Five Dock Station Hydrogeological Assessment Report

SMWSTCTP-AFJ-FDK-GE-RPT-000002 Revision 0

Sydney Metro West – Central Tunnelling Package

DOCUMENT APPROVAL

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REVISION HISTORY

Rev:	Date:	Pages:	By:	Description:
B.1	16/11/21	All	JTJV	CDR / Internal Review – Stage 2
B	13/12/21	All	JTJV	Issued for Stage 2 External Submission
C.1	02/03/22	All	JTJV	CDR / Internal Review – Stage 3
C	21/03/22	All	JTJV	Issued for Stage 3 External Submission
0	15/07/22	All	JTJV	For submission to DPE

CONTENTS

1. INTRODUCTION

1.1 OBJECTIVES AND SCOPE

The objective of this report is to provide hydrogeological advice for the design of the Five Dock Station box in support of the Stage 3 design.

The scope of this document includes:

- A review and update to the specifications and Minister's requirements as they pertain to Five Dock Station.
- A review and update of the hydrogeological conceptual site model to reflect additional bore logs, geological interpretations, permeability testing and groundwater level monitoring that have occurred.
- A review and update of the anticipated groundwater levels based on the above.
- A review of the groundwater quality at the site.
- Review of packer test data.
- Review of supplementary groundwater level data from monitoring bore, AF_BH17_w.
- Documentation of revised groundwater modelling that has occurred since Stage 2.
- An update to the groundwater inflow and impact assessment based on the above.
- A discussion of the design implications related to the above updates.

2. GENERAL SPECIFICATIONS, PARTICULAR SPECIFICATIONS AND MINISTERS' CONDITIONS

This report considers Sydney Metro West – Central Tunnel Package General Specification Requirements (V2.9) and Particular Specification Requirements (V7.0) as they pertain to Five Dock Station including:

General Specification Requirements:

3.8.1.3 Geotechnical Interpretive Report

(C) The GIR or other technical reports must include:

(iv) insitu testing results (such as in situ stress testing in rock) hydrogeological assessment at the principal features including:

A. Any underground stations and affected water crossings including the expected impact on the groundwater regime.
B. Groundwater levels and expected groundwater conditions, including baseline estimates of inflows and pumping rates
C. Anticipated ground behaviour and the influence of groundwater, with regard to methods of excavation and installation of ground support.

(vi) a detailed assessment of the design groundwater levels to be adopted during design, including areas where perched groundwater may be present.

<u>Particular Specification Requirements:</u>

4.1.7 Groundwater control

(a) The Tunnelling contractor must comply with the following for the drainage of assets:
    (Vii) Station Excavations –drained
(b) The Tunnelling Contractor must assess by modelling the impact on the groundwater table and specify control and monitoring measures to demonstrate compliance with Acceptable Effects.
(c) The Tunnelling Contractor must minimise the impacts of groundwater drawdown and demonstrate from modelling that there are only Acceptable Effects to adjacent structures.
(h) The groundwater seepage within each Station excavation and each Shaft Excavation must not exceed:

(i) 15,000 litres in any 24-hour period, measured over any square with an area of 10m², at any and all locations within the sides and bases of the shafts and excavations, except for The Bays Station Excavation where groundwater seepage must not exceed 50,000 litres in any 24-hour period, measured over any square with an area of 10m², at any and all locations within the sides and bases of the excavation; and [SM-W-CTP-PS-1040]

(ii) the volumes identified below in any 24-hour period: [SM-W-CTP-PS-1041]

E. Five Dock Station Shaft Excavation No.1: 82,000 litres; [SM-W-CTP-PS-1052]

F. Five Dock Station Shaft Excavation No.2: 25,000 litres; and [SM-W-CTP-PS-1053]

3. HYDROGEOLOGICAL CONCEPTUAL SITE MODEL

3.1 GEOLOGY

A geological section of Five Dock Station is illustrated in Figure 1 below, which shows five geological units at the station box:

- Fill
- Residual soil
- Ashfield Shale
- Mittagong Formation
- Hawkesbury Sandstone

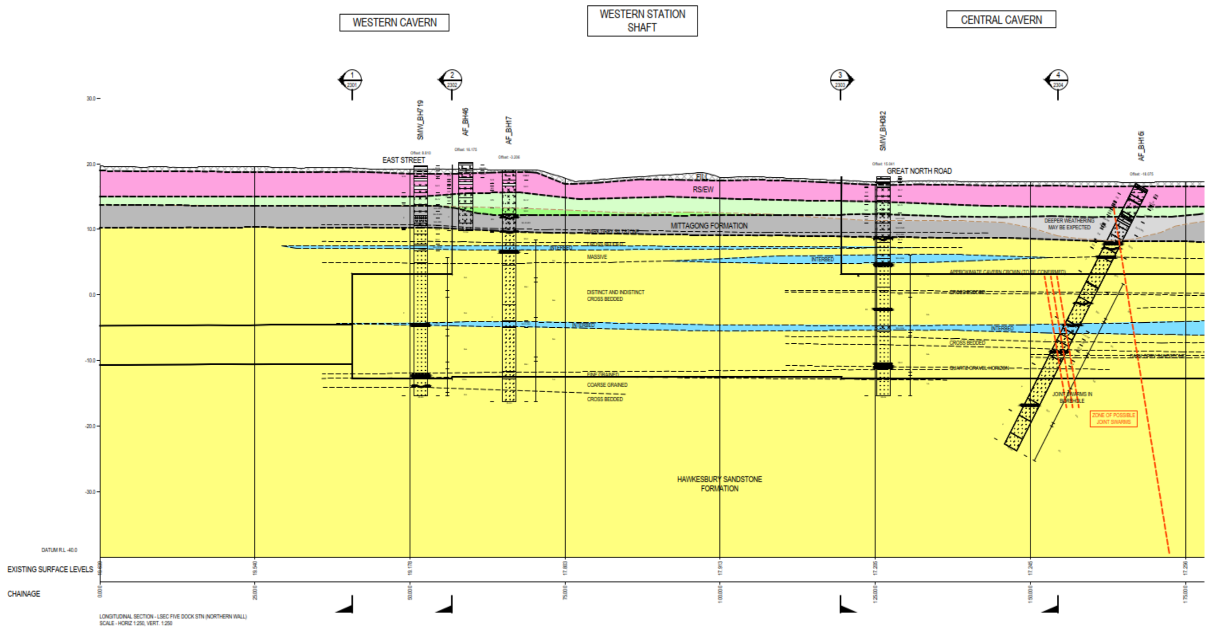


FIGURE 1 - GEOLOGICAL LONG SECTION OF FIVE DOCK STATION

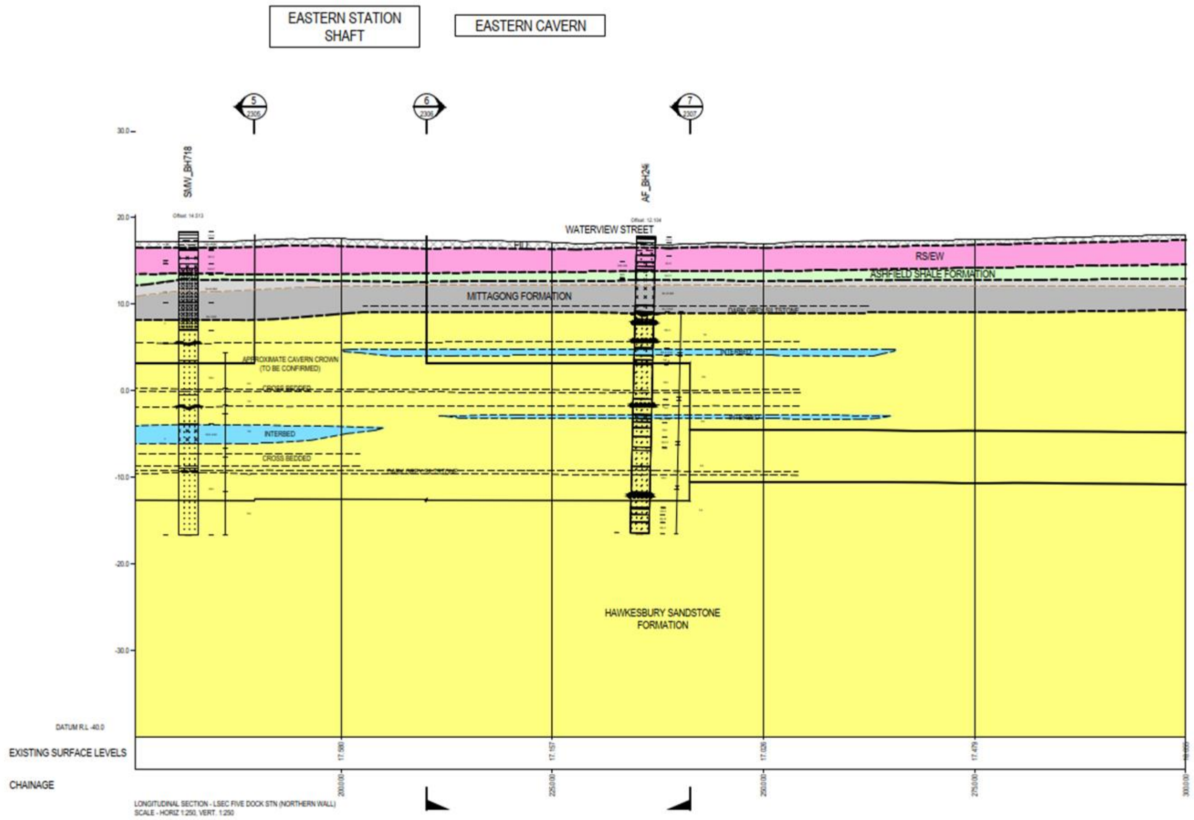


FIGURE 1 - GEOLOGICAL LONG SECTION OF FIVE DOCK STATION (CONTINUED)

The station comprises three separate cavern sections which are separated by two shafts, an eastern shaft and a western shaft. All three cavern sections lie within Hawkesbury Sandstone, whereas both shafts extend through all five of the geological units, with the majority of the shaft excavations being in Hawkesbury Sandstone.

Fill represents the dominant surficial deposits at Five Dock Station. It is highly variable in composition including clays, sands and gravels. While it dominates the surface material at the site, it is relatively thin, ranging from 0.4 m at SMW_BH082 to 1.0 m at SMW_BH719, and is absent at SMW_BH718.

Extremely weathered rock and residual soils underly fill across the Five Dock Station site. These are natural soils and dominated by high plasticity clays which represent weathering of the underlying Ashfield Shale and are around 1 m in thickness.

Ashfield Shale underlies the residual soils and is represented by dark grey siltstone with sandstone laminations. It is a relatively thin unit at the Five Dock Station box, ranging from approximately 5 m in thickness at SMW_BH719 to 2 m thickness at SMW_BH082 and SMW_BH719.

The Mittagong Formation forms a thin transition formation between the Ashfield Shale and the Hawkesbury Sandstone. It is characterised by interlaminated siltstone and sandstone at the Five Dock station box, and ranges in thickness from approximately 2 m at SMW_BH719 to 9 m at SMW_BH082.

The Hawkesbury Sandstone was deposited in a fluvial paleo-environment, likely to have been a braided river setting, and as such it is highly stratified. It is ubiquitous across the Sydney Basin and is up to some 300 metres thick. At Five Dock Station, the unit is characterised by variably sorted sands ranging from fine to coarse in size, interbedded with thin carbonaceous laminations.

3.2 GROUNDWATER LEVELS AND FLOW

Piezometers (all standpipe piezometers) near Five Dock Station box are shown in Figure 2. Typical groundwater levels observed in these piezometers are shown in Figure 3 in the datums of mAHD and mbgl, with the locations categorised by monitored strata.

Piezometer construction details and observed typical groundwater levels are tabulated in Table 1.

There is considerable variation in the groundwater levels and depths. Locations SMW_BH719_w and AF_BH17_w (both monitoring Hawkesbury Sandstone) and SMW_BH050_w (monitoring Hawkesbury Sandstone and Ashfield Shale) have observed groundwater levels significantly lower than other locations.

Due to the significant variation in groundwater level, to review potential hydraulic separation between the Ashfield Shale and Hawkesbury Sandstone, piezometer pressure head is compared to the minimum and maximum hydrostatic profile that would exist at these locations in Figure 4. Locations SMW_BH719_w, SMW_BH050_w and AF_BH17_w fall significantly below the minimum hydrostatic line, indicating a degree of hydraulic disconnection between the Hawkesbury Sandstone and Ashfield Shale at these locations. Conversely, two other locations, SMW_BH718_w and SMW_BH082_w, that both monitor Hawkesbury Sandstone, are below, but close to, the hydrostatic line. This suggests reasonable hydraulic connection between the Hawkesbury Sandstone and Ashfield Shale at these locations.

In addition to the piezometer pressure head analysis, the potential for existing structures to cause groundwater level drawdown has been reviewed to assess whether the lower groundwater levels at SMW_BH719_w, AF_BH17_w and SMW_BH050_w may be due to drawdown associated with drained structures. A supermarket with an underground carpark is situated near SMW_BH050_w, AF_BH17_w and SMW_BH719_w. The supermarket site has not been inspected and has been reviewed using Google Street View images. Therefore, the basement carpark footprint is not known. However, the building is about 10 m, 50 m and 65 m from SMW_BH050_w, SMW_BH719_w and AF_BH17_w, respectively. Also, the M4 East tunnel is located about 700 m south from SMW_BH050_w. Therefore, it is possible that the relatively lower groundwater levels at SMW_BH050_w, SMW_BH719_w and AF_BH17_w could be associated with the M4 East tunnel and/or the supermarket. The hydrograph for SMW_BH050_w (Annexure A) shows a consistent decline from October 2018 until May 2019, the end of the monitoring period. The total decline is 1.2 m and occurs during a period of below average rainfall. Given that the observed hydrograph decline at SMW_BH050_w correlates with a period of below average rainfall, and given that SMW_BH082_w, which is monitoring sandstone and positioned similarly to SMW_BH050_w and SMW_BH719_w relative to the discussed structures, does not have similarly low groundwater levels, the cause of the relatively low groundwater levels at SMW_BH050_w and SMW_BH719_w is not known and is assessed as not necessarily due to surrounding drained structures. The same is considered for AF_BH17_w, which only has a recent (20/12/2021 to 03/02/2022) hydrograph record of 45 days, during which the groundwater levels are relatively stable.

The current data indicates variable hydraulic connection between the Hawkesbury Sandstone and Ashfield. At the four closest locations to the station box, the data indicates relatively poor connection at SMW_BH719_w and AF_BH17_w and reasonable connection at SMW_BH718_w and SMW_BH082_w. The current data does not support the presence of a consistent perched water table near the station box.

FIGURE 2 - PIEZOMETER AND BOREHOLE LOCATIONS AT FIVE DOCK STATION

FIGURE 3 - GROUNDWATER LEVELS AT FIVE DOCK STATION

TABLE 1 - SUMMARY OF GROUNDWATER LEVELS AND BORE DETAILS AT FIVE DOCK STATION

Location ID	Grid	Easting	Northing	Approx. surface elevation	Filter pack from	Filter pack to	GWL[1]	GWL[1]	Formation screened
				m AHD	m bgl	m bgl	m AHD	m bgl	
SMW_BH050_w	MGA94	326914	6250868	24.3	8.5	26.1	4	20.3	AS/HSS
SMW_BH050_s	MGA94	326912	6250868	24.4	0.4	1.3	24	0.4	Residual Soils / Clay
SMW_BH051_w	MGA94	327065	6250869	21.7	5.0	11.0	19.0	2.7	AS
SMW_BH051_s	MGA94	327065	6250868	21.7	0.7	2.0	19.8	1.9	Residual Soils / Clay
SMW_BH082_w	MGA94	327013	6250966	18.0	7.3	13.3	14.8	3.2	HSS
SMW_BH718_w	MGA94	327068	6250957	18.35	11.0	16.0	14.3	4.1	HSS
SMW_BH719_w	MGA94	326941	6250985	19.73	25.8	35.15	0.7	19.0	HSS
AF_BH17_w	MGA94	326957	6250995	19.03	27.05	35.05	0.73	18.3	HSS
R248_3103_BH141	MGA94	327085	6250740	16.8	19.9 [3]	25.9 [3]	4.0 [4]	12.8 [4]	HSS
R248_3103_BH141A	MGA94	327085	6250740	16.8	4.1 [3]	7.1 [3]	14.1 [4]	2.7 [4]	HSS

[1]GWL = Groundwater level. Levels reported reflect typical groundwater levels

[2]AS = Ashfield Shale and HSS = Hawkesbury Sandstone

[3] Screen interval depth

[4] Single groundwater level measurement, so may not be representative

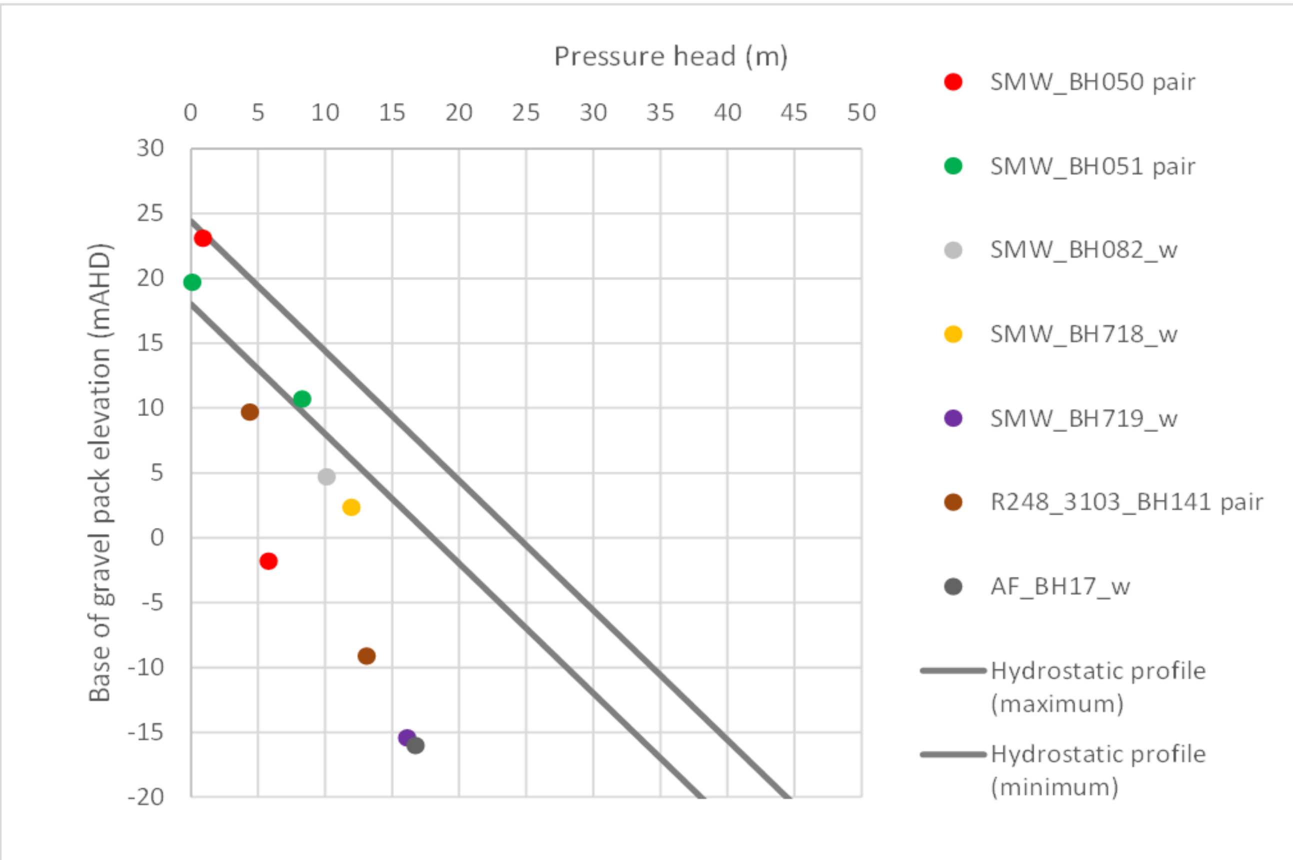


FIGURE 4 - GROUNDWATER PRESSURE HEAD IN PIEZOMETERS AT FIVE DOCK STATION

3.3 GROUNDWATER SYSTEM PROPERTIES

Groundwater system properties for hydrogeological units applicable to the whole CTP (aside from The Bays area) are covered in detail in Annexure B.

At Five Dock Station, the applicable hydrogeological units comprise Ashfield Shale, the Mittagong Formation and Hawkesbury Sandstone, with the latter two units having been grouped within Annexure B for the purpose of assigning parameter values. Fill and residual soil units are insignificant as hydrogeological units because the water table is situated below these units at the station.

The results of in-situ permeability (packer) tests at Five Dock Station are summarised in Table 2 and plotted by depth below ground in Figure 5. Figure 5 also includes all SMW packer test results outside of The Bays paleochannel, to enable a comparison of the Five Dock results to the broader CTP results.

The Five Dock results indicate generally low permeability, with 29 out of a total of 42 tests yielding a Lugeon (uL) value of less than 1 uL. Six tests yielded a Lugeon value of >2 uL and the largest and second largest Lugeon (uL) values were >100 uL and 5 uL, respectively. The median and average Lugeon value was 0.5 uL and 3.3 uL, respectively.

It should be noted that the single >100 uL value at SMW_BH719 represents a significant outlier. Packer tests completed at similar relative levels within other boreholes did not return similarly high Lugeon values.

The single >100 uL value at SMW_BH719 for the tested interval of 30 m to 35.15 m is inferred to be associated with a seam logged as less than 40 mm thick, and bedding partings either side of the seam. Water loss of 100% is noted on the corelog to commence at the approximate depth of these features.

The depth level (-6.93 mAHD to -12.67 mAHD) of the second largest Lugeon (uL) test value of 5 uL, which occurred at AF_BH16i, coincides with a zone of possible joint swarms noted on the geological long section.

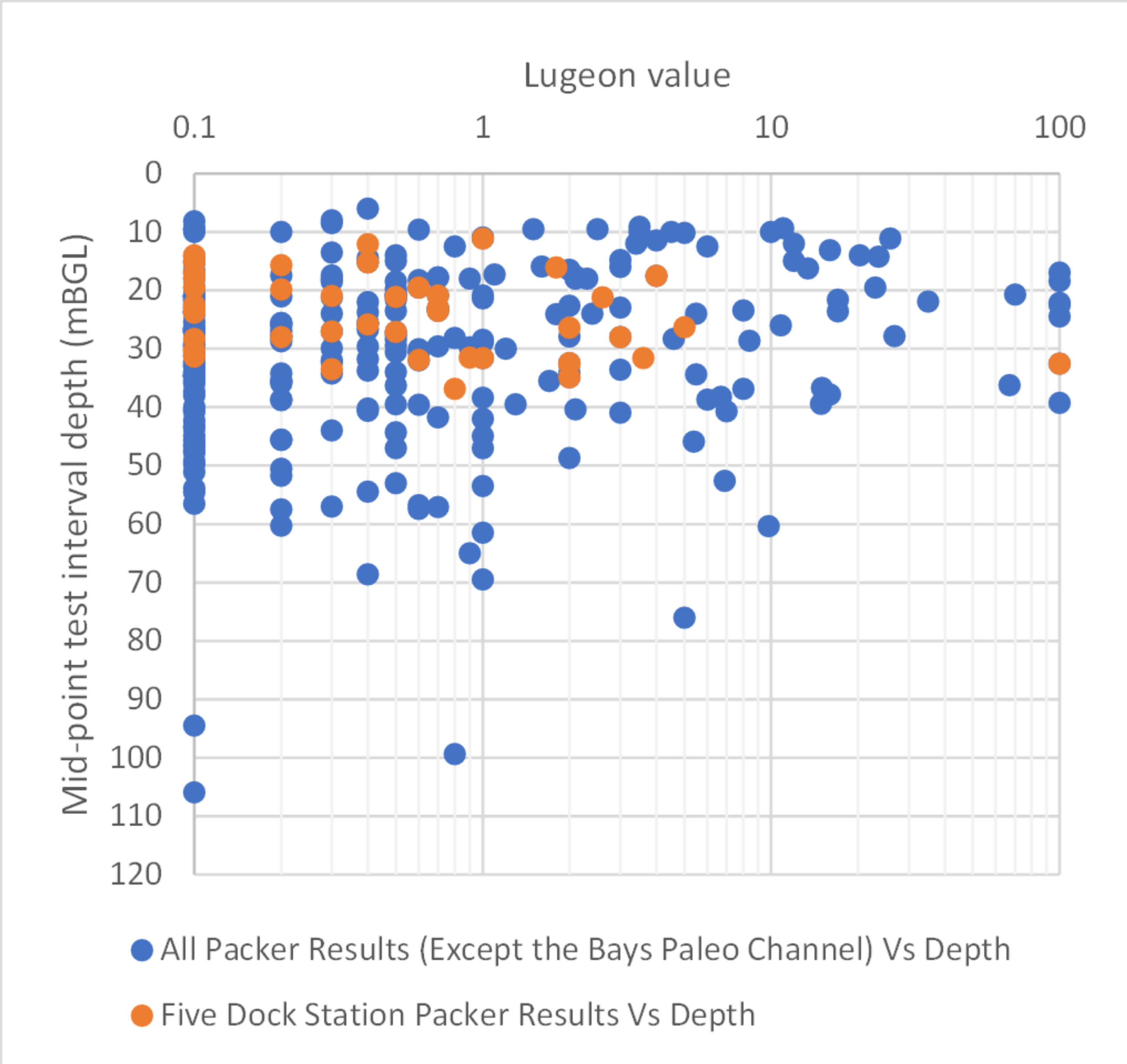


FIGURE 5 - PACKER TEST RESULTS BY DEPTH (VALUES PLOTTED BY MIDDLE DEPTH OF PACKER INTERVAL)

TABLE 2 - SUMMARY OF PACKER TESTING AT FIVE DOCK STATION

ID and [dip°]	Depth (mbgl)		Level (mAHD)		Unit [1]	Result		
	Top	Bottom	Top	Bottom		Qualifier	uL	m/day
SMW_BH050 [90]	20.6	27.1	3.74	-2.76	HSS	<	0.1	8.6×10^{-4}
	26.6	36.13	-2.26	-11.79	HSS	=	0.1	8.6×10^{-4}
SMW_BH051 [90]	18	21	3.68	0.68	HSS	=	0.1	8.6×10^{-4}
	20.05	27	1.63	-5.32	HSS	=	0.7	6.0×10^{-3}
	26.5	34	-4.82	-12.32	HSS	=	0.1	8.6×10^{-4}
SMW_BH082 [90]	12	18.33	6.04	-0.29	HSS	<	0.1	8.6×10^{-4}
	17.8	24.35	0.24	-6.31	HSS	=	0.5	4.3×10^{-3}
	23.8	30.38	-5.76	-12.34	HSS	=	0.5	4.3×10^{-3}
	29.8	33.4	-11.76	-15.36	HSS	=	1	8.6×10^{-3}
SMW_BH084 [90]	12	18.34	4.6	-1.74	HSS	=	0.4	3.5×10^{-3}
	17.5	24.3	-0.9	-7.7	HSS	=	0.7	6.0×10^{-3}
	23.8	30.3	-7.2	-13.7	HSS	=	0.3	2.6×10^{-3}
	29.8	35.27	-13.2	-18.67	HSS	=	2	1.7×10^{-2}
SMW_BH718 [90]	14	21	4.35	-2.65	HSS	=	4	3.5×10^{-2}
	18	21	0.35	-2.65	HSS	=	0.6	5.2×10^{-3}
	20	26.04	-1.65	-7.69	HSS	=	0.7	6.0×10^{-3}
	25	31	-6.65	-12.65	HSS	=	3	2.6×10^{-2}
	30	34.98	-11.65	-16.63	HSS	=	2	1.7×10^{-2}
SMW_BH719 [90]	14	20.04	5.73	-0.31	HSS	<	0.1	8.6×10^{-4}
	19	26	0.73	-6.27	HSS	<	0.1	8.6×10^{-4}
	25	31	-5.27	-11.27	HSS	=	0.2	1.7×10^{-3}
	30	35.15	-10.27	-15.42	HSS	>	100	8.6×10^{-1}
	34.6	35.15	-14.87	-15.42	HSS	=	2	1.7×10^{-2}
AF_BH16i [62]	12.80	18.59	3.67	-2.12	HSS	=	0.2	1.7×10^{-3}
	18.10	23.80	-1.63	-7.33	HSS	=	0.3	2.6×10^{-3}
	23.40	29.14	-6.93	-12.67	HSS	=	5	4.3×10^{-2}
	28.70	34.47	-12.23	-18.00	HSS	=	3.6	3.1×10^{-2}
	33.99	39.73	-17.52	-23.26	HSS	=	0.8	6.9×10^{-3}
AF_BH17 [90]	11.00	17.04	8.03	1.99	HSS	<	0.1	8.7×10^{-4}
	16.50	23.16	2.53	-4.13	HSS	=	0.2	1.7×10^{-3}
	22.50	29.14	-3.47	-10.11	HSS	=	0.4	3.5×10^{-3}
	28.50	35.28	-9.47	-16.25	HSS	=	0.6	5.2×10^{-3}
AF_BH24i [59]	8.57	13.71	9.24	4.10	HSS	=	1	8.7×10^{-3}
	13.29	18.86	4.52	-1.05	HSS	=	1.8	1.6×10^{-2}
	18.43	24.00	-0.62	-6.19	HSS	=	2.6	2.3×10^{-2}
	23.57	29.14	-5.76	-11.33	HSS	=	2	1.7×10^{-2}
	28.72	34.29	-10.91	-16.48	HSS	=	0.9	7.8×10^{-3}
AF_BH25i [60]	8.66	15.50	11.91	5.07	MIT + HSS	=	0.4	3.5×10^{-3}
	15.07	20.74	5.50	-0.17	HSS	<	0.1	8.7×10^{-4}

ID and [dip°]	Depth (mbgl)		Level (mAHD)		Unit [1]	Result		
	Top	Bottom	Top	Bottom		Qualifier	uL	m/day
	20.26	25.94	0.31	-5.37	HSS	=	0.1	8.7×10^{-4}
	25.46	31.09	-4.89	-10.52	HSS	<	0.1	8.7×10^{-4}
	30.66	36.29	-10.09	-15.72	HSS	=	0.3	2.6×10^{-3}
								Statistical summary (m/d)
							Arithmetic mean	2.9×10^{-2}
							Median	4.3×10^{-3}
							Geometric mean	4.6×10^{-3}

Notes: [1] HSS = Hawkesbury Sandstone. MIT = Mittagong Formation.

