASURING TAPE, AFTER SUFFICIENT TIME HAS BEEN ALLOWED FOR THE SAND TO SETTLE, THE SAND WILL BE LIGHTLY TAMPED IF NECESSARY.
6.8 A READING WITH READOUT WILL BE TAKEN TO ENSURE THAT THE INSTRUMENT IS SENSIBLY INDICATING THE KNOWN HEAD OF WATER WITHIN THE BOREHOLE.
6.9 THE BENTONITE PELLETS WILL BE DROPPED THROUGH THE WATER AND TAMPED AS NECESSARY.

FREQUENCY OF MONITORING

1. CLOSER FREQUENCY OF MONITORING MAY REQUIRE IF THE READING SHOW UNFAVOURABLE TREND UPON INSTRUCTION OF THE DESIGNER.
2. MONITORING SHALL BE TERMINATED AFTER THREE (3) MONTHS OF STABLE READINGS FOLLOWING THE COMPLETION OF CONSTRUCTION WORKS.

TRIGGER LEVEL	TRIGGER VALUE (% OF PREDICTED VALUE)	CONTROL PROCEDURES / REQUIREMENT
ALERT	70-80	1 NOTIFY THE DESIGNER IMMEDIATELY WITH DESCRIPTION OF RELATED WORKS IN THE VICINITY OF THE INSTRUMENT. 2 CONFIRM THAT THE READING IS RELIABLE i.e. NOT RELATED TO INSTRUMENT AND HUMAN ERRORS OR ABNORMALITIES, AND RECTIFY THE CAUSES OF THE ERRONEOUS READINGS, IF ANY. 3 CONDUCT A VISUAL INSPECTION OF THE AFFECTED BUILDING / STRUCTUR... 4 INITIATE RELEVANT TRIGGER ACTION PLAN. 5 MONITORING FREQUENCY AND APPLICABILITY OF TRIGGER LEVELS TO BE REVIEWED.
ALARM	80	1 NOTIFY THE DESIGNER IMMEDIATELY WITH DESCRIPTION OF RELATED WORKS IN THE VICINITY OF THE INSTRUMENT. 2 REVIEW FREQUENCY AND APPLICABILITY OF TRIGGER LEVELS. IF NECESSARY, INSTALL ADDITIONAL INSTRUMENTS OR INCREASE THE FREQUENCY OF MONITORING. 3 INITIATE RELEVANT TRIGGER ACTION PLAN INCLUDING REPORTING PROCEDURE. 4 DISCUSSION BETWEEN THE RELEVANT PARTIES (i.e. DESIGNER, CONTRACTOR AND OTHER STAKEHOLDERS) TO ESTABLISH THE NEXT STEP FORWARD AND APPROPRIATE RESPONSE TO THE ALERT.
ACTION	>100	1 SUSPEND ALL CONCERNED WORKS WITHIN THE AGREED ZONE OF DANGER. 2 NOTIFY THE DESIGNER IMMEDIATELY WITH DESCRIPTION OF RELATED WORKS IN THE VICINITY OF THE INSTRUMENT. 3 CONDUCT A JOINT INSPECTION OF THE AFFECTED BUILDING / STRUCTURE AND RELATED WORKS WITH THE DESIGNER. 4 INITIATE RELEVANT TRIGGER ACTION PLAN INCLUDING REPORTING PROCEDURE AND CONTINGENCY PLAN. 5 REVIEW MONITORING FREQUENCY AND APPLICABILITY OF TRIGGER LEVELS. 6 RECOMMENCE THE AFFECTED WORKS UPON DEMONSTRATING TO THE DESIGNER THAT IT IS SAFE TO DO SO; UPON AGREEMENT WITH ALL RELEVANT PARTIES.

INSTRUMENT	PRIOR TO EXCAVATION	DURING EXCAVATION	AFTER BASE SLAB COMPLETION
SURFACE SETTLEMENT MARKERS	WEEKLY	DAILY	WEEKLY
OPTICAL PRISM	WEEKLY	DAILY	WEEKLY
WALL INCLINOMETER	WEEKLY	DAILY	WEEKLY
SANDPIPE PIEZOMETER	REAL TIME	REAL TIME	REAL TIME
VW PIEZOMETER	REAL TIME	REAL TIME	REAL TIME
VW STRAIN GAUGE	NA	REAL TIME	REAL TIME
FORMER RTA DEPOT FAÇADE, EXISTING UTILITIES AND TEMPORARY SEWER	WEEKLY	DAILY	WEEKLY

