Sydney Olympic Park Station Hydrogeological Assessment Report

SMWSTCTP-AFJ-OLP-EN-RPT-000005 Revision 00

Sydney Metro West – Central Tunnelling Package

DOCUMENT APPROVAL

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CONTENTS

1. INTRODUCTION

1.1. OBJECTIVES AND SCOPE

The objective of this report is to provide hydrogeological advice for the design of the Sydney Olympic Park 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 Sydney Olympic Park Station.
- A review and update of the hydrogeological conceptual site model to reflect additional bore logs, geological interpretations and permeability testing that has 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.
- 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 Sydney Olympic Park 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.

Particular Specification Requirements:

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 $10m^2$, 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 $10m^2$, 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]

A. Sydney Olympic Park Station Excavation: 139,000 litres; [SM-W-CTP-PS-1044]

3. HYDROGEOLOGICAL CONCEPTUAL SITE MODEL

3.1 GEOLOGY

The location of Sydney Olympic Park Station and interpreted geological features is shown in Figure 3-1. A geological long section of Sydney Olympic Park Station is illustrated in Figure 3-2, which shows five geological units at the station box:

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

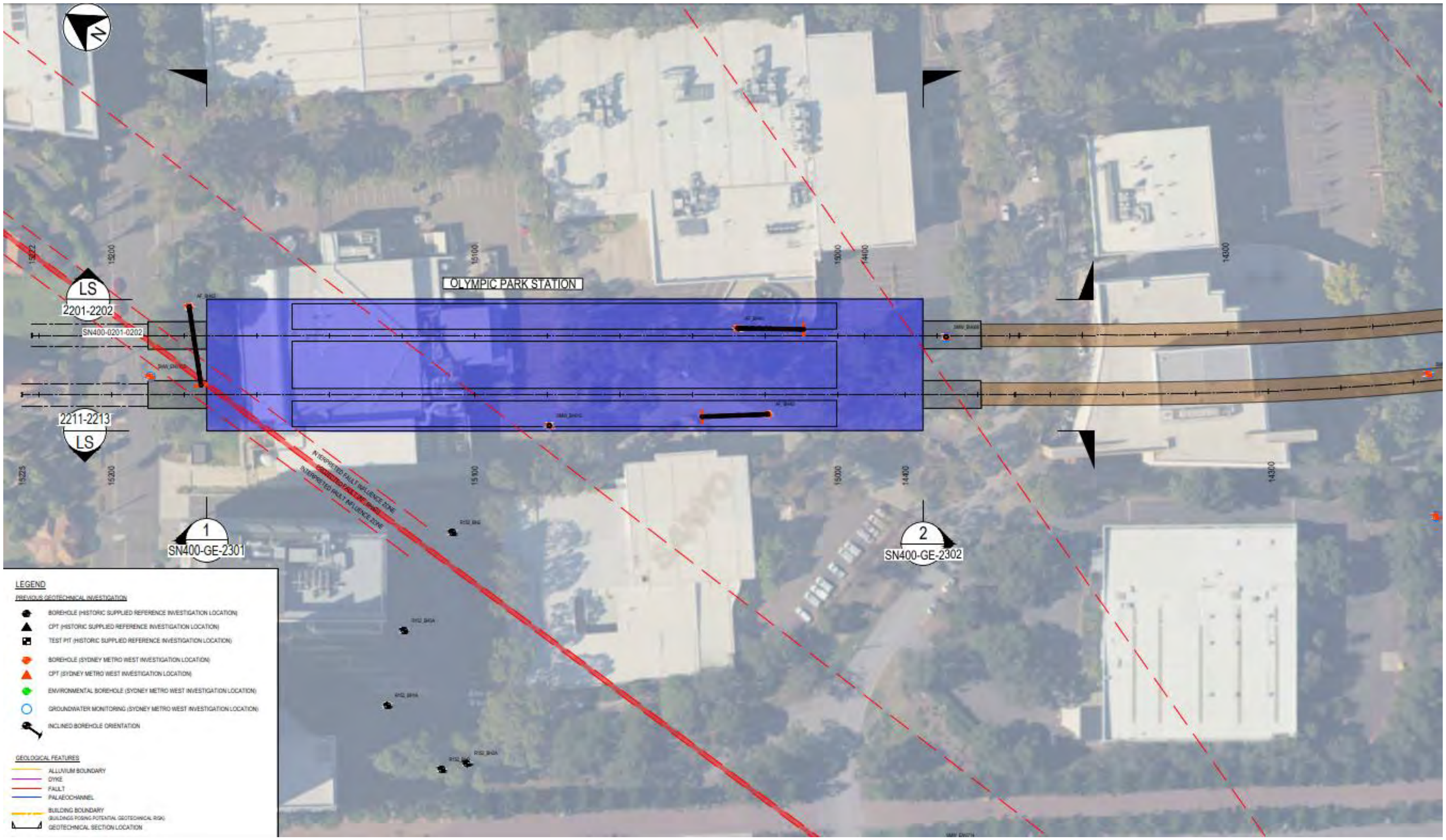


FIGURE 3-1: PLAN VIEW OF SYDNEY OLYMPIC PARK STATION, SHOWING INTERPRETED GEOLOGICAL FEATURES

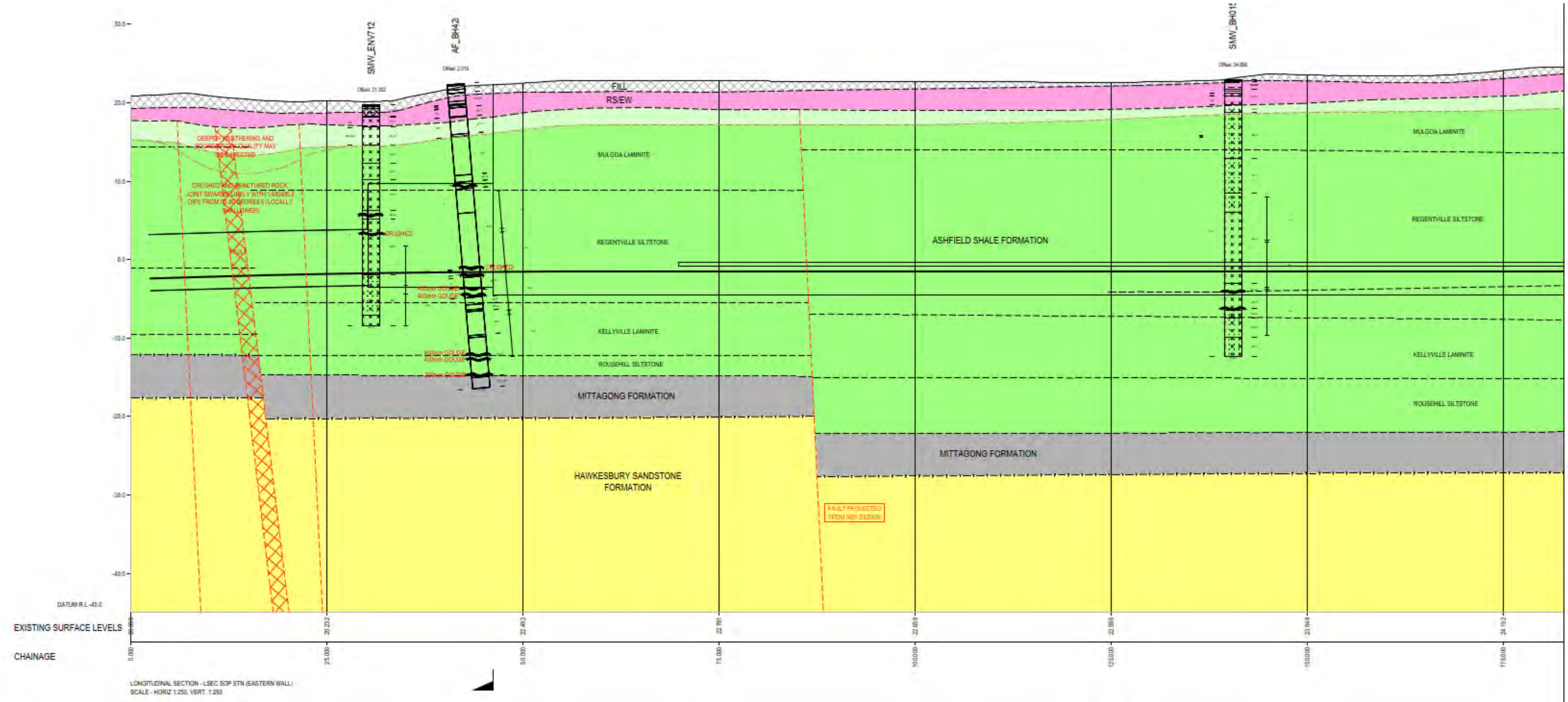


FIGURE 3-2: GEOLOGICAL LONG SECTION (LS 2201-2202) OF SYDNEY OLYMPIC PARK STATION

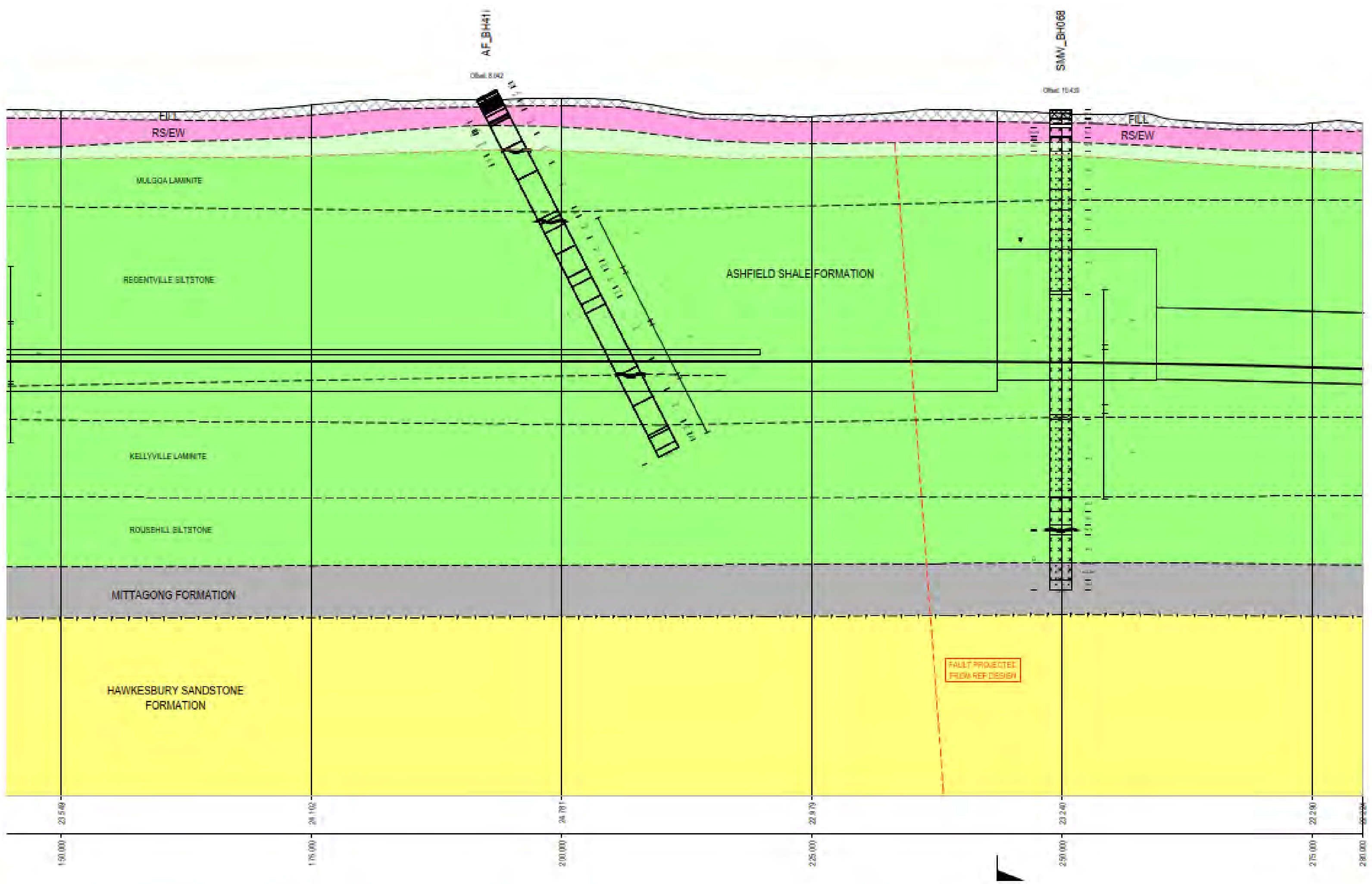


FIGURE 3-3: GEOLOGICAL LONG SECTION (LS 2201-2202) OF SYDNEY OLYMPIC PARK STATION

Of the five geological units, the station box will be excavated through the fill, residual soil and Ashfield Shale geological units.

The fill and residual soils are relatively thin, having a typical combined thickness of about 3 metres.The Ashfield Shale is about 30 m to 43 m thick in the area of the station, with the base of the unit at a level of about -23 m AHD over the majority of the station length, although the level decreases in the north western area of the station to about -12 m AHD to -15 m AHD. The decreases in the base level of the Ashfield Shale coincide with interpreted faults.

The station box has an excavation floor level of about -5 m AHD. Thus, an approximate 7 m to 10 m thick interval of Ashfield Shale exists between the station box excavation floor level and the next underlying geological unit, the Mittagong Formation.

The Mittagong Formation is thin (about 5 m thick) and is underlain by Hawkesbury Sandstone.

3.2 GROUNDWATER LEVELS AND FLOW

There are currently 13 SMW groundwater monitoring locations (Figure 3-4) in the vicinity of the Sydney Olympic Park Station. The locations are all standpipe piezometers.

Additionally, there are 11 reference site standpipe piezometers within 500 metres of the station, as shown in Figure 3-4. Data from these reference piezometers is not considered further in this memorandum. This is because:

- Hydrographs are not available for any of the piezometers
- Single groundwater level records for R152_BH2A and R152_BH3A are stated to be potentially influenced by drilling water
- The reported survey surface level is erroneous (surface level reported as 112.5 m AHD) for piezometer R154_BH6 and this piezometer is located relatively far from the station box excavation
- The R336_GA series reference site piezometers are all shallow landfill gas piezometers installed in fill and are located relatively far from the station box excavation. As such, groundwater levels from these piezometers are of little relevance.

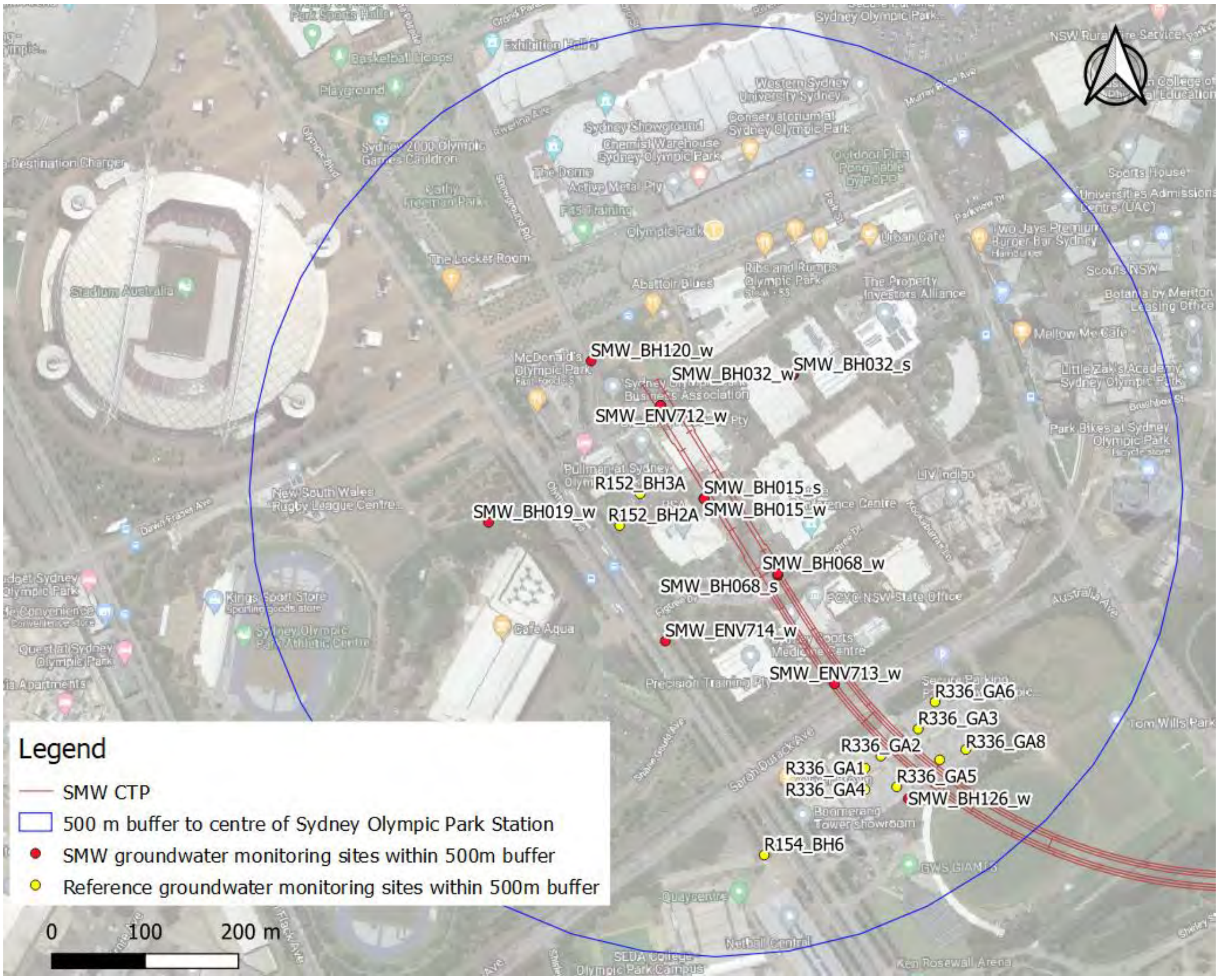


FIGURE 3-4: SMW AND REFERENCE SITE GROUNDWATER MONITORING LOCATIONS NEAR SYDNEY OLYMPIC PARK STATION

Piezometer construction details and recorded groundwater levels are provided in Table 1 and Table 2, respectively. Table 3 summarises groundwater levels and notes the formation in which the groundwater level lies for the piezometers. The effective screened material is summarised in detail in Table 1 and distinguished by hydrogeological unit in Table 2 and Table 3.

Monitored typical groundwater levels at Sydney Olympic Park Station range between approximately 0 m AHD and 12 m AHD, and between about 4 m and 19 m below ground surface. Excluding piezometers which are intermittently dry (SMW_BH015_s and SMW_BH068_s), the shallowest monitored groundwater level was about 14.3 m AHD (9.4 m bgl) and occurred at SMW_BH068_w. Piezometers SMW_BH015_s and SMW_BH068_s are intermittently dry but have maximum water levels of about 19.8 m AHD (3.12 m bgl) and 20.7 m AHD (2.66 m bgl), respectively. Piezometers SMW_BH015_s and SMW_BH068_s are interpreted to be monitoring perched local ephemeral groundwater systems and not the regional non-perched Ashfield Shale groundwater system.

The groundwater levels recorded in all piezometers except SMW_BH126_w lie within Ashfield Shale. The SMW_BH126_w groundwater level lies within clay fill.