Typical ranges and adopted representative hydrogeological parameter values to represent the Ashfield Shale and Hawkesbury Sandstone hydrogeological units for the CTP as a whole (excluding The Bays area) are summarised in Table 3. The horizontal hydraulic conductivity values for the Ashfield Shale and Hawkesbury Sandstone units reflect the 75th percentile values of the packer test datasets, as discussed in Annexure B.

These values are consistent with the arithmetic mean and median/geometric mean values of the Five Dock station packer test data. Thus, the Five Dock Station hydraulic conductivity test values derived from packer testing are considered typical for the relevant units.

Table 3 Summary of hydrogeological parameter values for Ashfield Shale and Hawkesbury Sandstone/Mittagong Formation, and adopted representative values for CTP as a whole (except The Bays area).

TABLE 3 - SUMMARY OF HYDROGEOLOGICAL PARAMETER VALUES FOR ASHFIELD SHALE AND HAWKESBURY SANDSTONE/MITTAGONG FORMATION, AND ADOPTED REPRESENTATIVE VALUES FOR CTP AS A WHOLE (EXCEPT THE BAYS AREA)

Hydrogeological unit	Typical hydraulic conductivity range (m/day)	K_v/K_h range	Specific storage range (m^{-1})	Specific yield range (-)
	Typical range			
Ashfield Shale	4.45×10^{-3} to 1.84×10^{-2} (0.5 to 2.1 Lugeons) (geomean to 75th percentile)	0.1 to 1.0	5.00×10^{-6} to 1.00×10^{-5}	0.01 to 0.025
Mittagong Formation and Hawkesbury Sandstone	6.03×10^{-3} to 5.62×10^{-2} (0.7 to 2.0 Lugeons) (geomean to 75th percentile)	0.01 to 1	1.00×10^{-6} to 1.00×10^{-5}	0.02 to 0.05
	Adopted representative value			
Ashfield Shale	1.84×10^{-2} (2.1 Lugeons; 75th percentile)	0.1	5.00×10^{-6}	0.02
Mittagong Formation and Hawkesbury Sandstone	1.72×10^{-2} (2.0 Lugeons; 75th percentile)	0.1	5.00×10^{-6}	0.05

4. PROJECT GROUNDWATER LEVELS

4.1 REQUIREMENTS

Design related to groundwater levels must consider the requirements of the Particular Specifications listed in Table 4.

TABLE 4 - PARTICULAR SPECIFICATIONS FOR DESIGN RELEVANT TO GROUNDWATER LEVELS

Particular Specification
1. The following design codes, in order of precedence: a. AS 5100 Bridge Design Series [SM-W-CTP-PS-703]. AS5100.2 requires that variation in groundwater levels shall be taken into account by using design levels based on a return period of 1000 years for the ULS (0.1% AEP) and 100 years for the SLS (1% AEP) b. AS/NZS 1170 Structural Design Actions Series for imposed loads and other actions that are not specified in AS 5100 Bridge Design Series; [SM-W-CTP-PS-704]. AS/NZS1170.1 requires that the hydrostatic pressure shall be the value assuming water level at the ground surface; unless there are groundwater level data available, in which case, a groundwater level with an annual exceedance probability (AEP) of 1 in 50 (2% AEP, or 50 year ARI) shall be adopted c. AS 4678 Earth - retaining structures for ground loadings, for free-standing retaining walls; and [SM-W-CTP-PS-705] d. AS 1657 Fixed Platforms, walkways, stairways and ladders - Design, Construction and installation. [SM-W-CTP-PS-706]
2. The design action resulting from hydrostatic pressure of water acting on surfaces below ground level (Fgw) for all underground structures considers a water level at ground level [SM-W-CTP-PS-910]; or, where information is available, the ground water level with an annual probability of exceedance of 1 in 100. [SM-W-CTP-PS-911]
3. The potential impact of groundwater levels and hydrostatic pressures of floodwater plains or a burst water main where existing or new water utilities are within proximity to the Project Works and Temporary Works [SM-W-CTP-PS-709]
4. Foreseeable differences in groundwater table level between opposite sides of the completed underground structures for the applicable Design Life [SM-W-CTP-PS-711]
5. Application of a minimum difference in groundwater level table of 5 m, for the exceptional or temporary load case, to represent a burst water pipe or groundwater flow differential loading condition, unless an alternate value can be demonstrated from hydrogeological analysis. [SM-W-CTP-PS-712]
6. The Tunnelling Contractor must not allow for any reduction in hydrostatic loadings due to localised lowering of groundwater levels *[due to existing drained structures]* in the design of the Works. The reduction of hydrostatic loading due to localised lowering of groundwater levels is permitted in the design of the support of Station Excavations and Station Shaft Excavations that are drained in accordance with the requirements in Section 4.1.7(a). [SM-W-CTP-PS-715]
7. The Tunnelling Contractor must design for the risk of water pressure build-up as a result of blocked drainage. [SM-W-CTP-PS-1030]
8. For the design of tunnels, caverns and adits, consider long term variations in groundwater levels [SM-W-CTP-PS-1389]

4.2 FACTORS POTENTIALLY AFFECTING GROUNDWATER LEVELS

The factors that have been considered as potential causes of future rises in groundwater levels (some of which are discounted as being of negligible impact to the project) include:

- Short term changes

- High rainfall events
- Flooding

- Long term changes
  - Sea level rise caused by climate change
  - Prolonged wet periods (long term above average rainfall)
  - Annual seasonal variation

4.2.1 GROUNDWATER LEVEL RISE IN RESPONSE TO RAINFALL

The potential for long term increases in groundwater levels due to prolonged wetter periods has been considered. However, there are no bores near the site with long term (decadal) groundwater monitoring data.

Available hydrographs for the piezometers listed in Table 1 are provided in Annexure A.

To assess the potential for short term fluctuations in groundwater levels resulting from prolonged and intense rainfall events (e.g., high rainfall over several days), monitoring of water levels at a daily or sub-daily frequency is required. A few data logger records are available for the site.

Figure 6 below shows groundwater levels monitored in residual soils (BH050_s). This is a shallow bore (screened 0.4-1.3 m bgl) and has been selected due to its relatively high responsiveness to rainfall compared to other bores in the area). Because groundwater levels in this bore have a strong response to rainfall, and the bore is screened in the shallow residual soils, it provides a conservative estimate of the groundwater level increase which may be observed in response to high rainfall events. Accordingly, the data indicate that the groundwater response to significant rainfall events at Five Dock Station could be up to approximately 0.5 m as observed in response to rainfall events of >40 mm in October 2018, December 2018 and March 2019.

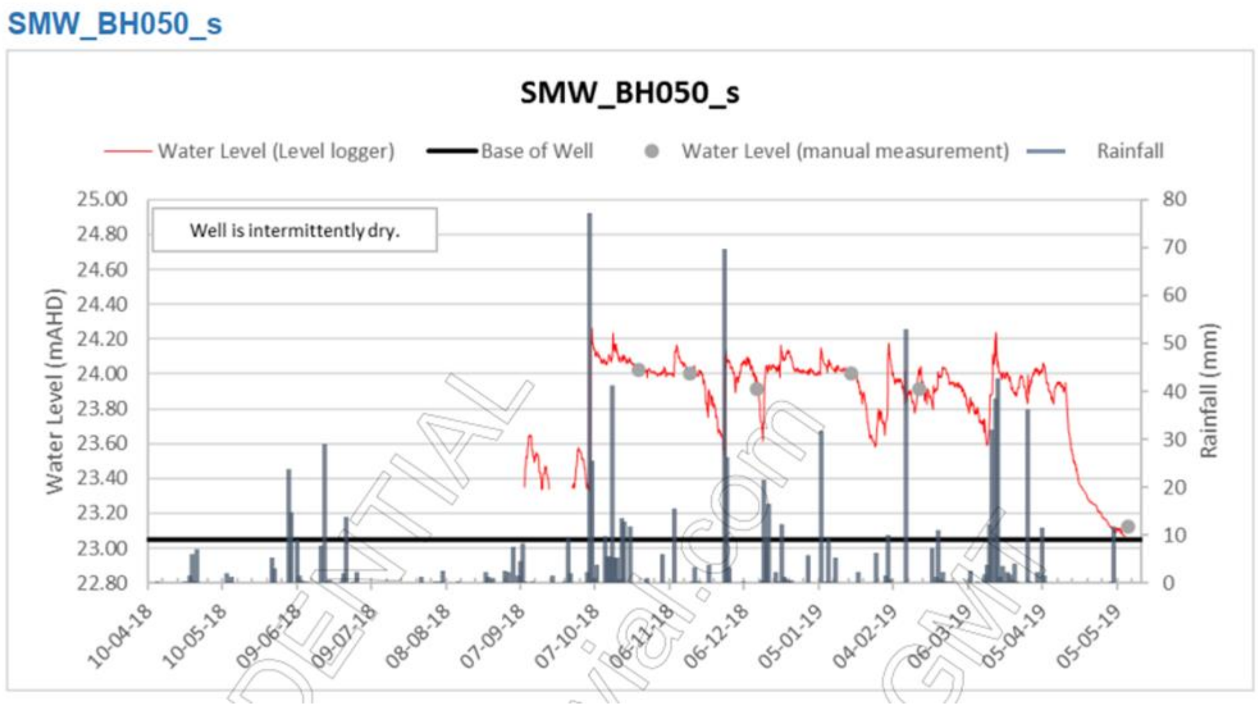


FIGURE 6 - GROUNDWATER MONITORING IN SMW_BH050_S

Seasonal variations in the Hawkesbury Sandstone at Five Dock Station as illustrated by trends in Figure 7 suggest seasonal responses of the order of 0.5 m. Seasonal trends are less prevalent in shallower bores which tend to respond more rapidly to individual rainfall events.

Cumulative mean monthly rainfall deviation since the year 2000 is shown in Figure 8 for rainfall data extracted from the SILO database for a point located at latitude -33.85 and longitude 151.05, located near Sydney Olympic Park, about 7.5 km northwest of the Five Dock station box. For recent years, a period of below average rainfall commencing in early 2017 and extending to December 2019 is evident. After December 2019, the cumulative monthly rainfall trend is generally increasing or fairly stable.

Except for SMW_BH050_w, the hydrographs do not appear to correlate with the cumulative mean monthly rainfall deviation trend.

Based on the short term and seasonal fluctuation analysis, and the cumulative mean monthly rainfall deviation analysis, a potential rise in groundwater level of 1 m is adopted for the construction period to the end of 2024.

SMW_BH719

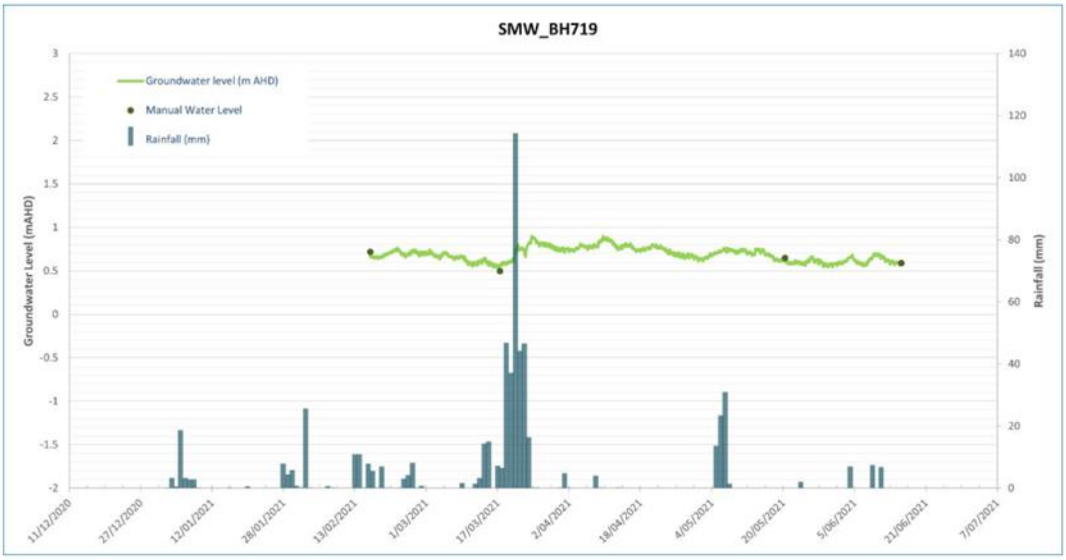


FIGURE 7 - GROUNDWATER MONITORING IN SMW_BH719

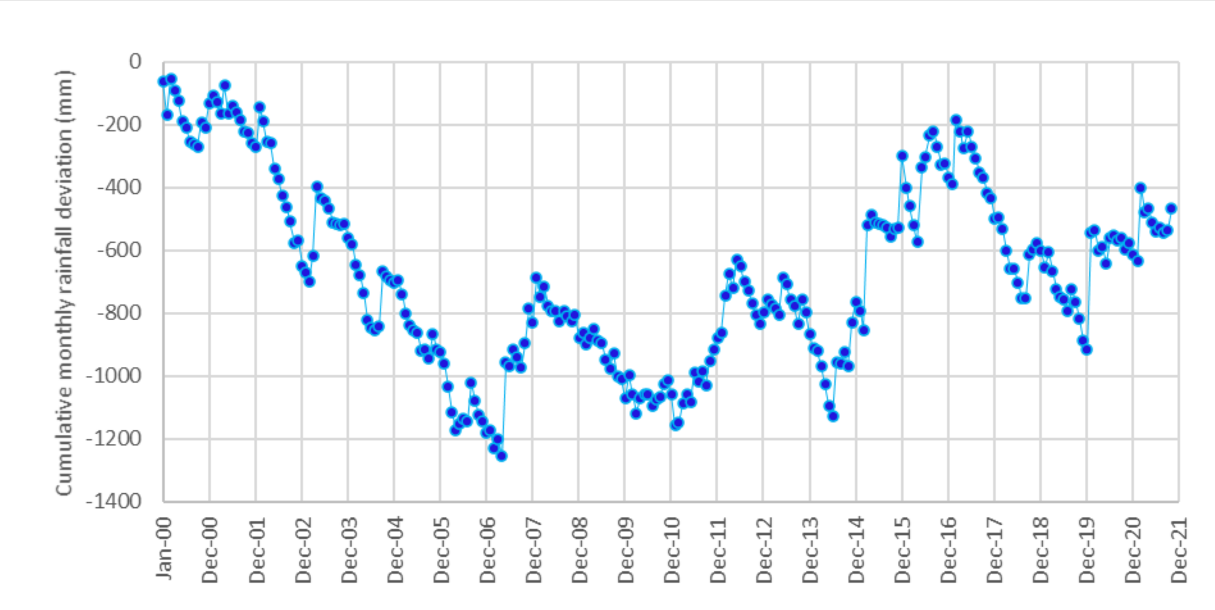


FIGURE 8 - CUMULATIVE DEVIATION FROM MEAN MONTHLY RAINFALL AT SILO POINT, LATITUDE -33.85, LONGITUDE 151.05, LOCATED ABOUT 7.5 KM NORTH WEST OF FIVE DOCK STATION

4.2.2 FLOODING

Flooding can cause a temporary rise in groundwater levels as water is transferred into the ground across a wider surface area. The effect of flooding of waterways on groundwater levels is influenced by the area inundated by flood waters, the duration of the flood event, and the hydraulic connection between the surface water and the relevant aquifer(s).

Flood depths are included in the overall Five Dock design report.

4.2.3 SEA LEVEL RISE FROM CLIMATE CHANGE

The dominant effect that future climate change could have on groundwater levels is via sea level rise, which will affect groundwater levels by both driving a higher groundwater level inland, and also by increasing surface water levels in streams and rivers. There is no standard for determining impact on groundwater level from sea level rise. Other potential impacts on groundwater levels due to potentially higher intensity rainfall events associated with future climate change were not specifically estimated for this assessment (i.e., short- and medium-term high rainfall), due in part to the high uncertainty associated with climate change rainfall predictions.

Guidance from NSW Government for assessing climate change impacts on potential sea level rise has been estimated based on Representative Concentration Pathways (RCP) 8.5. This refers to the upper range projection of greenhouse gas concentrations in the atmosphere as adopted by the Intergovernmental Panel on Climate Change (IPCC) in 2014 for the assessment of climate change impacts by the year 2100. The sea level rise associated with this scenario is 0.9 meters. Over a 10-year project design life, to the year 2032, this equates to a sea level rise of 0.1 m.

The impact of the rise in sea level on groundwater levels is anticipated to diminish moving inland from the coast. Given the proximity of Five Dock Station to the Paramatta River, a rise in the base level of the regional groundwater can be expected. The effect of this impact is likely to involve an increase in the base level for all groundwater levels, with the existing variation of background groundwater levels inland from the coast likely being maintained. Over a 10-year project design life, the impact of the rise in sea level from climate change on groundwater levels has therefore been estimated at a maximum rise of 0.1 m at Five Dock Station.

4.3 ANTICIPATED GROUNDWATER LEVELS

4.3.1 EXISTING CONDITIONS

This section discusses the potential rises in groundwater levels under existing conditions (i.e., in the absence of excavation dewatering due to CTP works or other drained structures).

The current typical groundwater levels observed at SMW_BH082 and SMW_BH718, located close to the station footprint, are 3.2 m and 4.1 m below ground surface, respectively. The shallowest groundwater levels observed to date at these locations are 3.0 mbgl (SMW_BH082) and 3.9 mbgl (SMW_BH718).

Based on the above discussion, the following potential increases to the currently observed groundwater levels at SMW_BH082 and SMW_BH718 are considered possible over a 10 year design life (assuming drained excavations, including CTP works and any others are not present):

- An increase of 1.0 m due to rainfall
- No increase for flooding at the western end of the station box
- An increase of 0.1 m for climate induced sea level rise effects on groundwater levels

The effects considered above have been summarised in Table 5 below.

Table 5 Summary of factors and groundwater levels at Five Dock Station in absence of CTP excavation works.

TABLE 5 - SUMMARY OF FACTORS AND GROUNDWATER LEVELS AT FIVE DOCK STATION IN ABSENCE OF CTP EXCAVATION WORKS

Area	Surface elevation[1] (m AHD)	Shallowest current GWL (m bgl)	Rise due to rainfall (m)	Rise due to rising sea level (m)	Possible existing groundwater level (m bgl)
Western Shaft (Shaft 1)	Ranges approx. 17.5 to 21 (18.0 at SMW_BH082)	3.0 (SMW_BH082)	1.0	0.1	1.9 (16.1 mAHD)
Eastern Shaft (Shaft 2)	Ranges approx. 17.5 to 20 (18.4 at SMW718)	3.9 (SMW_BH718)	1.0	0.1	2.8 (15.6 mAHD)

[1]Values presented rounded to 0.5 m

4.3.2 CTP WORKS CONDITIONS

The Five Dock Station excavation will be drained.

Over the long-term, groundwater levels immediately surrounding the excavation will be close to the excavation floor level (or the deepest passive dewatering level). For the permanent (10 year design life) condition, it can therefore be assumed that there is no hydrostatic pressure on the retaining walls.

Design can exploit this, consistent with Particular Specification SM-W-CTP-PS-715.

4.3.3 CTP WORKS EXCEPTIONAL CONDITIONS

Design is required to consider groundwater levels in response to burst water mains and blocked drainage (Particular Specification SM-W-CTP-PS-709 and SM-W-CTP-PS-1030). See Annexure C for more details on this.

5. GROUNDWATER QUALITY

5.1 INSIDE STATION BOX

A project-wide Groundwater Monitoring Event (GME) undertaken by Golder/Douglas in May 2021 comprised sampling of groundwater monitoring wells located within the Five Dock Station box footprint including SMW_BH718 and SMW_BH719.

Groundwater was typically of a slightly alkaline pH (7.62 to 8.02) with electrical conductivity (EC) recorded between 5,050 to 11,700 µS/cm. Exceedances of ecological guideline trigger values for 95% protection of freshwater ecosystems (slightly – moderately disturbed), as stipulated in ANZG (2018), are generally restricted to slightly elevated levels of metals nickel and zinc in sample SMW_BH719. Both samples collected from SMW_BH718 and SMW_BH719 were also identified to contain concentrations of manganese exceeding the adopted criteria of recreation and aesthetics.

Other notable detections included TRHs and toluene in sample SMW_BH718, and toluene and carbon disulfide in sample SMW_BH719. However, no exceedances of the adopted criteria (where applicable) were reported. No detections of PFAS above the laboratory reporting limit were identified in either samples collected from SMW_BH718 and SMW_BH719.

5.2 OUTSIDE STATION BOX

Two groundwater monitoring wells outside the station box were sampled as part of a groundwater monitoring event in September 2018, including SMW_BH050 and SMW_BH051. Groundwater was typically slightly alkaline pH (7.38 to 8.26) with EC recorded between 5,750 to 7,940 µS/cm. Exceedances of ecological guideline trigger values for 95% protection of freshwater ecosystems (slightly – moderately disturbed) are associated with elevated levels of metals, including arsenic, chromium, copper, manganese, nickel and zinc for adopted criteria of recreation and aesthetics. No other notable detections were identified. PFAS was not assessed as part of the groundwater sampling undertaken in September 2018.

6. GROUNDWATER INFLOW AND DRAWDOWN

6.1 OVERVIEW

Two separate 2D cross section models were developed to predict potential groundwater inflow rates into the Five Dock Station excavations and associated propagation of groundwater level drawdown. The two models represented, separately, the shaft and cavern geometries.

The models were developed using Geoslope's Geostudio SEEP/W, a finite difference modelling software package for modelling groundwater flow in porous media.

Details of the modelling are covered in Annexure D.

6.2 MODEL LAYERS

Two hydrogeological units were represented in the model: Ashfield Shale and Hawkesbury Sandstone. Fill and residual soil units were not included in the model because the water table is situated below these units at the station. The Mittagong Formation was not explicitly represented in the model and is instead represented by the Hawkesbury Sandstone unit. This approach was adopted because the Mittagong Formation is thin (e.g. 2 m to 5 m thick) and conceptualised to be characteristically similar to the Hawkesbury Sandstone.

6.3 ADOPTED HYDROGEOLOGICAL PARAMETER VALUES FOR MODELLING

Aside from hydraulic conductivity values, hydrogeological parameter values adopted for the modelling were as per the adopted representative values outlined in Section 3.3.

Hydraulic conductivity values applied in the model for both the Ashfield Shale and Hawkesbury Sandstone (and Mittagong Formation) were equivalent to the 75th percentile value of CTP packer testing (excluding The Bays area) for siltstone and sandstone test intervals, respectively. The applied Ashfield Shale and Hawkesbury Sandstone horizontal hydraulic conductivity values were 0.0184 m/d and 0.0173 m/d, respectively.

These horizontal hydraulic conductivity values adopted for modelling are somewhat conservative.

6.4 GROUNDWATER INFLOWS

6.4.1 INFLOW RATES

Groundwater inflow rates estimated by the modelling are shown in Figure 9 and are summarised as follows:

- Eastern shaft – up to 19 m^3/d
- Western shaft – up to 58 m^3/d
- Caverns – up to 94 m^3/d
- Shafts and caverns combined – up to 170 m^3/d

As shown in Figure 9, the modelled groundwater inflow rates vary with time. It is noted that the early time groundwater inflow rates are considered to be higher than are likely to occur in practice under the assumed hydrogeological conditions because the model assumes that full excavation is instantaneous (i.e., the excavation is "wished in place"). In practice, the excavation will be deepened progressively, and peak groundwater inflows are likely to be lower than those reported here.