NOTES

1. FOR GENERAL NOTES REFER TO SMWSTWTP-GLO-RSH-SF500-RS-DRG- 10010 TO 10011

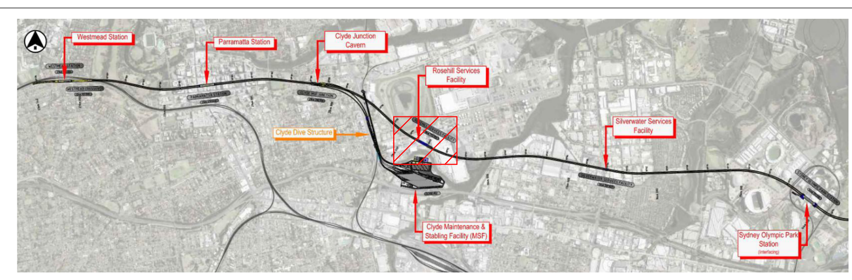
OFFICIAL

FOR REVIEW AND COMMENT

REV	AMENDMENT DESCRIPTION	Design by	Verified by	Approved Initial/Date	
B	STAGE 3 DETAILED DESIGN RE-SUBMISSION	DL	MC	RS	10/06/22
A	STAGE 3 DETAILED DESIGN SUBMISSION	DL	MC	RS	28/04/22

SCALE: 1:100 @ A1 0 1 2 3 4 5 m

KEYPLAN:



CLIENT: NSW GOVERNMENT / sydney METRO

INTEGRATING DESIGN AEO: GHD / SMEC

SMEC | GHD DESIGN JOINT VENTURE

The information shown on this drawing is for the purposes of the Sydney Metro Project only. No warranty is given or implied as to its suitability for any other purpose. The Service Providers accept no liability arising from the use of this drawing and the information shown thereon for any purpose other than the Sydney Metro Project.

SERVICE PROVIDERS: GAMUDA Australia / LAING O'ROURKE / aurecon

AURECON
RETAINING STRUCTURES

DRAWN	YOLANDE BLIGNAUT	10.06.2022
DESIGNED	FIONA MA	10.06.2022
DRG CHECK	DAVID STREVENS	10.06.2022
DESIGN CHECK	SHARMEELEE SUBRAMANIAM	10.06.2022
APPROVED	ROB SMITHSON	10.06.2022

SYDNEY METRO WEST
SERVICE FACILITY - ROSEHILL
RETAINING STRUCTURES
INSTRUMENTATION FOR EXCAVATION WORKS
SHEET 1

DOCUMENT No: - SHEET: 1 OF 4 ©
STATUS: STAGE 3 DETAILED DESIGN EDMS NO: -
DRG No.SMWSTWTP-GLO-RSH-SF500-RS-DRG-010115 REV B VER -

A1 Original | Co-ordinate System: - | Height Datum: - | This sheet may be prepared using colour and may be incomplete if copied | NOTE: Do not scale from this drawing.

PIEZOMETER COORDINATES		
PIEZOMETER	EASTING	NORTHING
ENV282_w	317055.700	6255217.100
ENV283_w	317145.200	6255207.800
ENV284_w	317142.400	6255142.500
ENV293_w	316920.700	6255138.200
ENV812_w	317336.900	6255189.300
ENV813_w	317362.000	6255210.300
ENV814_w	317311.400	6255174.100
WTP_BH13_w	317215.000	6255249.300
WTP_BH16_w	317316.800	6255135.100
WTP_BH17_w	317353.800	6255064.000
WTP_BH18_w	317164.400	6255141.200

BOREHOLE COORDINATES		
BOREHOLE	EASTING	NORTHING
RSF_BH1	317242.500	6255182.200
RSF_BH1(P)	317243.400	6255180.900
RSF_BH2	317325.400	6255114.700
RSF_BH2(P)	317326.300	6255113.500
RSF_BH4	317268.900	6255144.200
RSF_BH5	317282.200	6255126.000
RSF_BH6	317203.300	6255230.500
RSF_BH7	317263.700	6255039.000

INCLINOMETER COORDINATES		
INCLINOMETER	EASTING	NORTHING
INC1	317249.318	6255200.045
INC2	317271.576	6255184.869
INC3	317316.671	6255119.757
INC4	317323.064	6255092.771
INC5	317270.899	6255149.179