Groundwater level contours developed based on representative non-perched Ashfield Shale groundwater level observations in project piezometers are shown in Figure 3-4. The relatively steep hydraulic gradient has likely been induced by the Olympic Park Rail loop and the relatively low permeability of the surrounding rock.

TABLE 1: SUMMARY OF GROUNDWATER MONITORING PIEZOMETERS AT SYDNEY OLYMPIC PARK STATION

Bore ID	Ground Surface Elevation (m AHD)	Effective Screen Depth Top (m bgl)	Effective Screen Depth Bottom (m bgl)	Material adjacent to effective screen interval	Monitoring Period
SMW_BH015_s	22.92	1.30	4.50	Siltstone and interlaminated siltstone and sandstone	30/08/2018 – 13/08/2019 (bore reported as intermittently dry)
SMW_BH015_w	22.94	23.00	29.30	Siltstone	30/08/2018 – 06/08/2019
SMW_BH019_w	17.33	20.50	26.50	Siltstone	14/08/2018 – 06/08/2019
SMW_BH032_s	19.76	3.30	7.25	Siltstone	06/05/2019 – 06/08/2019 (bore reported as dry)
SMW_BH032_w	19.74	16.00	23.00	Siltstone	2/09/2018 – 06/08/2019
SMW_BH068_s	23.36	0.60	4.40	Fill, gravelly clay, siltstone, siltstone, interlaminated siltstone and sandstone	01/09/2018 – 06/08/2019 (bore reported as intermittently dry)
SMW_BH068_w	23.64	19.30	26.10	Interlaminated siltstone and sandstone	30/08/2018 – 06/08/2019
SMW_BH120_w	17.38	20.00	26.50	Siltstone, interlaminated siltstone and sandstone	5/10/2019 – 01/09/2020
SMW_BH126_w	11.40	7.20	12.50	Fill, siltstone	5/12/2019 – 01/09/2020
SMW_ENV712_s	19.72	12.00	16.50	Siltstone	1/04/2021 – 11/06/2021
SMW_ENV712_w	19.72	19.90	28.04	Siltstone, interlaminated	1/04/2021 – 11/06/2021

Bore ID	Ground Surface Elevation (m AHD)	Effective Screen Depth Top (m bgl)	Effective Screen Depth Bottom (m bgl)	Material adjacent to effective screen interval	Monitoring Period
				siltstone and sandstone	
SMW_ENV713_w	13.67	13.10	23.10	Siltstone	22/02/2021 – 15/03/2021
SMW_ENV714_w	19.47	10.50	21.40	Siltstone	1/04/2021 – 11/06/2021

TABLE 2: SUMMARY OF GROUNDWATER LEVELS AND DEPTHS AT SYDNEY OLYMPIC PARK STATION

Bore ID	Unit(s) [1]	Monitoring Period	Groundwater Elevation (m AHD)		Groundwater Depth (m bgl)	
			Approx. Typical	Highest	Approx. Typical	Shallowest
SMW_BH015_s	Ash	30/08/2018 – 13/08/2019	NA – ID [2]	19.80	NA – ID [2]	3.12
SMW_BH015_w	Ash	30/08/2018 – 06/08/2019	7.10	7.20	15.84	15.74
SMW_BH019_w	Ash	14/08/2018 – 06/08/2019	1.70	1.95	15.63	15.38
SMW_BH032_s	Ash	06/05/2019 – 06/08/2019	Well reported as dry			
SMW_BH032_w	Ash	2/09/2018 – 06/08/2019	12.00	13.05	7.74	6.69
SMW_BH068_s	Fill and Ash (maximum water level resides in Ash)	01/09/2018 – 06/08/2019	NA – ID [2]	20.70	NA – ID [2]	2.66
SMW_BH068_w	Ash	30/08/2018 – 06/08/2019	12.00	14.25	11.64	9.39
SMW_BH120_w	Ash	5/10/2019 – 01/09/2020	0.40	0.52	16.98	16.86

Bore ID	Unit(s) [1]	Monitoring Period	Groundwater Elevation (m AHD)		Groundwater Depth (m bgl)	
			Approx. Typical	Highest	Approx. Typical	Shallowest
SMW_BH126_w	Fill and Ash (water levels reside in clay fill)	5/12/2019 – 01/09/2020	7.20	7.30	4.2	4.1
SMW_ENV712_s	Ash	1/04/2021 – 11/06/2021	5.50	5.65	14.22	14.07
SMW_ENV712_w	Ash	1/04/2021 – 11/06/2021	0.30	0.43	19.42	19.29
SMW_ENV713_w	Ash	22/02/2021 – 15/03/2021	10.00	10.10	3.67	3.57
SMW_ENV714_w	Ash	1/04/2021 – 11/06/2021	4.25	4.40	15.22	15.07

Notes: 1 ASH means Ashfield Shale. 2 NA – ID means not applicable – intermittently dry. Water levels of intermittently dry piezometers are summarised more comprehensively in Table 3.

TABLE 3: SUMMARY OF GROUNDWATER LEVELS AND DEPTHS AT SYDNEY OLYMPIC PARK STATION, INCLUDING STRATIGRAPHIC LOCATION OF TYPICAL GROUNDWATER LEVEL

Bore ID	Effective Screened Unit(s) [1]	Typical Groundwater Level (m AHD)	Typical Groundwater Level (m bgl)	Stratigraphic Location of Typical Groundwater Level
SMW_BH015_s	Ash	NA – ID [2]	NA – ID [2]	Variable from dry (>4.5 mBGL or <18.42 mAHD), which is >3.3 m into ASH, to 3.12 mBGL or 19.8 mAHD (shallowest level), which is 1.92 m into ASH
SMW_BH015_w	Ash	7.10	15.84	14.64 m into ASH
SMW_BH019_w	Ash	1.70	15.63	11.13 m into ASH
SMW_BH032_s	Ash	<12.51 (dry)	>7.25 (dry)	>4.15 m into ASH
SMW_BH032_w	Ash	12.00	7.74	4.74 m into ASH
SMW_BH068_s	Fill and Ash	NA – ID [2]	NA – ID [2]	Variable from dry (>4.3 mBGL or <19.06 mAHD), which is >3.35 m into ASH, to 2.66 mBGL or 20.70 mAHD (shallowest level), which is 1.71 m into ASH

Bore ID	Effective Screened Unit(s) [1]	Typical Groundwater Level (m AHD)	Typical Groundwater Level (m bgl)	Stratigraphic Location of Typical Groundwater Level
SMW_BH068_w	Ash	12.00	11.64	10.69 m into ASH
SMW_BH120_w	Ash	0.40	16.98	12.49 m into ASH
SMW_BH126_w	Fill and Ash	7.20	4.20	4.20 m into clay fill, 5.70 m above interface of clay fill and ASH
SMW_ENV712_s	Ash	5.50	14.22	12.82 m into ASH
SMW_ENV712_w	Ash	0.30	19.42	18.02 m into ASH
SMW_ENV713_w	Ash	10.00	3.67	2.07 m into ASH
SMW_ENV714_w	Ash	4.25	15.22	13.52 m into ASH

Notes: 1 ASH means Ashfield Shale. 2 NA – ID means not applicable – intermittently dry

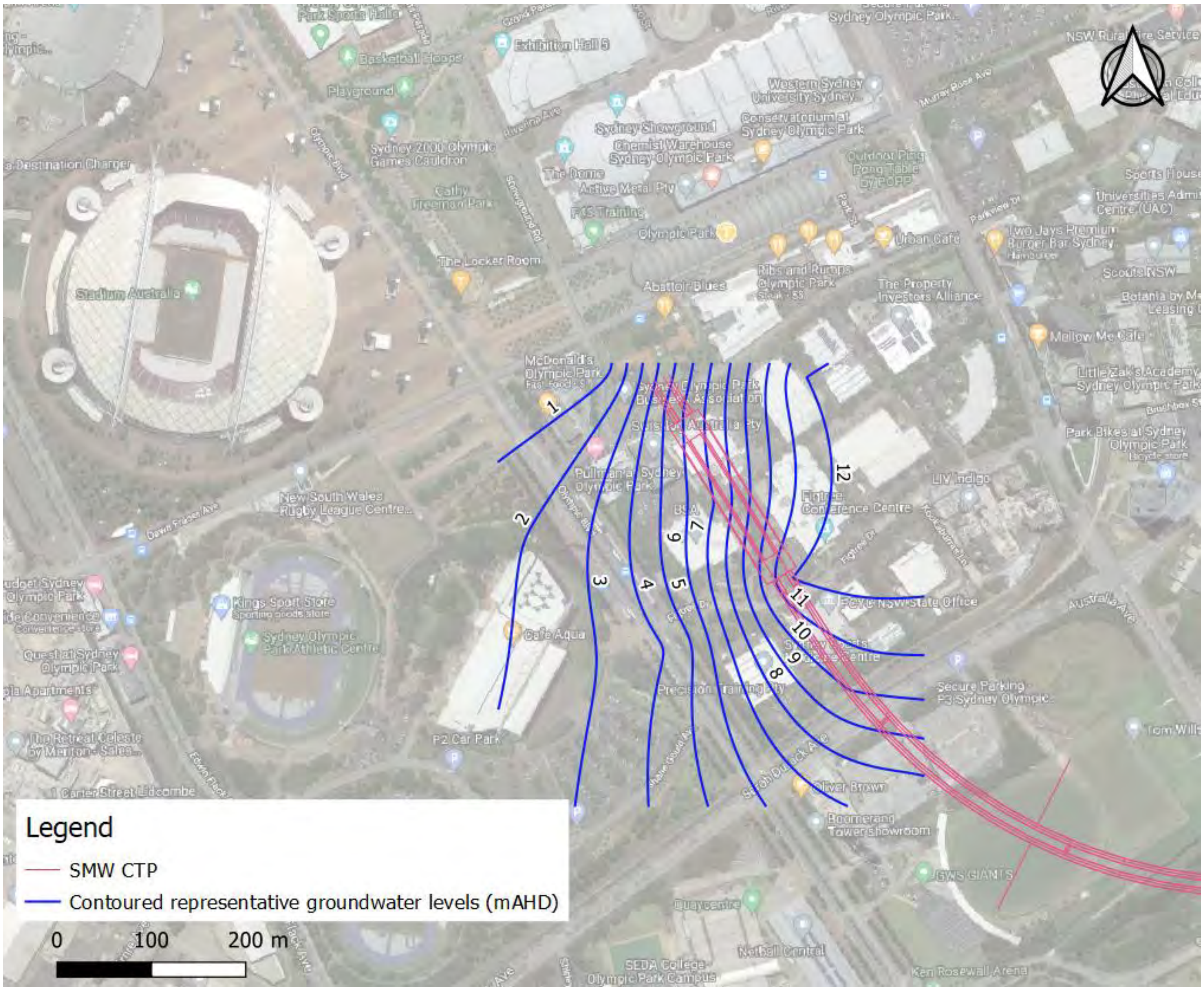


FIGURE 3-5: CONTOURED REPRESENTATIVE NON-PERCHED ASHFIELD SHALE GROUNDWATER LEVELS

The groundwater level data suggests that perched groundwater systems may exist in the vicinity of Sydney Olympic Park Station. However, the shallowest groundwater levels associated with these perched groundwater systems remained in the same hydrogeological unit as the hydrogeological unit which typically hosts the regional water table, Ashfield Shale. Only one monitoring location had groundwater levels which did not lie in Ashfield Shale and instead were situated in clay fill, and the station excavation floor level is situated in Ashfield Shale. Thus, Ashfield Shale is considered the key hydrogeological unit relevant to assessment of groundwater inflows and potential groundwater level drawdown associated with the station box excavation.

Figure 3-6 shows groundwater pressure head against the elevation of the base of the piezometer screen for all piezometers listed in Table 2 (except for those piezometers which were dry). The figure shows the maximum and minimum hydrostatic profiles that would exist for these piezometers as well as the pressure head in all piezometers, including a coupled (shallow and deep) piezometer. In all cases, the trend in pressure profile is close to a hydrostatic trend, suggesting the potential for a hydraulically connected groundwater system within the Ashfield Shale.

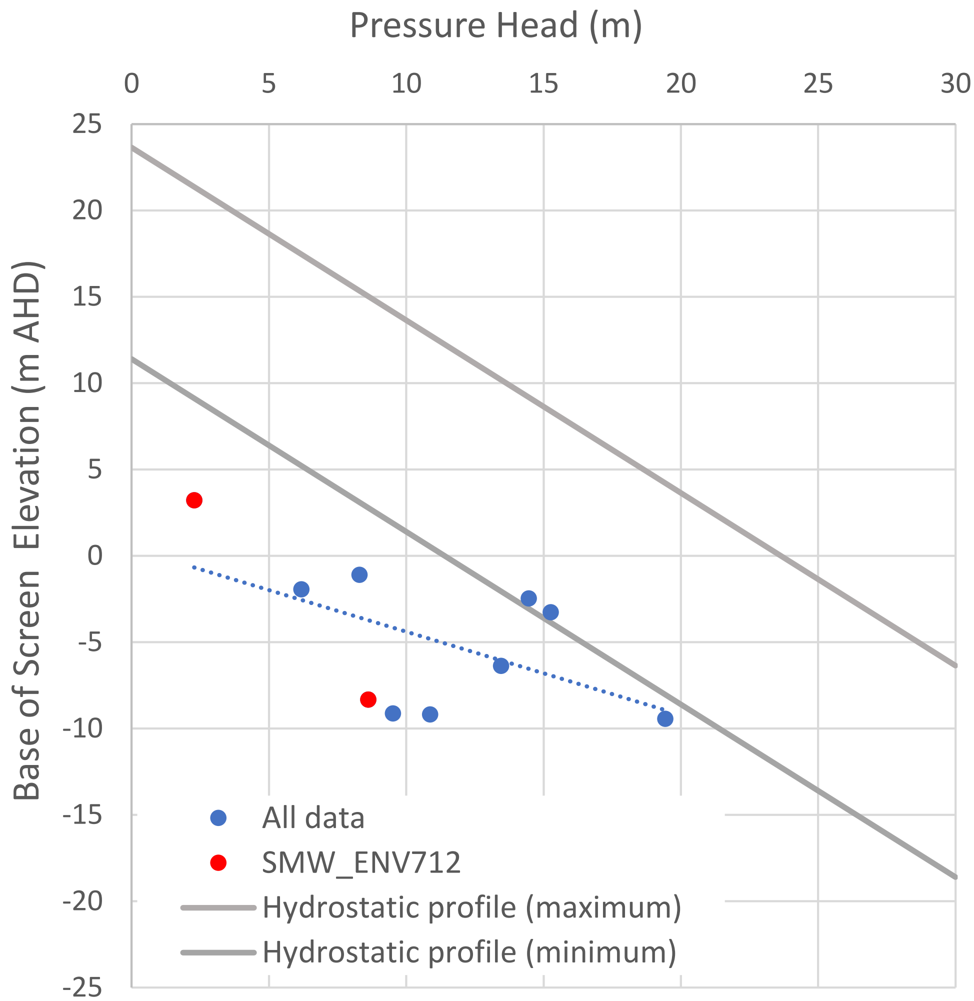


FIGURE 3-6: GROUNDWATER PRESSURE HEAD IN PIEZOMETERS AT SYDNEY OLYMPIC PARK STATION

The following existing groundwater sinks (drained structures/excavations) may be causing a depressed groundwater table: the Olympic Park Rail loop (tunnelled portions and non-tunnelled but excavated portions), adjacent building basements and the brick pit excavation (Figure 3-7). The likelihood of these features causing depressed groundwater levels in the vicinity of the station box is summarised below.

- Olympic Park Rail loop – it is considered likely that the western tunnelled portion of the Olympic Park Rail loop is lowering groundwater levels in the vicinity of the station box to some degree. The minimum rail level of 1.61 mAHD (19.1 mbgl) is below the representative groundwater level observed at all SMW monitoring sites except SMW_BH120w, which had a representative level of 0.4 mAHD. Contouring of the representative groundwater levels suggests that the groundwater flow direction is towards the western tunnelled portion of the Olympic Park Rail loop, indicating the structure may be behaving as a groundwater sink.
- With respect to non-tunnelled portions of the Olympic Park Rail loop, to the east and south of the station box, there is relatively less potential for these portions to be behaving as significant groundwater sinks. This is because the eastern portion is generally either relatively close to ground level or above ground level, and the southern portion has rail levels relatively close to ground level, ranging from 1.4 mbgl (11.3 mAHD) to 6.35 mbgl (8.25 mAHD). In the southern portion, the representative groundwater level contours do not suggest groundwater level

drawdown. However, the points used to generate the groundwater levels contours are somewhat sparse and may not show minor groundwater level drawdown.

- Adjacent buildings and basements – the likelihood of surrounding buildings and basements causing significant groundwater level drawdown is considered relatively low. The relatively small footprints of such buildings and basements, if constructed as drainage structures below the water table, is unlikely to manifest significant drawdown. However, there is some uncertainty regarding building basements in the area. Basement level elevations of existing buildings would be required to confirm this.
- Brickpit excavation – the likelihood of the brickpit excavation causing significant groundwater level drawdown in the vicinity of the station box is considered low. Whilst ELVIS LIDAR spot height elevations indicate the water level in the brickpit is relatively low at about -18.2 mAHD, the representative groundwater level contours suggest this feature is currently not causing groundwater level drawdown in the vicinity of the station box. The contoured representative groundwater levels suggest groundwater flow is to the west, away from the brickpit.

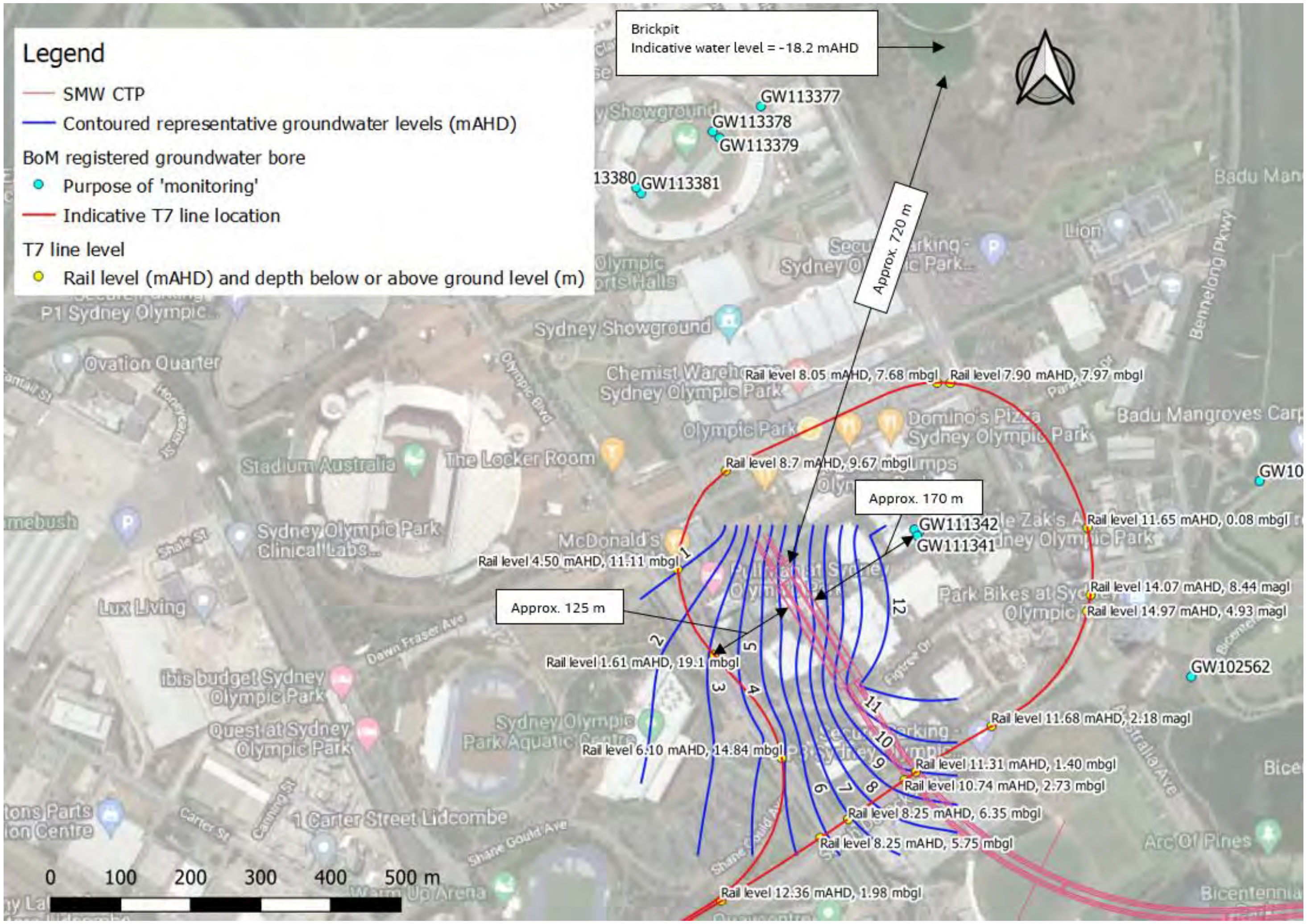


FIGURE 3-7: PLAN VIEW OF SYDNEY OLYMPIC PARK STATION, SHOWING INTERPRETED GEOLOGICAL FEATURES

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 Sydney Olympic Park Station, the pertinent hydrogeological units comprise Ashfield Shale, and to a lesser degree, 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 generally situated below these units at the station and the units are thin.