Modelled peak groundwater inflow rates are compared to the Particular Specifications in Table 6. The modelled peak groundwater inflow rates are below the Particular Specification criteria for both shafts.

With respect to Particular Specification 4.1.7 (h) (ii), which states that groundwater seepage must not exceed 15,000 litres in any 24-hour period, measured over any square with an area of 10 m^2;

inflows over any given 10 m² area of excavation face will depend on the water-bearing features encountered during excavation.

There is a possibility that hydraulic conductivity may be higher than the values adopted for the modelling, particularly in zones of possible joint swarms, or in other as-yet unidentified zones. Should water-bearing features be encountered during excavation, groundwater inflows may be higher than estimated, and localised grouting during excavation may be required to limit groundwater inflows to the Particular Specification criteria.

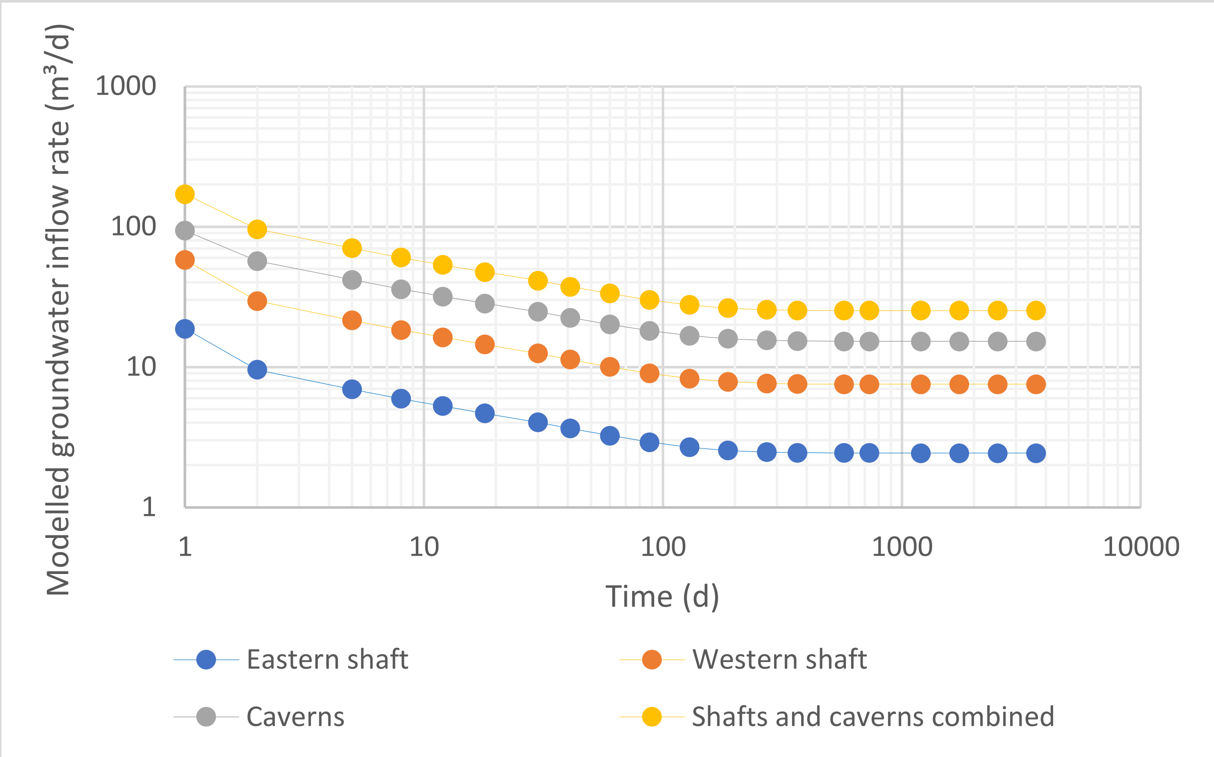


FIGURE 9 - GROUNDWATER INFLOW RATES CALCULATED BY MODEL

TABLE 6 - SUMMARY OF GROUNDWATER INFLOWS ESTIMATED BY MODELLING

Feature	Modelled groundwater inflow rate (m³/d)	Particular Specification criteria (m³/d)
Western shaft (Shaft 1)	Up to 58	82
Eastern shaft (Shaft 2)	Up to 19	25
Any square with an area of 10m², at any and all locations within the sides and bases of the shafts and excavations	Inflows over a given 10 m² area will be dependent on water-bearing features encountered during excavation and will require localised grouting during excavation should inflows exceed criteria	15

6.4.2 CUMULATIVE INFLOW VOLUMES COMPARED TO EIS

The cumulative groundwater inflow volume calculated by the model is compared to the EIS cumulative inflow prediction Table 7. The cumulative inflow calculated by the model is less than the EIS prediction.

TABLE 7 - CUMULATIVE GROUNDWATER INFLOW FOR WHOLE STATION COMPARED TO EIS PREDICTION

Cumulative groundwater inflow at 2 years (ML)	Cumulative groundwater inflow at 2 years predicted by EIS (ML)
20	53

6.5 DRAWDOWN AND COMPARISON TO EIS

Drawdown of the watertable predicted by the model is shown in Figure 10 and compared to the drawdown predicted in the EIS.

The watertable is conceptualised to lie within the Ashfield Shale prior to excavation, and is drawn down to lie within the Ashfield Shale / Hawkesbury Sandstone.

Significant drawdown of the watertable is not expected in the alluvium that lies over 200 m to the north of the station site.

There is negligible difference between the modelled water table drawdown at a time of two years and 10 years, which is why drawdown for both output times is not visible in Figure 10.

The predicted drawdown is generally similar to the drawdown predicted in the EIS, although somewhat larger in the southern region of the model domain.

In the southern region, the 2 m drawdown contour extends about 70 m further from the station than the 2 m drawdown predicted in the EIS. In the southern region, at the location of the 2 m drawdown predicted in the EIS, the predicted drawdown is about 6 m larger. Moving towards the station, the difference in drawdown compared to the EIS drawdown diminishes.

The somewhat larger drawdown in the southern area of the model domain primarily occurs due to differences in boundary conditions compared to those used in the EIS model.

Note also that at the time of the EIS, the design for Five Dock Station comprised two shafts with a single caverns between them. The current design comprises two shafts and three sections of cavern.

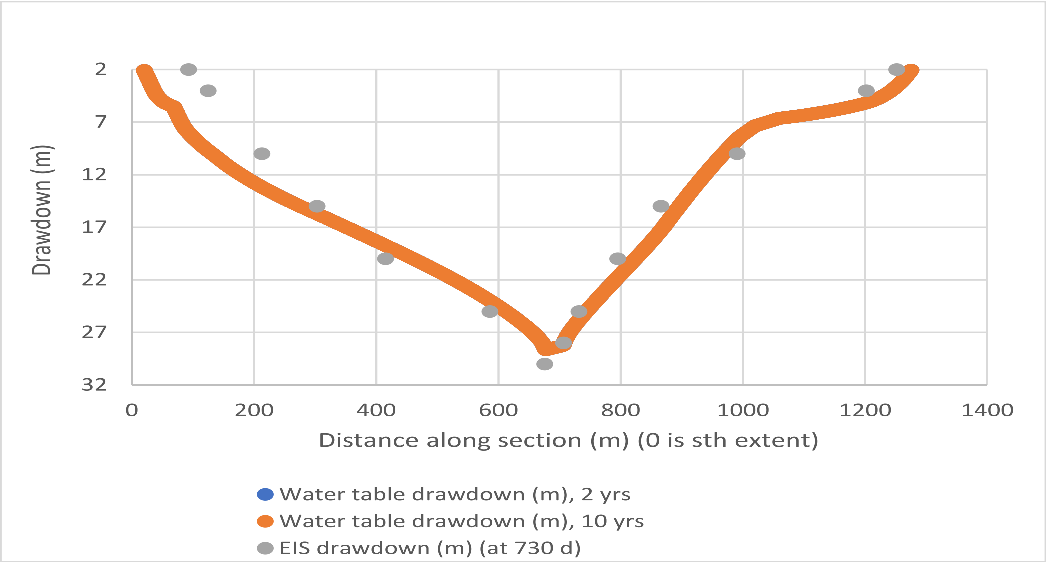


FIGURE 10 - GROUNDWATER LEVEL DRAWDOWN PREDICTED BY MODEL COMPARED TO THAT IN EIS

There is a possibility that hydraulic conductivity values may be relatively higher in the zone of possible joint swarms identified in the geological long section, in the vicinity of faults or in other as-yet unidentified zones. Should these features act as conduits to groundwater flow, groundwater level drawdown could propagate further from the station compared to the model-predicted drawdown.

7. GROUNDWATER IMPACTS

7.1 GROUNDWATER USERS AND RECEPTORS

Figure 11 below illustrates potential groundwater receptors surrounding Five Dock Station and the drawdown predicted by the EIS (Jacobs, 2020). As outlined in Section 6.5, drawdown is generally like that predicted in the EIS (Jacobs, 2020). There are no groundwater users within the estimated 2 m drawdown extent predicted by the modelling discussed in Section 6.5. Furthermore, all the existing bores located beyond the estimated 2 m drawdown extent up to a distance of about 1.2 km from the station have a purpose of 'monitoring'.

A small tract of Turpentine Forest represents a potential groundwater dependent ecosystem and falls within the drawdown extent predicted in this report and in the EIS. This potential GDE is situated about 350 m east of the Five Dock Station.

It is noted that the ecosystem is situated in a suburban park, is immediately adjacent to an excavated skateboard park facility and has no understorey. Thus, the ecosystem currently exists in a highly modified and urbanised setting.

This ecosystem grows on Wianamatta Shale and the rootzone is likely to lie within residual clay soils of the shale and/or the shale itself (where the shale is shallow). As discussed above, these geological units are likely to be of relatively low permeability. Station induced groundwater level drawdown within the sandstone is considered unlikely to impact this ecosystem.

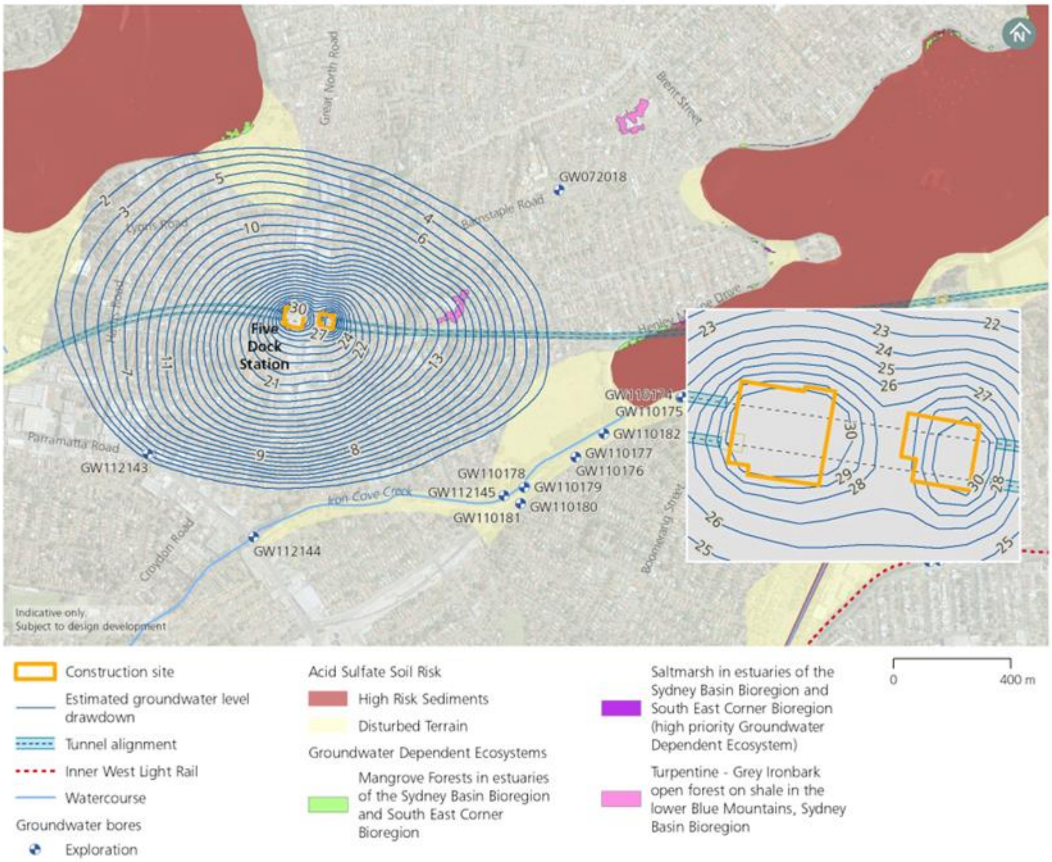


FIGURE 11 - GROUNDWATER RECEPTORS NEAR FIVE DOCK STATION, AND DRAWDOWN PREDICTED IN THE EIS (JACOBS, 2020)

7.2 ACID SULFATE SOILS

As noted in the Contamination Report, a nearby sample location at SMW_BH051_0.6-0.9 consisting of silty clay displayed an exceedance for categorisation as an acid sulfate soil (ASS). Similar soil materials including silty to gravelly clays, grey and containing sub-angular iron-stained gravels/rock fragments are present in all boreholes (SMW_BH718, SMW_BH719 and SMW_BH082) situated within the Station box footprint. The results from SMW_BH051_0.6-0.9 represent natural soils across the site. However, the soils were below detection for the presence of sulfides or sulfate and thus are more likely to represent naturally occurring acidic soils. As such, disturbance of acid sulfate soils via dewatering near the station box appears to be a low risk.

7.3 SETTLEMENT

Settlement related to groundwater drawdown has been considered as part of a separate technical memorandum.

7.4 CONTAMINANT MOVEMENT

No significant contaminants have been identified in groundwater within the station box footprint or its surrounds and, as such, contaminant movement is considered a low risk at Five Dock Station. However, TRHs, toluene and metals including arsenic, chromium, copper, manganese, nickel and zinc have been detected above the ecological guideline of freshwater 95% species protection, and as such, treatment of groundwater seepage to the excavation prior to discharge may require consideration as part of groundwater and construction management plans.

7.5 SALINE INTRUSION

The drawdown presented in Figure 10 suggests that up to 2 m of drawdown may occur within approximately 25 m of the Parramatta River. The modelled drawdown is considered conservative as higher permeability Quaternary alluvium situated adjacent to Parramatta River is not represented in the model. Since this Quaternary alluvium would have relatively elevated permeability and be in hydraulic connection with the Parramatta River, significant drawdown is unlikely to occur near the Parramatta River.

The Parramatta River is about 610 m from Five Dock Station. Based on a water level of 1 mAHD at the Parramatta River and the station excavation base level of -12.5 mAHD, the point-to-point single value hydraulic gradient (i.e. excludes lower and higher gradients that would develop near the Parammatta River and the station, respectively) is 0.02 m/d. Accordingly, assuming the hydraulic conductivity and specific yield values of the Hawkesburry Sandstone, based on Darcy’s Law, the average linear groundwater velocity would be 0.007 m/d. Based on this, saline water from Parramatta River would not reach the station within the design life. It is noted that velocities could be higher if geological structures act as conduits for groundwater flow.

8. CONSTRUCTION PHASE MONITORING

Table 8 lists proposed groundwater level monitoring locations during the construction phase, and includes existing representative groundwater levels, predicted groundwater level drawdown and groundwater drawdown trigger levels. These locations are shown in Figure 12.

Construction details of existing bores listed in Table 8 are provided in Table 1, in Section 3.2.

As noted in Section 7.1, the Turpentine Forest potential groundwater dependent ecosystem located 350 m to 450 m east of the station, is considered unlikely to be impacted by the CTP. Should it be necessary to confirm this, groundwater level monitoring would be required during construction in the vicinity of the ecosystem because no existing monitoring bores are located near the ecosystem.

A paired monitoring site, consisting of a shallow monitoring bore monitoring residual soil and upper weathered shale, with an accompanying deeper monitoring bore which monitors the lower 3 m extent of the Ashfield Shale formation is recommended. The surface level elevation in the area of the ecosystem is about 22 mAHD and based on CTP geotechnical long sections, the Ashfield Shale / Mittagong Formation transition is expected to be at a level of about 10 mAHD. Thus, the relatively deeper proposed monitoring bore would have a depth of about 12 m below ground level and be screened from about 9 m to 12 m below ground level. The ecosystem is considered unlikely to draw from groundwater at levels deeper than this.

It is assumed that the existing piezometers listed are accessible and in suitable working order. In the event that the existing piezometers listed are inaccessible or destroyed, alternative monitoring locations will need to be constructed.

TABLE 8 - SUMMARY OF CONSTRUCTION PHASE MONITORING LOCATIONS AND TRIGGER LEVELS

Location I.D.	Existing / proposed monitoring location	Reason for monitoring	Approx. distance from station (m)	Existing representative groundwater level (mAHD)	Existing representative groundwater saturated bore thickness (m)	Predicted groundwater level drawdown, two years after excavation commenced (m)	Predicted groundwater level drawdown, two years after excavation commenced (mAHD)
SMW_BH050_w	Existing	Drawdown monitoring	103	4	5.8	23.5	-6.35
SMW_BH050_s	Existing	Drawdown monitoring	103	24	0.9	NA	NA
SMW_BH051_w	Existing	Drawdown monitoring	75	19.0	8.3	24.5	-7.6
SMW_BH051_s	Existing	Drawdown monitoring	75	19.8	0.1	NA	NA
AF_BH17	Existing	Drawdown monitoring	<1	0.73	18.3	28	-12.6
R248_3103_BH141	Existing	Drawdown monitoring	200	4.0 [a]	13.1 [a]	20.5	-3.1
R248_3103_BH141A	Existing	Drawdown monitoring	200	14.1 [a]	4.4 [a]	20.5	-3.0
TBC (paired monitoring bores)	Proposed	Drawdown monitoring at potential GDE	350 to 450	Unknown	Unknown	Up to 16	Unknown

Notes: [a] Percentage drawdown criteria from Stage 2 Instrumentation and Monitoring Report applied to bore.

FIGURE 12 - CONSTRUCTION PHASE GROUNDWATER LEVEL MONITORING LOCATIONS

9. SUMMARY

9.1 DESIGN GROUNDWATER LEVELS

Over the long-term, groundwater levels immediately surrounding the excavation will be close to the excavation floor level (or the deepest passive dewatering level). For the permanent (10 year design life) condition, it can therefore be assumed that there is no hydrostatic pressure on the retaining walls.

Design is required to consider groundwater levels in response to burst water mains and blocked drainage (Particular Specification SM-W-CTP-PS-709 and SM-W-CTP-PS-1030).

9.2 GROUNDWATER INFLOWS

Groundwater inflow rates calculated by the model are summarised as follows:

- Eastern shaft – up to 19 m^3/d.
- Western shaft – up to 58 m^3/d.
- Caverns – up to 94 m^3/d.
- Shafts and caverns combined – up to 170 m^3/d.

The modelled groundwater inflow rates vary with time. During earlier phases of excavation, groundwater inflow rates are likely to be lower than those listed above (because the model assumes that full excavation is instantaneous). In practice, the excavation will be deepened progressively, and peak groundwater inflows are likely to be lower than those reported here.

The modelled peak groundwater inflow rates are below the Particular Specification criteria for the western shaft (criteria = 82 m^3/d) and eastern shaft (criteria = 25 m^3/d).

With respect to Particular Specification 4.1.7 (h) (ii), which states that groundwater seepage must not exceed 15,000 litres in any 24-hour period, measured over any square with an area of 10 m^2; inflows over any given 10 m^2 area of excavation face will depend on the water-bearing features encountered during excavation.

There is a possibility that hydraulic conductivity may be higher than the values adopted for the modelling, particularly in zones of possible joint swarms, or in other as-yet unidentified zones. Should water-bearing features be encountered during excavation, groundwater inflows may be higher than estimated, and localised grouting during excavation may be required to limit groundwater inflows to the Particular Specification criteria.

The cumulative inflow (groundwater "take") after two years is estimated to be 20 ML. This is less than the cumulative inflow of 53 ML predicted in the EIS.

9.3 DRAWDOWN AND GROUNDWATER IMPACTS

The estimated groundwater drawdowns associated with inflows indicate that:

- Groundwater users are unlikely to be affected by drawdown
- There is a low risk to groundwater dependent ecosystems (the Turpentine Forest) located to the east of the station box
- There is a low risk of disturbing acid sulphate soils via dewatering as the only acidic soils identified did not indicate the presence of sulphides or sulphate
- The movement of contaminated groundwater is considered a low risk
- Treatment of groundwater seepage to the excavations prior to disposal may be necessary, depending on the disposal options proposed
- Saline intrusion from the coastal aquifers near the Parramatta is considered to be a low risk.

Construction-phase groundwater level monitoring is required to confirm groundwater level drawdown predicted in this report.

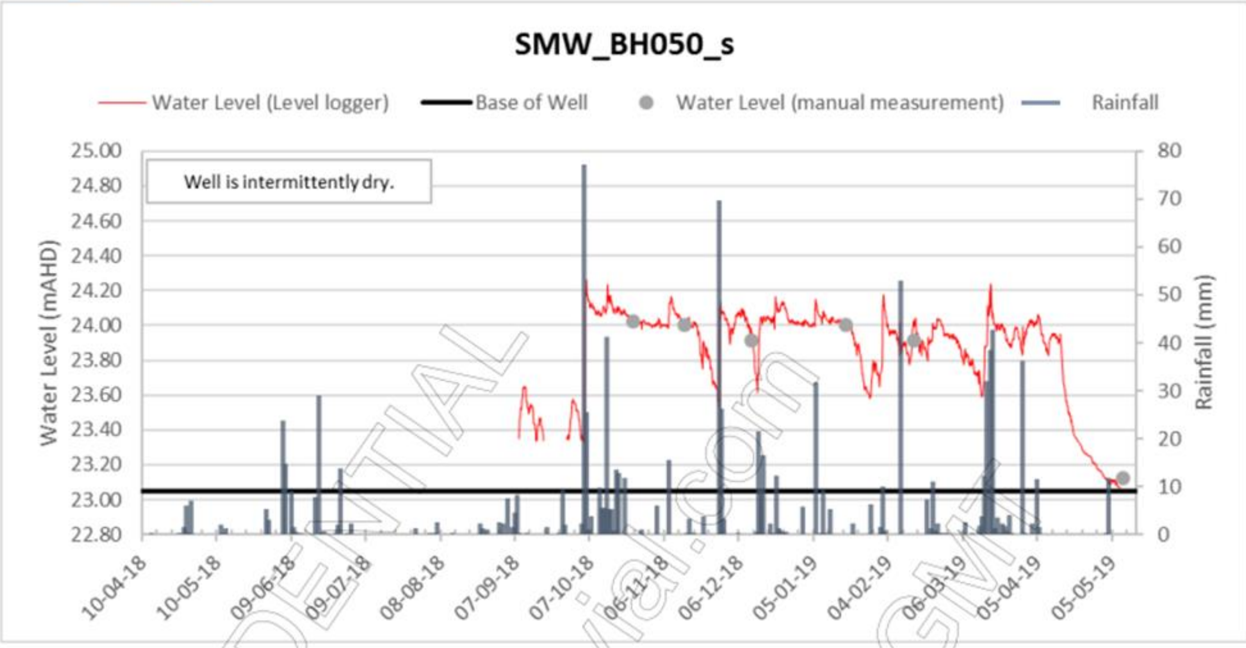
REFERENCES

Jacobs (2020), Westmead to The Bays and Sydney CBD Environmental Impact Statement Concept and Stage 1 Technical Paper 7 Hydrogeology. April 2020.