SM COORDINATES		
SM	EASTING	NORTHING
SM1	317315.537	6255228.485
SM2	317287.278	6255239.469
SM3	317259.624	6255227.525
SM4	317197.869	6255252.019
SM5	317167.132	6255237.652
SM6	317188.876	6255169.731
SM7	317151.053	6255173.859
SM8	317346.462	6255152.535
SM9	317365.659	6255150.553
SM10	317207.394	6255120.627
SM11	317261.983	6255034.456
SM12	317378.179	6255102.299
SM13	317259.133	6255108.839

SG COORDINATES		
SG	EASTING	NORTHING
SG1	317313.680	6255156.213
SG2	317269.336	6255220.106
SG3	317229.213	6255205.533
SG4	317222.749	6255165.509

GENERAL ARRANGEMENT PLAN GEOTECHNICAL MONITORING

SCALE 1 : 500

LEGEND:

- LOCATION OF OPTICAL PRISMS (OP1 TO OP4 IN PAIR)
- LOCATION OF ADDITIONAL BOREHOLES WITH STANDPIPE PIEZOMETER (RSF_BH1, BH2, BH5, BH6 AND BH7)
- LOCATION OF BOREHOLES (RSF_BH4) WITH VWP AND WTP_BH18 WITH VWP
- LOCATION OF UTILITY SETTLEMENT GAUGE FOR SEWER (SG1 TO SG4)
- LOCATION OF SETTLEMENT MARKERS (SM1 TO SM13)
- LOCATION OF EXISTING PIEZOMETER
- INCLINOMETER (INC1 TO INC5)
- 10mm ESTIMATED SETTLEMENT CONTOUR

NOTES

1. FOR GENERAL NOTES REFER TO SMWSTWTP-GLO-RSH-SF500-RS-DRG- 10010 TO 10011
2. FOR GENERAL NOTES ON INSTRUMENTATION REFER TO SMWSTWTP-GLO-RSH-SF500-RS-DRG- 10115
3. DRAWING SHOWN ALIGNMENT FOR TEMPORARY SEWER DATED 20/05/2022

OFFICIAL

FOR REVIEW AND COMMENT

REV	AMENDMENT DESCRIPTION	Design by	Verified by	Approved Initial/Date	
C	STAGE 3 DETAILED DESIGN RE-SUBMISSION	DL	MC	RS	07/07/22
B	STAGE 3 DETAILED DESIGN RE-SUBMISSION	DL	MC	RS	10/06/22
A	STAGE 3 DETAILED DESIGN SUBMISSION	DL	MC	RS	28/04/22

SCALE: 1:500 @ A1 0 5 10 15 20 25 m

KEYPLAN:

A1 Original | Co-ordinate System: - | Height Datum: - | This sheet may be prepared using colour and may be incomplete if copied | NOTE: Do not scale from this drawing.

CLIENT: NSW GOVERNMENT | sydney METRO

INTEGRATING DESIGN AEO: GHD | SMEC

SMEC | GHD DESIGN JOINT VENTURE

The information shown on this drawing is for the purposes of the Sydney Metro Project only. No warranty is given or implied as to its suitability for any other purpose. The Service Providers accept no liability arising from the use of this drawing and the information shown thereon for any purpose other than the Sydney Metro Project.

SERVICE PROVIDERS: GAMUDA Australia | LAING O'ROURKE

aurecon

AURECON
RETAINING STRUCTURES

DRAWN	YOLANDE BLIGNAUT	07.07.2022
DESIGNED	FIONA MA	07.07.2022
DRG CHECK	DAVID STREVENS	07.07.2022
DESIGN CHECK	SHARMEELEE SUBRAMANIAM	07.07.2022
APPROVED	ROB SMITHSON	07.07.2022

SYDNEY METRO WEST

SERVICE FACILITY - ROSEHILL
RETAINING STRUCTURES
INSTRUMENTATION FOR EXCAVATION WORKS
SHEET 2

DOCUMENT No: - | SHEET: 2 OF 4 | ©
STATUS: STAGE 3 DETAILED DESIGN | EDMS NO.: -
DRG No. SMWSTWTP-GLO-RSH-SF500-RS-DRG-010116 | REV C | VER -

100mm AT FULL SIZE Plot Date & Time 7/7/2022 2:48:03 pm

File Path: \\aursfnefs011\Projects\Projects\BR\520326 WTP ROSEHILL\PAC01503 DEL00 ISS\07072022 Stage 3 re-submission\SMWSTWTP-GLO-RSH-SF500-RS-M3D-010002.rvt

OFFICIAL

FOR REVIEW AND COMMENT

REV	AMENDMENT DESCRIPTION	Design by	Verified by	Approved Initial/Date	
B	STAGE 3 DETAILED DESIGN RE-SUBMISSION	DL	MC	RS	10/06/22
A	STAGE 3 DETAILED DESIGN SUBMISSION	DL	MC	RS	28/04/22

A1 Original | Co-ordinate System: - | Height Datum: - | This sheet may be prepared using colour and may be incomplete if copied | NOTE: Do not scale from this drawing.