Packer tests were undertaken in 10 boreholes at the site, as listed in Table 4. Additionally, piezometer SMW_BH126_w has been slug tested. All of the packer test intervals are interpreted to be within Ashfield Shale. The effective screen interval material of the sole slug tested piezometer, SMW_BH126_w, is clay fill and Ashfield Shale.

The results of in-situ permeability (packer) tests and slug tests at Sydney Olympic Park Station are summarised in Table 4 and the packer test results plotted by depth below ground in Figure 3-8. Figure 3-8 also includes all SMW packer test results outside of the Bays paleo channel, to enable a comparison of the Sydney Olympic Park results to the broader CTP results.

Packer test results in the vicinity of Sydney Olympic Park Station range between less than 0.1 Lugeon (<8.6×10-4 m/day) and 2 Lugeons (1.7×10-2 m/day). The median and average values of all the data are 0.1 and 0.3 Lugeons, respectively. There is a potential trend with depth at Sydney Olympic Park Station, with Lugeon decreasing with depth. However, the correlation is not strong.

The maximum packer test result at Sydney Olympic Park Station of 2 Lugeons (1.7×10-2 m/day) is similar to the average and 75th percentile statistic values of 1.6 Lugeons and 1.4 Lugeons, respectively, for all packer tests completed within siltstone for the whole project (excluding tests completed in palaeochannel at The Bays Station site). The Sydney Olympic Park Station packer test median value of 0.1 is similar to, but slightly less than, the median value of 0.3 Lugeons for all packer tests completed within siltstone for the whole project (excluding tests completed in the palaeochannel at The Bays Station site).

Overall, the Sydney Olympic Park Station packer test data is similar to the results for all packer tests completed within siltstone for the whole project (excluding tests completed in palaeochannel at The Bays Station site), indicating typical conditions.

TABLE 4: SUMMARY OF PERMEABILITY TEST RESULTS AT SYDNEY OLYMPIC PARK STATION AND INFERRED HYDRAULIC CONDUCTIVITY

Bore ID	Depth Top	Depth Bottom	Unit(s) [1]	Result	
	m BGL (vertical)	m BGL (vertical)		uL	Hydraulic conductivity m/day
SMW_BH015_w	20.50	26.72	ASH	<0.1	<8.7×10-4
SMW_BH015_w	26.50	32.60	ASH	0.7	6.1×10-3
SMW_BH019_w	29.80	35.00	ASH	<0.1	<8.7×10-4
SMW_BH019_w	11.00	18.00	ASH	0.4	3.5×10-3
SMW_BH032_w	15.00	18.00	ASH	2.4	2.1×10-2
SMW_BH032_w	17.50	24.00	ASH	<0.1	<8.7×10-4

Bore ID	Depth Top	Depth Bottom	Unit(s) [1]	Result	
	m BGL (vertical)	m BGL (vertical)		uL	Hydraulic conductivity m/day
SMW_BH032_w	23.50	30.10	ASH	0.1	8.7×10-4
SMW_BH068_w	18.00	24.00	ASH	<0.1	<8.7×10-4
SMW_BH068_w	23.50	30.35	ASH	0.2	1.7×10-3
SMW_BH068_w	29.50	39.00	ASH	<0.1	<8.7×10-4
SMW_BH120_w	12.00	17.94	ASH	0.5	4.3×10-3
SMW_BH120_w	17.50	23.92	ASH	0.5	4.3×10-3
SMW_BH120_w	23.50	30.02	ASH	<0.1	<8.7×10-4
SMW_BH120_w	29.50	35.97	ASH	<0.1	<8.7×10-4
SMW_BH126_w	12.00	17.97	ASH	0.1	8.7×10-4
SMW_BH126_w	17.77	26.18	ASH	<0.1	<8.7×10-4
SMW_ENV712_w	18.00	24.13	ASH	<0.1	<8.7×10-4
SMW_ENV712_w	23.00	28.13	ASH	0.2	1.7×10-3
SMW_BH126_w [2]	7.20	12.50	Fill (clay) and ASH		8.4×10-4
AF_BH40i	14.00	19.79	ASH	<0.1	8.7×10-4
AF_BH40i	19.25	25.14	ASH	0.1	8.7×10-4
AF_BH40i	24.65	30.44	ASH	<0.1	8.7×10-4
AF_BH40i	29.99	35.64	ASH	1.7	1.5×10-2
AF_BH41i	14.26	19.74	ASH	<0.1	8.7×10-4
AF_BH41i	19.16	24.95	ASH	<0.1	8.7×10-4
AF_BH41i	24.50	30.29	ASH	0.6	5.2×10-3
AF_BH41i	29.85	35.72	ASH	0.1	8.7×10-4
AF_BH42i	13.99	19.29	ASH	0.3	2.6×10-3
AF_BH42i	18.80	24.31	ASH	0.9	7.8×10-3
AF_BH42i	23.61	29.87	ASH	<0.1	8.7×10-4
AF_BH42i	29.30	35.22	ASH	0.2	1.7×10-3
Summary statistics					
				Median of all packer tests	8.7×10-4
				Maximum of all packer tests	1.7×10-2

Notes: 1 ASH means Ashfield Shale. 2 Slug tested piezometer. All other tests in table are packer tests. Hydraulic conductivity value report for bore is average of six slug test results.

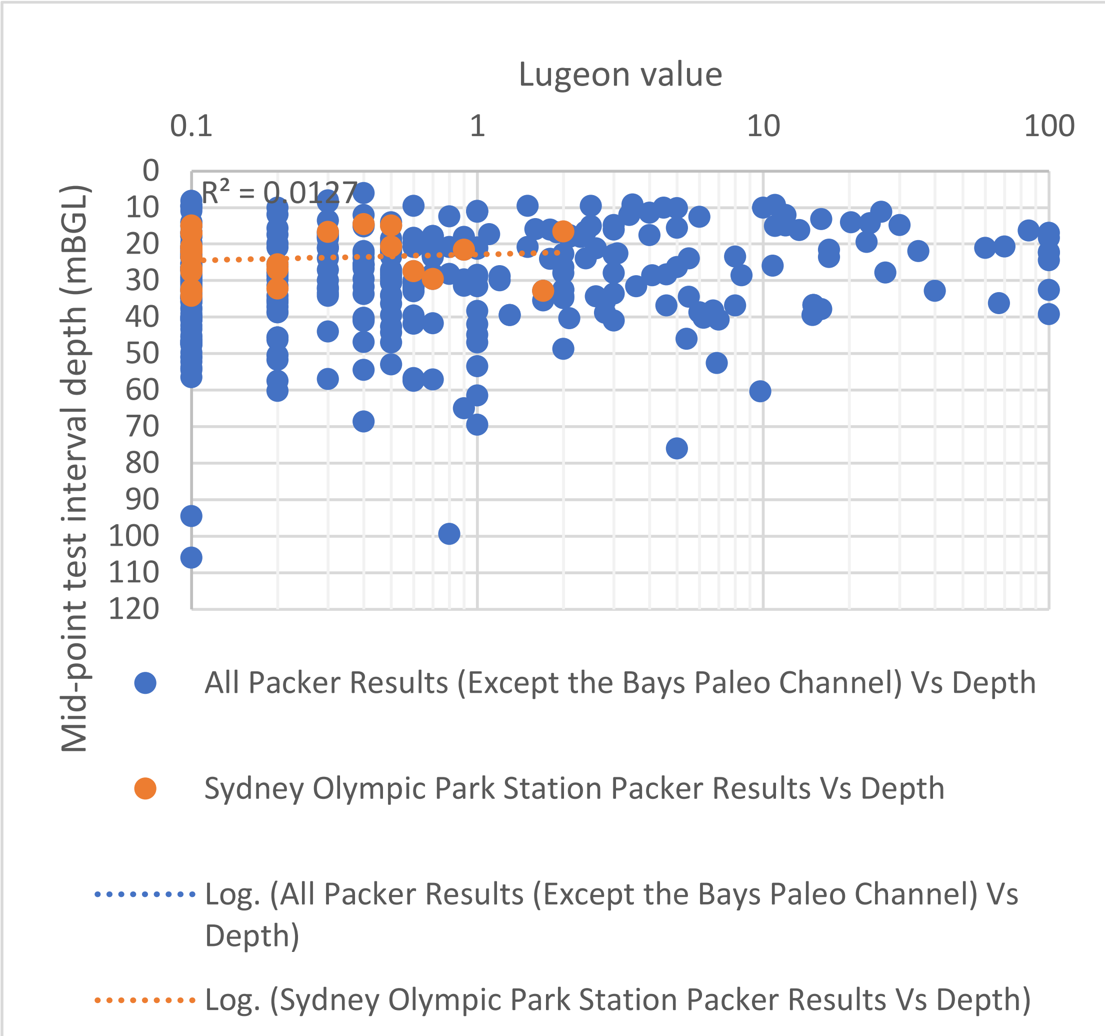


FIGURE 3-8: LUGEON VALUES WITH DEPTH BELOW GROUND SURFACE

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 5. 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 higher than the arithmetic mean and median/geometric mean values of the Sydney Olympic Park Station packer test data.

TABLE 5: 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)	Kv/Kh range	Specific storage range (m-1)	Specific yield range (-)
	Typical range			
Ashfield Shale	3.80×10-3 to 1.20×10-2 (0.4 to 1.4 Lugeons) (geomean to 75th percentile) (Log-normally distributed arithmetic mean is 1.64×10-2 = 1.9 Lugeons; K3D value is 6.05×10-3 m/d = 0.7 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	5.27×10-3 to 1.73×10-2 (0.6 to 2.0 Lugeons) (geomean to 75th percentile) (Log-normally distributed arithmetic mean is 2.65×10-2 m/d = 3.1 Lugeons; K3D value is 9.06×10-3 m/d = 1.0 Lugeons)	0.01 to 1	1.00×10-6 to 1.00×10-5	0.02 to 0.05
	Adopted representative value			
Ashfield Shale	1.20×10-2 (1.4 Lugeons; 75th percentile)	0.1	5.00×10-6	0.02
Mittagong Formation and Hawkesbury Sandstone	1.73×10-2 (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 6.

TABLE 6: 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.

Based on review of the hydrographs (Annexure A), the highest short term groundwater level increases occurred at piezometers SMW_BH015_s, SMW_BH068_s and SMW_BH068_w and were increases of about 0.85 metres, 1.6 metres and 1.5 metres, respectively. However, these groundwater level increases are interpreted to not have been caused by high rainfall events (SMW_BH068_s and SMW_BH068_w), or in the case of SMW_BH015_s, may have been caused by high rainfall but are not relevant to Ashfield Shale regional groundwater system levels because SMW_BH015_s is interpreted to have groundwater levels associated with a perched groundwater system.

Based on review of the hydrographs, groundwater level increases in the non-perched Ashfield Shale groundwater system due to rainfall are minor. A groundwater level increase of about 0.25 m at piezometer SMW_BH126_w (Figure 4-1) is interpreted to have occur due to a daily rainfall of about 79 mm.

The short-term groundwater level increase of about 0.25 m, which occurred at piezometer SMW_BH126_w, is taken to inform potential short term increases to non-perched Ashfield Shale groundwater levels at Sydney Olympic Park Station which could occur due to high rainfall events.

Seasonal variations in the Ashfield Shale at Sydney Olympic Park Station, as shown by trends in Figure 4-2, suggest that seasonal groundwater level rises up to 0.5 m occur. Seasonal trends are less prevalent in shallower bores, which tend to respond more rapidly to individual rainfall events.

Overall, the data suggests that non-perched Ashfield Shale groundwater levels are not significantly influenced by short to medium term rainfall.

Cumulative mean monthly rainfall deviation since the year 2000 is shown in Figure 4-3 for rainfall data extracted from the SILO database for a point located at latitude -33.85 and longitude 151.05, located about 1.8 km west of the 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. Considering the timing of the piezometer groundwater level data occurs near the end of a prolonged period of below average rainfall, based on the hydrograph trends, a potential rise in groundwater level of 1 m is adopted for the construction period to the end of 2024.