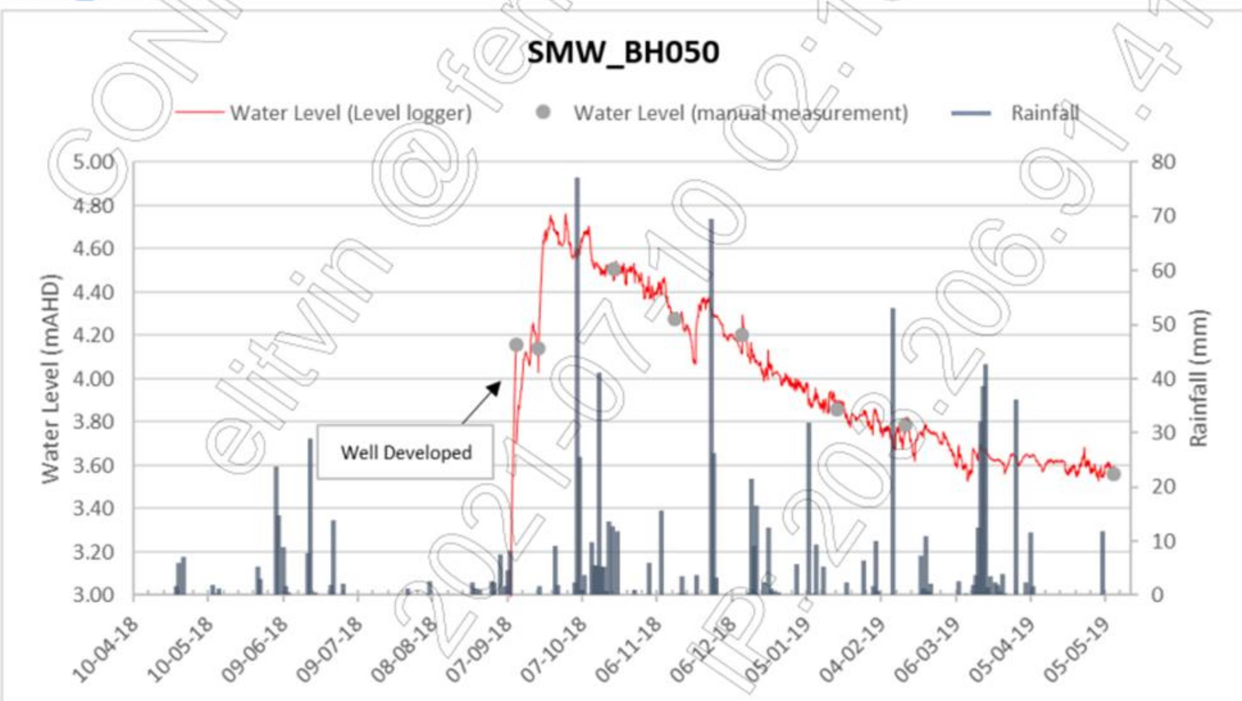
10. ANNEXURES

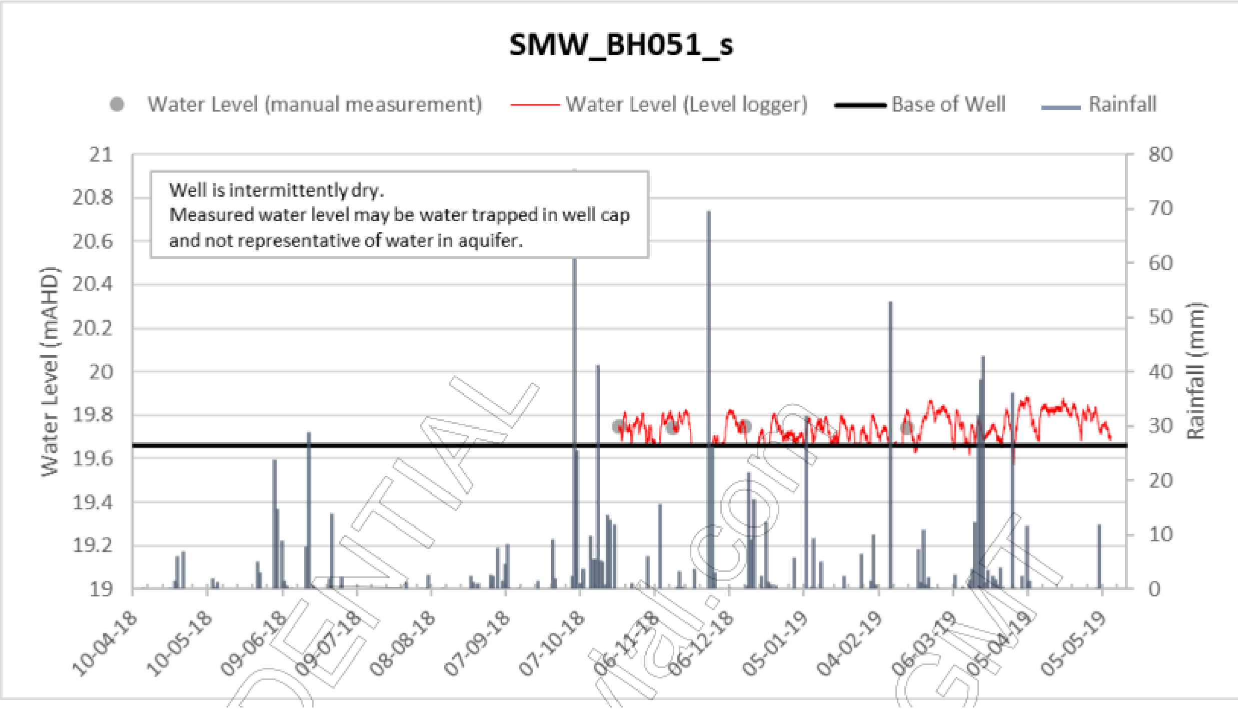
ANNEXURE A. HYDROGRAPHS

SMW_BH050_s

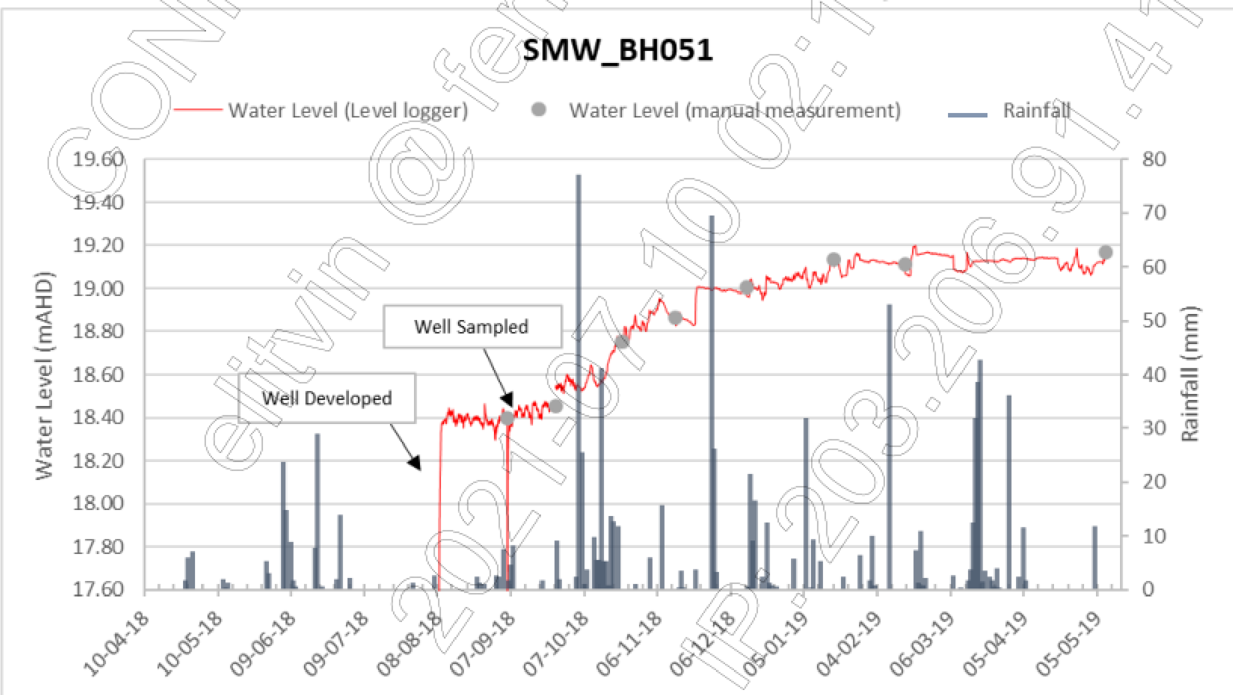


SMW_BH050




SMW_BH051_s
Water Level (manual measurement)
Water Level (Level logger)
Base of Well
Rainfall
Well is intermittently dry.
Measured water level may be water trapped in well cap
and not representative of water in aquifer.
Water Level (mAHD)
Rainfall (mm)
21
20.8
20.6
20.4
20.2
20
19.8
19.6
19.4
19.2
19
80
70
60
50
40
30
20
10
0
10-04-18
10-05-18
09-06-18
09-07-18
08-08-18
07-09-18
07-10-18
06-11-18
06-12-18
05-01-19
04-02-19
06-03-19
05-04-19
05-05-19

SMW_BH051


SMW_BH051
Water Level (Level logger)
Water Level (manual measurement)
Rainfall
Well Sampled
Well Developed
Water Level (mAHD)
Rainfall (mm)
19.60
19.40
19.20
19.00
18.80
18.60
18.40
18.20
18.00
17.80
17.60
80
70
60
50
40
30
20
10
0
10-04-18
10-05-18
09-06-18
09-07-18
08-08-18
07-09-18
07-10-18
06-11-18
06-12-18
05-01-19
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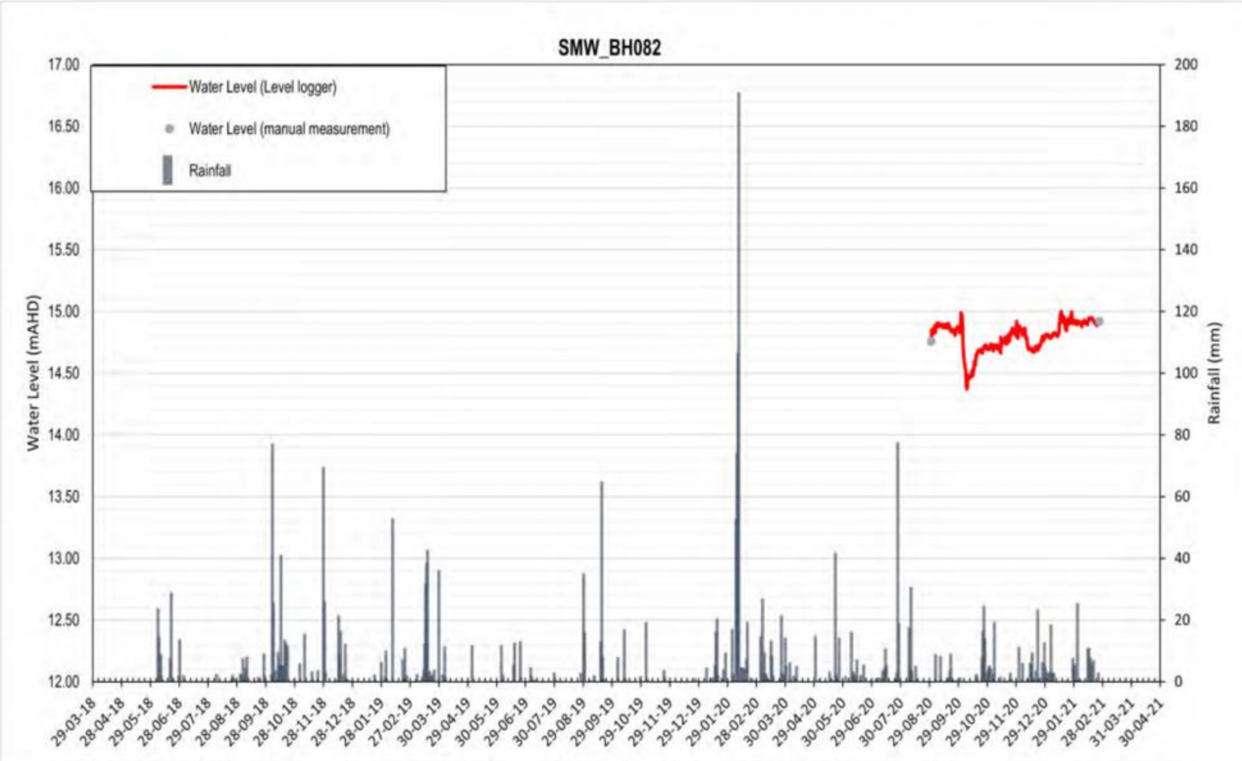
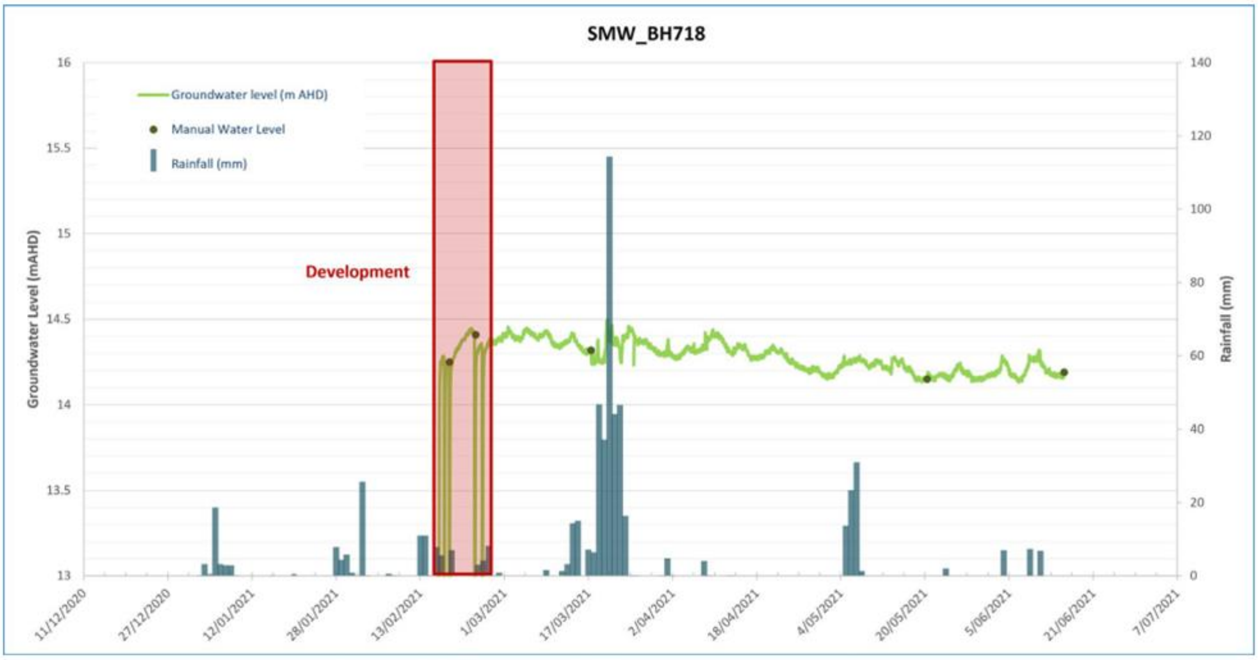
05-04-19
05-05-19



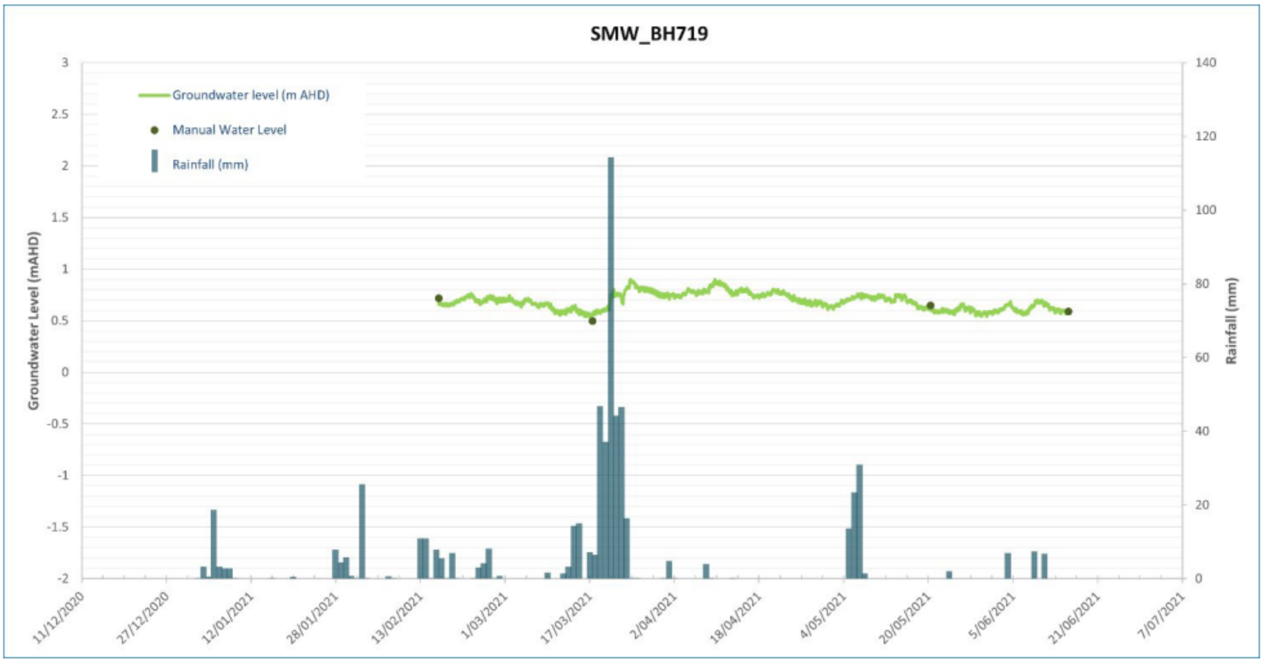
SMW_BH082
Water Level (Level logger)
Water Level (manual measurement)
Rainfall
Water Level (mAHD)
Rainfall (mm)

SMW_BH718

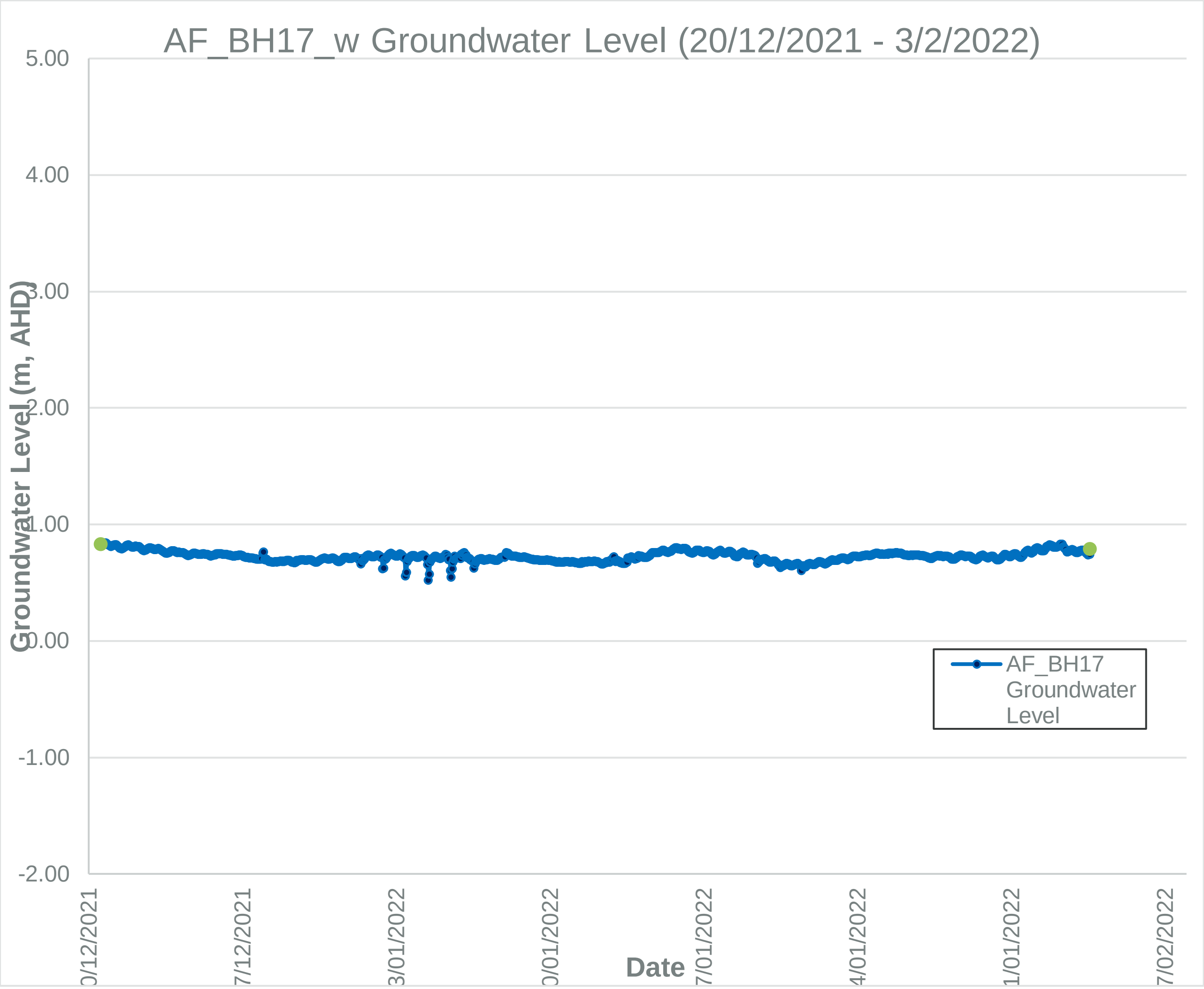


SMW_BH718
Groundwater level (m AHD)
Manual Water Level
Rainfall (mm)
Development
Groundwater Level (mAHD)
Rainfall (mm)

SMW_BH719



SMW_BH719
Groundwater level (m AHD)
Manual Water Level
Rainfall (mm)
Groundwater Level (mAHD)
Rainfall (mm)
3
2.5
2
1.5
1
0.5
0
-0.5
-1
-1.5
-2
140
120
100
80
60
40
20
0
11/12/2020
27/12/2020
12/01/2021
28/01/2021
13/02/2021
1/03/2021
17/03/2021
2/04/2021
18/04/2021
4/05/2021
20/05/2021
5/06/2021
21/06/2021
7/07/2021


AF_BH17_w Groundwater Level (20/12/2021 - 3/2/2022)
5.00
4.00
3.00
2.00
1.00
0.00
-1.00
-2.00
Groundwater Level (m, AHD)
AF_BH17 Groundwater Level
Date
0/12/2021
7/12/2021
3/01/2022
0/01/2022
7/01/2022
4/01/2022
1/01/2022
7/02/2022

ANNEXURE B. HYDROGEOLOGICAL PROPERTIES

Annexure B: Hydrogeological units and parameter values

Document History and Status

Revision	Date	Description	Author	Checked	Reviewed	Approved
A	28/02/2022	Draft Report	Ben Rose, Ben Rotter	Ben Rotter	Richard Evans	Fernando Lopez Asensio

Contents

1. Introduction

1.1. Objective and scope

The objective of this memorandum is to summarise key hydrogeological units, and parameter values applicable to the CTP project, for all CTP works locations with the exception of The Bays Station area. The Bays Station area is covered separately in the The Bays Station Hydrogeological Design Report due to its unique characteristics.

1.2. Basis of memorandum

This memorandum has been prepared based on ground profile data and hydraulic testing results from investigations specifically undertaken for the CTP project, as well as hydrogeological unit properties published in studies and reports for other major projects undertaken in Sydney.

The other major projects include:

- WestConnex – New M4
- WestConnex – M4-M5 Link
- WestConnex – New M5
- Beaches Link and Gore Hill Freeway Connection
- Western Harbour Tunnel and Warringah Freeway Upgrade
- Rozelle Interchange
- Hydrogeological resource investigations to supplement Sydney's water supply at Leonay, Western Sydney
- North Strathfield Rail Underpass

Studies that were not directly associated with specific major projects included:

- Groundwater Control for Sydney Rock Tunnels and geotechnical aspects of tunnelling for infrastructure projects reported by Hewitt (2005)
- Hydrogeological properties of Hawkesbury Sandstone in the Sydney region summarised by Tammetta and Hewitt (2004)
- A summary of hydrologic and physical properties of rock and soil materials by Morris and Johnson (1967)

2. Hydrogeological units

2.1. Overview

There are seven key hydrogeological units applicable to project:

- Fill
- Quaternary alluvium
- Residual soil
- Ashfield Shale
- Mittagong Formation
- Hawkesbury Sandstone
- Dykes

Fault zones are also discussed.

Not all seven hydrogeological units are present throughout the entire project area. In some settings, the shallower hydrogeological units (fill, quaternary alluvium and/or residual soil) may be unsaturated. For discussion purposes, dykes and faults have been grouped.

2.2. Fill

Fill of variable thickness is present across much of the project area and may host perched or permanent water tables, or be unsaturated, depending on specific-site conditions. The hydraulic properties for fill are conceptualised to be highly variable, owing to highly variable composition, ranging from gravel to clay.

Groundwater flow through the fill is controlled by the primary permeability of the units with areas of coarse material (gravels and sands) yielding higher permeabilities and finer grained material (silts and clays) yielding lower permeabilities.

2.3. Quaternary alluvium

With the exception of The Bays, alluvium is not present at the location of the station boxes. Alluvium is generally not considered a significant hydrogeological unit for the project.

However, alluvium is present to the east of the Burwood North Station site and is of potential relevance to the impacts of groundwater level drawdown.

Approximate minimum distances from the station boxes to alluvium mapped by the Geological Survey of NSW (1983) are as follows:

- Sydney Olympic Park Station – 260 m
- North Strathfield Station – 400 m
- Burwood North Station – 25 m
- Five Dock Station – 400 m

JTJV has inferred, based on limited available geotechnical field data, that the alluvium in the vicinity of Burwood North Station is about 40 m from the eastern end of the station box. The alluvium at this location is up to 4 m thick, as shown in Figure 2-2 and Figure 2-2.

FIGURE 2-1: LOCATION OF ALLUVIUM AT BURWOOD NORTH STATION IN PLAN

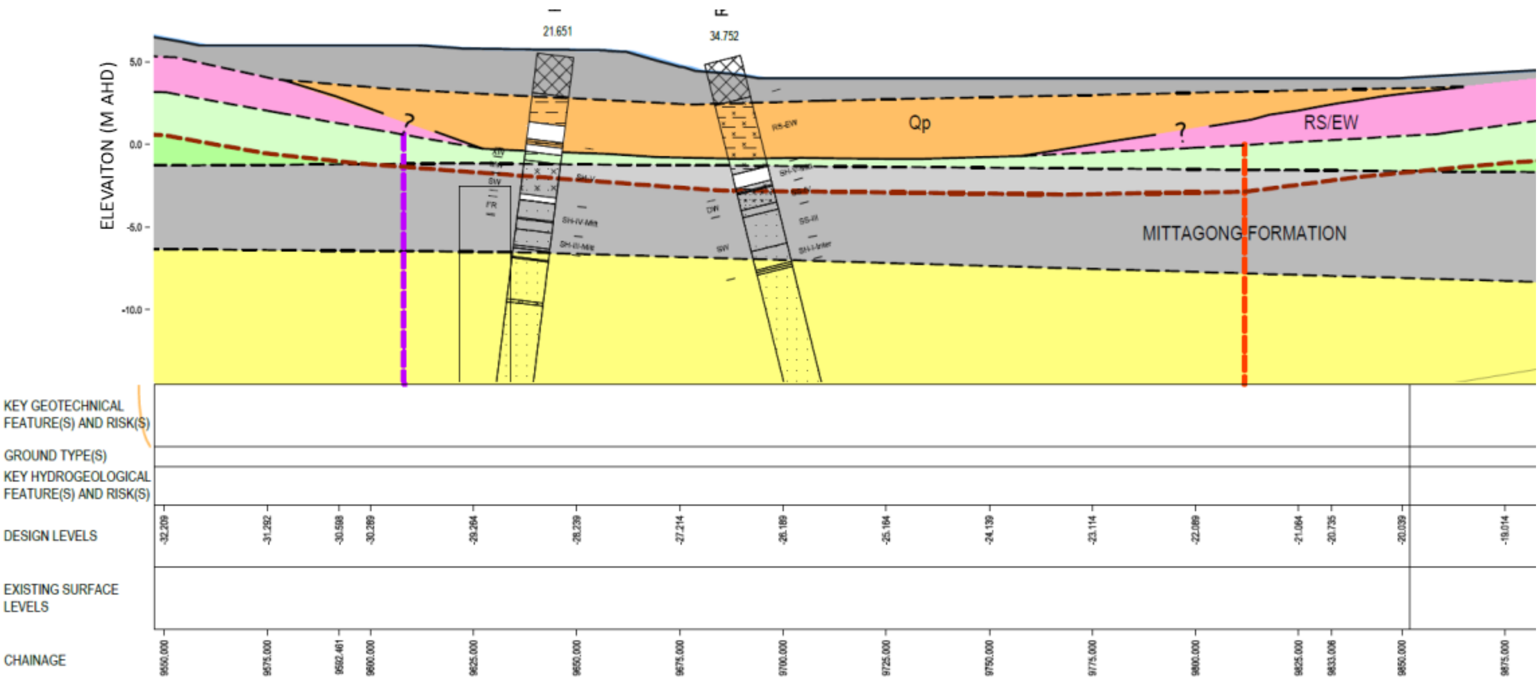


FIGURE 2-2: LOCATION OF ALLUVIUM AT BURWOOD NORTH STATION IN SECTION

2.4. Residual soil

Residual soil is not considered a significant hydrogeological unit for the project as it is typically relatively thin, typically occurs relatively close to existing ground levels and is often unsaturated. Additionally, excluding The Bays area, much of the residual soils are derived from weathered Ashfield Shale, which results in clayey material of relatively low permeability.

In locations where the unit is unsaturated (typical case), except for influences on groundwater recharge, the unit will have no direct influence on groundwater inflows to project excavations and associated groundwater level drawdowns. Indirectly, the unit could influence recharge rates, which could influence groundwater inflow rates and drawdown.