SCALE: 1:2 @ A1; 1:25 @ A1; 1:200 @ A1

KEYPLAN:

CLIENT: NSW GOVERNMENT — Sydney METRO

INTEGRATING DESIGN AEO: GHD | SMEC — SMEC | GHD DESIGN JOINT VENTURE

The information shown on this drawing is for the purposes of the Sydney Metro Project only. No warranty is given or implied as to its suitability for any other purpose. The Service Providers accept no liability arising from the use of this drawing and the information shown thereon for any purpose other than the Sydney Metro Project.

SERVICE PROVIDERS: GAMUDA Australia | LAING O'ROURKE | aurecon

AURECON
RETAINING STRUCTURES

DRAWN	YOLANDE BLIGNAUT	10.06.2022
DESIGNED	FIONA MA	10.06.2022
DRG CHECK	DAVID STREVENS	10.06.2022
DESIGN CHECK	SHARMEELEE SUBRAMANIAM	10.06.2022
APPROVED	ROB SMITHSON	10.06.2022

SYDNEY METRO WEST

SERVICE FACILITY - ROSEHILL
RETAINING STRUCTURES
INSTRUMENTATION FOR EXCAVATION WORKS
SHEET 3

DOCUMENT No: - | SHEET: 3 OF 4 | ©
STATUS: STAGE 3 DETAILED DESIGN | EDMS NO: -
DRG No. SMWSTWTP-GLO-RSH-SF500-RS-DRG-010117 | REV B | VER -

100mm AT FULL SIZE Plot Date & Time 10/6/2022 7:14:51 am

File Path: G:\Projects\BR\520326 WTP ROSEHILL\PAC01503 DEL\00 ISS\10062022 Stage 3 Detailed Design Re-Submission\SMWSTWTP-GLO-RSH-SF500-RS-M3D-010002.rvt

TRIGGER VALUES FOR INSTRUMENTATION AND MONITORING

1. SETTLEMENT MARKER (SM) & UTILITIES SETTLEMENT GAUGE (SG)

		SETTLEMENT VALUE (mm)		
		70%	80%	100%
INSTRUMENT ID	TYPE OF STRUCTURE/SERVICES	ALERT	ACTION	ALARM
SM1	ROAD PAVEMENT	14	16	20
SM2				
SM4				
SM3	GROUND ADJACENT TO THE DEEP EXCAVATION ZONE	21	24	30
SM5				
SM6				
SM7				
SM8				
SM9				
SM10				
SM11				
SM12				
SM13				
SG1	TEMPORARY SEWER PIPE			
SG2				
SG3				
SG4				

2. OPTICAL PRISM (OP)

		SETTLEMENT VALUE (mm)			VERTICAL DISTORTION		
		70%	80%	100%	70%	80%	100%
INSTRUMENT ID	TYPE OF STRUCTURE	ALERT	ACTION	ALARM	ALERT	ACTION	ALARM
OP1(a) & (b)	FAÇADE WALL	14	16	20	1:715	1:625	1:500
OP2(a) & (b)							
OP3(a) & (b)							
OP4(a) & (b)							

3. INCLINOMETER (INC)

	WALL DEFLECTION (mm)		
	70%	80%	100%
INSTRUMENT ID	ALERT	ACTION	ALARM
INC 1	23	26	32
INC 2	24	28	34
INC 3	18	20	25
INC 4	17	19	23
INC 5	19	21	26

4. VIBRATING WIRE STRAIN GAUGE (VWSG)

	AVERAGE STRUT FORCE (kN)		
	70%	80%	100%
INSTRUMENT ID	ALERT	ACTION	ALARM
VWSG 1 (a) & (b)	4270	4880	6100
VWSG 2 (a) & (b)	3430	3920	4900
VWSG 3 (a) & (b)	15610	17840	22300
VWSG 4 (a) & (b)	21350	24400	30500
VWSG 5 (a) & (b)	14840	16960	21200
VWSG 6 (a) & (b)	26950	30800	38500
VWSG 7 (a) & (b)	24850	28400	35500
VWSG 8 (a) & (b)	23520	26880	33600