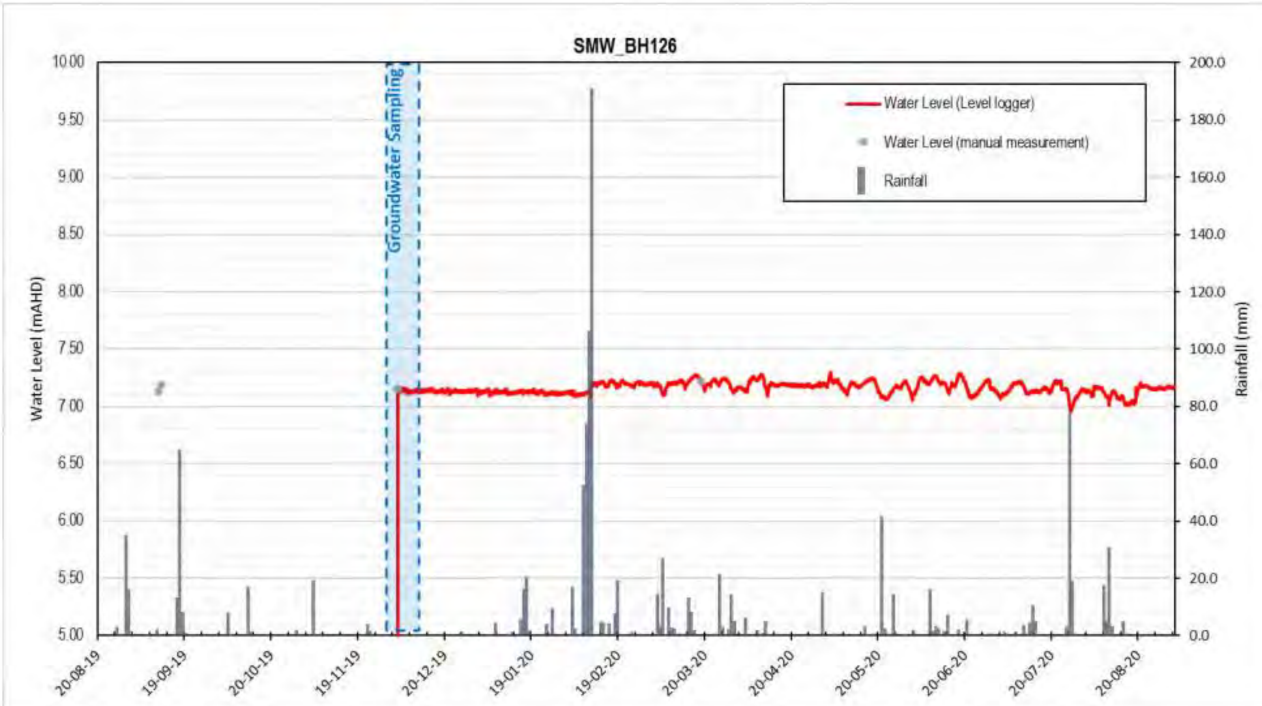


FIGURE 4-1: SMW_BH125_W HYDROGRAPH

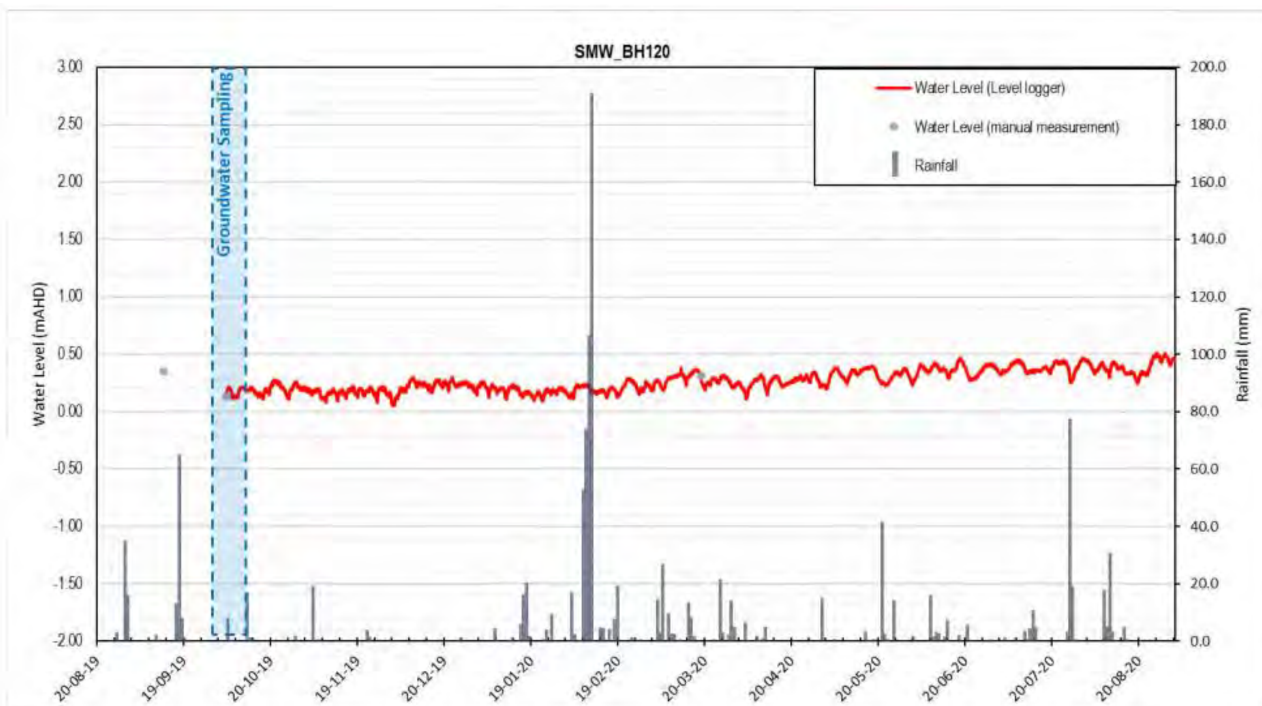


FIGURE 4-2: SMW_BH120_W HYDROGRAPH

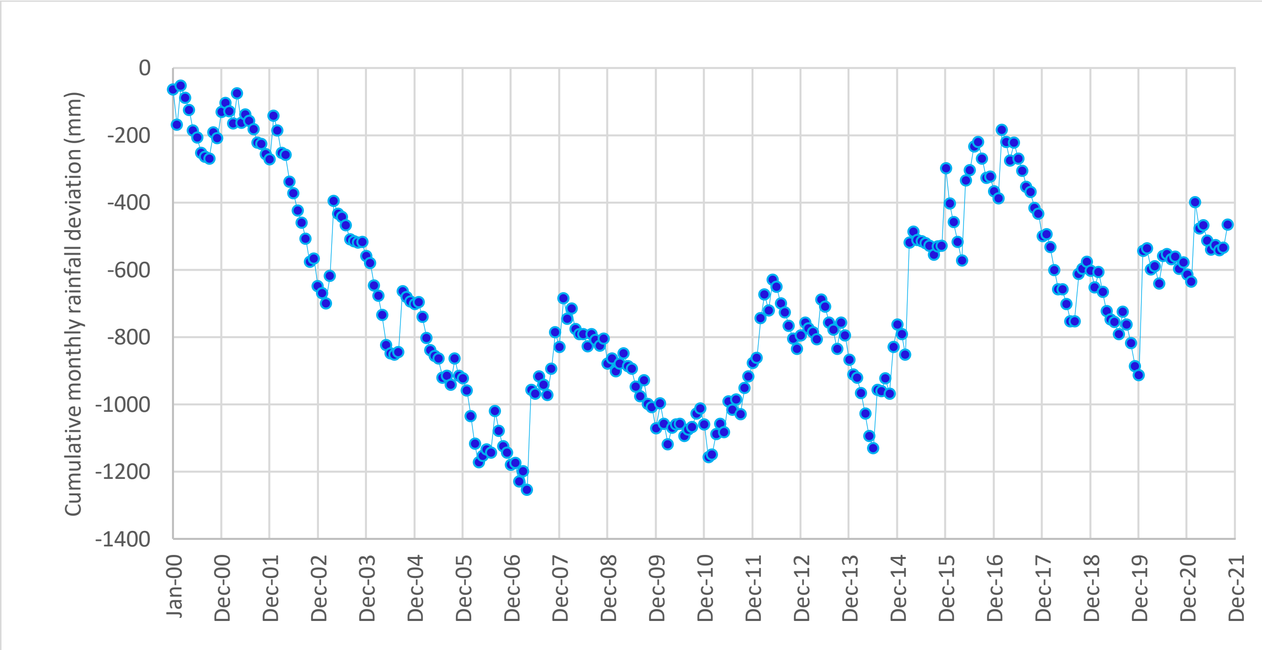


FIGURE 4-3: CUMULATIVE DEVIATION FROM MEAN MONTHLY RAINFALL AT SILO POINT, LATITUDE -33.85, LONGITUDE 151.05, LOCATED ABOUT 1.8 KM WEST OF SYDNEY OLYMPIC PARK STATION

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 addressed in a seperate 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 Sydney Olympic Park Station to the Paramatta River and its tributaries, 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 Sydney Olympic Park 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 maximum groundwater level observed at SMW_BH068_w, located close to the station footprint, is 14.3 m AHD, which is 9.3 m below ground surface.

Based on the above discussion, the following potential increases to the currently observed maximum groundwater level at SMW_BH068_w are possible:

- An increase of 1.0 m due to rainfall
- No increase for flooding
- An increase of 0.1 m for climate induced sea level rise effects on groundwater levels

Accordingly, within the design life, the highest possible non-perched groundwater level at the Sydney Olympic Park Station is inferred to be 15.4 m AHD, which is 8.2 m below ground surface at the location of SMW_BH068_w.

The effects considered above are summarised in Table 7 below.

TABLE 7: SUMMARY OF FACTORS AND GROUNDWATER LEVELS AT SYDNEY OLYMPIC PARK STATION IN ABSENCE OF CTP EXCAVATION WORKS

Surface elevation[1] (m AHD)	Maximum observed GWL (m AHD) [m bgl]	Rise due to rainfall (m)	Rise due to rising sea level (m)	Possible groundwater level (m AHD) [m bgl]
23.6 (at SMW_BH068_w)	14.3 [9.3] at SMW_BH068_w	1.0	0.1	15.4 [8.2]

1 Values presented rounded to 0.5 m

4.3.2 CTP WORKS CONDITIONS

The Sydney Olympic Park 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

Results in the Contamination Assessment Report indicate that:

- pH is generally slightly acidic to slightly alkaline. Excluding an elevated outlier (pH 11.8) interpreted to be associated with grout contamination and a low value outlier of pH 5 at R154_BH6, the pH ranged from 6.6 to 7.7.
- In accordance with salinity classes outlined in Freeze and Cherry (1979), the groundwater ranges from brackish (minimum TDS of 1,000 mg/L) to saline (maximum TDS of 11,300 mg/L). The median TDS concentration was 7,610 mg/L, which is characterised as brackish.

Groundwater quality is discussed further in the context of contamination in Section 7.4.

6. GROUNDWATER INFLOW AND DRAWDOWN

6.1 OVERVIEW

A 2D cross section model was developed to predict potential groundwater inflow rates into the Sydney Olympic Park Station excavation and associated propagation of groundwater level drawdown.

The model was 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 generally 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. 5 m thick) and conceptualised to be characteristically similar to the Hawkesbury Sandstone.

6.3 ADOPTED HYDROGEOLOGICAL PARAMETER VALUES FOR MODELLING

Hydrogeological parameter values adopted for the modelling were as per the adopted representative values outlined in Table 5, Section 3.3.

The horizontal hydraulic conductivity values adopted for modelling are considered somewhat conservative.

6.4 GROUNDWATER INFLOWS

6.4.1 INFLOW RATES

Groundwater inflow rates to the station excavation calculated by the model are shown in Figure 6-1 and were up to 55 m^3/d.

As shown in Figure 6-1, 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 occurs instantaneously (the excavation is "wished in place") in the model. In practice, the excavation would deepen progressively, and peak groundwater inflows would be lower than those reported here.

MODELLED PEAK GROUNDWATER INFLOW RATES ARE COMPARED TO THE PARTICULAR SPECIFICATIONS IN

Table 8. The modelled peak groundwater inflow rates are below the Particular Specification criteria for the station excavation as a whole.

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

Sydney Olympic Park Station is on the periphery of the Homebush Bay Fault Zone. Geotechnical interpretations indicate that possible faults (see Figure 3-1) may be present in the vicinity of the site, likely represented as joint swarms. Despite this, currently available packer testing data does not indicate that the shale is of relatively high permeability. There is a possibility that hydraulic conductivity values may be relatively higher in the vicinity of joint swarms, faults 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 Geotechnical Interpretive Report interprets a fault zone to be present at the northwestern corner of the station box excavation. The interpreted width of this zone is approximately 10 m, intersecting the northern and western station box walls. Available packer test data in the vicinity of this feature (AF_BH42i) do not indicate significantly elevated permeability of the rock. However, it is possible that the borehole did not intersect the water-bearing fractures of this geological feature. The total inflow limit to the station box excavation as required by the Particular Specification is not expected to be exceeded. However, it is possible that, should this feature be significantly water-bearing, it may cause localised inflows to exceed the Particular Specification that requires inflows of less than 15 m3/day over any square with an area of 10 m². In such a case, localised grouting during excavation may be required to limit groundwater inflows to the Particular Specification criteria.

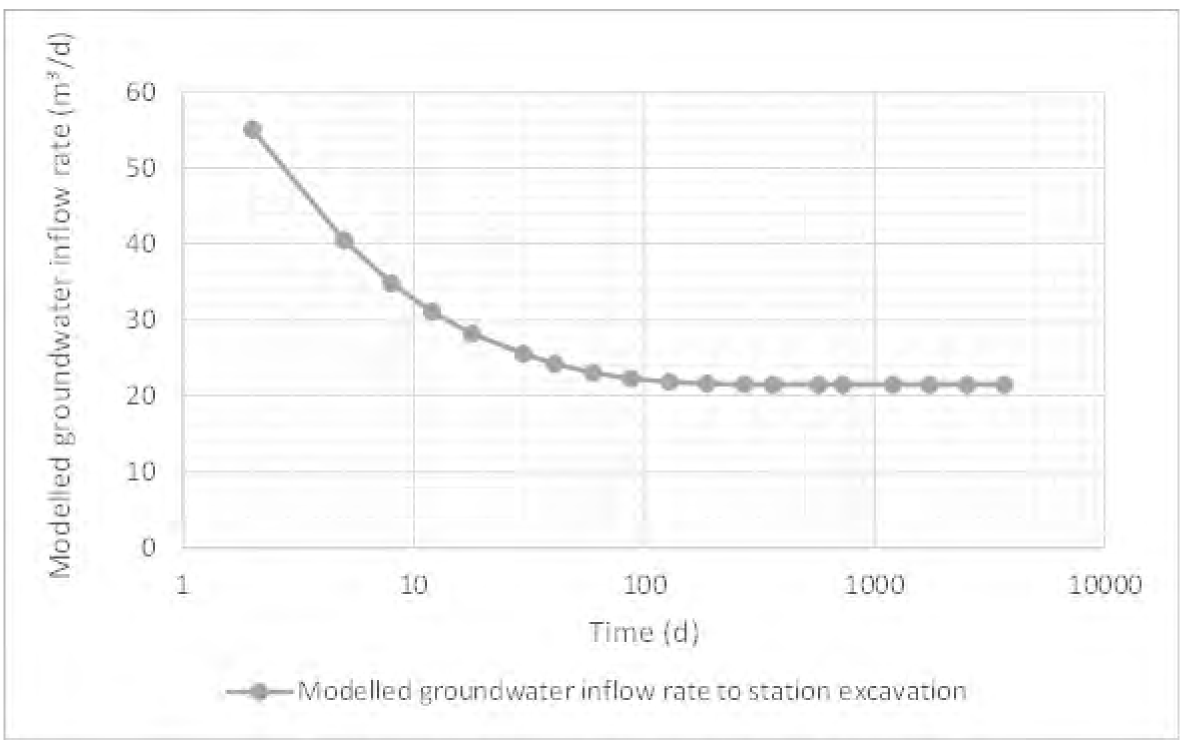


FIGURE 6-1: GROUNDWATER INFLOW RATES CALCULATED BY MODEL

TABLE 8: SUMMARY OF GROUNDWATER INFLOWS ESTIMATED BY MODELLING

Feature	Model-predicted groundwater inflow rate (m^3/d)	Maximum allowable inflow rate nominated in the Particular Specification (m^3/d)
Station excavation	Up to 55	139
Any square with an area of $10m^2$, at any and all locations within the sides and bases of the shafts and excavations	Not modelled. Inflows over a given 10 m^2 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 in Table 9. The cumulative inflow calculated by the model is less than the EIS prediction.

TABLE 9: 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)
16	25

6.5 DRAWDOWN AND COMPARISON TO EIS

Drawdown of the watertable predicted by the model is shown in Figure 6-2 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 in the model to lie within the Ashfield Shale.

Significant drawdown of the watertable is not expected in the alluvium that is interpreted to lie over 350 m to the east of the station site. Although Figure 6-2 indicates between about 4 m and 8 m of drawdown could occur at about 350 m from the station, such drawdown would primarily occur in the Ashfield Shale and is unlikely to manifest in material drawdown within the alluvium.

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 6-2.

The predicted drawdown is larger than the drawdown predicted in the EIS, in both maximum drawdown at the station and also areal extent.

At the station, the predicted drawdown is 3.5 m larger than predicted in the EIS. To the southwest and northeast of the station, respectively, the 2 m drawdown level extends about 535 m and 240 m further from the station than the 2 m drawdown contour predicted in the EIS.

Key differences between the model and the EIS model are summarised in Table 10, and are considered responsible for the differences between the modelled drawdown and the EIS predicted drawdown. The greater magnitude and more laterally extensive drawdown predicted by the model developed here for CTP works (compared to the drawdown predicted in the EIS) is likely due to the (1) deeper excavation level and wider station box footprint, (2) higher initial groundwater level prior to excavation, and (3) lower head/more distant boundary conditions.

There is a possibility that hydraulic conductivity values may be higher than the values modelled in the zone of possible joint swarms identified in the geological long/cross sections, and 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.

TABLE 10: SUMMARY OF KEY DIFFERENCES BETWEEN MODEL AND EIS MODEL

Element	Current model	EIS model
Target existing groundwater level (mAHD)	14	12
Station excavation floor (mAHD)	-5	-3.6
Station box width (m)	36	24
Northeastern boundary	Bennelong Pond, which connects to Powells Creek, 638 m from station, constant head boundary of 1 mAHD, applied to upper 1 m of model	Powells Creek, 1150 m from station, constant head boundary of 2 mAHD applied to whole model thickness
Southwestern boundary	Seepage face boundary applied at drainage line, 1870 m from station, boundary only applied to upper 1 m of model	No flow ridge, 1500 m from station
T7 line	Modelled	Not modelled

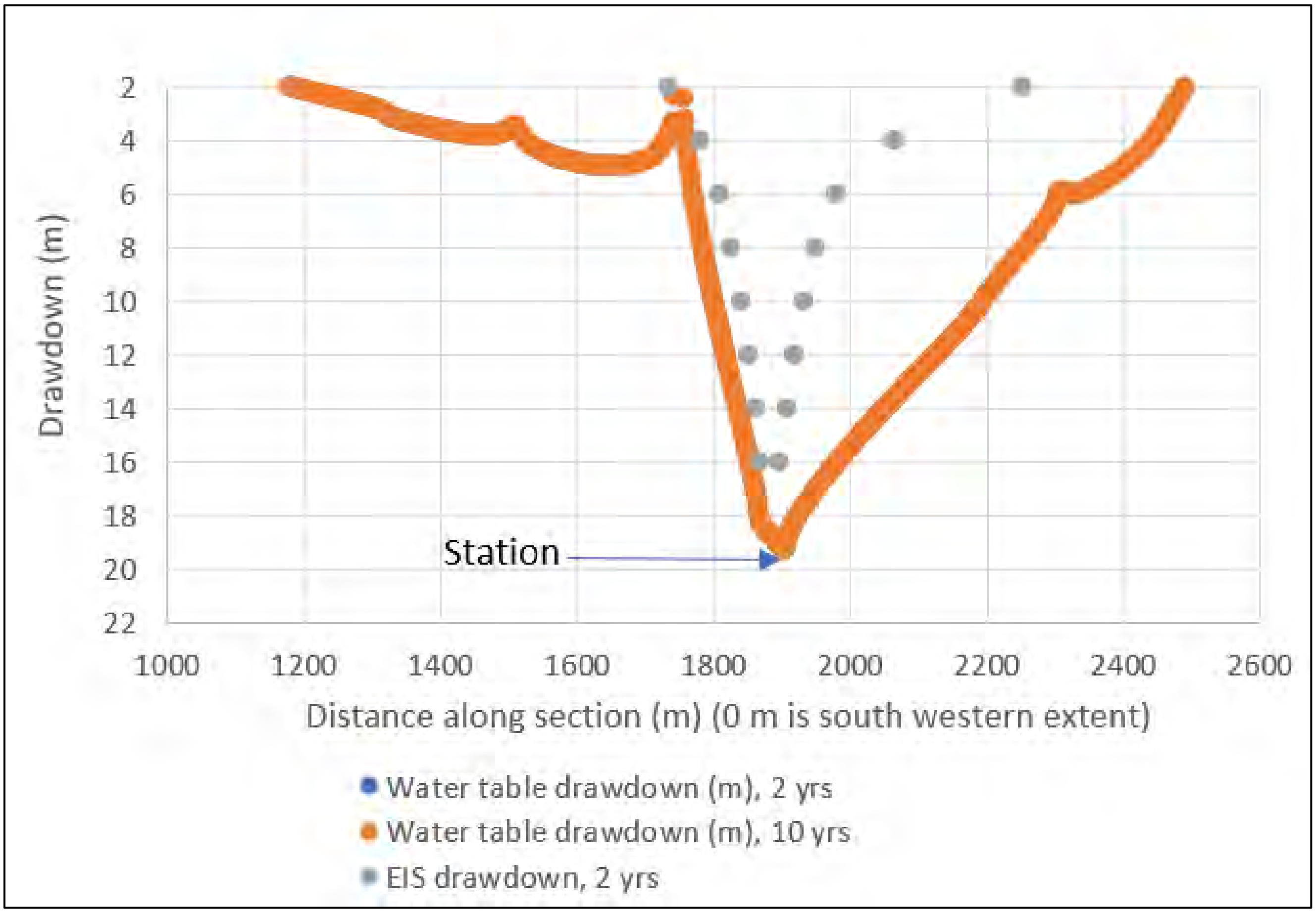


FIGURE 6-2: GROUNDWATER LEVEL DRAWDOWN PREDICTED BY MODEL COMPARED TO THAT IN EIS

7. GROUNDWATER IMPACTS

7.1 GROUNDWATER USERS AND RECEPTORS

Figure 7-1 below illustrates potential groundwater receptors surrounding Sydney Olympic Park Station and the drawdown predicted by the EIS (Jacobs, 2020).

With respect to existing registered groundwater bores, although the modelled drawdown is greater in extent than the drawdown predicted in the EIS, there are no existing registered bores within the predicted extent of drawdown except for monitoring bores.

The EIS (Jacobs, 2020) indicates swamp/wetland groundwater dependent ecosystems (GDEs) are present from about 650 m northeast and southeast of the station. Predicted drawdown at the northeastern model boundary, located about 640 m from the station, is about 0.3 m. This predicted drawdown is unlikely to occur in practice, as the wetland is relatively large and connected to Powells Creek / Parammatta River, and the CTP works will not reduce the level of the surface water within these systems. As such, drawdown associated with the CTP works is considered unlikely to impact the GDEs.