In locations where the unit is permanently saturated (atypical case), there may be implications associated with drawdown at groundwater receptors, if present. Additionally, there may be settlement implications.

2.5. Ashfield Shale

Ashfield Shale is relevant to the project and, where present, forms the uppermost hydrogeological rock unit, with the unit present over about half of the entire CTP project alignment length. The unit is characteristically of relatively low permeability. Groundwater flow primarily occurs through fractures and joints (secondary porosity) as the matrix effective porosity and hydraulic conductivity are very low.

The Sydney 1:100,000 Geological Series Sheet (Geological Survey of NSW, 1983) describes Ashfield Shale as black to dark grey shale and laminite. Residual soil, alluvium or alluvium and residual soil overly the unit. The Mittagong Formation underlies the unit.

The unit is variable in thickness. For example, at the project stations, the unit ranges from relatively thin (about 2 to 5 m thick) at Five Dock Station to relatively thick (about 40 m thick) at Sydney Olympic Park Station.

2.6. Mittagong Formation

The Mittagong Formation is a transitional unit between the Ashfield Shale and Hawkesbury Sandstone.

The Sydney 1:100,000 Geological Series Sheet (Geological Survey of NSW, 1983) describes the Mittagong Formation as interbedded shale, laminite and medium grained quartz sandstone.

The unit is generally thin and in the range of 1 m to 10 m thick.

2.7. Hawkesbury Sandstone

Hawkesbury Sandstone is relevant to the project and forms the basal groundwater system for the project.

The Sydney 1:100,000 Geological Series Sheet (Geological Survey of NSW, 1983) describes Hawkesbury Sandstone as medium to coarse grained quartz sandstone, very minor shale and laminite lenses.

Groundwater flow in the sandstone is typically controlled by secondary features such as fractures, joints, shears and bedding planes and effectively acts as a fractured rock aquifer. Areas where the unit is more fractured tend to yield greater permeabilities, while more competent sections typically yield low permeabilities.

2.8. Dykes

The CTP project alignment intersects dykes that are both known to be present and have been inferred as present based on published geological maps.

Where present, the dykes are expected to consist of linear doleritic rock body intruded into the surrounding country rock. Typical of dolerite dykes in the Sydney Basin, it is expected that the central core of the dyke at depth would be fresh, with country rock adjacent to the dyke being more deeply weathered in the uppermost bedrock, but fresh and of higher strength in the metamorphosed ("baked") margin adjacent to the dyke at depth. The more deeply weathered zones can be either of lower permeability, due to the presence of rock that has been weather to clay; or of higher permeability, where the extent of weathering is less than highly/extremely weathered and leads to more permeable fractures.

2.9. Fault zones

If present, faults zones can be associated with rock that exhibits joint swarms. It is possible that rock in the vicinity of inferred fault zones is relatively more fractured compared to surrounding rock and has higher permeability than the surrounding country rock.

3. Hydrogeological testing results and properties

3.1. Hydrogeological test data and literature

Hydrogeological unit parameter values were assessed for CTP project hydrogeological testing results, supplemented with individual hydrogeological testing results from other surrounding projects. Although incorporating some non-CTP project data, the dataset used in this assessment is hereafter referred to as CTP project data in text and summary tables. Statistical analysis was performed on this dataset.

In addition to the statistical analysis performed on the CTP project data, a literature review was undertaken for projects in the region. The hydrogeological parameter value ranges and statistics reported in the literature were summarised to compare against the CTP project dataset. This approach was taken because the literature typically did not contain individual test results and instead summarised results. For the literature review, in addition to hydrogeological parameter values associated with hydraulic testing, parameter values adopted for numerical groundwater models are summarised.

Outside of The Bays Station site, the following testing data has been used to characterise hydrogeological units and define hydrogeological parameter values:

- Hydrogeological testing for the Sydney Metro West (SMW) project:
  - 22 water pressure (packer) tests in Ashfield Shale, supplemented with 18 packer tests in Ashfield Shale, undertaken for North Strathfield Rail Underpass (SKM and Parsons Brinckerhoff, 2013)
  - Six packer tests incorporating either sandstone and breccia or dolerite
  - Six rising/falling head tests at a single location where the gravel packed zone encompassed fill, monitoring bore SMW_BH126_w, located at Sydney Olympic Park. The gravel packed zone consisted of generally clayey fill and siltstone
- Hydrogeological testing data from other projects:
  - 80 packer tests in siltstone and sandstone, supplemented with two packer tests undertaken for Western Harbour Tunnel
  - 115 packer tests in sandstone, with supplemented with four packer tests undertaken for Western Harbour Tunnel, and 31 packer tests undertaken for Rozelle Interchange.
- Generalised data from the literature:
  - 30 packer tests in Ashfield Shale (Aecom, 2015 and 2017), undertaken for WestConnex M4-M5 and New M5
  - 196 packer tests, undertaken for WestConnex M4-M5 Link (Aecom, 2017)
  - 205 packer tests, undertaken for New M5 (Aecom, 2015)
  - 363 packer tests, Sydney region, non-project specific (Hewitt, 2005)
  - 300 packer tests, undertaken for Western Harbour Tunnel and Warringah Freeway Upgrade (Jacobs, 2020)

3.2. Hydrogeological testing results and hydrogeological properties

3.2.1. Fill

To date, project hydraulic conductivity testing has only been completed at one location where the gravel packed zone encompassed fill, monitoring bore SMW_BH126_w, located at Sydney Olympic Park. The gravel packed zone consisted of generally clayey fill and siltstone. Six rising/falling head tests were completed in the monitoring well and returned an average and median hydraulic conductivity of 8.6×10^{-4} m/d and 8.4×10^{-4} m/d, respectively (Golder and Douglas Partners, 2021).

The fill is of little relevance to the CTP project with respect to its influence of groundwater inflow rates to excavations and potential groundwater level drawdown because the unit is typically unsaturated. In atypical areas where the fill is saturated, the fill is generally relatively shallow (less than a few metres thick).

3.2.2. Quaternary alluvium

Outside of The Bays region, hydraulic testing of alluvium has not been undertaken for the project. With the exception of The Bays Station site, alluvium is not present at the locations of the station boxes, except in the vicinity of Burwood North Station as noted above.

Alluvium hydrogeological properties derived from the literature are summarised in Table 3-1. As expected, there is considerable variation in the hydraulic conductivity and specific yield values, since alluvium can range from predominantly sandy to clayey, and incorporate a wide variety of deposits, including silts and gravels.

TABLE 3-1: QUATERNARY ALLUVIUM GROUNDWATER SYSTEM PROPERTIES FROM LITERATURE REVIEW

Statistic	Regional literature review	Non-geographic literature review	Numerical groundwater models			
			SS [a]	SS [a]	SS [a]/T [b]	SS [a]/T [b]
Horizontal hydraulic conductivity (m/d)						
Minimum	1.00×10^{-2}					
Single value		5.00×10^{-3} (clay)	4.32×10^{-1}	5.00×10^{-1}	1.00×10^{0}	1.00×10^{0}
Maximum	1.00×10^{0}					
K_v/K_h						
Minimum	0.01					
Single value			0.2	0.1		0.5
Maximum	0.1					
Specific storage range (m^{-1})						
Single value						1.00×10^{-5}
Specific yield (-)						
Single value		0.06 (clay)				0.20
Source						
	Golder (2016)	Morris and Johnson (1967)	Golder (2016)	CDM Smith (2016)	GHD (2015)	Hydro Simulations (2017)
Summary						
Parameter	**Minimum value**	**Maximum value**	**Representative value**			
Horizontal hydraulic conductivity (m/d)	1.00×10^{-2}	1.00×10^{0}	1.00×10^{0} (sandy) 5.00×10^{-3} (clayey)			
Kv/Kh	0.01	0.5	0.1			
Specific storage (m-1)	1.00×10^{-5}	1.00×10^{-5}	1.00×10^{-5}			
Specific yield (-)	0.20	0.20	0.20 (sandy) 0.06 (clayey)			

Notes: [a] SS = steady state. [b] T = transient.

3.2.3. Residual soil

Hydraulic testing of residual soil has not been undertaken for the project. As outlined in Section 2.4, residual soil is not considered a significant hydrogeological unit for the project. As such, hydrogeological properties have not been reviewed for this hydrogeological unit.

3.2.4. Ashfield Shale

Ashfield Shale groundwater system hydraulic properties derived from the literature review are summarised in Table 3-2.

Table 3-2: Ashfield Shale groundwater system properties from CTP project data and literature review

Statistic		Packer testing		Literature reviews		Groundwater models					
	CTP siltstone intervals	WestConnex M4-M5 Link	New M5			SS [a]	SS [a]	SS [a]	T [b]	SS [a]/T [b]	T [b]
Horizontal hydraulic conductivity (m/d)											
Minimum	8.67×10^{-4}	8.60×10^{-3}	1.00×10^{-4}	Weathered and fresh rock: 1.00×10^{-4}	1.00×10^{-4}				1.91×10^{-4}	1.00×10^{-3}	
5th percentile	8.67×10^{-4}										
10th percentile	8.67×10^{-4}										
25th percentile	8.67×10^{-4}										
Median	2.60×10^{-3}		3.00×10^{-3}							2.00×10^{-2}	
Harmonic mean	1.91×10^{-3}	1.00×10^{-2}									
Geomean	4.45×10^{-3}										
Average	1.65×10^{-2}	1.70×10^{-2}	2.00×10^{-2}							2.82×10^{-2}	
Single value						8.00×10^{-4}	1.00×10^{-3}	1.08×10^{-2}			4.32×10^{-3}
75th percentile	1.84×10^{-2}										
90th percentile	4.42×10^{-2}										
95th percentile	8.71×10^{-2}										
Maximum	1.39×10^{-1}	1.20×10^{-1}	7.00×10^{-2}	Weathered rock: 1.00×10^{-1} Fresh rock: 1.00×10^{-2}	1.00×10^{-2}				6.62×10^{-3}	6.00×10^{-2}	
N (number of tests)	40	24	6								
K_v/K_h											
Minimum										0.003	
Single value						1	0.1				0.1
Maximum										0.1	
Specific storage (m^{-1})											
Single value					1.00×10^{-5}					1.00×10^{-5}	5.00×10^{-6}
Specific yield (-)											
Minimum										0.02	
Single value					0.01						0.03
Maximum										0.025	
Source											
	CTP project data	Aecom (2017)	Aecom (2015)	Hewitt (2005)	Golder (2016)	Golder (2016)	CDM Smith (2016)	GHD (2015)	GHD (2015)	Hydro Simulations (2017)	LSBJV (2020)
Summary											
Parameter	**Minimum value**	**Maximum value**	**Adopted representative value**								
Horizontal hydraulic conductivity (m/d)	1.00×10^{-4}	1.20×10^{-1}	5.00×10^{-3}								
Kv/Kh	0.003	1	0.1								
Specific storage (m-1)	5.00×10^{-6}	1.00×10^{-5}	1.00×10^{-5}								
Specific yield (-)	0.01	0.03	0.02								

Notes: [a] SS = steady state. [b] T = transient.

Packer tests have been undertaken for the project and surrounding projects and results reviewed based on material type. The results for packer tests conducted in siltstone are summarised in Table 3-4. Figure 3-1 provides a plot of this data and additionally the results for sandstone and siltstone test intervals (i.e., interbedded material). It is noted that the results for the sandstone and siltstone test intervals were not statistically different to the results for the siltstone packer test intervals.

In Figure 3-1 the Lugeon values are plotted against depth.

Additionally, in accordance with Quinones-Rozo (2010), qualitative Lugeon and hydraulic conductivity classification, as well as qualitative rock mass discontinuity classifications, are noted on Figure 3-1. These test interval material types are considered to be generally representative of Ashfield Shale.

Qualitative Lugeon and hydraulic conductivity classification and description of rock mass discontinuities in accordance with Quinones-Rozo (2010) is as follows:

- The 75th percentile value for the sandstone and siltstone test intervals is classified as a very low (<1 Lugeon) Lugeon value, with the rock mass characterised as very tight
- The 75th percentile value for the siltstone test intervals is classified as a low Lugeon value (1 to 5 Lugeon), with the rock mass characterised as tight
- For the sandstone and siltstone test intervals, only two out of 88 tests surpassed the medium Lugeon range criteria (15 to 50 Lugeon). These two tests occurred in borehole SMW_BH502 and the recorded result was greater than 100 Lugeons for both tests, which is classified as a very high Lugeon value
- For the siltstone test intervals, only one out of 40 tests surpassed the moderate Lugeon range criteria (5 to 15 Lugeon), the maximum test value of 16 Lugeons

The packer test results are consistent with those reported in the literature and indicate that the bulk hydraulic conductivity for Ashfield Shale is very low. However, hydraulic conductivity can be, and is, elevated locally in some instances due to potential geological features.

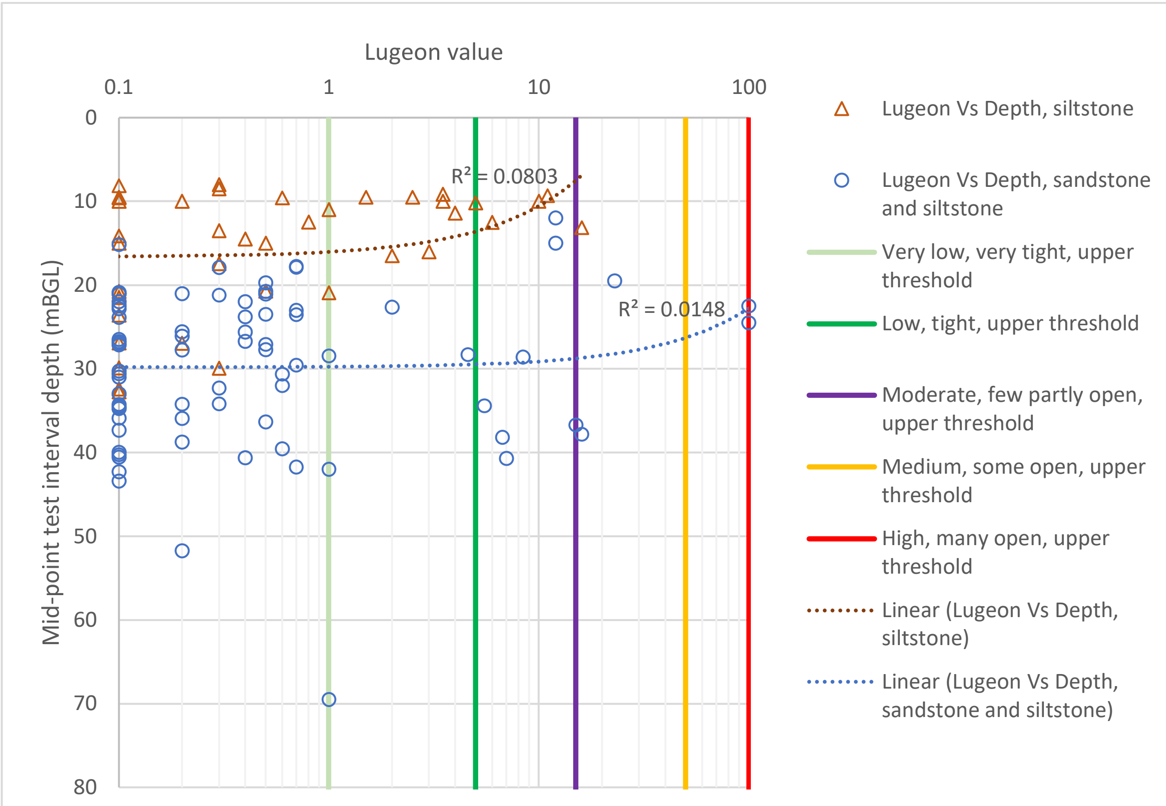


FIGURE 3-1: LUGEON VALUES FOR SILTSTONE TEST INTERVALS, AND SANDSTONE AND SILTSTONE TEST INTERVALS, CLASSED ACCORDING TO QUINONES-ROZO (2010)

The relationship between Ashfield Shale hydraulic conductivity and depth below ground surface has been assessed. The trend lines in Figure 3-1 suggest that hydraulic conductivity decreases with depth. However, the coefficients of determination for both trendlines are low, indicating the relationship is not strong.

Table 3-3 shows packer test result statistics (median, geometric mean and arithmetic mean) for siltstone test intervals by depth categories. A box and whisker plot of the siltstone packer test interval results is provided in Figure 3-2.

FIGURE 3-1,

Table 3-3 and Figure 3-2 indicate the hydraulic conductivity of Ashfield Shale generally decreases with depth. The trends also suggest that an initial upper layer may be present and have relatively higher hydraulic conductivity, which could be associated with weathering. Although a trend is established, the decreases in values are not considered significant for the purpose estimating groundwater inflows and associated impacts.

TABLE 3-3: LUGEON AND HYDRAULIC CONDUCTIVITY STATISTICS FOR SILTSTONE PACKER TEST INTERVALS BY DEPTH

Packer mid-point test interval depth category	Number of tests	Lugeon value			Horizontal hydraulic conductivity (m/d)		
		Median	Geometric mean	Arithmetic mean	Median	Geometric mean	Arithmetic mean
0 to <15 m	26	0.6	0.7	2.6	4.77×10^{-3}	6.47×10^{-3}	2.27×10^{-2}
15 to <30 m	12	0.3	0.3	0.7	2.17×10^{-3}	2.60×10^{-3}	5.63×10^{-3}
30 to <45 m	2	0.1	0.1	0.1	8.67×10^{-4}	8.67×10^{-4}	8.67×10^{-4}

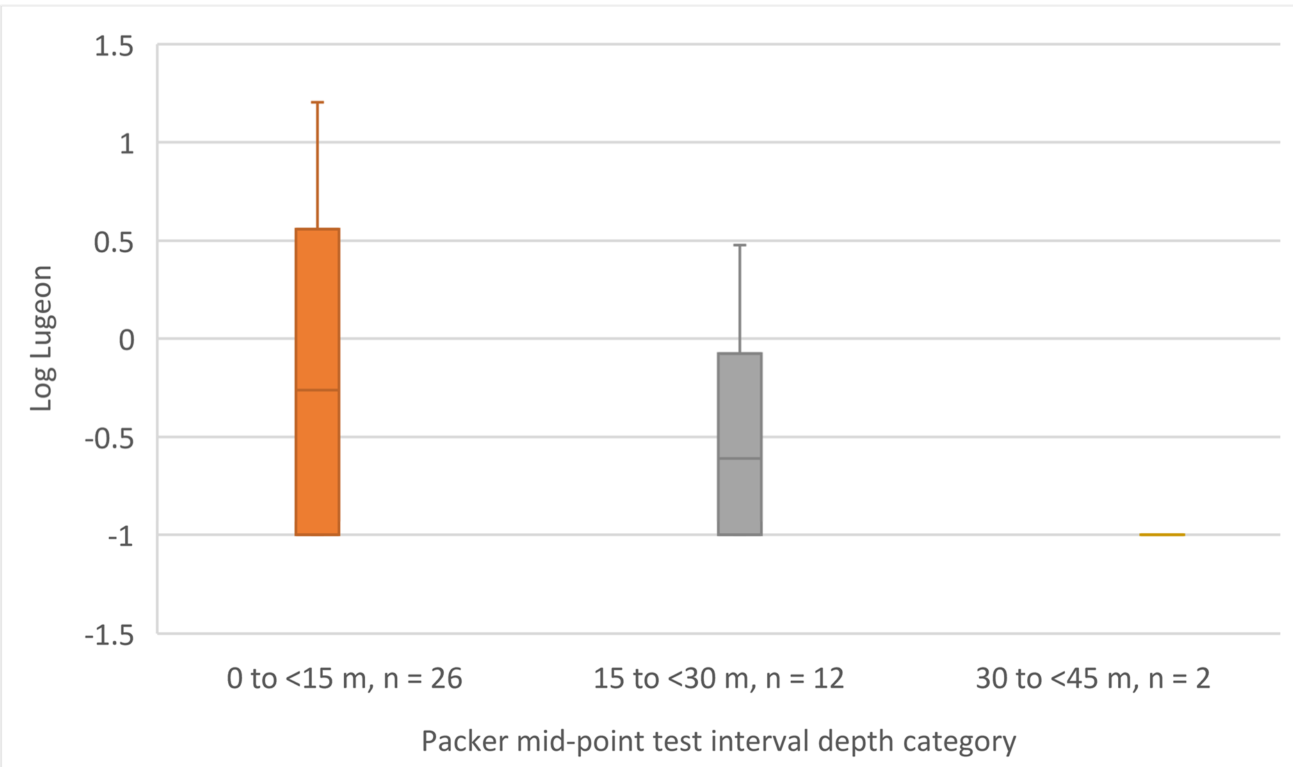


FIGURE 3-2: LOG LUGEON VALUES FOR SILTSTONE TEST INTERVALS BY DEPTH CATEGORY

It is well established that hydraulic conductivity test values are log-normally distributed. Figure 3-3 shows the cumulative distribution for the tests in siltstone.

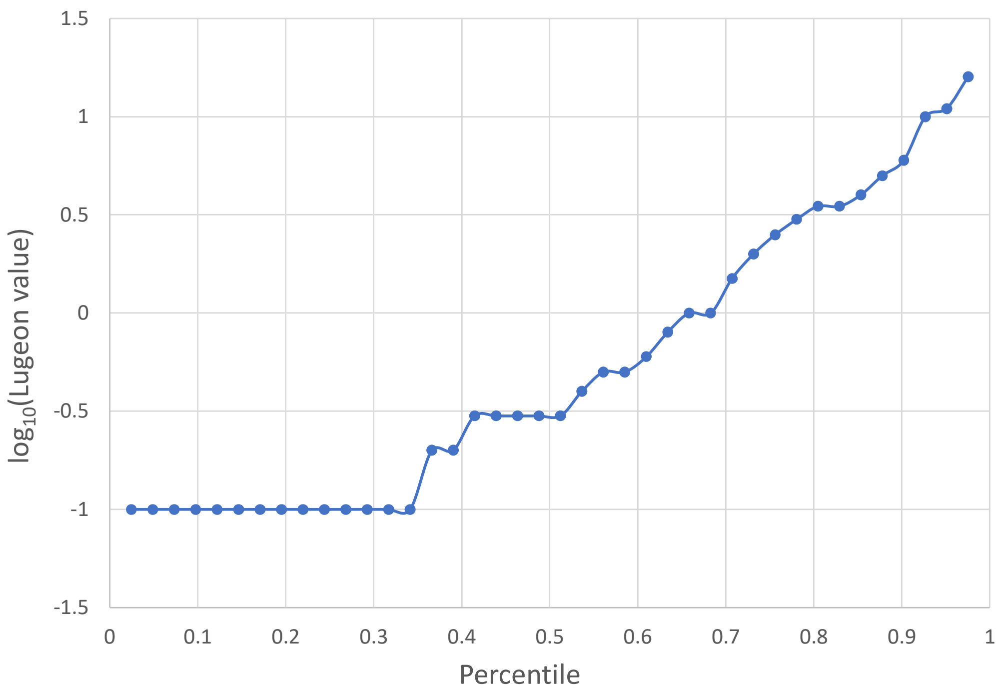


FIGURE 3-3: CUMULATIVE DISTRIBUTION OF LUGEON VALUES FOR SILTSTONE TEST INTERVALS

Since Darcy's Law uses an arithmetic mean hydraulic conductivity, the arithmetic mean of the log-normal distribution of the Lugeon values may be adopted in groundwater modelling as representative of the bulk rock.

Figure 3-4 shows the same cumulative distribution as in Figure 3-3, along with a normal distribution model fitted to the data. The model considers a 90% confidence interval and that the limits of measurement of the packer tests are 0.1 Lugeons and 100 Lugeons. Figure 3-5 shows a quantile plot for the Lugeon data and the model. The resulting mean value from the model is 2 Lugeons. This result is also shown in Table 3-4.

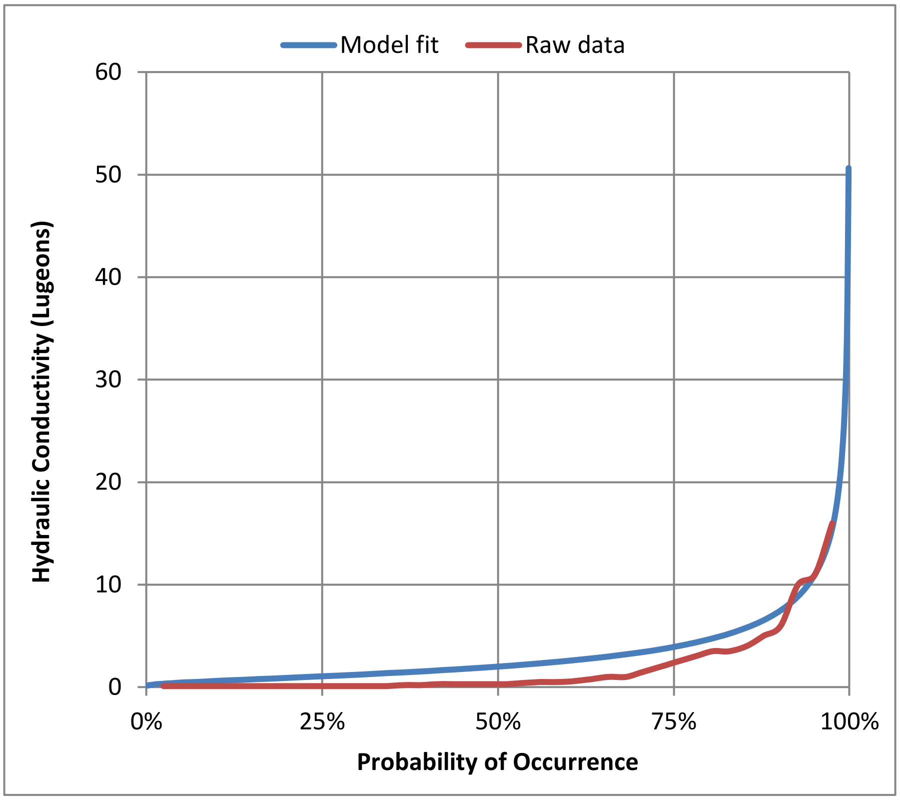


FIGURE 3-4: CUMULATIVE DISTRIBUTION OF LUGEON VALUES FOR SILTSTONE TEST INTERVALS AND NORMAL DISTRIBUTION MODEL FIT TO DATA

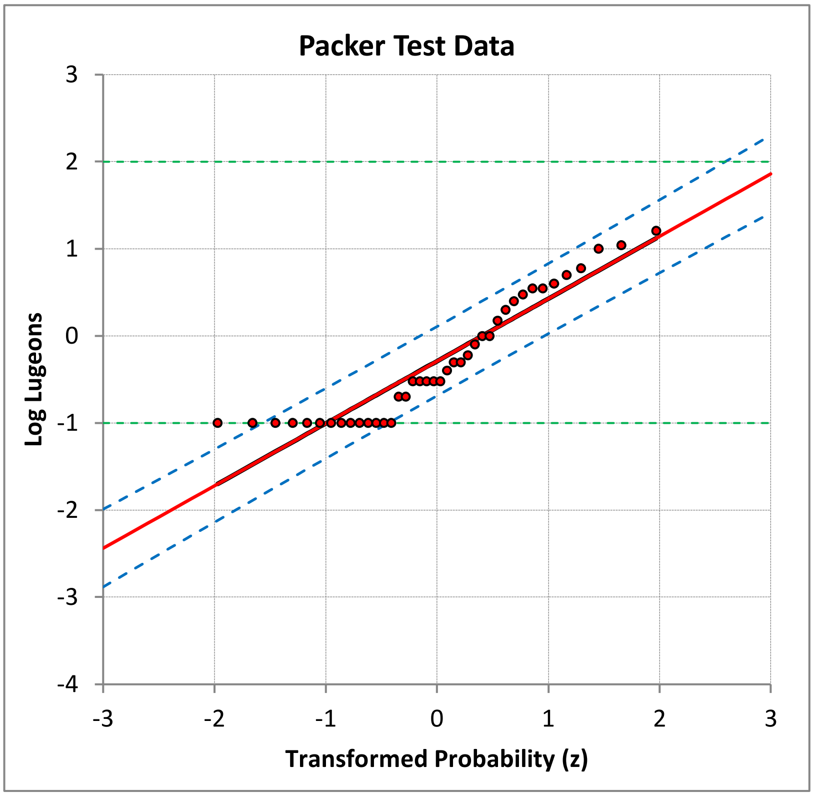


FIGURE 3-5: QUANTILE PLOT OF LUGEON VALUES FOR SILTSTONE TEST INTERVALS

However, this approach tends to potentially overestimate the regional hydraulic conductivity because the high-end values dominate log-normally distributed properties. In addition, packer tests tend to engage a relatively small volume of aquifer, meaning that the test scale is relatively small, and potentially underestimates the regional/bulk hydraulic conductivity of the rock.