5. VIBRATING WIRE PIEZOMETER (VWP)

		DRAWDOWN IN PORE WATER PRESSURE BELOW BASE READING (kPa)		
		70%	80%	100%
INSTRUMENT ID	PIEZOMETER	ALERT	ACTION	ALARM
WTP_BH18	1	-7	-8	-10
	2			
	3			
RSF_ BH04	1			
	2			
	3			

6. EXISING STANDPIPE PIEZOMETER (SP)

	DRAWDOWN BELOW BASE READING (m)		
	70%	80%	100%
INSTRUMENT ID	ALERT	ACTION	ALARM
ENV282_W	-0.7	-0.8	-1.0
ENV283_W			
ENV284_W			
ENV293_W			
ENV812_W	-1.0	-1.2	-1.5
ENV813_W			
ENV814_W			
WTP_BH13_W	-0.7	-0.8	-1.0
WTP_BH16_W			
WTP_BH17_W			
WTP_BH18_W			
RSF_BH01			
RSF_BH01(P)			
RSF_BH02			
RSF_BH02(P)			
RSF_BH04			
RSF_BH05			
RSF_BH06			
RSF_BH07			

NOTES

1. FOR GENERAL NOTES REFER TO SMWSTWTP-GLO-RSH-SF500-RS-DRG-10010 TO 10011
2. FOR GENERAL NOTES ON INSTRUMENTATION REFER TO SMWSTWTP-GLO-RSH-SF500-RS-DRG-10115

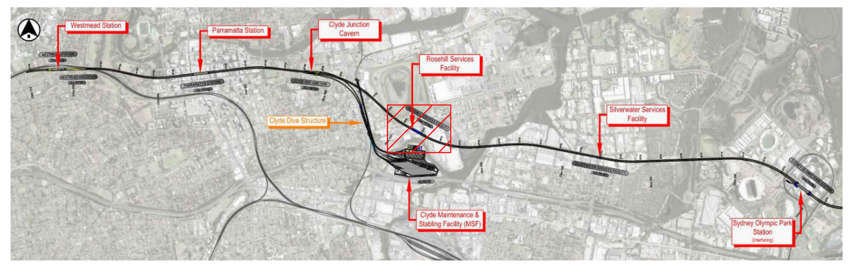
OFFICIAL

FOR REVIEW AND COMMENT

REV	AMENDMENT DESCRIPTION	Design by	Verified by	Approved Initial/Date	
C	STAGE 3 DETAILED DESIGN RE-SUBMISSION	DL	MC	RS	07/07/22
B	STAGE 3 DETAILED DESIGN RE-SUBMISSION	DL	MC	RS	24/06/22
A	STAGE 3 DETAILED DESIGN RE-SUBMISSION	DL	MC	RS	10/06/22

SCALE: 1:100 @ A1 0 1 2 3 4 5 m

KEYPLAN:



CLIENT: NSW GOVERNMENT, sydney METRO

INTEGRATING DESIGN AEO: SMEC | GHD DESIGN JOINT VENTURE

The information shown on this drawing is for the purposes of the Sydney Metro Project only. No warranty is given or implied as to its suitability for any other purpose. The Service Providers accept no liability arising from the use of this drawing and the information shown thereon for any purpose other than the Sydney Metro Project.

SERVICE PROVIDERS: GAMUDA Australia, LAING O'ROURKE

AURECON RETAINING STRUCTURES

DRAWN	YOLANDE BLIGNAUT	07.07.2022
DESIGNED	FIONA MA	07.07.2022
DRG CHECK	DAVID STREVENS	07.07.2022
DESIGN CHECK	SHARMEELEE SUBRAMANIAM	07.07.2022
APPROVED	ROB SMITHSON	07.07.2022

SYDNEY METRO WEST
SERVICE FACILITY - ROSEHILL
RETAINING STRUCTURES
INSTRUMENTATION FOR EXCAVATION WORKS
SHEET 4

DOCUMENT No: - SHEET: 4 OF 4 ©
STATUS: STAGE 3 DETAILED DESIGN EDMS NO: -
DRG No.SMWSTWTP-GLO-RSH-SF500-RS-DRG-010118 REV C VER -

Co-ordinate System: - Height Datum: This sheet may be prepared using colour and may be incomplete if copied NOTE: Do not scale from this drawing.

100mm AT FULL SIZE Plot Date & Time 7/7/2022 2:48:04 pm

File Path: \\aursfnefs01\Projects\Projects\BR\520326 WTP ROSEHILL\PAC01503 DEL\00 ISS\07072022 Stage 3 re-submission\SMWSTWTP-GLO-RSH-SF500-RS-M3D-010002.rvt