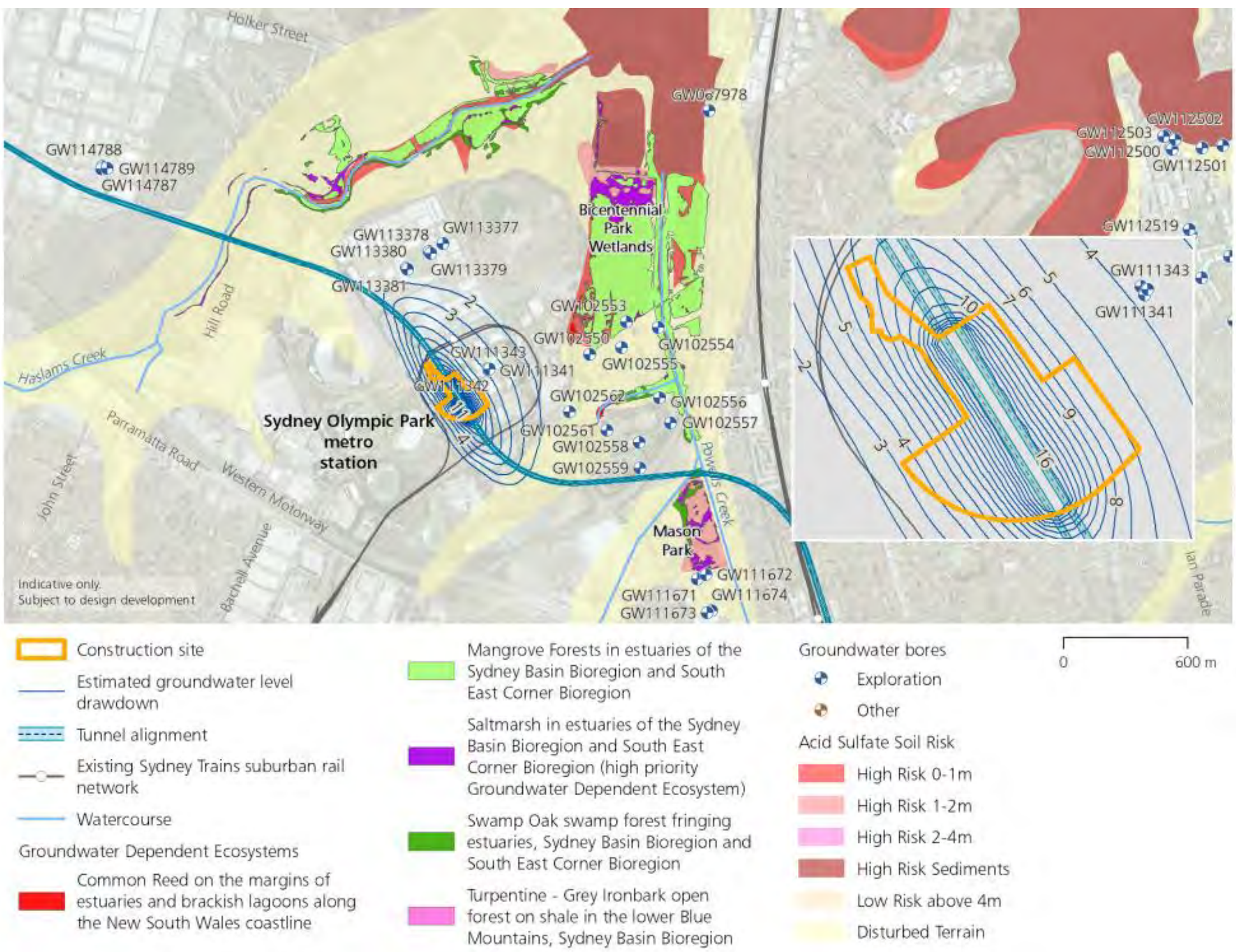


FIGURE 7-1: GROUNDWATER RECEPTORS NEAR SYDNEY OLYMPIC PARK STATION, AND DRAWDOWN PREDICTED IN THE EIS (JACOBS, 2020)

7.2 ACID SULFATE SOILS

The Contamination Assessment Report indicates that based on the site's elevation and underlying geology, there is a low potential for the occurrence of actual or potential acid sulfate soils, and that this is further supported by NSW Government online mapping (eSPADE) that does not identify the site as lying within an acid sulfate soil risk area.

Fill exists in the area of Sydney Olympic Park. If perched or permanent groundwater systems reside in the fill, it is possible that groundwater level drawdown could cause oxidation of acid sulfate soils, if they are present within the fill. However, this is considered a low risk as significant perched groundwater systems within the fill have not been identified, and based on piezometer monitoring, permanent groundwater was only identified to reside in fill at one location, SMW_BH126_w.

Additionally, groundwater systems residing in fill, if present, are likely to be poorly hydraulically connected to the Ashfield Shale groundwater system, the unit considered most relevant to assessment of groundwater inflows and drawdown at the site.

At distance from the station, the EIS (Jacobs, 2020) indicates high risk sediments are present from about 650 m northeast and southeast of the station. As discussed in Section 7.1, groundwater level drawdown is not anticipated at these locations based on the model predictions, and because these areas are expected to be hydraulically connected to the Parramatta River. Additionally, groundwater systems residing in alluvium are likely to be poorly hydraulically connected to the Ashfield Shale groundwater system, the unit considered most relevant to assessment of groundwater inflows and drawdown at the site.

There is a low risk that the CTP works could oxidise sediments at distance from the site via groundwater level drawdown. If the sediments are actual or potential acid sulfate soils, then pH and dissolved oxygen levels could be lowered and heavy metals/metalloids released.

7.3 SETTLEMENT

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

7.4 CONTAMINATION

7.4.1 GROUNDWATER SEEPAGE TO STATION BOX EXCAVATION

Based on the Contamination Assessment Report, groundwater seepage is likely to require dilution or treatment prior to discharge to surface waters. Groundwater quality treatment or dilution would also require consideration if the discharge is to stormwater or sewer. The base case modelled groundwater inflow rate is considered low (0.06 L/s), particularly given the evaporation could be about 0.15 L/s if half of the excavation floor area is assumed to behave as an evaporative surface. It is possible that the low groundwater inflow rate and relatively high evaporation rate could negate the need for groundwater discharge. However, higher groundwater inflow rates, as demonstrated by the high permeability scenario, are possible, and exceed the evaporation rate of 0.3 L/s, thereby necessitating the need for discharge (or re-use) of the groundwater.

7.4.2 LANDFILLS

The Sydney Olympic Park Station excavation will act as a groundwater sink. It is therefore possible that contaminated groundwater at distance from the site, such as that associated with nearby former landfills, will be drawn towards the excavation.

Figure 7-2 shows the landfills in the vicinity of the station as identified by the EIS (Jacobs, 2020). The Area of Environmental Interest (AEI) number of each landfill site in EIS is also shown in red.

The two closest landfills to the station are the Aquatic Centre Carpark Landfill (AEI 31), located some 180 m from the station box; and the Former Golf Driving Range Landfill (AEI 32), located some 190 m from the station box.

Sydney Metro's response to the JTJV RFI titled "Sydney Olympic Authority Landfill leachate and groundwater data" (JTJV reference SMWSTCTP-JTJ-SWD-TU350-EV-RFI-1059, i2CX reference DRFI#0066) included information on the management of the landfills.

Sydney Olympic Park Authority's Remediated Lands Management Plan from January 2009 and an extract from the Remediated Lands Management Plan for the Golf Driving Range Landfill (date unknown).

The Aquatic Centre Carpark landfill (AEI 31) comprises two waste cells that were sealed with a double-layered liner system designed to entirely contain the waste and any leachate generated. The cells contain gypsum, aluminium slag, gasworks residue, building rubble, asbestos and potential PFAS. The integrity of one of the cell's liners was under investigation in 2009. The outcome of the investigation is unknown. The key leachate contaminants reported are ammonia and cyanide.

The Former Golf Driving Range Landfill (AEI 30) was a consolidation of three separate landfills from the State Sports Centre precinct, and includes various waste types (including putrescible waste). The waste is contained by a residual clay base, low permeability clay cap (reportedly 650 mm thick) and a clay cut off wall with a bentonite liner along the northern edge of Boundary Creek. The landfill includes a subsurface leachate and landfill gas management system. A study in December 2008 identified that there were no containment issues associated with the landfill. The key leachate contaminants reported are ammonia and BOD, although heavy metals, nitrate, PAH, sulfate and TPH are also reported in the leachate quality data from 2007.

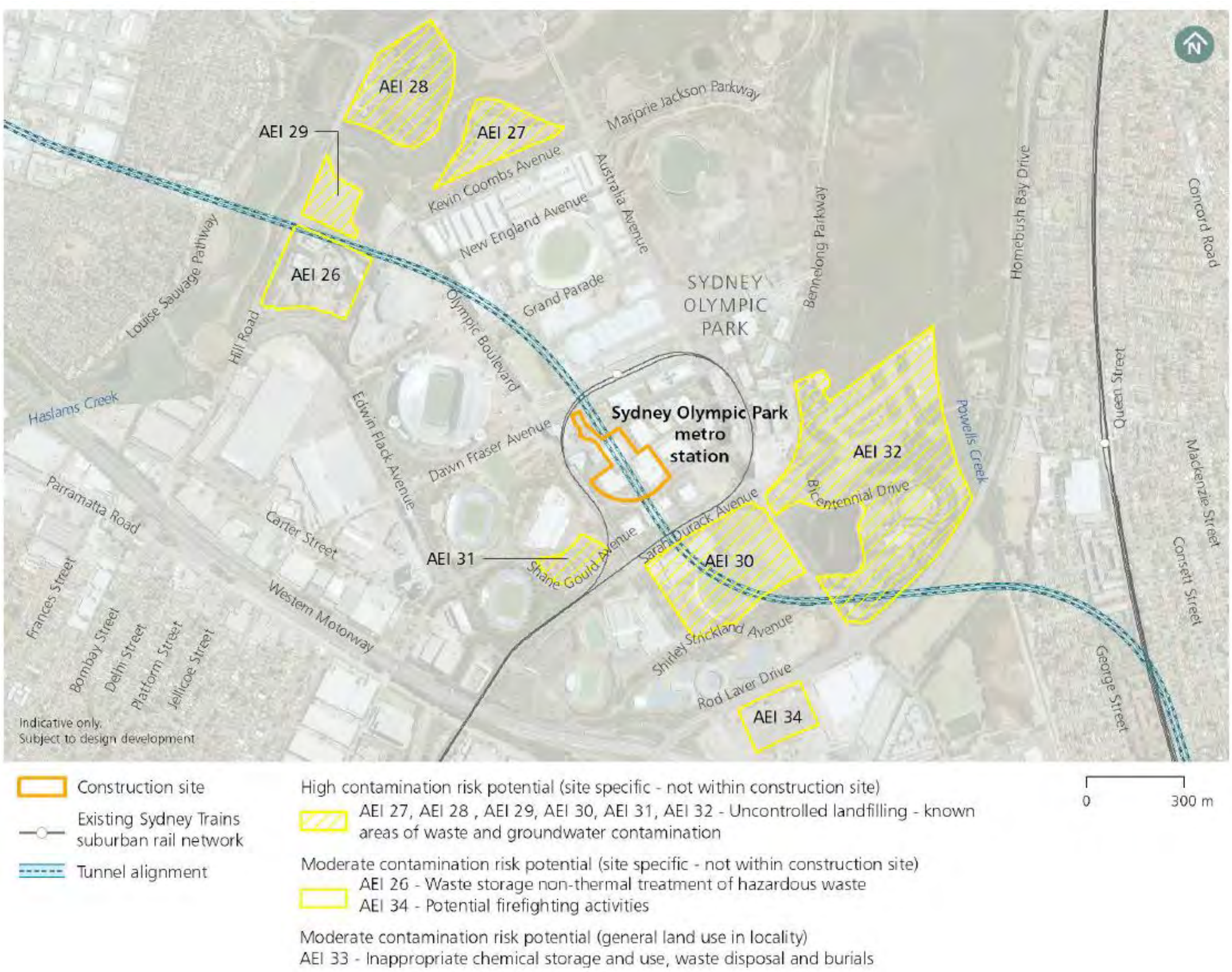


FIGURE 7-2: LANDFILLS IN THE VICINITY OF SYDNEY OLYMPIC PARK STATION IDENTIFIED IN THE EIS (FROM JACOBS, 2020)

Piezometers in the vicinity of the landfills are shown in Figure 7-3. There are only piezometers present in the AEI 30 landfill site. Groundwater quality data are available for these piezometers, with the exception of AF_BH38 which was yet to be sampled at the time of reporting. The Contamination Assessment Report provides summaries of the groundwater quality data for these piezometers.

Of the piezometers located within the AEI 30 landfill footprint, piezometer SMW_BH126 is screened across the deeper fill and shallow shale, and has likely been impacted by landfill leachate, as indicated by elevated ammonia (267 mg/L), PFAS (0.34 µg/L), and chloroform (61 µg/L). It is unknown how representative this is of conditions more generally in the area.

Piezometers SMW_BH033 and SMW_BH069 are screened from some 7 m and 10 m, respectively, below the top of shale. Considering that there is unlikely to be a significant vertical (downward) groundwater flow gradient at SMW_BH033 and SMW_BH069, it’s unlikely that groundwater at these depths has been impacted by the landfill.

Piezometer AF_BH37s (screened in fill) shows elevated concentrations of some metals, ammonia, PFAS and methane, suggesting that the fill is impacted by the waste materials. Piezometer AF_BH37w (screened from 5 m below the top of shale) shows lower concentrations of these analytes than AF_BH37s (screened in fill), suggesting that the shallow shale may not be significantly impacted by the landfill leachate in the fill.

FIGURE 7-3: LANDFILLS AND GROUNDWATER MONITORING LOCATIONS IN THE VICINITY OF SYDNEY OLYMPIC PARK STATION (LANDFILL AEI NUMBER IN RED)

Given that the details of the landfill liners are unknown, an assessment of the potential timeframes for contaminated groundwater to reach the station from closest landfills (AEI 30 and 31) was undertaken. This assessment assumes that the liners are of equal or higher permeability than the natural siltstone and clay (as relevant), and that any leachate management system is not fully removing contaminated leachate. It is therefore a conservative assessment.

The assessment:

- Excludes sorption, dispersion and decay and is purely based on advection
- Adopts a hydraulic gradient based on the shortest distance between the station excavation and the landfill, and considers the head differential between the station and landfill to be equal to the difference between excavation floor level and ground surface.
- Considers available groundwater level data. There is no groundwater level monitoring in landfills AE I30 and AEI 32. Groundwater level monitoring data for shallow piezometer AF_BH38s in the fill within the AEI 30 landfill footprint was not available. However, the groundwater level recorded piezometer AF_BH37s (also located within the fill) was approximately 1.5 m AHD in January 2022, some 13 m below the ground surface of the landfill. All other piezometers in the area (SMW_ENV713_w, SMW_BH126_w, SMW_ENV715B, SMW_BH069, SMW_BH033), including outside the landfills, are screened in the deeper shale (with the exception of SMW_ENV81A for which there are no groundwater level data – this is a soil vapour monitoring location), and are unlikely to reflect the groundwater levels in the landfill fill material. The boreholes log for AF_BH38, located within the AE30 landfill fooprint, indicates a groundwater level was encountered around 5 m bgl. These may reflect groundwater levels in landfill fill AEI 30.

  Boreholes logs from boreholes drilled in the P3 carpark footprint in 2015 (reference site R336) show typical groundwater levels in the fill at approximately 4 mbgl.

  Boreholes logs from boreholes drilled immediately west of the P3 carpark in 2016 (reference site R709) show groundwater levels in the fill ranging from approximately 1.5 mbgl to 5 mbgl.

  The depth to the groundwater table of up to approximately 5 mbgl has been considered in the analyses as a conservative estimate (noting that it may be significantly deeper, such as over 10 mbgl as observed at AF_BH37s).

  The more conservative assumption that groundwater level lies at ground surface in the landfill has also been considered in the analysis, and has been adopted for locations where groundwater levels are not known at the landfill site (e.g., at AEI 31)

- Considers the adopted horizontal hydraulic conductivity for the Ashfield Shale nominated in Annexure X of 1.21×10-2 m/d (75th percentile value) and 6.05×10-3 m/d (K3D value) and effective porosity values of between 1% and 2%

The range parameter values reflects uncertainty in hydrogeological conditions. Based on these assumptions, there would be potential for contaminated groundwater within the closest landfills (AEI 30 and 31) to reach the station excavation between 3 years and 20 years post-excavation.

Potentially contaminated groundwater from other landfills in the area (AEI 26 to 29, 32 and 34) would not be expected to reach the station excavation within the 10 year design life.

Given that landfills AEI 30 and 31 have been designed with liners, and include a leachate management system, it is considered unlikely that contaminated groundwater from them would migrate to the station. However, it is noted that the integrity of one of the cell's liners at the Aquatic Centre Carpark landfill (AEI 31) was under investigation, suggesting a possible leak from the cell; and the permeability of the liner at the Former Golf Driving Range Landfill (AEI 32) is unknown.

In the case that contaminated groundwater did escape the landfill systems, it is possible that it could be drawn to the station box during the project's 10 year design life. This risk is considered low. There are no sensitive receptors in-between the landfill and station box that could be impacted by potential contaminant migration towards the station box. However, if contaminated groundwater were to enter the excavation, it could pose a potential human health risk. Contaminated groundwater in the vicinity of the station retaining wall and anchors could also have the potential to corrode concrete and steel elements.

The groundwater contamination risk must be re-assessed after additional information on the landfill designs is available, and when additional groundwater quality data is acquired.

7.4.3 SALINE INTRUSION

The maximum change in groundwater head between the nearest saltwater bodies, the Bicentennial wetlands to the east of the station, and the station box excavation is approximately 7 m. The wetlands are located over 500 m from the station. The hydraulic gradient induced by the station box excavation is insufficient to cause migration of saline water within the Ashfield Shale to reach the station site over the 10 year design life.

7.5 CUMULATIVE IMPACTS

With the exception of the Olympic Park Rail loop, there are no known significant drained structures in the vicinity the station excavation. The modelling undertaken in this assessment accommodates the existing groundwater level drawdown induced by the Olympic Park Rail loop. Based on this, cumulative groundwater level drawdown is not expected to be significantly different to the drawdown predicted in this assessment, and potential groundwater-related cumulative impacts are expected to be consistent with the impacts predicted for the CTP excavation works.

8. CONSTRUCTION PHASE MONITORING

Table 11 lists recommended groundwater level monitoring locations during the construction phase, and includes existing representative groundwater levels and predicted groundwater level drawdown. These locations are shown in Figure 8-1.