Stille (2015) notes that the effective hydraulic conductivity through a three-dimensional volume of blocks can be calculated according to 'Matheron's conjecture' and depends on the geometric mean and the variance of the hydraulic conductivity test data as follows:

$$K_{3D} = e^{\left(\mu + \frac{\sigma^2}{6}\right)}$$

Where K_{3D} is the three-dimensional hydraulic conductivity as noted, μ is the mean, and σ is the standard deviation, of the natural log of the hydraulic conductivity. The K_{3D} value reflects the hydraulic conductivity of a rock volume through which flow occurs, consistent with the conceptual flow regime of groundwater flow into a parallelogram/rhombus-shaped excavations. However, since the K_{3D} value is based on packer tests undertaken at a relatively small scale, it may not reflect the larger-scale (local/regional) hydraulic conductivity of the rock.

Considering this, the 75th percentile value, which is slightly greater than the log-normally distributed arithmetic mean, is considered to represent a relatively conservative representative hydraulic conductivity value; and the K_{3D} value is considered to represent a more likely representative hydraulic conductivity value.

TABLE 3-4: LUGEON AND HYDRAULIC CONDUCTIVITY RESULTS FOR SILTSTONE TEST INTERVALS

Statistic	Siltstone test intervals	
	Lugeon value	Horizontal hydraulic conductivity, *K* (m/d)
Raw data		
Minimum	0.10	8.67×10^{-4}
5th percentile	0.10	8.67×10^{-4}
10th percentile	0.10	8.67×10^{-4}
25th percentile	0.10	8.67×10^{-4}
Median	0.30	2.60×10^{-3}
Harmonic mean	0.22	1.91×10^{-3}
Geomean	0.51	4.45×10^{-3}
Arithmetic mean	1.90	1.65×10^{-2}
75th percentile	2.13	1.84×10^{-2}
90th percentile	5.10	4.42×10^{-2}
95th percentile	10.05	8.71×10^{-2}
Maximum	16.00	1.39×10^{-1}
Log-normally distributed fit		
Arithmetic mean	2.00	1.73×10^{-2}
K_{3D}	0.80	7.00×10^{-3}
N (number of tests)	40	

3.2.5. Mittagong Formation

The Mittagong Formation generally behaves consistent with Hawkesbury Sandstone. For the purposes of the project and assigning hydrogeological properties, because of this reason, the unit being thin, and lying immediately above the Hawkesbury Sandstone; the Mittagong Formation has been lumped with Hawkesbury Sandstone.

3.2.6. Hawkesbury Sandstone

Hawkesbury Sandstone groundwater system hydraulic properties derived from a literature review are summarised in Table 3-5.

Table 3-5: Hawkesbury Sandstone groundwater system properties from CTP project data and literature review

Statistic	Packer testing					Lit					Groundwater models					
	CTP sandstone intervals	WestConnex M4-M5 Link	New M5	Sydney region	WHT and Warringah Freeway Upgrade (land based/water based)	Literature regional range or single value					SS [a]	SS [a]	SS [a]	T [b]	SS [a] /T [b]	T [b]
Horizontal hydraulic conductivity (m/d)																
Minimum	8.67×10^{-4}	8.60×10^{-3}	1.00×10^{-4}		4.00×10^{-6} / 1.40×10^{-4}		1.00×10^{-3}	1.00×10^{-2}					1.00×10^{-3}	1.00×10^{-3}	1.50×10^{-3}	8.64×10^{-4} (deeper zones)
5th percentile	8.67×10^{-4}															
10th percentile	8.67×10^{-4}															
25th percentile	8.67×10^{-4}															
Median	4.33×10^{-3}		3.00×10^{-3}		1.00×10^{-3} / 1.70×10^{-2}										6.00×10^{-3}	
Harmonic mean	2.16×10^{-3}	1.10×10^{-2}														
Geomean	6.03×10^{-3}															
Average	5.65×10^{-2}	9.30×10^{-2}	8.00×10^{-2}	1.00×10^{-1} near surface 2.00×10^{-3} at 50m depth	5.30×10^{-2} / 1.87×10^{-1}										3.02×10^{-2}	
Single value											1.00×10^{-2}	1.00×10^{-2}				8.64×10^{-3} (excludes 'deeper zones'
75th percentile	1.73×10^{-2}															
90th percentile	1.17×10^{-1}															
95th percentile	2.71×10^{-1}															
Maximum	8.67×10^{-1}	1.17×10^{-0}	4.30×10^{0}		2.25×10^{0} / 4.04×10^{0}		1.00×10^{0}	1.00×10^{0}					5.16×10^{-3}	5.00×10^{-2}	1.30×10^{-1}	6.91×10^{-3} (deeper zones)
N (number of tests)	150	196	205	363	300											
K_v/K_h																
Minimum							0.01								0.02	
Single value											1	0.05				0.1
Maximum							0.10								0.50	
Specific storage range (m^{-1})																
Minimum						5.00×10^{-6}	5.00×10^{-6}		1.00×10^{-5}	3.70×10^{-3}					1.00×10^{-6}	
Single value																5.00×10^{-6}
Maximum						1.00×10^{-5}	5.00×10^{-5}		1.00×10^{-4}	1.00×10^{-1} [c]					1.00×10^{-5}	
Specific yield (-)																
Minimum						0.02									0.02	
Single value							0.025									0.01
Maximum						0.05									0.05	
Source	CTP project data	Aecom (2017)	Aecom (2015)	Hewitt (2005)	Jacobs (2020)	Jacobs (2020)	Golder (2016)	McKibbin and Smith (2000)	Hawkes, Ross and Gleeson (2009)	Tammetta and Hewitt (2004)	Golder (2016)	CDM Smith (2016)	GHD (2015)	GHD (2015)	Hydro Simulations (2017)	LSBJV (2020)
Summary																
Parameter	**Minimum value**	**Maximum value**	**Adopted representative value**													

Horizontal hydraulic conductivity (m/d)	4.00×10^{-6}	4.30×10^{0}	1.00×10^{-2}													
Kv/Kh	0.01	1	0.1													
Specific storage (m^{-1})	1.00×10^{-6}	3.70×10^{-3}	1.00×10^{-5}													
Specific yield (-)	0.01	0.05	0.05													

Notes: [a] SS = steady state. [b] T = transient. [c] Value atypically high and not from original reference. Value may be erroneous and has been excluded from summary maximum statistic calculation. Kv/Kh means the ratio of vertical hydraulic conductivity to horizontal hydraulic conductivity

Packer tests have been undertaken for the project and results reviewed based on material type. The results for sandstone packer test intervals are summarised in Table 3-7 and plotted in Figure 3-6.

In Figure 3-6 the Lugeon results are plotted against depth. Additionally, in accordance with Quinones-Rozo (2010), qualitative Lugeon and hydraulic conductivity classification, as well as qualitative rock mass discontinuity classifications, are noted on Figure 3-6. The test interval material type of sandstone is considered to be generally representative of Hawkesbury Sandstone.

Qualitative Lugeon and hydraulic conductivity classification and description of rock mass discontinuities in accordance with Quinones-Rozo (2010) is as follows:

- The 75th percentile value is classified as a very low Lugeon value, with the rock mass characterised as very tight.
- The median, geometric mean and mean value is 0.5 Lugeons, 0.7 Lugeons and 6.5 Lugeons, respectively. The median and geometric mean values are classified as very low Lugeon values, with the rock mass characterised as very tight. The mean value is classified as a moderate Lugeon value, with the rock mass characterised as having 'a few partly open' discontinuities.
- Out of a total of 150 tests, the maximum test result of >100 Lugeons occurred for three tests at SMW_BH502, a single test at SMW_BH717 and a single test at SMW_BH719

The project's packer test results align with those reported in the literature review of hydraulic conductivity values, and indicate that the bulk hydraulic conductivity for Hawkesbury Sandstone is very low. However, hydraulic conductivity can be, and is, elevated locally in some instances. The statistics clearly indicate that the hydraulic conductivity for Hawkesbury Sandstone is higher than that for Ashfield Shale.

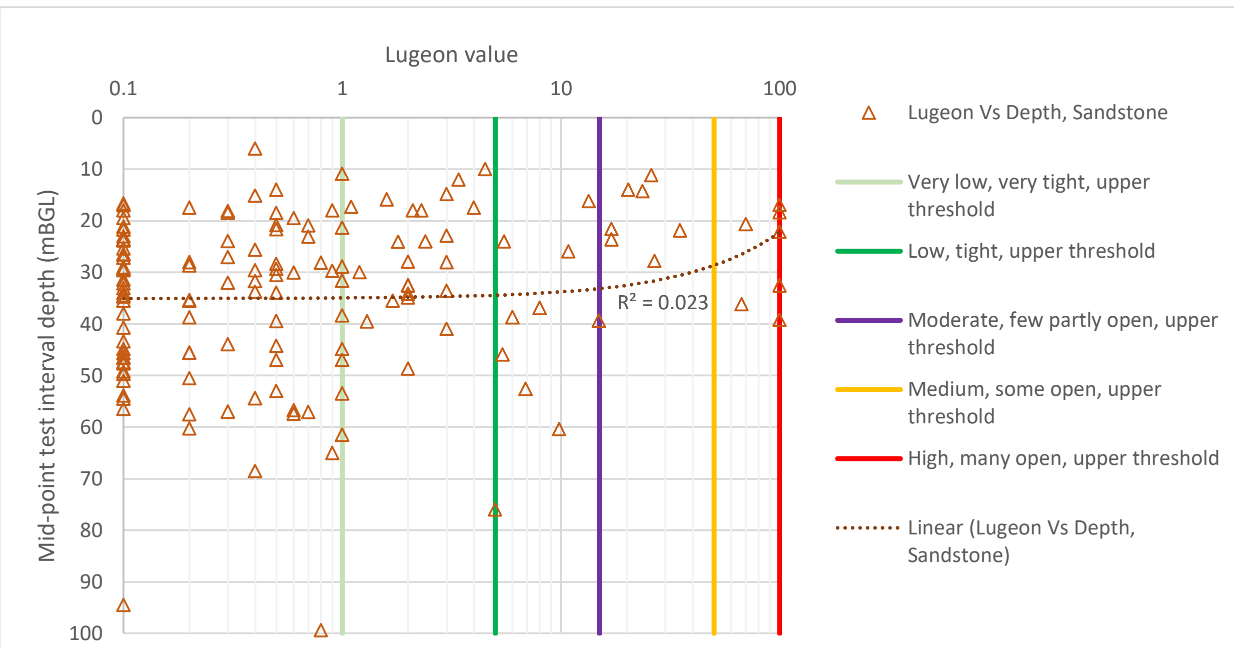


FIGURE 3-6: LUGEON VALUES FOR SANDSTONE TEST INTERVALS, CLASSED ACCORDING TO QUINONES-ROZO (2010)

The relationship between Hawkesbury Sandstone hydraulic conductivity and depth below ground surface has been assessed. The trend lines in Figure 3-6 suggest that hydraulic conductivity decreases with depth. However, the coefficient of determination is low, indicating the relationship is not strong. Table 3-6 shows packer test result statistics (median, geometric mean and arithmetic mean) for sandstone test intervals by depth categories. A box and whisker plot of the sandstone packer test interval results is provided in Figure 3-7:.

Figure 3-6, Table 3-6 and Figure 3-7: indicate the hydraulic conductivity of Hawkesbury Sandstone generally decreases with depth. They also suggest that an initial upper layer may be present and have relatively higher hydraulic conductivity, which could be associated with weathering. Although a trend is established, the decreases are not considered significant for the purpose estimating groundwater inflows and associated impacts.

TABLE 3-6: LUGEON AND HYDRAULIC CONDUCTIVITY STATISTICS FOR SANDSTONE PACKER TEST INTERVALS BY DEPTH

Packer mid-point test interval depth category	N	Lugeon value			Horizontal hydraulic conductivity (m/d)		
		Median	Geometric mean	Arithmetic mean	Median	Geometric mean	Arithmetic mean
0 to <15 m	9	3.4	3.6	9.2	2.95×10^{-2}	3.16×10^{-2}	7.95×10^{-2}
15 to <30 m	64	0.5	0.7	8.3	4.33×10^{-3}	6.50×10^{-3}	7.22×10^{-2}
30 to <45 m	41	0.5	0.8	7.9	4.33×10^{-3}	6.51×10^{-3}	6.82×10^{-2}
45 to <60 m	27	0.2	0.3	0.8	1.73×10^{-3}	2.62×10^{-3}	7.00×10^{-3}
60 to 105.9 m (max)	9	0.8	0.7	2.0	6.93×10^{-3}	5.83×10^{-3}	1.76×10^{-2}

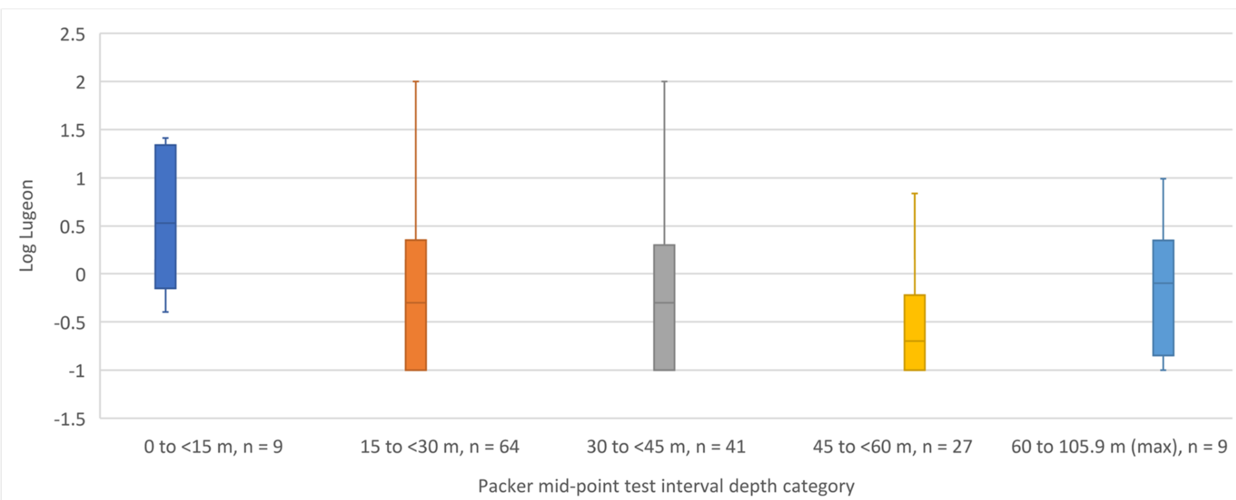


FIGURE 3-7: LOG LUGEON VALUES FOR SANDSTONE TEST INTERVALS BY DEPTH CATEGORY

As noted in Section 3.2.4, it is well established that hydraulic conductivity test values are log-normally distributed. Figure 3-8 shows the cumulative distribution for the tests in sandstone. The following discussion mirrors the discussion of log-normally distributed hydraulic conductivity values in Section 3.2.4, but for the sandstone.

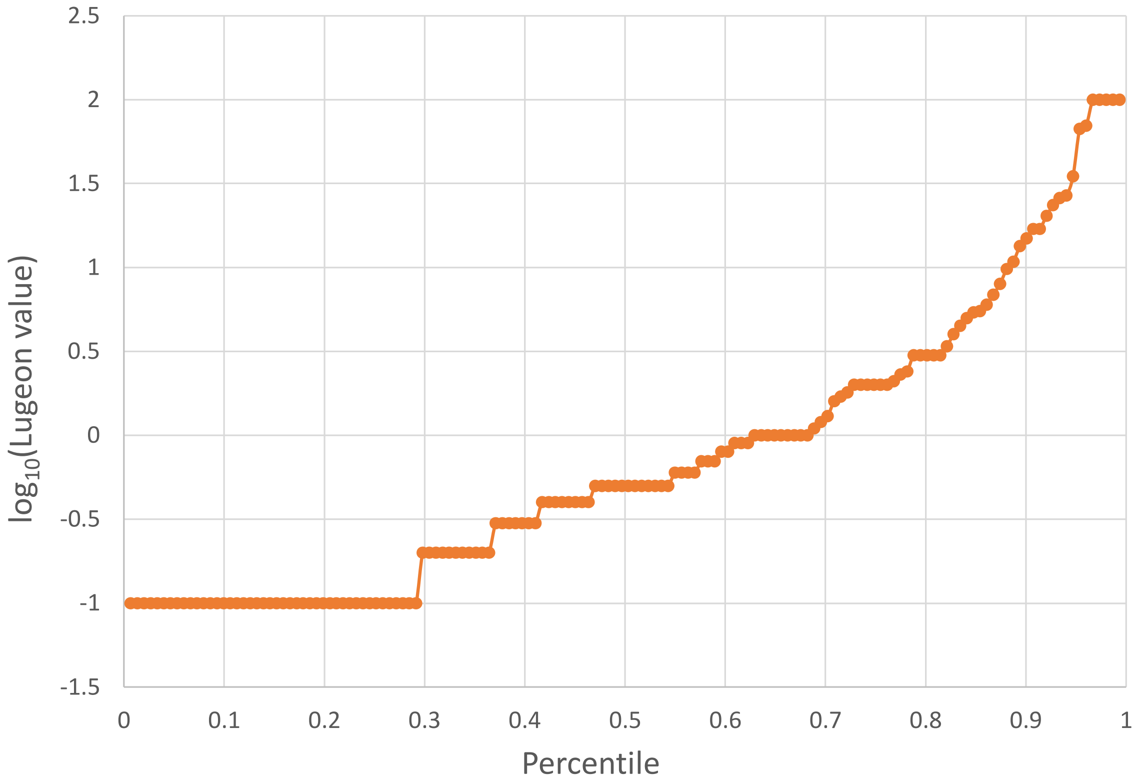


FIGURE 3-8: CUMULATIVE DISTRIBUTION OF LUGEON VALUES FOR SANDSTONE TEST INTERVALS

Figure 3-9 shows the same cumulative distribution as in Figure 3-8, along with a normal distribution model fitted to the data. The model considers a 90% confidence interval and that the limits of measurement of the packer tests are 0.1 Lugeons and 100 Lugeons. Figure 3-10 shows a quantile plot for the Lugeon data and the model. The resulting mean value from the model is 2 Lugeons. This result is also shown in Table 3-7.

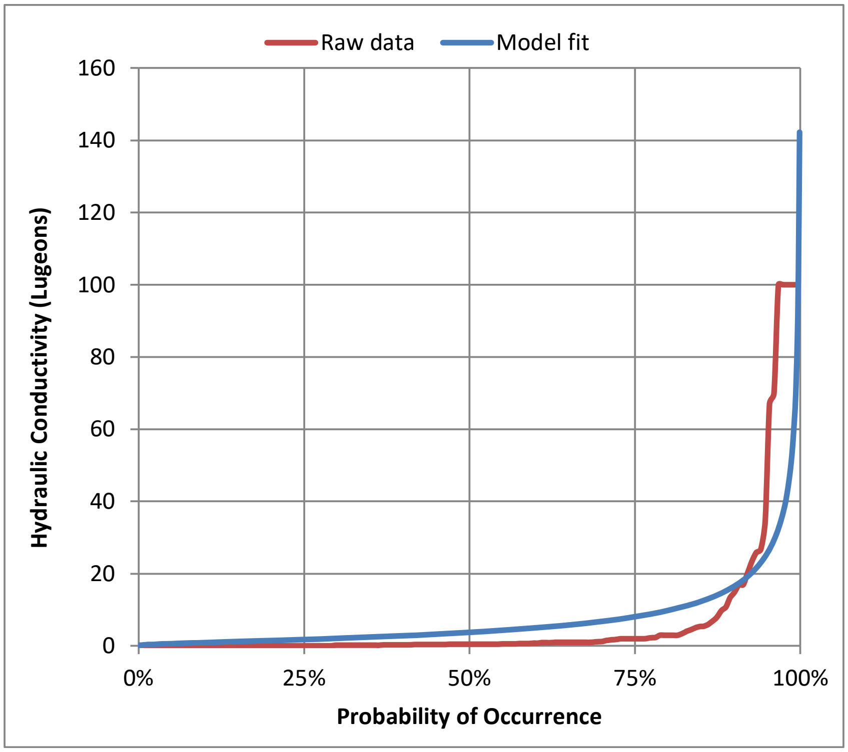


FIGURE 3-9: CUMULATIVE DISTRIBUTION OF LUGEON VALUES FOR SILTSTONE TEST INTERVALS AND NORMAL DISTRIBUTION MODEL FIT TO DATA

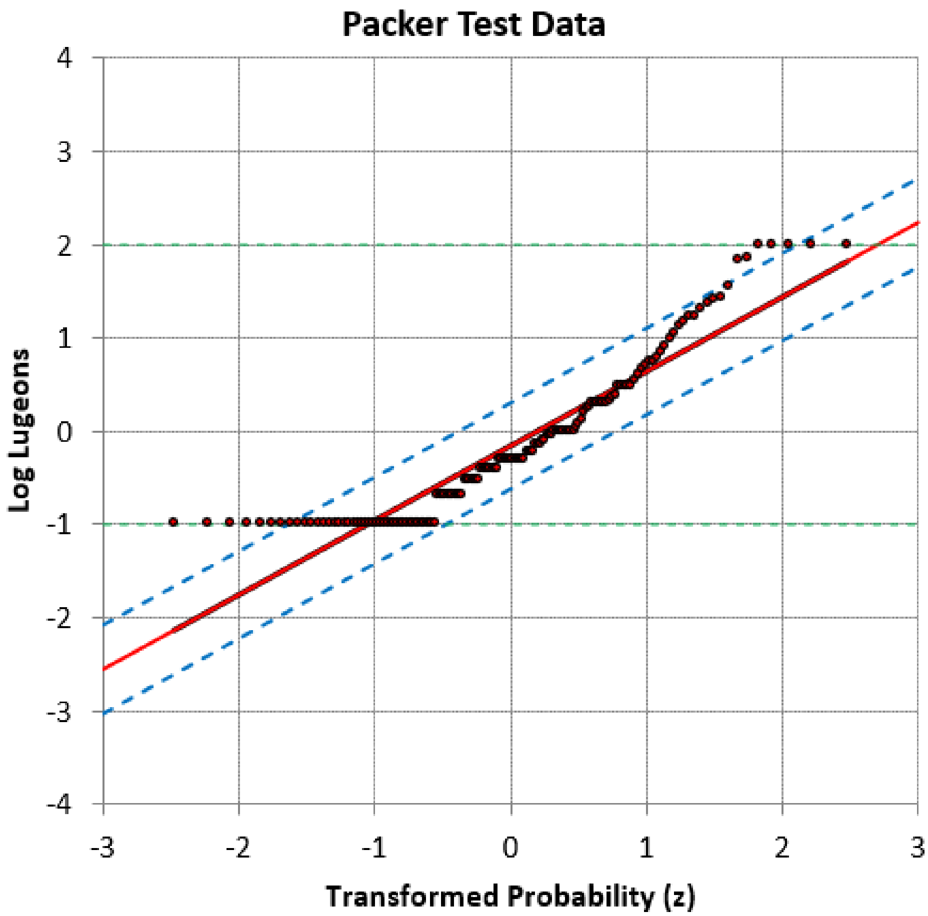


FIGURE 3-10: QUANTILE PLOT OF LUGEON VALUES FOR SILTSTONE TEST INTERVALS

Again, this approach tends to potentially overestimate the regional hydraulic conductivity because the high-end values dominate log-normally distributed properties. Table 3-7 lists the K_{3D} value.

!@#Considering this, the 75th percentile value, which is slightly greater than the log-normally distributed arithmetic mean, is considered to represent a relatively conservative representative hydraulic conductivity value; and the K_{3D} value is considered to represent a more likely representative hydraulic conductivity value.

TABLE 3-7: LUGEON AND HYDRAULIC CONDUCTIVITY RESULTS FOR SANDSTONE TEST INTERVALS

Statistic	Sandstone test intervals	
	Lugeon	Horizontal hydraulic conductivity (m/d)
Minimum	0.10	8.67×10^{-4}
5th percentile	0.10	8.67×10^{-4}
10th percentile	0.10	8.67×10^{-4}
25th percentile	0.10	8.67×10^{-4}
Median	0.50	4.33×10^{-3}
Harmonic mean	0.25	2.16×10^{-3}
Geomean	0.70	6.03×10^{-3}
Average	6.52	5.65×10^{-2}
75th percentile	2.00	1.73×10^{-2}
90th percentile	13.55	1.17×10^{-1}
95th percentile	31.31	2.71×10^{-1}
Maximum	100.00	8.67×10^{-1}
Log-normally distributed fit		
Arithmetic mean	6.50	5.62×10^{-2}
K_{3D}	1.20	1.06×10^{-2}
N (number of tests)	150	

3.2.1. Dykes and Faults

Dykes and fault zones may exhibit enhanced permeability. These are reviewed on a case by case basis for each relevant CTP project works location.

4. Adopted representative hydrogeological parameter values

Based on the review of hydrogeological testing results and properties documented in Section 3, a summary of hydrogeological parameter values for pertinent CTP project hydrogeological units, as well as the representative parameter values adopted in the groundwater modelling, is provided in Table 4-1.