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 11: SUMMARY OF RECOMMENDED CONSTRUCTION PHASE MONITORING LOCATIONS AND PREDICTED DRAWDOWN

Location I.D.	Existing / proposed monitoring location	Easting	Northing	Ground level (mAHD)	Effective screen interval (m BGL)	Monitored formation	Approx. distance from station (m)	Existing representative groundwater level (mAHD)	Existing representative saturated thickness of aquifer in monitoring piezometer (m)	Predicted groundwater level drawdown, two years after excavation commenced (m)
SMW_BH019_w	Existing	321150	6252872	17.33	20.50 to 26.50	Ash	210, southwest of station	1.70	10.87	5.0
SMW_BH032_s	Existing	321477	6253030	19.76	3.30 to 7.25	Ash	155, northeast of station	<12.51 (dry)	NA – bore typically dry	NA – bore typically dry
SMW_BH032_w	Existing	321476	6253031	19.74	16.00 to 23.00	Ash	155, northeast of station	12.00	15.26	13.8
SMW_BH120_w	Existing	321260	6253045	17.38	20.00 to 26.50	Ash	100, northwest of station	0.40	9.52	10.7 a
SMW_BH126_w	Existing	321600	6252576	11.40	7.20 to 12.50	Fill and Ash	285, southeast of station	7.20	8.3	7.5 a
SMW_ENV714_w	Existing	321339	6252745	19.47	10.50 to 21.40	Ash	115, southwest of station	4.25	6.18	4.5

Notes: a Monitoring piezometer located perpendicular to model section. Average of drawdown to northeast and southwest of station adopted. NA means not applicable. Ash means Ashfield Shale

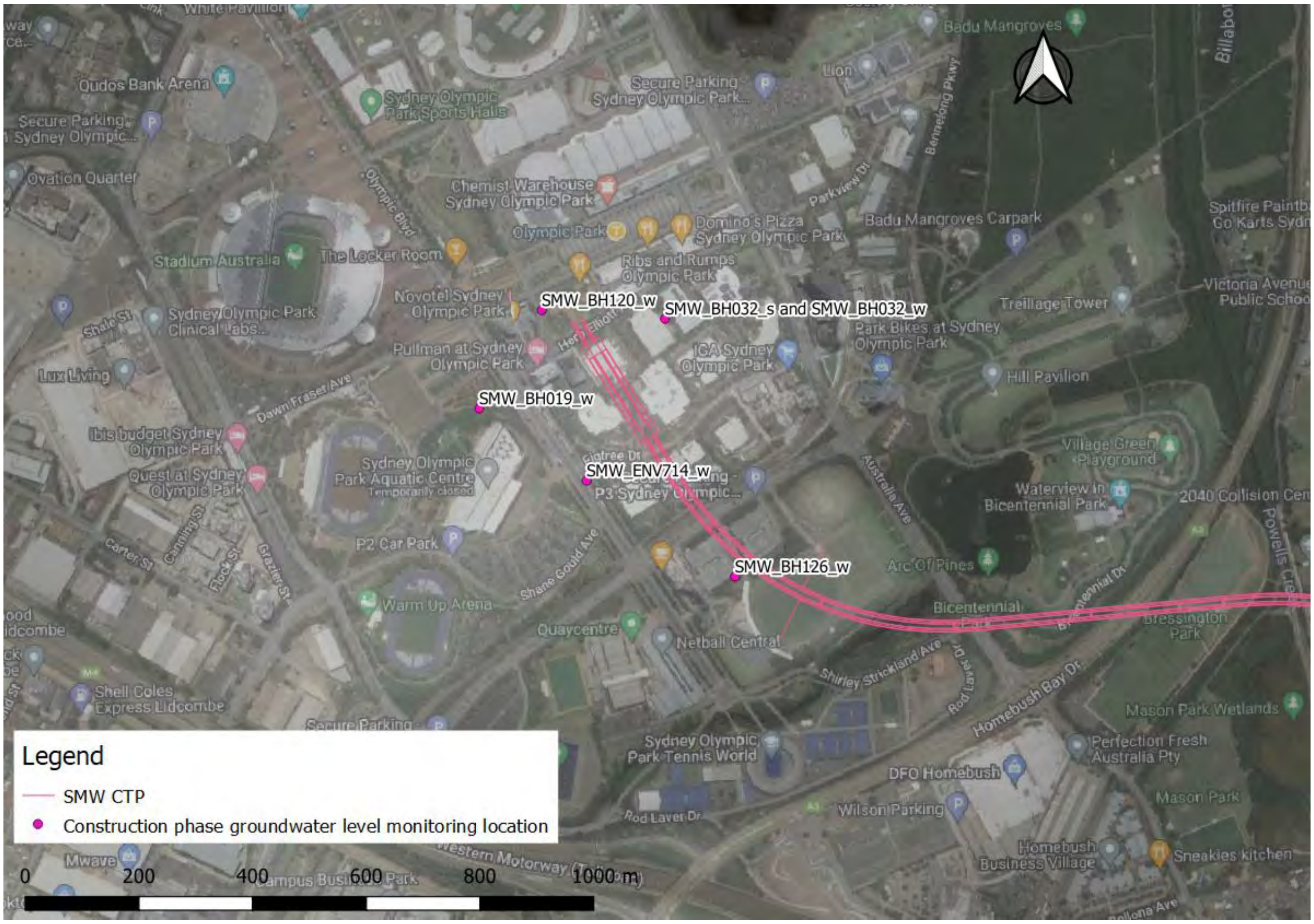


FIGURE 8-1: CONSTRUCTION PHASE GROUNDWATER LEVEL MONITORING LOCATIONS

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 (ten-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). See Annexure C for more details on this.

9.2 GROUNDWATER INFLOWS

Groundwater inflows to the station excavation of up to 55 m^3/d were predicted. These are below the Particular Specification criteria for the station excavation of 139 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.

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.

There is a possibility that hydraulic conductivity may be higher than the values adopted for the modelling, particularly in zones of possible joint swarms and faults, 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 16 ML. This is less than the cumulative inflow of 25 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 located to the north east and south east of the station box
- There is a low risk of disturbing acid sulfate soils via dewatering as significant drawdown within alluvium is not anticipated.
- The migration of contaminated groundwater is considered a low risk. However, there is some uncertainty regarding the design and performance of the landfills. The groundwater contamination risk must be re-assessed after additional information on the landfill designs is available, and when additional groundwater quality data is acquired.
- 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.

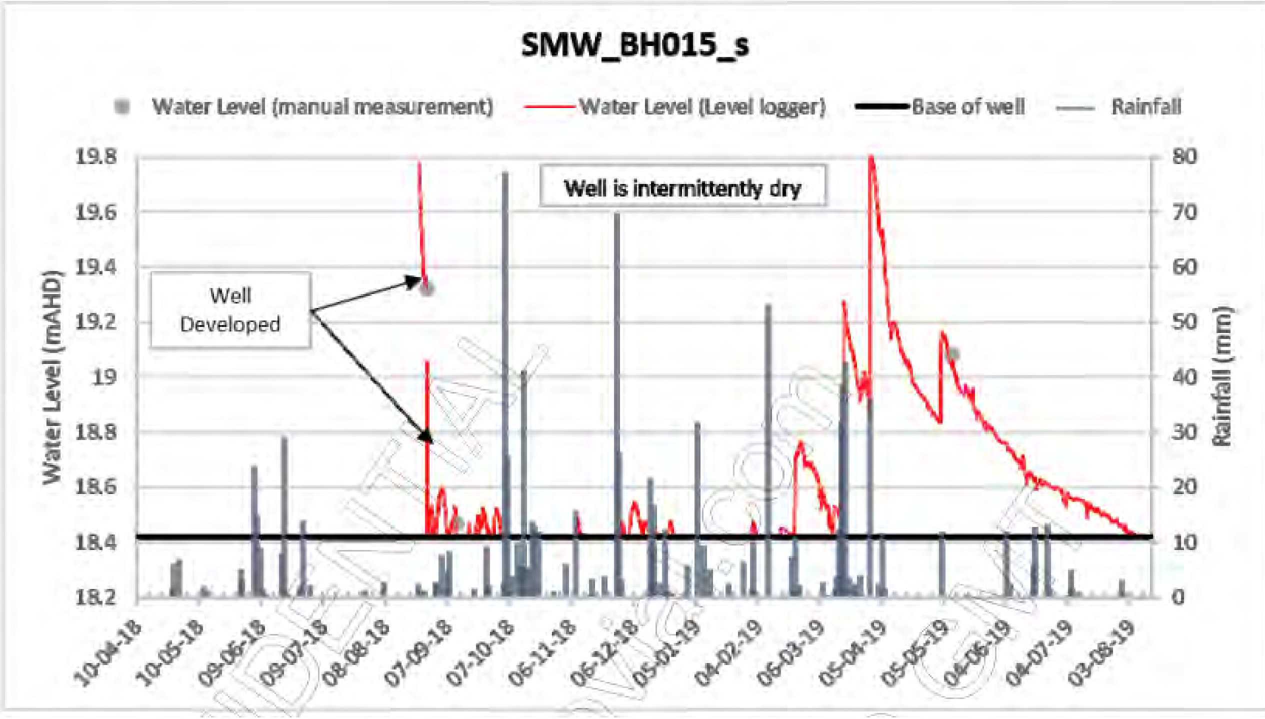
Recommended construction-phase groundwater level monitoring has been provided in this report to support verification during construction-phase that the potential groundwater-related impacts of the CTP works are consistent with expectations.

10. REFERENCES

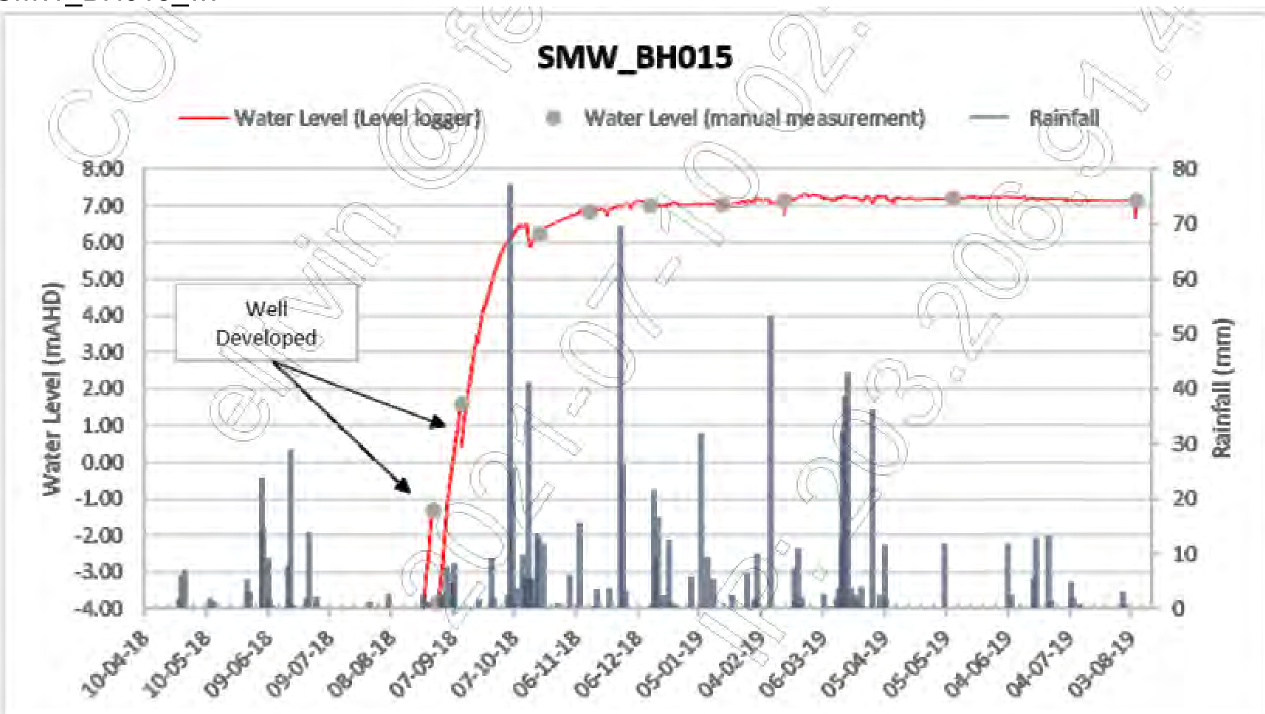
Jacobs (2020), Westmead to The Bays and Sydney CBD Environmental Impact Statement Concept and Stage 1, April 2020.

11. ANNEXURES

Annexure A. Hydrographs



SMW_BH015_w:



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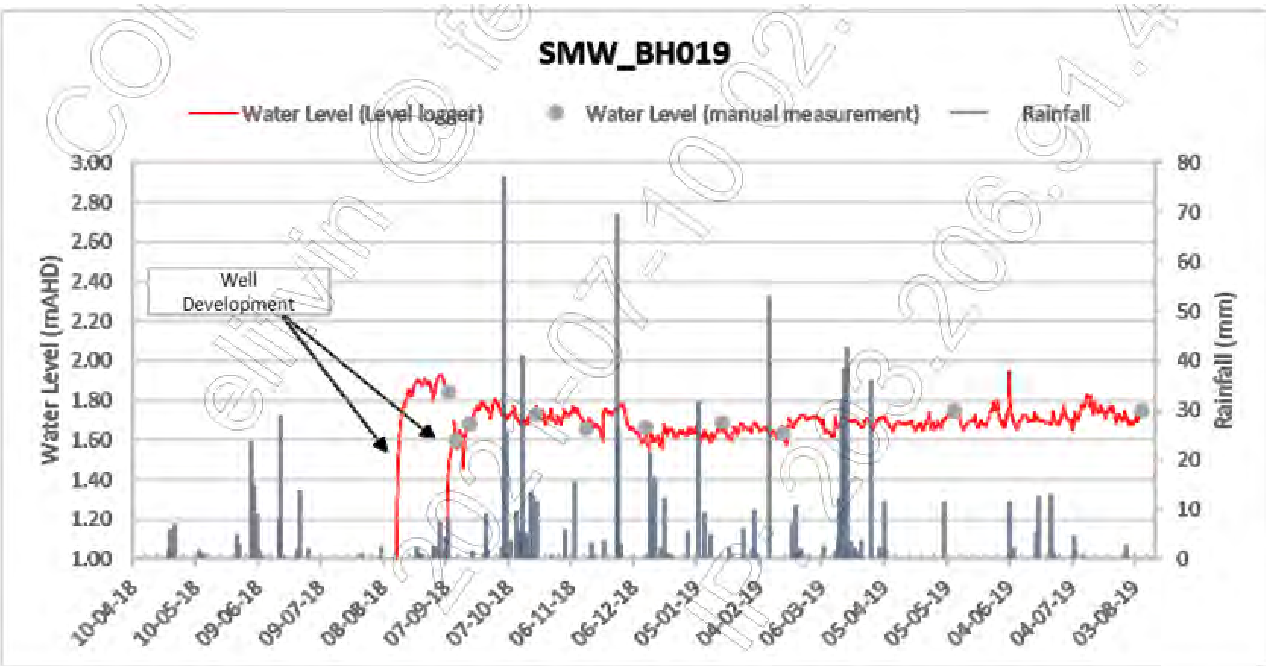
SMW_BH019
Water Level (Level logger)
Water Level (manual measurement)
Rainfall
Water Level (mAHD)
Rainfall (mm)
Well Development
3.00
2.80
2.60
2.40
2.20
2.00
1.80
1.60
1.40
1.20
1.00
80
70
60
50
40
30
20
10
0
10-04-18
10-05-18
09-06-18
09-07-18
08-08-18
07-09-18
07-10-18
06-11-18
06-12-18
05-01-19
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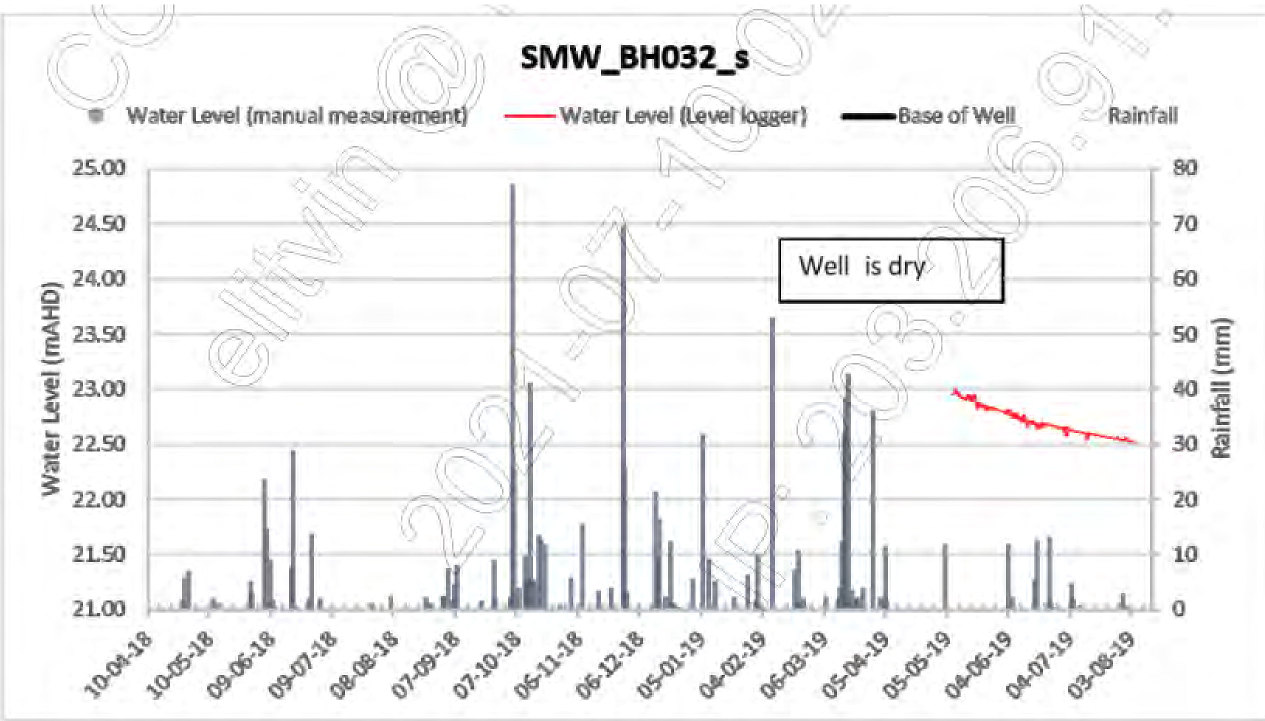
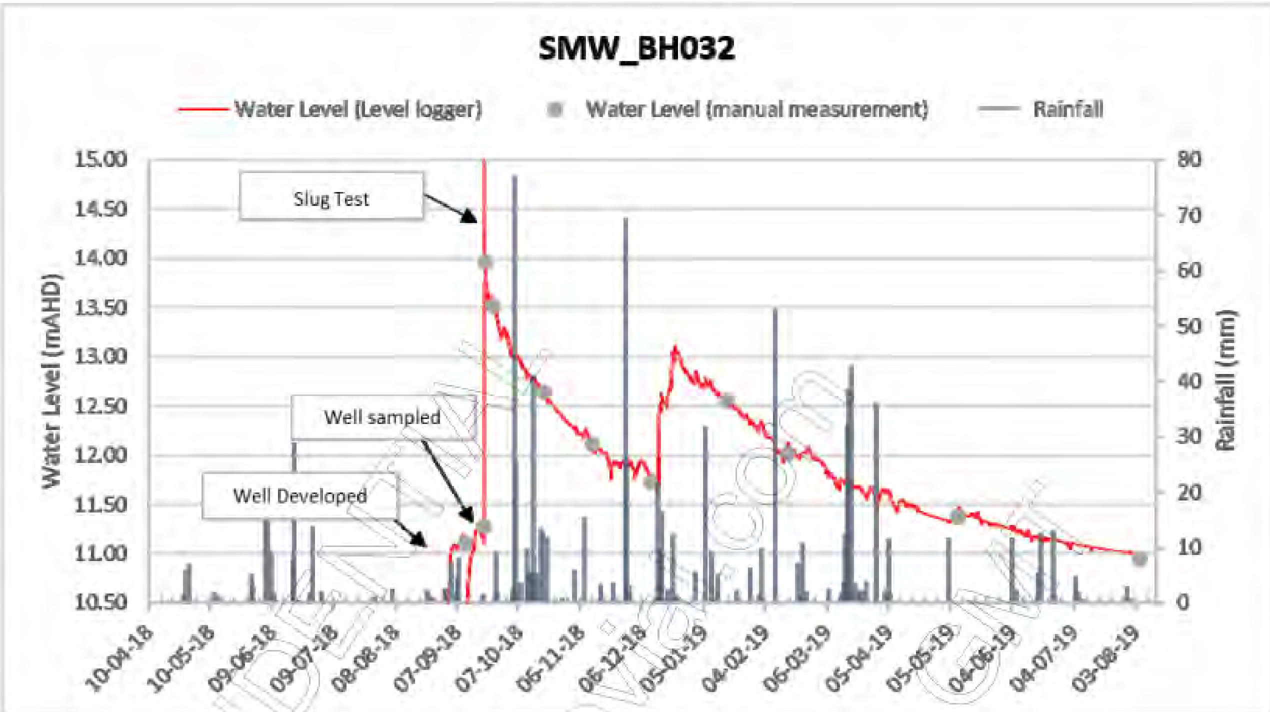
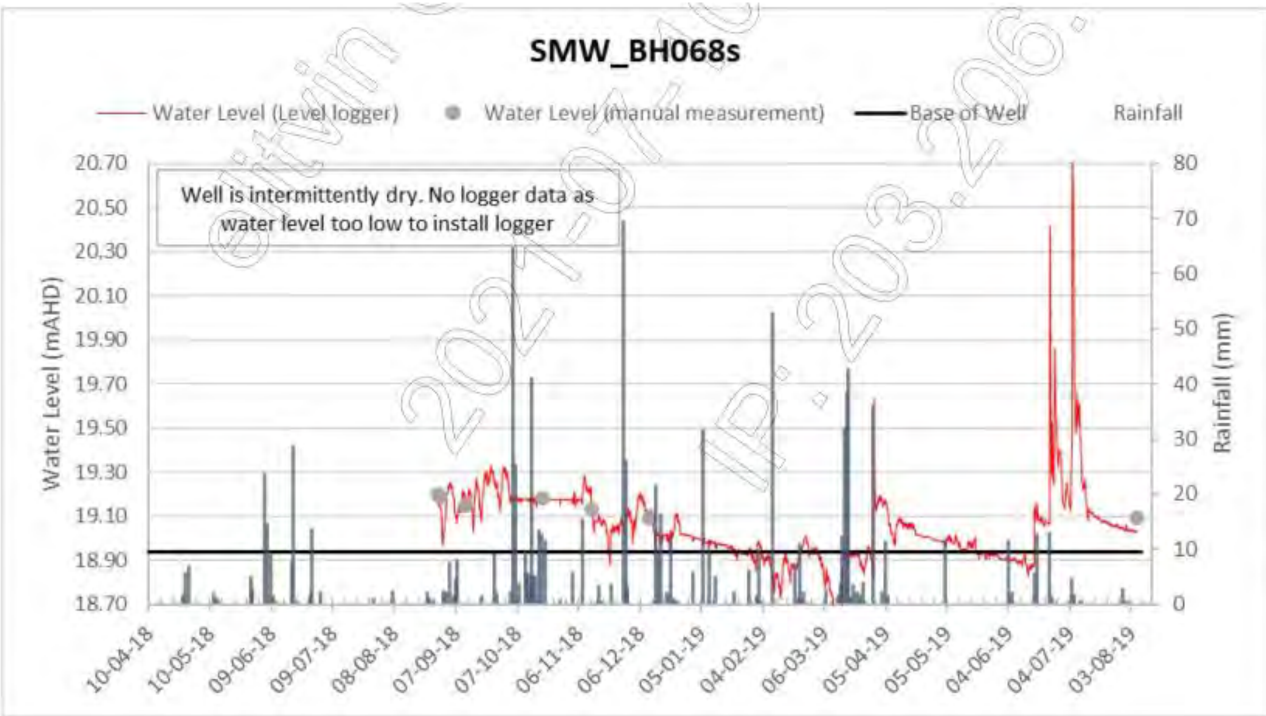
04-07-19
03-08-19