TABLE 4-1: SUMMARY OF HYDROGEOLOGICAL PARAMETER VALUES FOR PROJECT HYDROGEOLOGICAL UNITS, AND ADOPTED REPRESENTATIVE VALUES

Hydrogeological unit	Typical Horizontal hydraulic conductivity range (m/day)	K_v/K_h range	Specific storage range (m^{-1})	Specific yield range (-)
Typical range				
Quaternary alluvium	5.00×10^{-3} to 1.00×10^{0}	0.1 to 0.5	1.00×10^{-5}	0.06 to 0.20
Ashfield Shale	4.45×10^{-3} to 1.84×10^{-2} (0.5 to 2.1 Lugeons) (geomean to 75th percentile) (Log-normally distributed arithmetic mean is 1.72×10^{-2} = 2.0 Lugeons; K_{3D} value is 7.00×10^{-3} m/d = 0.8 Lugeons)	0.1 to 1.0	5.00×10^{-6} to 1.00×10^{-5}	0.01 to 0.025
Mittagong Formation and Hawkesbury Sandstone	6.03×10^{-3} to 5.62×10^{-2} (0.7 to 2.0 Lugeons) (geomean to 75th percentile) (Log-normally distributed arithmetic mean is 5.62×10^{-2} m/d = 6.5 Lugeons; K_{3D} value is 1.06×10^{-2} m/d = 1.2 Lugeons)	0.01 to 1	1.00×10^{-6} to 1.00×10^{-5}	0.02 to 0.05
Adopted representative value				
Quaternary alluvium	1.00×10^{0} (predominantly sandy) 5.00×10^{-3} (predominantly clayey)	0.1	1.00×10^{-5}	0.20 (predominantly sandy) 0.06 (predominantly clayey)
Ashfield Shale	Conservative: 1.84×10^{-2} (2.1 Lugeons; 75th percentile) Likely: 7.00×10^{-3} m/d (0.8 Lugeons; K_{3D} value)	0.1	5.00×10^{-6}	0.02
Mittagong Formation and Hawkesbury Sandstone	Conservative: 1.72×10^{-2} (2.0 Lugeons; 75th percentile) Likely: 1.06×10^{-2} m/d (1.2 Lugeons; K_{3D} value)	0.1	5.00×10^{-6}	0.05

Note: K_v/K_h is the ratio of vertical to horizontal hydraulic conductivity.

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ANNEXURE C. DESIGN GROUNDWATER LOADS FOR STATION SOIL RETAINING WALLS – ACCIDENTAL LOAD CASES

Technical Memo

To	Fernando Martinez Ceballos Eduardo Torralba Marsutha Somasundaram	Date	23 February 2022
Copies	Fernando Lopez Asensio Dan Worrall Jiun Dar Ong	Document ID	SMWSTCTP-AFJ-SWD-SN250-ST-RPT-003020 Appendix G Annexure C
From	Ben Rotter	Revision	C
Subject	Design groundwater loads for station soil retaining walls – accidental load cases – burst water main and blocked drainage		

1. Introduction

This memorandum provides hydrogeological advice in support of the accidental load scenarios for geotechnical and structural design of the station retaining walls for the Sydney Metro West – Central Tunnel Package works.

2. Particular Specifications

The Sydney Metro West – Central Tunnel Package Particular Specification Requirements (V7.0) state the following requirements in relation to design groundwater loads for civil and structural design:

> **4.1 Civil and Structural**
>
> **4.1.3 Design Loading**
>
> **4.1.3.1 General**
>
> (d) The Tunnelling Contractor must design all civil and structural works to accommodate the potential impact of groundwater levels and hydrostatic pressures of floodwater plains or a burst water main where existing or new water utilities are within proximity to the Project Works and Temporary Works. [SM-W-CTP-PS-709]
>
> (i) The Tunnelling Contractor must not allow for any reduction in hydrostatic loadings due to localised lowering of groundwater levels in the design of the Works. The reduction of hydrostatic loading due to localised lowering of groundwater levels is permitted in the design of the support of Station Excavations and Station Shaft Excavations that are drained in accordance with the requirements in Section 4.1.7(a). [SM-W-CTP-PS-715]
>
> **4.1.8 Groundwater Seepage**
>
> (b) The Tunnelling Contractor must design for the risk of water pressure build-up as a result of blocked drainage. [SM-W-CTP-PS-1030]

3. Design groundwater load conditions

3.1. CTP project works conditions

The Bays Station excavation is undrained above the soil retention system toe level [Particular Specification SM-W-CTP-PS-1022]. Design groundwater levels for The Bays Station are provided in Section 4.4. of Appendix G of The Bays Retaining Walls Stage 3 Design Report (document number SMWSTCTP-AFJ-TBY-SN200-ST-RPT-003000 Appendix-G[D]| REV1).

The Five Dock Station, Burwood North Station, North Strathfield Station and Sydney Olympic Park Station excavations will be drained. Groundwater levels surrounding the excavation will decline as excavation progresses. Over the long-term, groundwater levels immediately surrounding the excavation will be close to the excavation floor level (or the deepest passive dewatering level). For the permanent (10 year design life) condition, it can therefore be assumed that there is no hydrostatic pressure on the retaining walls.

Design can exploit this, as Particular Specification SM-W-CTP-PS-715 allows for design to consider a reduction of hydrostatic loading due to localised lowering of groundwater levels for drained station and shaft excavations.

3.2. CTP project works exceptional conditions

Design is required to consider groundwater levels in response to burst water mains and blocked drainage (Particular Specification SM-W-CTP-PS-709 and SM-W-CTP-PS-1030).

See the relevant Structural and Geotechnical Design Reports for the design load conditions associated with flooding.

4. Exceptional load condition: burst water mains

It is possible that a burst water main could saturate the soils adjacent to station retaining walls, imposing hydrostatic load on the retaining wall.

The soils present at the station sites comprise fill and residual soils derived from Ashfield Shale. The residual soils derived from Ashfield Shale are typically clayey in nature, and have relatively low permeability. Given the relatively short duration (less than one day) of a burst water main released water into the soils, it is expected that the water released would saturate the fill of the trench within which the burst water main lies, but would not saturate the underlying soils.

A conservative assumption from a design load perspective is to assume that the fill material is of relatively high permeability (e.g., is sandy/gravelly in nature) and lies immediately adjacent to the retaining wall.

The burst water main would then saturate the soils.

Two scenarios have been considered:

1. The entire fill material to ground surface is saturated. This is illustrated in Figure 1
2. The fill material below the pipe invert level is saturated. This is illustrated in Figure 2

Note that these scenarios are provide an unrealistically conservative pressure profile, which assumes that the retaining wall drainage system is not working and that the fill is highly permeable. In practice, the retaining wall drainage system will (at least partially) drain the fill, and lower permeability soils would take time to saturate resulting in only partial saturation of the fill. The actual pressure experienced by the wall would therefore not be as high as shown in Figure 1 or Figure 2. It is therefore reasonable to consider a lower pressure than that shown in Figure 1 or Figure 2 in design.

See the relevant Structural and Geotechnical Design Reports for the specific conditions, and adopted loads, at each station site.

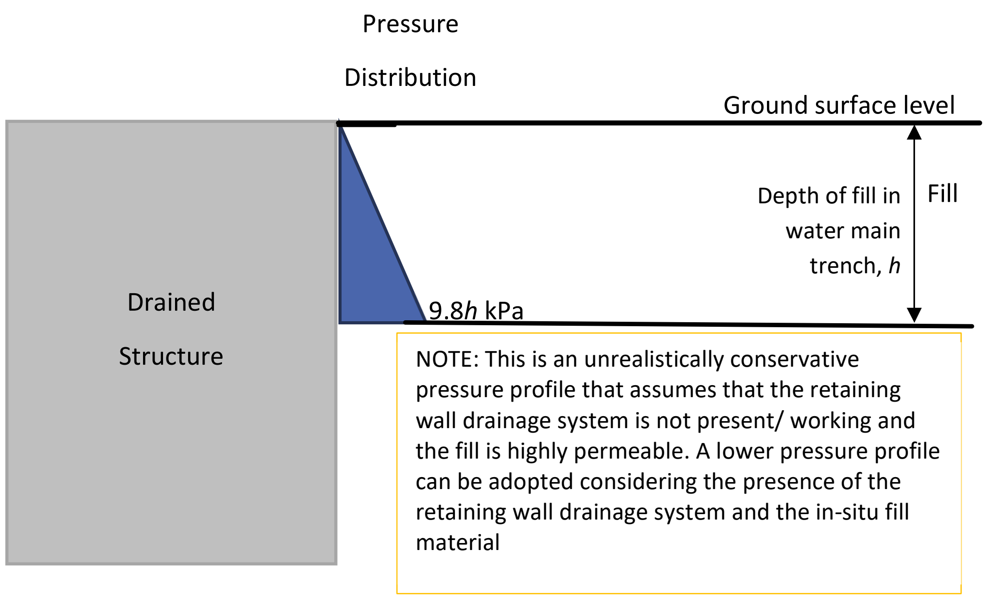

FIGURE 1: EXCEPTIONAL GROUNDWATER PRESSURE CONDITION FOR BURST WATER MAIN

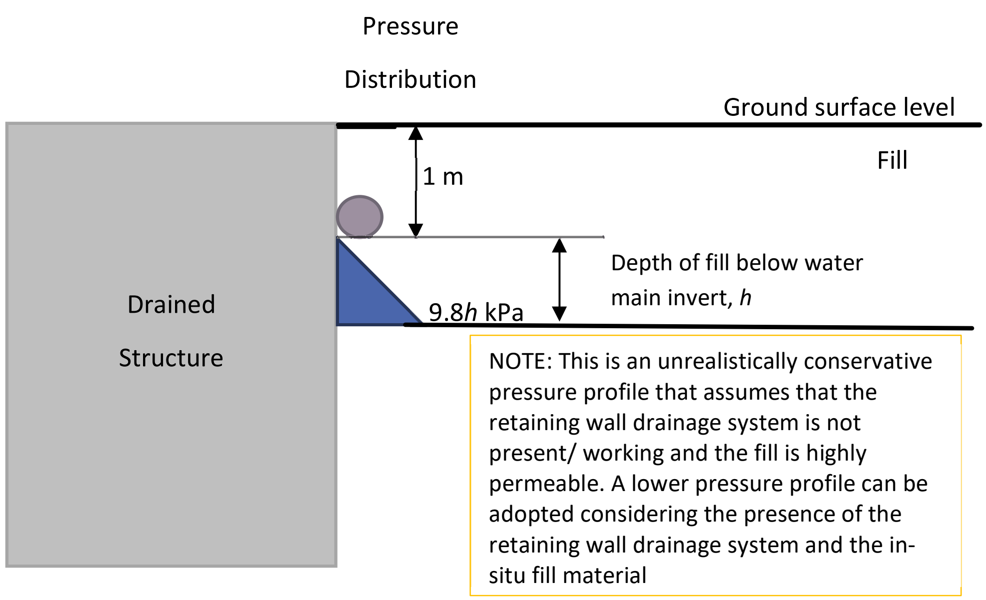

FIGURE 2: EXCEPTIONAL GROUNDWATER PRESSURE CONDITION FOR BURST WATER MAIN CONSIDERING PIPE INVERT LEVEL

5. Exceptional load condition: flood

It is possible that a flood could saturate the soils adjacent to station retaining walls, imposing hydrostatic load on the retaining wall.

Consistent with the approach for burst water mains (see Section 4), a conservative assumption from a design load perspective is to assume that the fill material is of relatively high permeability and lies immediately adjacent to the retaining wall. This fill becomes fully saturated during a Probable Maximum Flood (PMF) event and the pressure distribution on the retaining wall is therefore as shown in Figure 3.

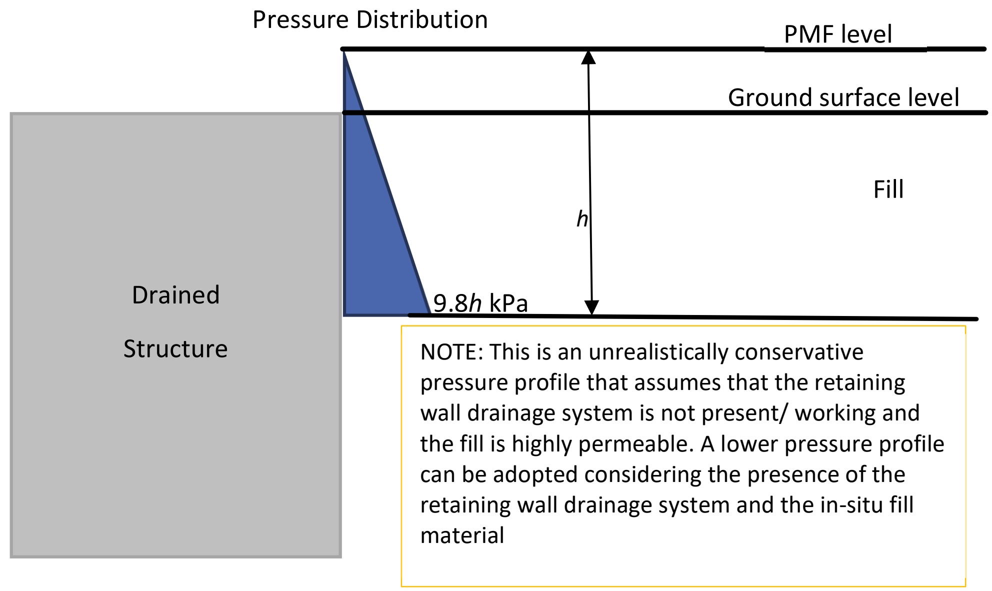

FIGURE 3: EXCEPTIONAL GROUNDWATER PRESSURE CONDITION FOR FLOOD SCENARIO

6. Exceptional load condition: blocked drainage

A general load condition is adopted to represent a blocked drainage scenario for the retaining walls at Five Dock Station, Burwood North Station, North Strathfield Station and Sydney Olympic Park Station.

This section describes the development of the general load condition.

6.1. Retaining wall design

The retaining walls at these stations typically comprise a solider pile wall with alternating piles of two 750 mm-diameter short piles spaced at 1.8 m centres and 750 mm-diameter long piles spaced at 5.4 m centres. Shotcrete is applied across the soil/rock between the piles. Vertical strip drains are centred between every pile couple. The layout is illustrated in Figure 4.

For the purposes of general representation, a particular piled wall layout has been adopted that considers the short piles to be 11 m deep (and the long piles to extend 1 m below the floor of the excavation). This represents a conservative scenario, where both pile types are deeper and therefore reduce the potential release of groundwater pressure behind the piled wall by reducing the opportunity for groundwater to flow between the piles to the face of the excavation.

6.1. Approach to developing load condition

The approach adopts conditions that are conservative with regard to inducing higher water pressures on the retaining wall, including:

- Consideration of the deepest excavation (30 m deep), to reflect a scenario where groundwater would be blocked across a tall drainage system (greatest retaining wall height)
- Consideration of a shallower excavation (20 m deep), for which the groundwater heads that drive groundwater flow would be lower, and therefore pressure release behind the wall is slower

- The retained soils and rock have a relatively low permeability. This is conservative because it allows for a greater build-up of pressure behind the wall

6.2. Modelling approach

Two-dimensional numerical models were developed in the GeoStudio software package SEEP/W to estimate the potential groundwater pressure on the retaining walls. The modelling approach considered the following:

- Transient groundwater flow analysis
- A two-dimensional cross section through the wall is modelled
- An initial condition in which the excavation is at the finished floor level, and the groundwater system is at approximately steady state, with the groundwater table drawndown to excavation level at the retaining wall
- Seepage occurs through excavation wall and floor
- The retaining wall has an equivalent net permeability, considering the presence of concrete piles and rock
- The equivalent length of retaining wall that is modelled by this equivalent net permeability is shown in Figure 4
- An extreme rainfall event occurs, causing infiltration of water into the groundwater system. Groundwater flow is modelled during the rainfall event, and the groundwater pressure experienced at the rear of the retaining wall is modelled
- A blocked drain is represented by reduced equivalent net permeability of the retaining wall during the rainfall event. It is assumed that no seepage occurs through the zone between two adjacent piles (at 1.8 m spacing) along the entire depth of the piled wall, i.e., no seepage occurs through the blocked zone as shown Figure 4

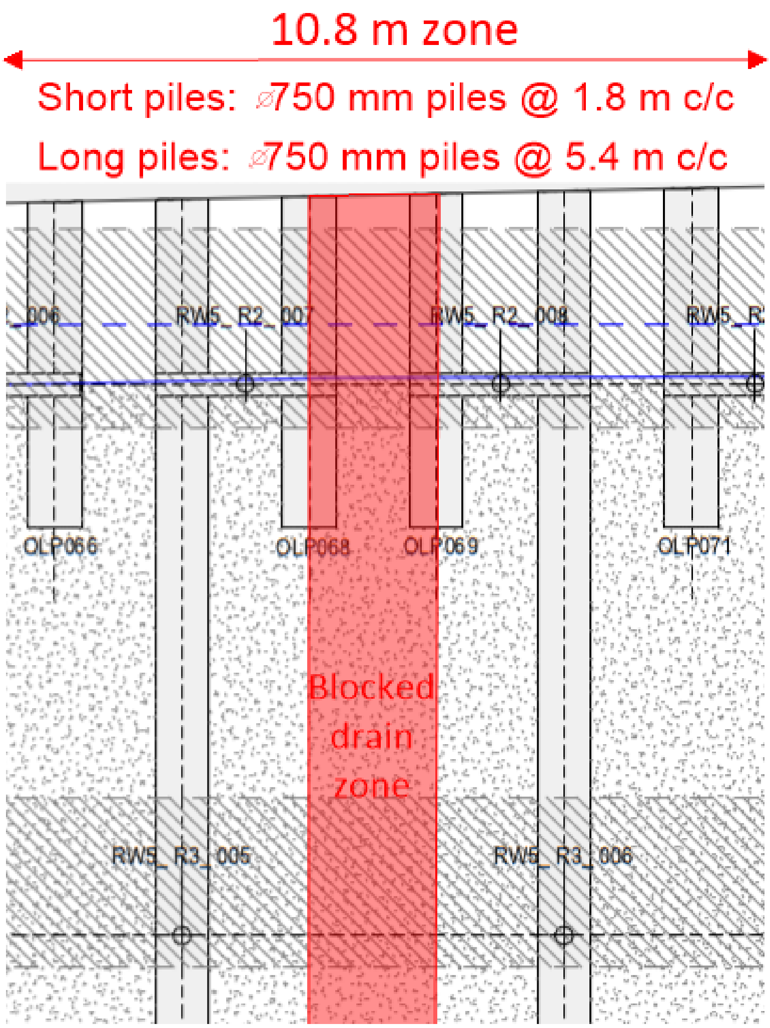

FIGURE 4: TYPICAL PILE LAYOUT AND BLOCKED DRAINAGE ZONE

6.3. Model parameter values

Adopted hydrogeological parameter values are provided in Table 1.

Two extreme rainfall events were considered based on the Bureau of Meterology's Design Rainfall Data System (2016) (http://www.bom.gov.au/water/designRainfalls/revised-ifd/):

- 1 day-duration, 1% AEP event (284 mm)
- 7 day-duration, 1% AEP event (482 mm)

A rainfall recharge rate of 2% was adopted. These conditions result in infiltration that is greater than the modelled ground can receive. Therefore, a constant head boundary conditions was applied in the model at ground surface level to replicate extreme rainfall.

The model domain is shown in Figure 5 and an example model output (showing pore water pressure in kPa) is shown in Figure 6.

TABLE 1 ADOPTED HYDROGEOLOGICAL PARAMETER VALUES

Material	Horizontal hydraulic conductivity (m/d)	Ratio of vertical to horizontal hydraulic conductivity (-)	Specific storage (m^{-1})	Specific yield (-)
Soil/rock	2.6×10^{-3} (0.3 Lugeons)*	0.1	5×10^{-6}	0.02
Concrete	8.6×10^{-8}	0.1	N/A	0.01
Short piles in free seepage zone	1.5×10^{-3}	0.1	5×10^{-6}	0.016
Long piles in free seepage zone	2.2×10^{-3}	0.1	5×10^{-6}	0.019
Short piles in blocked drained zone	1.1×10^{-3}	0.1	5×10^{-6}	0.014
Long piles in blocked drained zone	1.8×10^{-3}	0.1	5×10^{-6}	0.017

*This is the median value of all packer test results within Ashfield Shale available outside of The Bays Station site

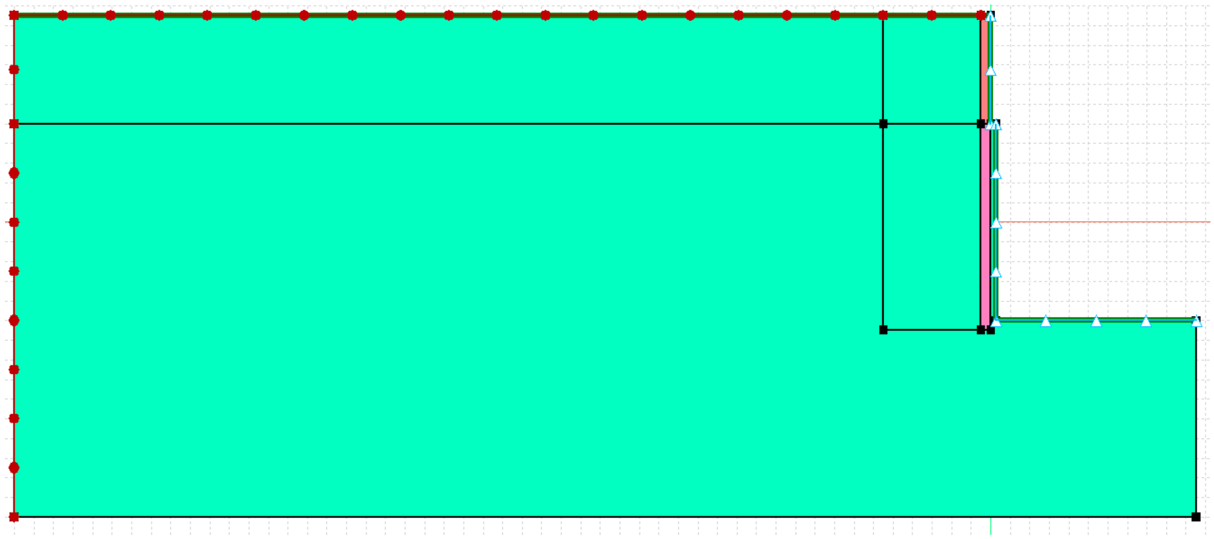

FIGURE 5: MODEL DOMAIN

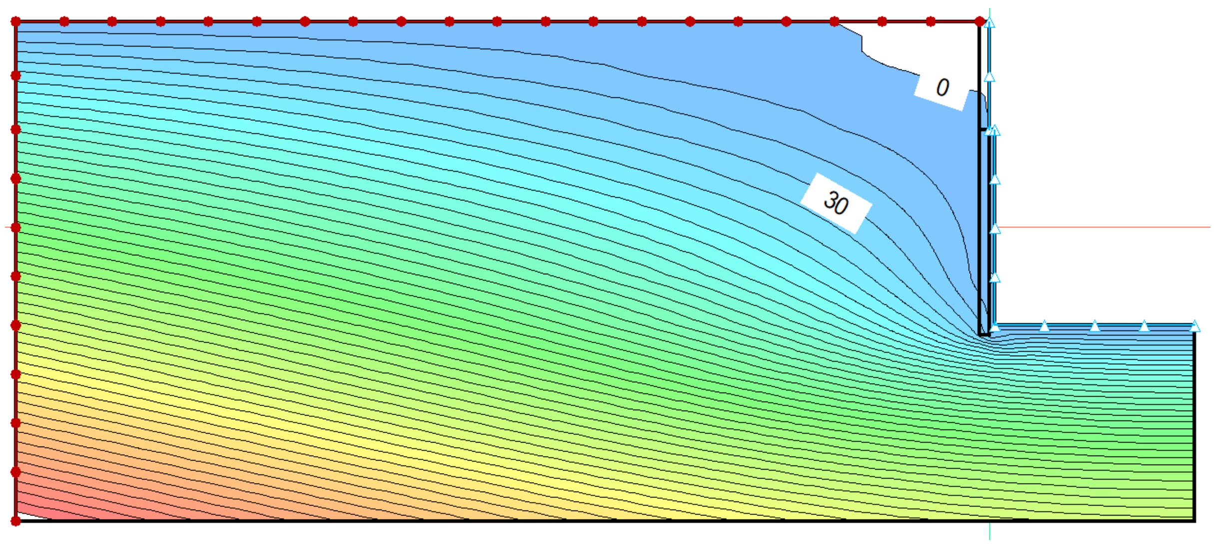

FIGURE 6: EXAMPLE MODEL OUTPUT

6.1. Modelling results

Figure 7 and Figure 8 summarise the key modelling results for the one day and seven day-duration rainfall events for the shallow and deep excavations.

The predicted groundwater pressures on the rear of the piled wall that retains soil/shallow rock are less than 5 kPa. Pressures across the deeper horizon, in the rock, are not discussed here, as the focus of this advice is on the soil retaining wall.

Figure 9 shows a simplified pressure profile for the soil retaining wall.

Because the modelling is two-dimensional, the results shown in Figure 9 reflect the averaged pressures on a representative length of wall (which is averaged in the two-dimensional model in the direction of the wall). In practice, these pressures would be experienced at the blocked drain itself, and would reduce laterally due to operating drains either side of the blocked drain. This means that the maximum equivalent pressure experienced by a pile located either side of the blocked drainage zone would be for the closest spaced piles (1.8 m centres) as shown in Figure 10.

Based on this, the pressure experienced by a pile adjacent to the blocked drainage zone is shown in Figure 11.

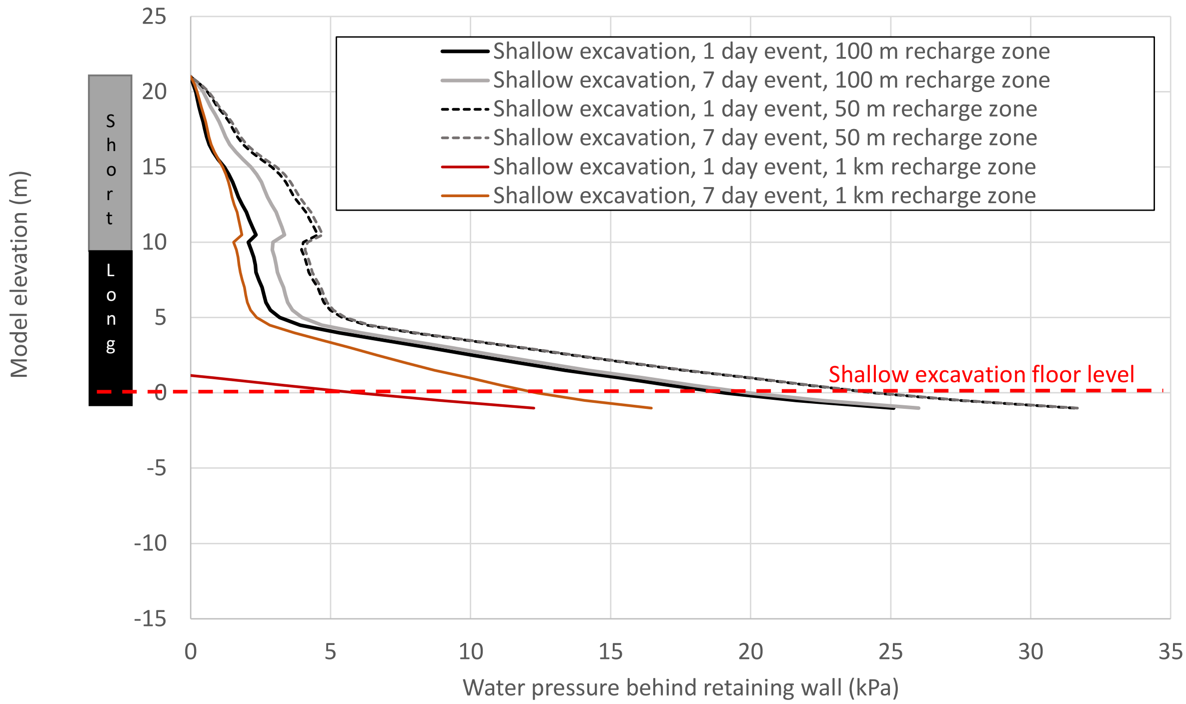

FIGURE 7: MODEL RESULTS – GROUNDWATER PRESSURE PROFILE ALONG PILED WALL – SHALLOW EXCAVATION

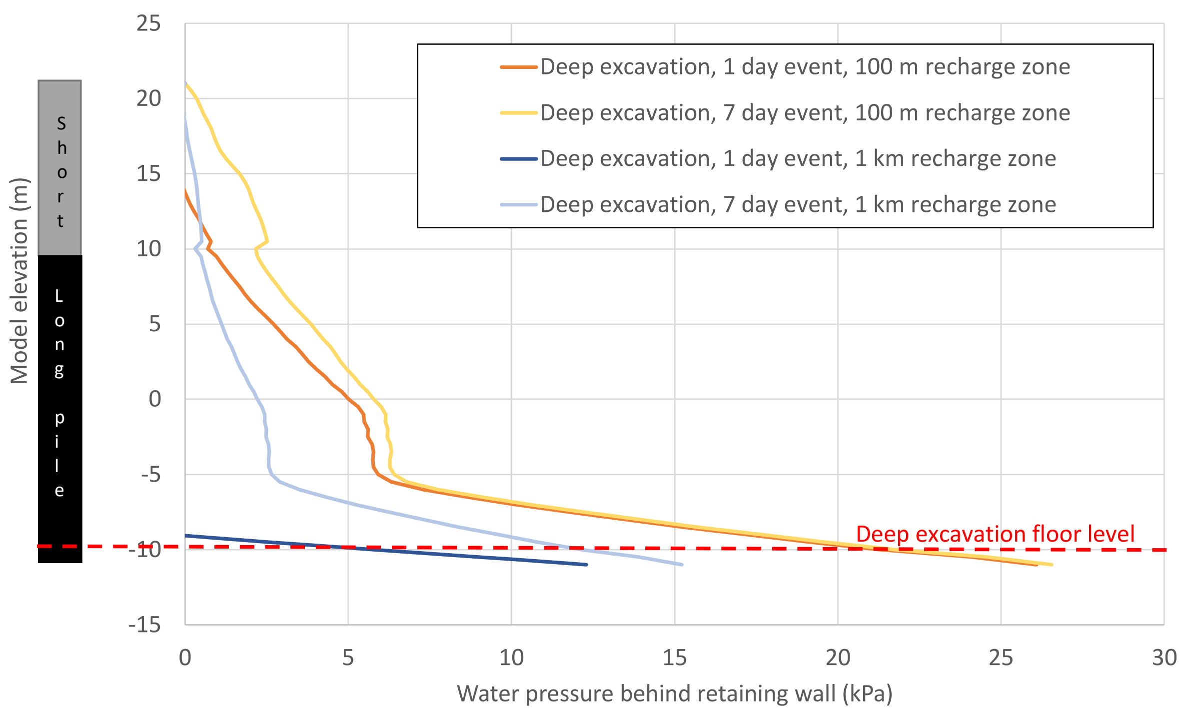

FIGURE 8: MODEL RESULTS – GROUNDWATER PRESSURE PROFILE ALONG PILED WALL – DEEP EXCAVATION

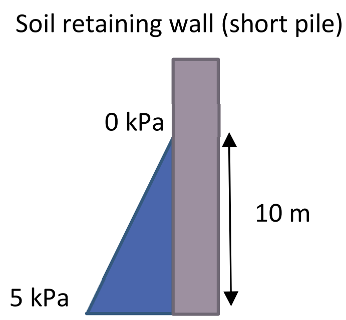

FIGURE 9: PRESSURE PROFILE DIAGRAM BASED ON MODEL RESULTS

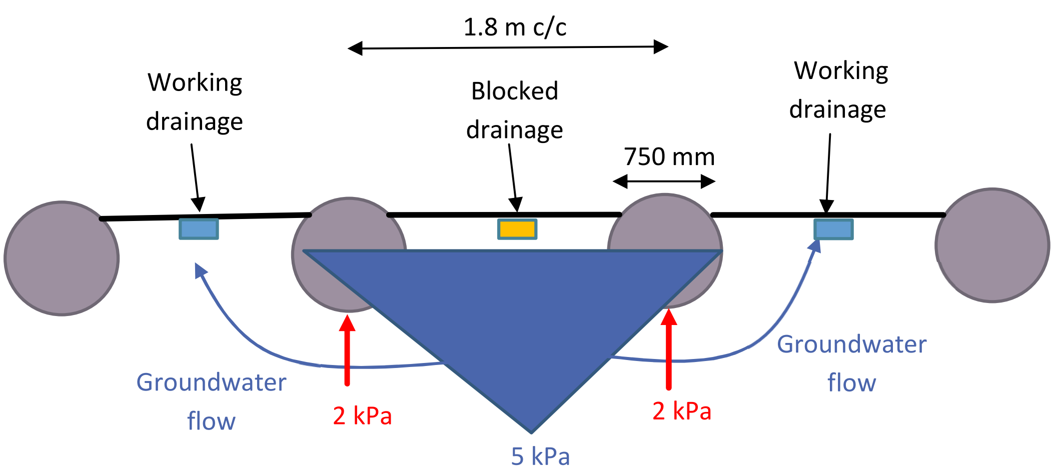

FIGURE 10: PRESSURE PROFILE DIAGRAM (IN PLAN VIEW)

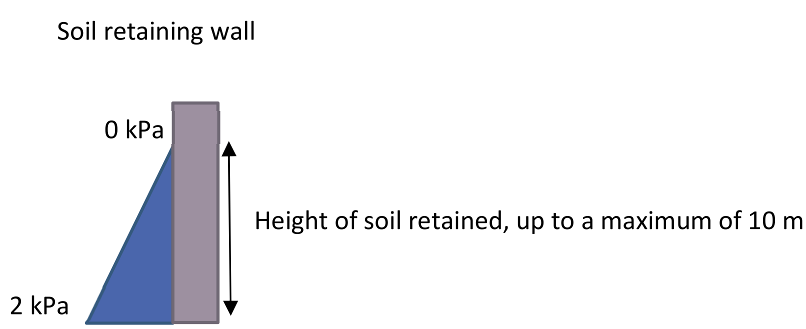

FIGURE 11: PRESSURE PROFILE TO ADOPT IN DESIGN OF SOIL RETAINING WALLS FOR EXCEPTIONAL LOAD CONDITION (GROUNDWATER) REPRESENTING BLOCKED DRAINAGE

ANNEXURE D. GROUNDWATER MODELLING

Jacobs TYPSA
Jacobs Typsa Joint Venture

Technical Memo

To	Dan Worrall	Date 2 March 2022
Copies		Document ID SMWSTCTP-AFJ-FDK-SN250-ST-RPT-003020 Appendix G Annexure D
From	Ben Rose	Revision A
Subject	Five Dock Station Groundwater Modelling – Stage 3 – Annexure D	

1. Introduction

The objective of this memorandum is to summarise groundwater modelling undertaken in support of the Stage 3 Five Dock Station design.

The scope of this document is limited to:

- Reporting of the groundwater modelling method.
- Reporting of modelled groundwater inflow rates and associated groundwater level drawdown.

Potential implications associated with the model results and evaluation of the results is not covered in this memorandum and is instead covered in the main respective Stage 3 Five Dock Station hydrogeological assessment report.

2. Groundwater modelling

2.1. Model objectives

A numerical groundwater flow model (GFM) has been developed in support of the Stage 3 Five Dock Station design. The modelling objectives were to:

- Predict groundwater inflow rates to Five Dock Station excavations.
- Predict associated propagation of groundwater level drawdown.

2.2. Adopted model type and program

The GFM has been developed in the Geostudio software package, SEEP/W (v2019). SEEP/W is a finite element modelling package for modelling groundwater flow in porous media.

A 2D cross section style model(s) was developed.

2.3. Modelling method summary

A 2D cross section model was developed approximately south to north through Five Dock Station and extended to appropriate boundaries. The model was calibrated to existing representative groundwater levels at Five Dock Station in steady state by adjusting the recharge rate. Upon achieving suitable calibration, a transient model was developed, which incorporated a boundary condition to simulate groundwater drainage associated with the station excavations. This boundary condition enabled prediction of groundwater inflow rates into the station excavations and also calculation of groundwater level drawdown (by comparison to existing groundwater level conditions as assessed by the steady state calibration model).

Shaft and cavern inflow rates were estimated in separate versions of the model, which represented the shaft and cavern geometries, respectively. Drawdown was conservatively calculated using the version of the model which included the shaft. As the shaft is a relatively large excavation, this provides a relatively conservative assessment of drawdown.

The cross section model was established to be 1 m thick. Thus, groundwater inflow rates were calculated by multiplying shaft and cavern lengths with the relevant modelled groundwater inflow rates.

To account for potential groundwater inflows to the eastern and western walls of the station caverns, a multiplier of 1.1 was applied to the net inflow to the cavern. This multiplier was adopted based on past experience with similar projects.

2.4. Model set up

2.4.1. Model cross section

The location of the cross section represented in the SEEP/W model(s) is shown in Figure 1. The cross section extends from Iron Cove Creek in the far south to near the Parramatta River in the far north and is approximately 1.3 km long. This cross section was selected to provide reasonable representation of distant boundary conditions.

At the station site, the ground profiles reported in the Geotechnical Interpretive Report were considered, with particular focus on the conditions relevant to the station shafts and caverns as shown conceptually in Figure 2.

FIGURE 1 FIVE DOCK STATION SEEP/W CROSS SECTION LOCATION

FIGURE 2 GROUND PROFILES CONSIDERED IN FIVE DOCK STATION SEEP/W CROSS SECTION MODEL (SECTION LINES IN RED)

2.4.2. Model layers

Two hydrogeological units are represented in the model: Ashfield Shale and Hawkesbury Sandstone. Fill and residual soil units are not included in the model because the water table is situated below these units at the station. The Mittagong Formation is not explicitly represented in the model and is instead represented by the Hawkesbury Sandstone unit. This approach was adopted because the Mittagong Formation is thin (e.g. 2 m to 5 m thick) and conceptualised to be characteristically similar to the Hawkesbury Sandstone.

The Ashfield Shale layer is represented from ground surface level to a uniform depth of 6 m along the entire section and is based on the depth to the Ashfield Shale/Mittagong Formation interface at the approximate centre of the station. The Hawkesbury Sandstone/ Mittagong Formation layer occurs beneath the Ashfield Shale layer and its base is represented at a level of -100 mAHD. This base level is 87.5 m below the base of the station excavation (-12.5 mAHD) and therefore provides sufficient model thickness to enable interaction of the station excavations with the underlying groundwater system.

The model layers are shown in Figure 3.

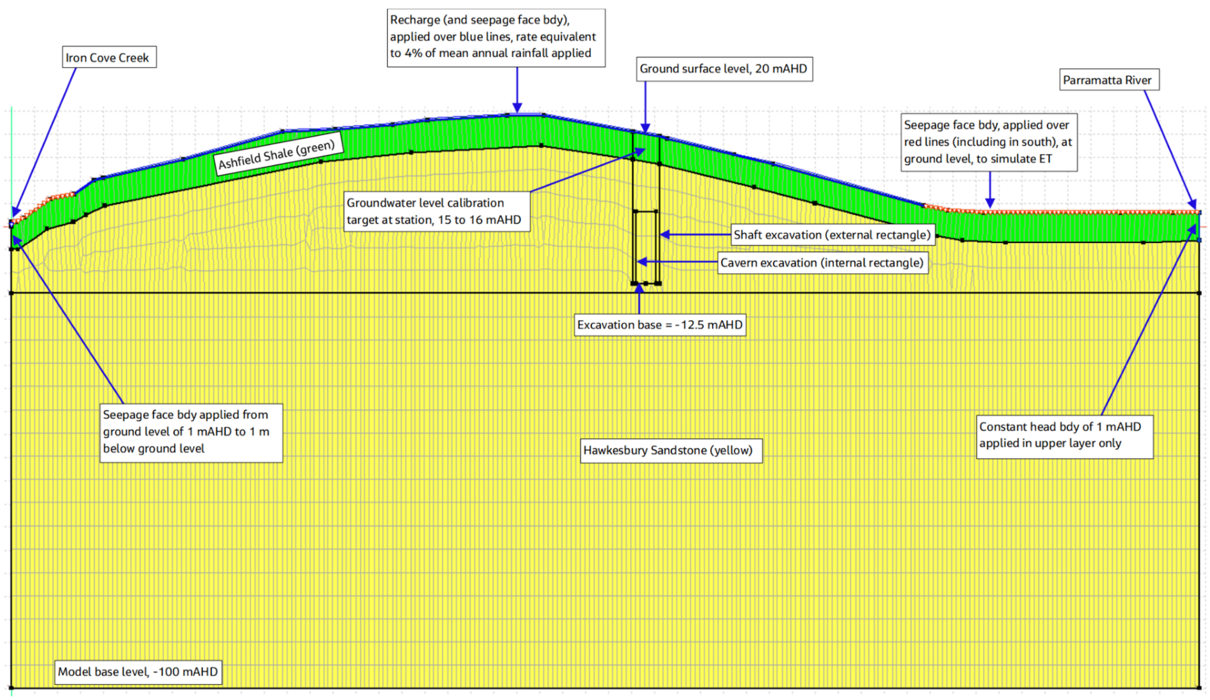

FIGURE 3 FIVE DOCK STATION SEEP/W MODEL SET UP. NOTE VERTICAL EXAGGERATION (VE) = 5.

2.4.1. Flow mode

Saturated flow conditions were simulated. Representation of unsaturated flow within the fill and residual soil was not required because these units are relatively thin, unsaturated at the station and are not significant with respect to the groundwater flow regime.

2.4.2. Model layer hydrogeological properties

Hydrogeological parameter values applied in the models are shown in Table 1. A brief justification for the applied parameter values is included in Table 1. Hydrogeological parameter values are covered in detail in the hydrogeological property annexure (Annexure B of the Stage 3 Hydrogeological Assessment Report).

TABLE 1 HYDROGEOLOGICAL PARAMETER VALUES APPLIED IN MODELS

Parameter	Ashfield Shale	Hawkesbury Sandstone	Justification
Saturated horizontal hydraulic conductivity (m/d)	0.0184	0.0173	Equivalent to 75th percentile of project packer testing for siltstone intervals, as documented in hydrogeological properties annexure, Annexure B
Saturated hydraulic conductivity (m/d) applied over excavation	100	100	Applied over excavation area to represent free drainage within the excavation that would occur during excavation
Kv/Kh [1]	0.1	0.1	Based on regional literature review, as documented in hydrogeological properties annexure, Annexure B
Specific yield	0.02	0.05	Based on regional literature review, as documented in hydrogeological properties annexure, Annexure B
Coefficient of volume compressibility (kPa^{-1})	5.1×10^{-7}	5.1×10^{-7}	Calculated based specific storage values derived from regional literature review, as documented in hydrogeological properties annexure, Annexure B

[1]Kv = vertical hydraulic conductivity, Kh = horizontal hydraulic conductivity.

2.4.3. Mesh resolution

A global mesh resolution of 5 m was applied to the models. The applied mesh is shown in Figure 3.

2.4.4. Boundary conditions

Boundary conditions are shown in Figure 3 and included:

- External constant head – applied at a level of 1 mAHD, in Ashfield Shale layer only, at northern extent of model, to represent the Parramatta River.
- External potential seepage face – applied from ground level (1 mAHD) to a depth of 1 m, at southern extent of model, to represent potential discharge to Iron Cove Creek.
  A potential seepage face was also applied at ground level in northern and southern portions of the model that have relatively low ground surface elevations, to simulate potential evapotranspiration (ET).
- Recharge applied at a rate equivalent to 4% of mean annual long term rainfall over whole section except where ET simulated. This recharge rate was arrived at iteratively, through trial and error, whilst matching modelled groundwater levels to existing conditions.
- Internal potential seepage face applied around shaft and cavern, in separate shaft and cavern models. This boundary condition simulates dewatering due to the station excavations.
- No flow boundaries applied at base and southern and northern extents of Hawkesbury Sandstone, and in Ashfield Shale at southern extent from a depth of 1 m below ground level until base of shale level (-5 mAHD). The no flow boundaries at the southern and northern extents represent groundwater flow divides.

2.4.5. Approach

The model calibration to existing groundwater levels was solved in steady state mode. A cloned transient model was developed and used the initial head from the steady state model to begin the transient simulation and ran for a duration of 3,650 days (10 years).

The only differences between the steady state model and predictive transient models were the internal seepage face boundaries applied around either the shaft or cavern excavations, and the hydraulic conductivity within the station excavation area being increased to a value of 100 m/d, to simulate efficient drainage.

2.5. Results

2.5.1. Calibration to existing representative groundwater levels

In both separate shaft and cavern models, the models were calibrated by adjusting the recharge rate to achieve the targeted existing representative water table level of 15 to 16 mAHD at the centre of the station. The watertable level target was achieved and the calibrated watertable level is shown in Figure 4.

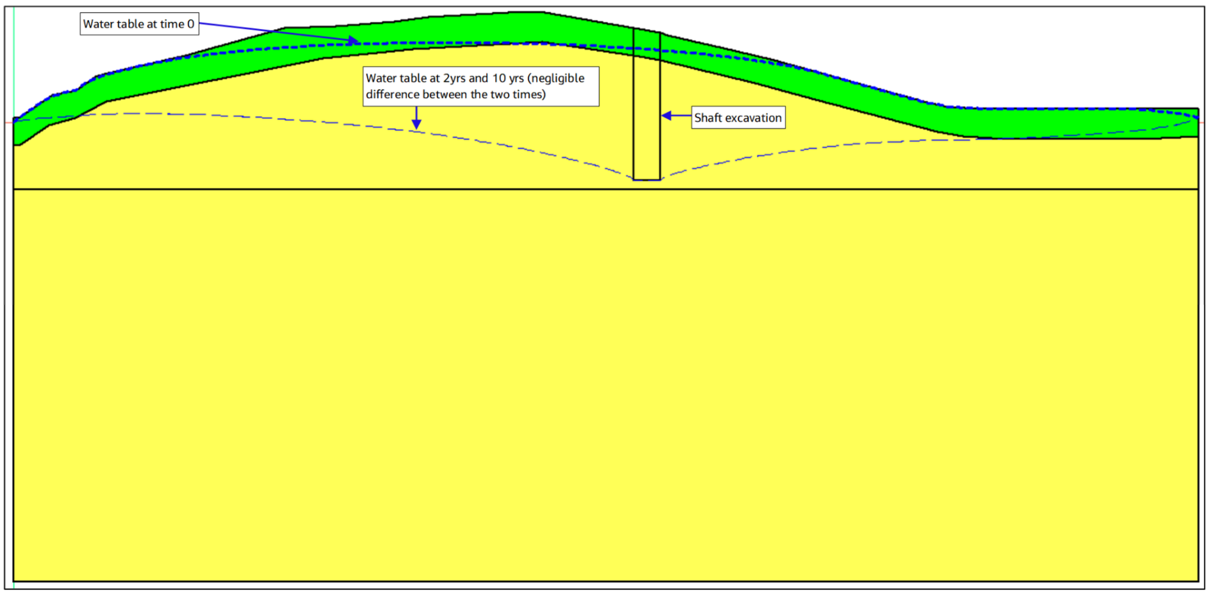

FIGURE 4 CALIBRATED WATERTABLE LEVEL (BLUE DASHED LINE). NOTE VERTICAL EXAGGERATION OF 5:1

2.5.2. Groundwater inflows

Groundwater inflow rates calculated by the model are shown in Figure 5 and are summarised as follows:

- Eastern shaft – up to 19 m^3/d.
- Western shaft – up to 58 m^3/d.
- Caverns – up to 94 m^3/d.
- Shafts and caverns combined – up to 170 m^3/d.

As shown in Figure 5, the modelled groundwater inflow rates vary with time. It is noted that the early time groundwater inflow rates are considered to be higher than would occur in reality under the assumed hydrogeological conditions and are considered to be elevated, in part, because the full excavation is assumed to occur instantaneously (the excavation is “wished in place”). In reality, the excavation would deepen progressively, and peak groundwater inflows would be lower than those reported here.

As discussed in the main body of the Five Dock Station Hydrogeological Assessment Report, there is a possibility that hydraulic conductivity values may be relatively higher in a zone of possible joint swarms identified in the geological long section, or in other not-yet identified zones. If this is the case, then groundwater inflows may be higher than modelled. The potential implications of this are discussed in the main body of the Hydrogeological Assessment Report.

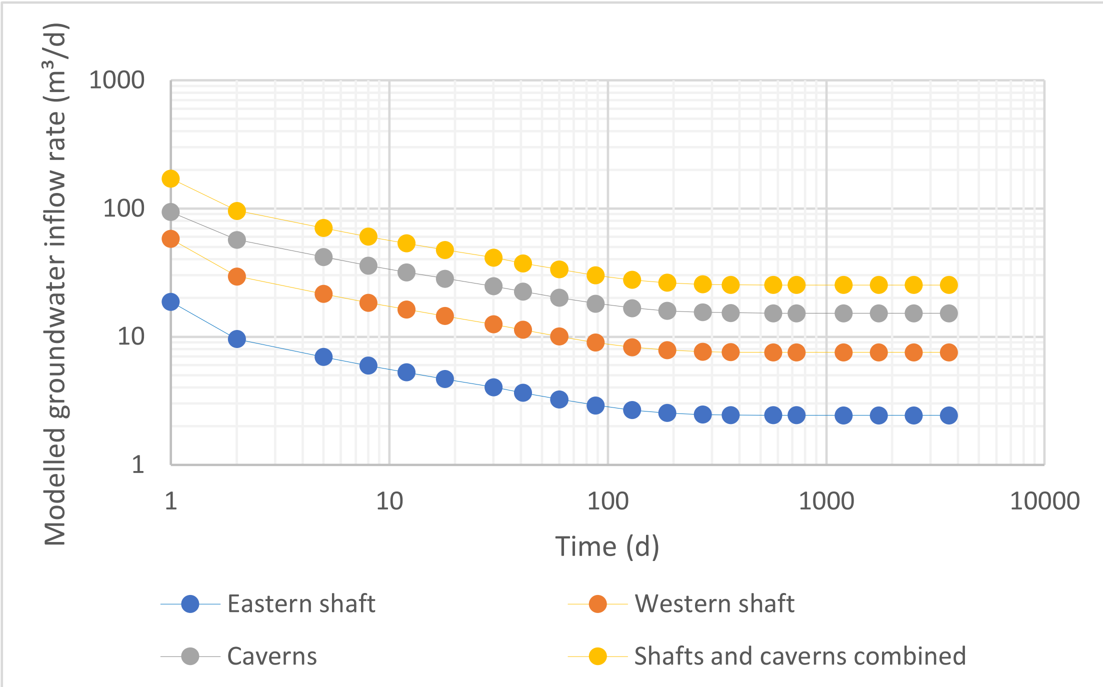

FIGURE 5 GROUNDWATER INFLOW RATES CALCULATED BY MODEL

2.5.3. Watertable drawdown

The modelled watertable surfaces are shown in Figure 6, and drawdown of the watertable is shown in Figure 7. In Figure 7, the distance of 0 m is at the southern extent of the modelled section.

There is negligible difference between the modelled water table drawdown at a time of two years and 10 years since wished-in-place excavation (i.e., steady state is reached within two years), which is why drawdown for both output times is not visible in Figure 7.

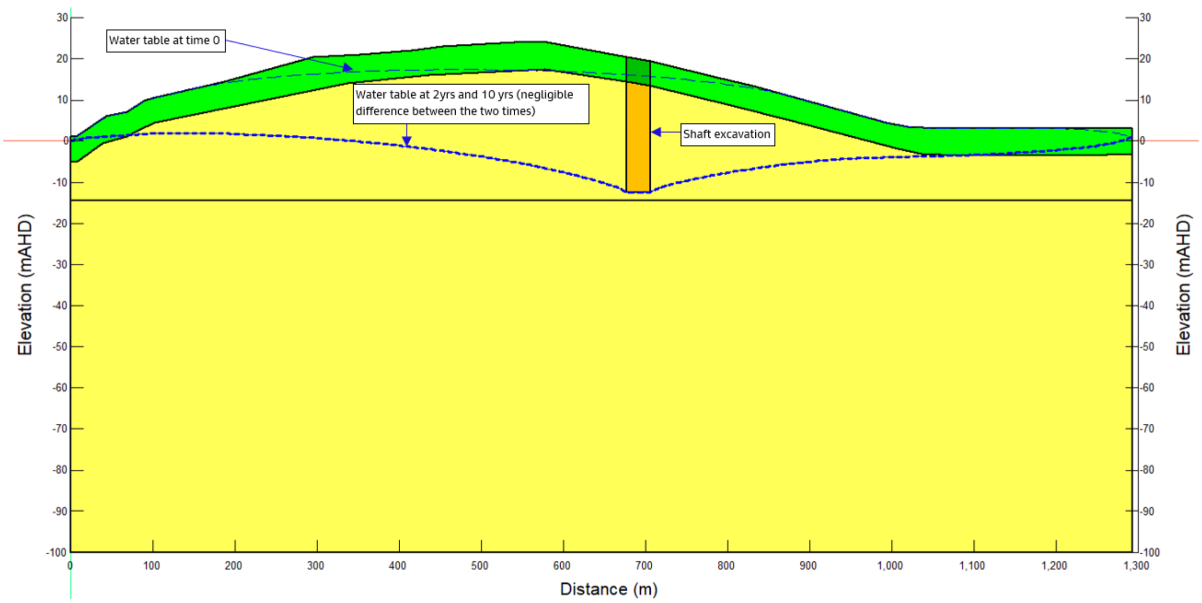

FIGURE 6 MODELLED WATER TABLE LEVELS. NOTE VERTICAL EXAGGERATION OF 5:1

FIGURE 7 MODELLED DRAWDOWN TO WATER TABLE