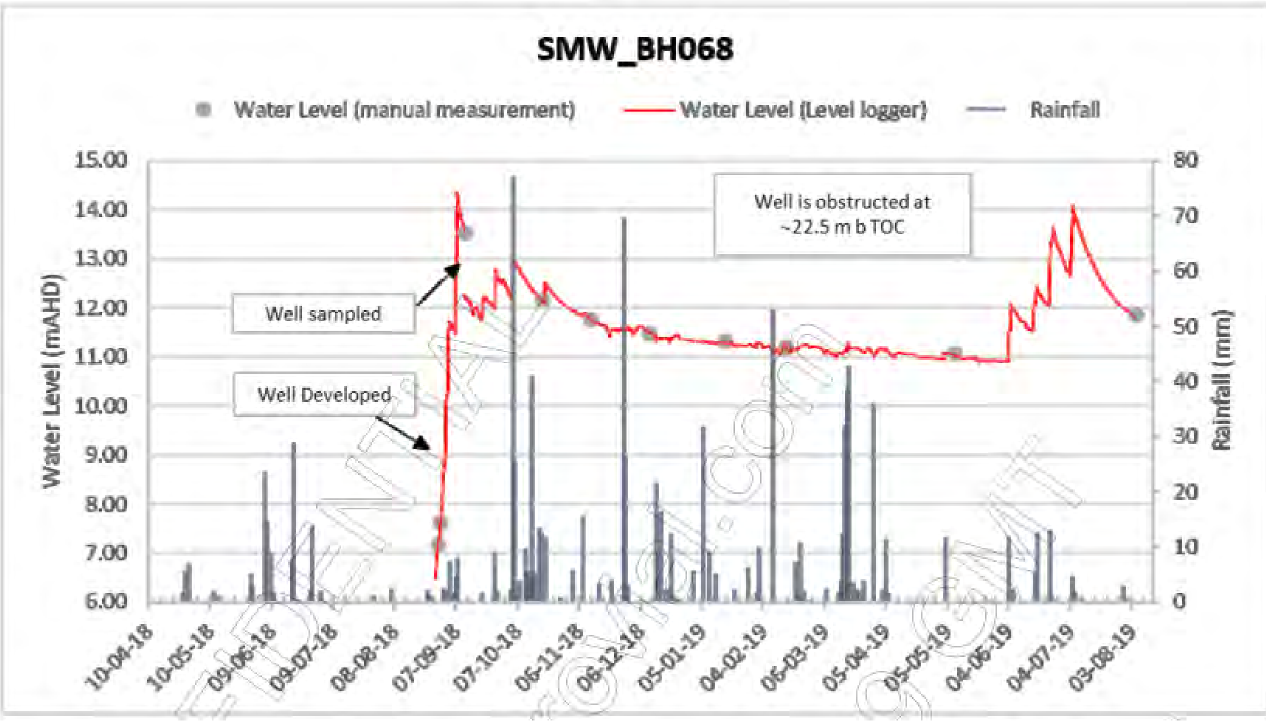

SMW_BH032_s
Water Level (manual measurement)
Water Level (Level logger)
Base of Well
Rainfall
Water Level (mAHD)
Rainfall (mm)
Well is dry

SMW_BH032_w:

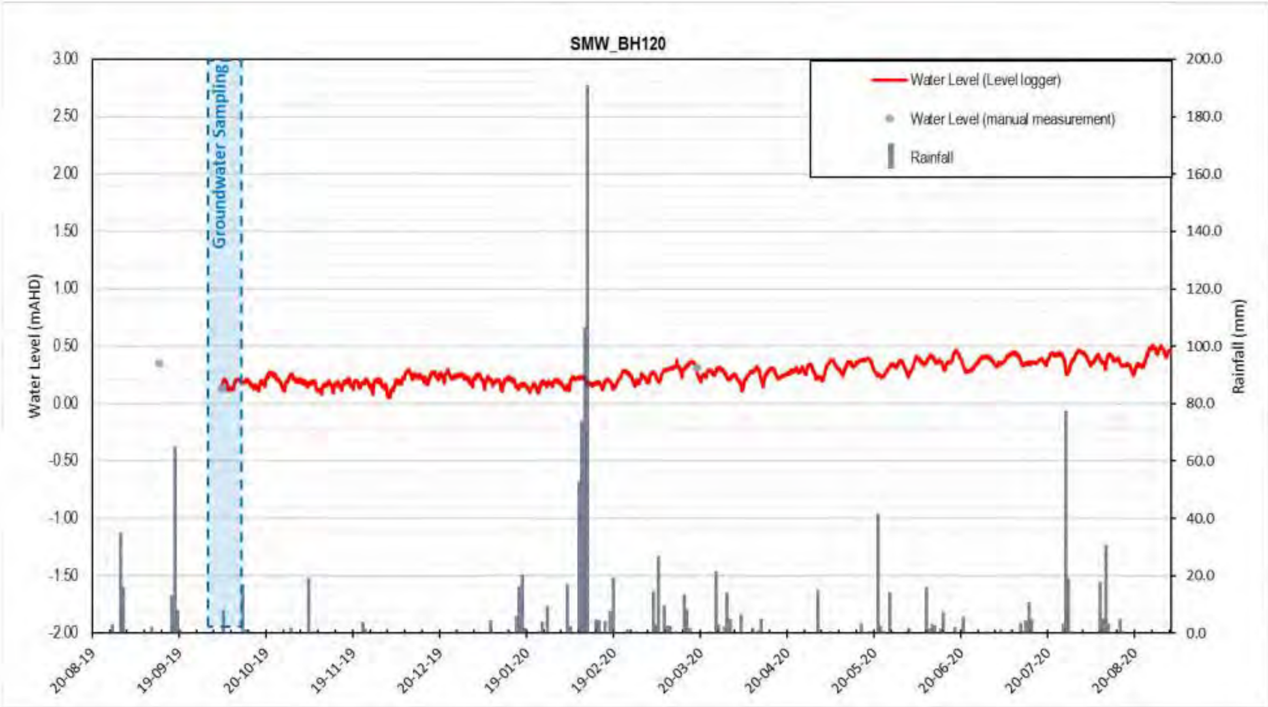

SMW_BH032
Water Level (Level logger)
Water Level (manual measurement)
Rainfall
Water Level (mAHD)
Rainfall (mm)
Slug Test
Well sampled
Well Developed


SMW_BH068s
Water Level (Level logger)
Water Level (manual measurement)
Base of Well
Rainfall
Well is intermittently dry. No logger data as water level too low to install logger
Water Level (mAHD)
Rainfall (mm)

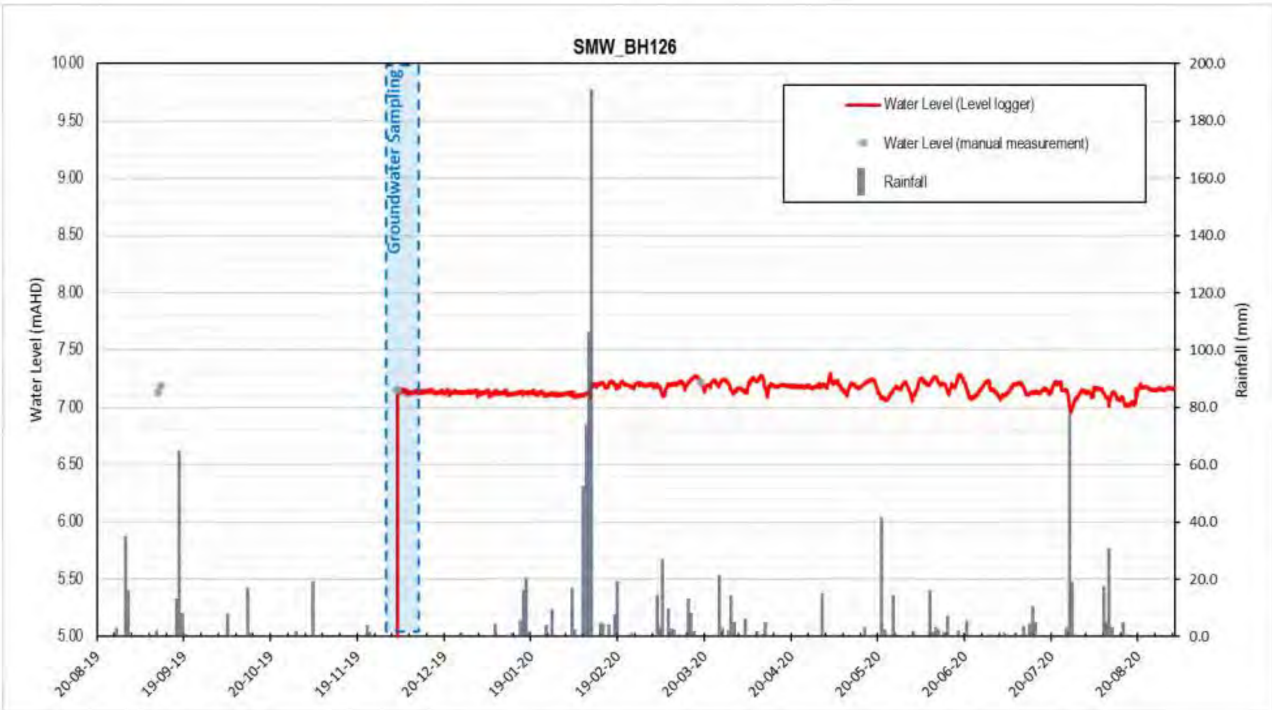
SMW_BH068_w:

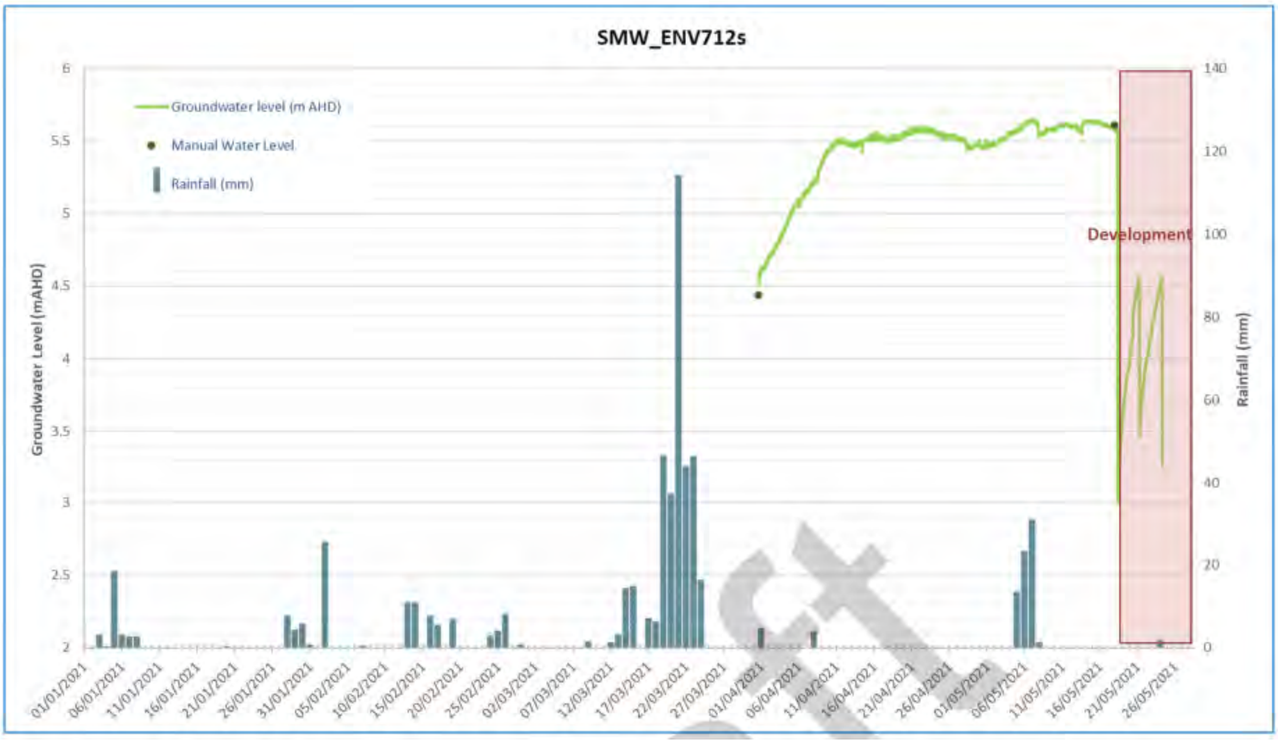

SMW_BH068
Water Level (manual measurement)
Water Level (Level logger)
Rainfall
Well is obstructed at ~22.5 m b TOC
Well sampled
Well Developed
Water Level (mAHD)
Rainfall (mm)

SMW_BH120_w:

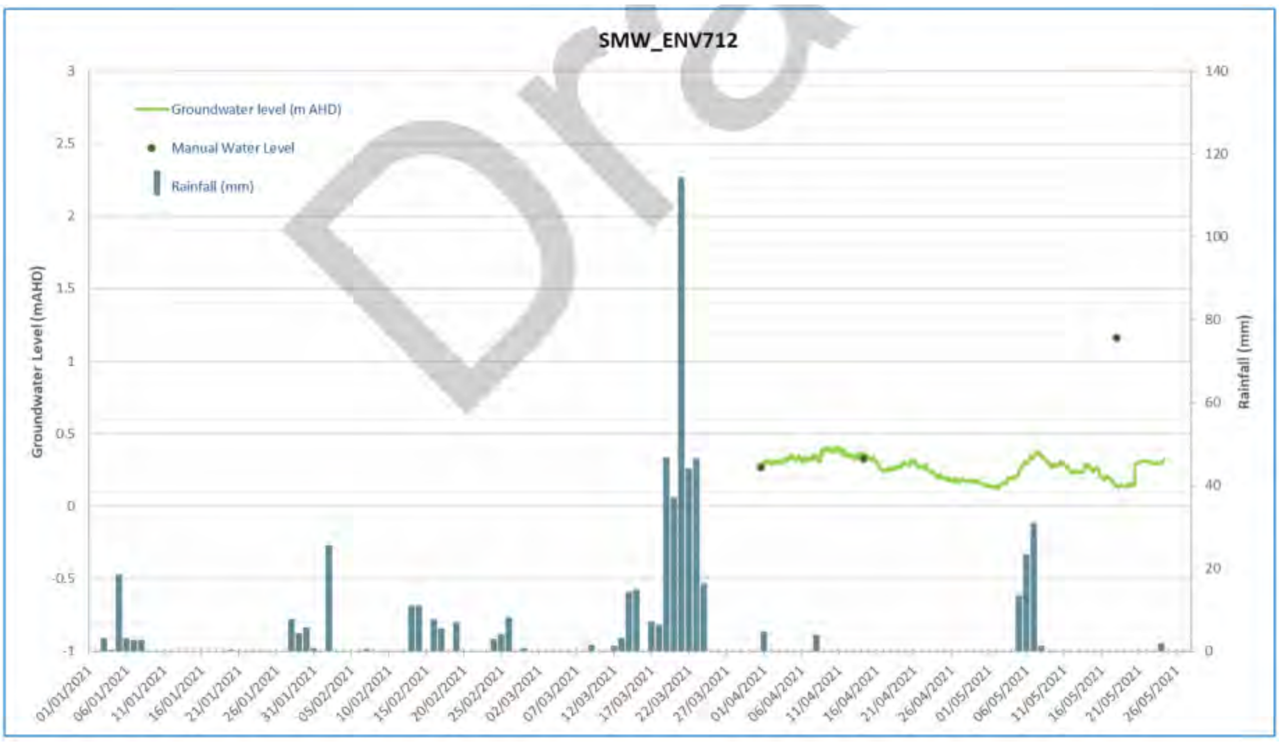


SMW_BH126_w:

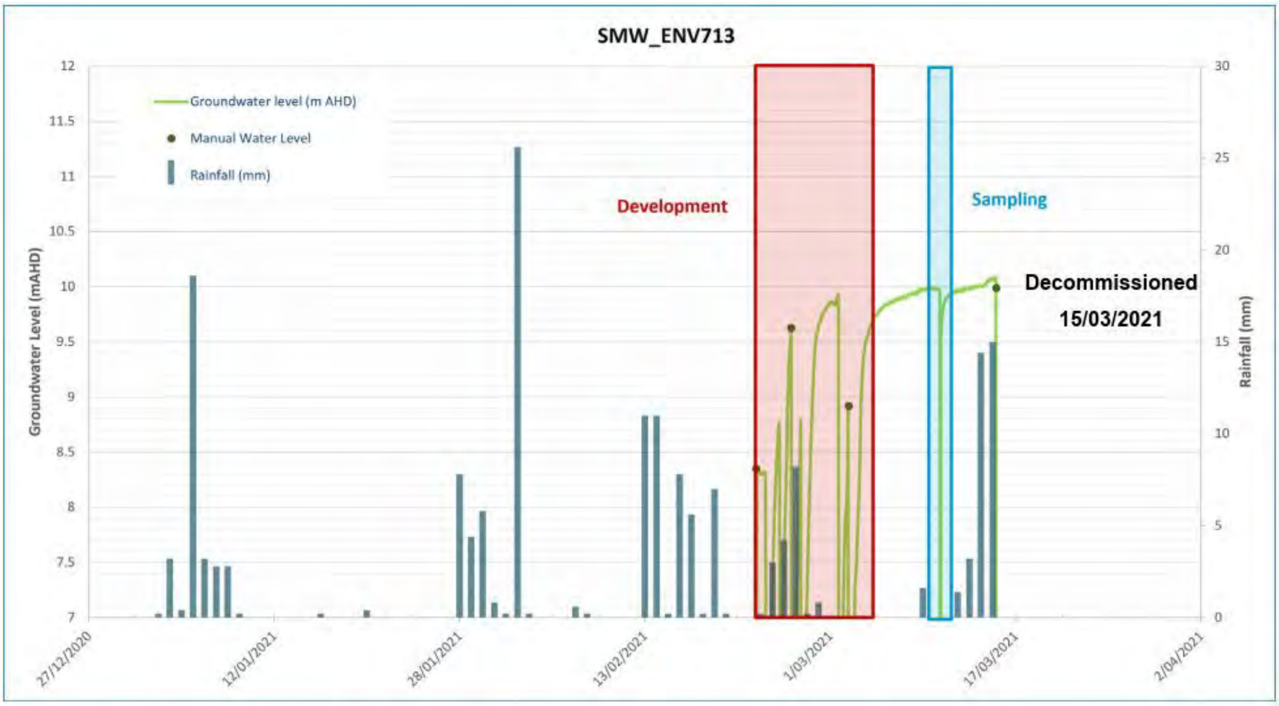



SMW_ENV712s
Groundwater level (m AHD)
Manual Water Level
Rainfall (mm)
Development
Groundwater Level (mAHD)
Rainfall (mm)

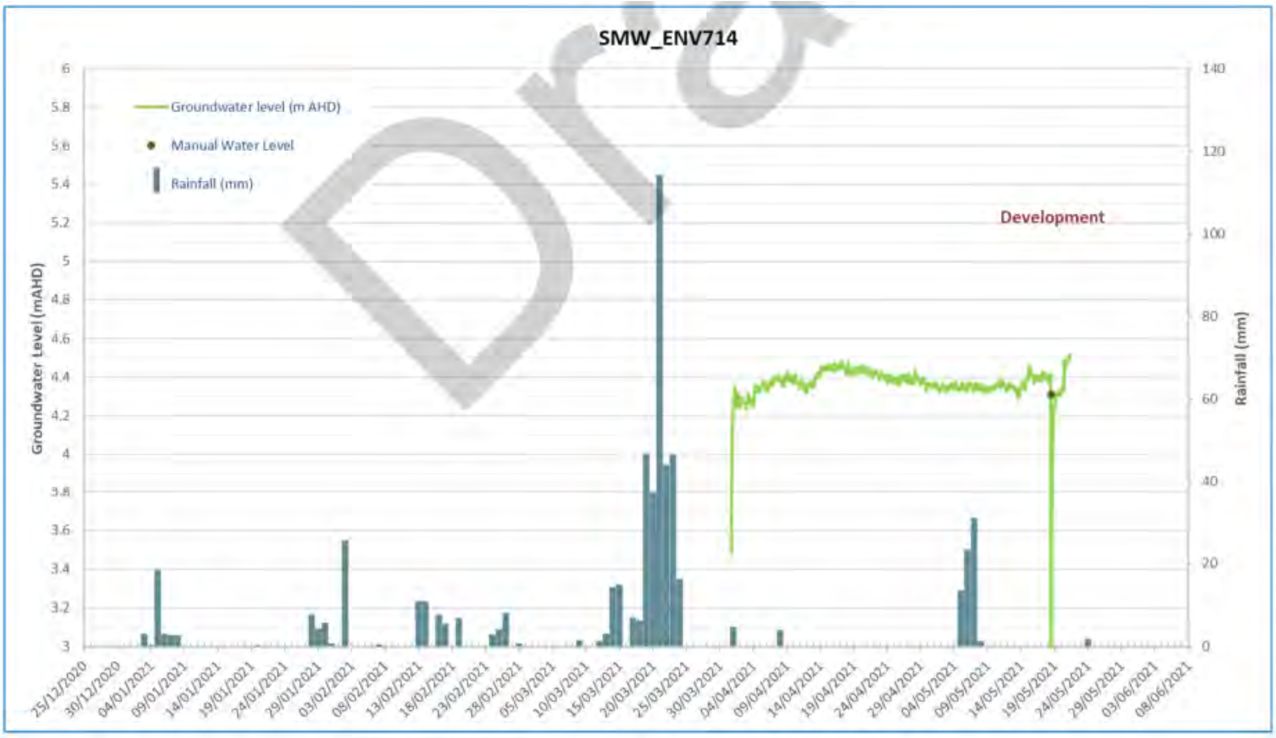
SMW_ENV712_w:

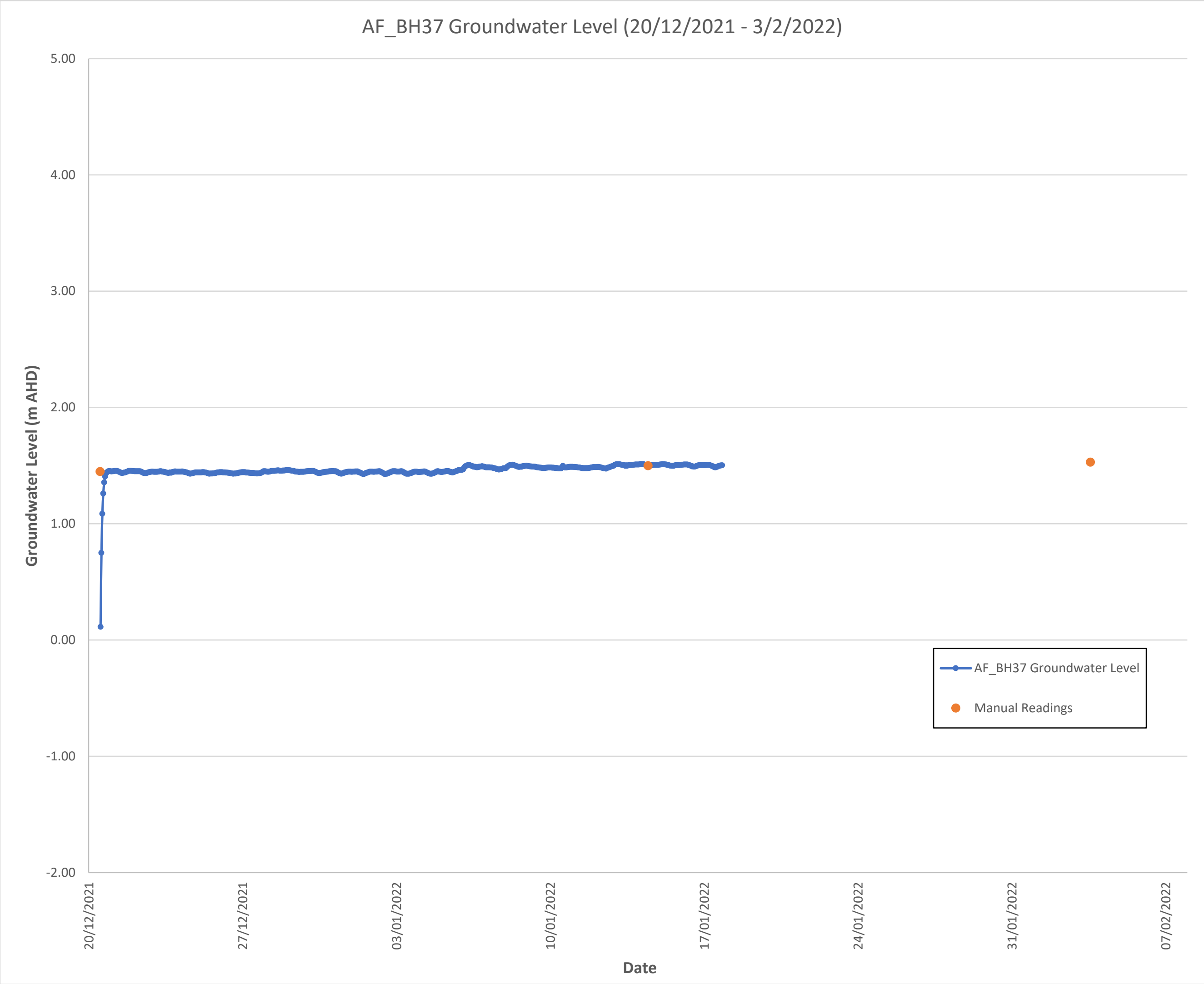

SMW_ENV712
Groundwater level (m AHD)
Manual Water Level
Rainfall (mm)
Groundwater Level (mAHD)
Rainfall (mm)

SMW_ENV713_w:



SMW_ENV714_w:




AF_BH37 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)
20/12/2021
27/12/2021
03/01/2022
10/01/2022
17/01/2022
24/01/2022
31/01/2022
07/02/2022
Date
AF_BH37 Groundwater Level
Manual Readings

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
B	8/03/2022	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

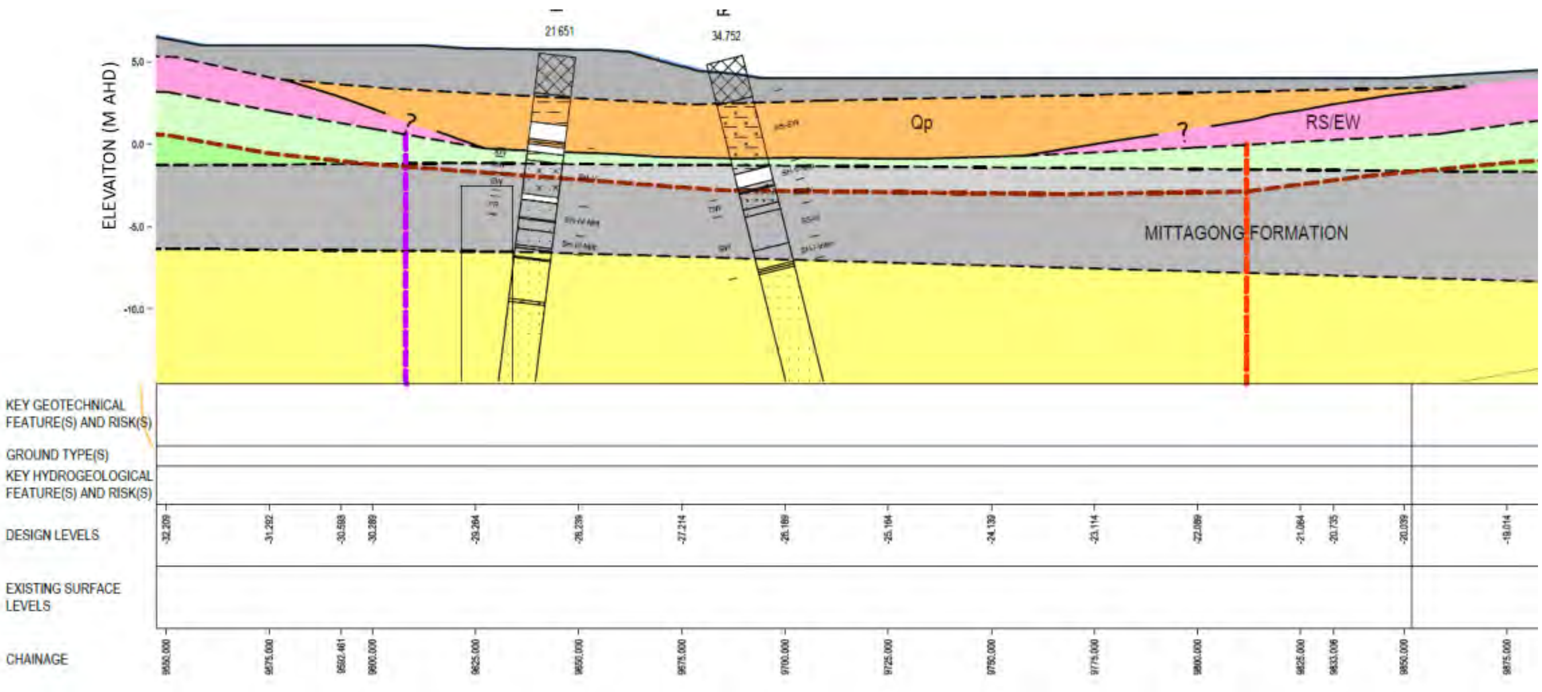


FIGURE 2-2: LOCATION OF ALLUVIUM AT BURWOOD NORTH STATION IN SECTION

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:
  - 36 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
  - 101 packer tests in siltstone and sandstone, supplemented with two packer tests undertaken for Western Harbour Tunnel
  - 176 packer tests in sandstone, 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 54 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.

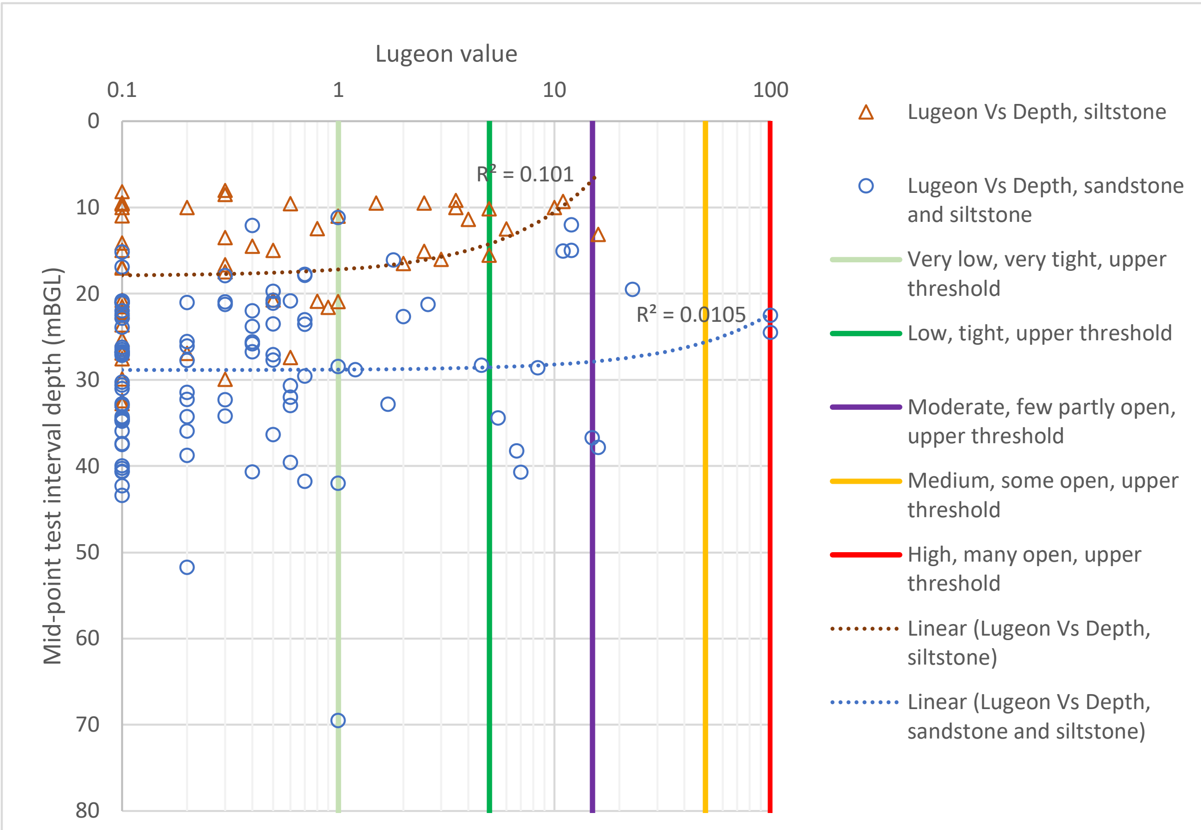


FIGURE 3-1: LUGEON VALUES FOR SILTSTONE TEST INTERVALS, AND SANDSTONE AND SILTSTONE TEST INTERVALS, CLASSED ACCORDING TO QUINONES-ROZO (2010)

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	27	0.6	0.8	2.6	5.20×10^{-3}	6.81×10^{-3}	2.28×10^{-2}
15 to <30 m	25	0.1	0.2	0.6	8.67×10^{-4}	1.99×10^{-3}	5.10×10^{-3}
30 to <45 m	2	0.6	0.5	0.6	5.20×10^{-3}	4.50×10^{-3}	5.20×10^{-3}

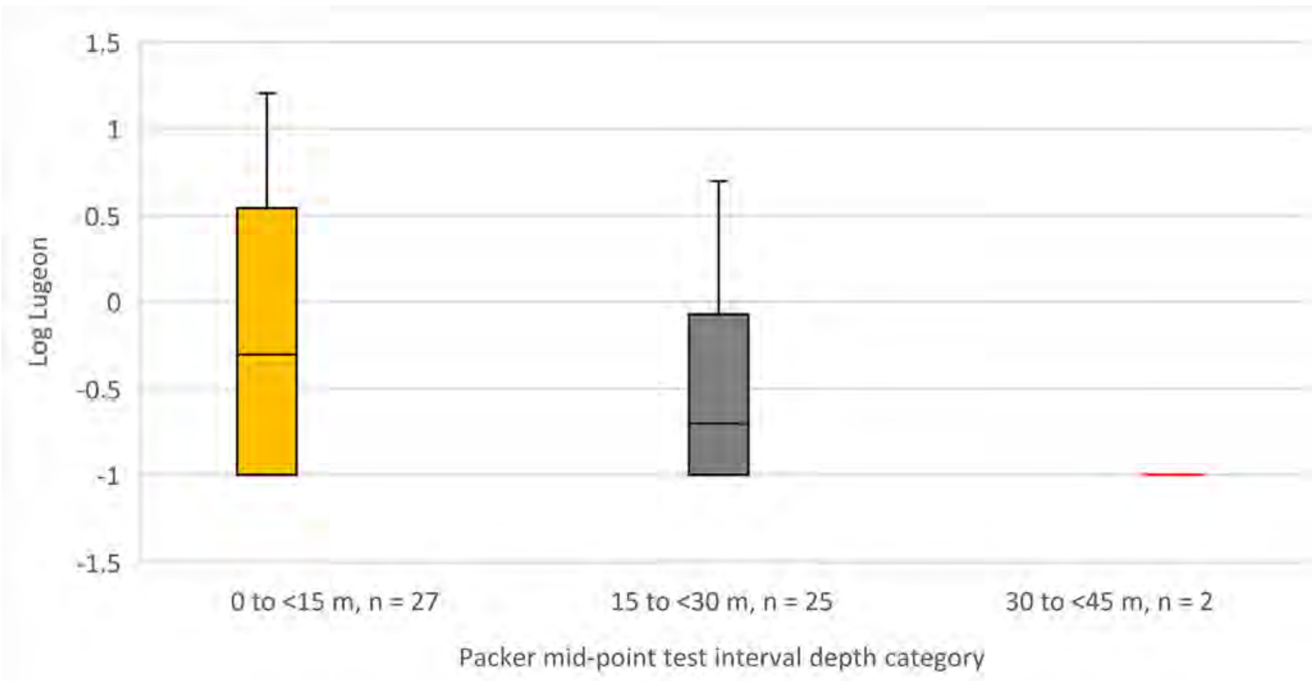


FIGURE 3-2: LOG LUGEON VALUES FOR SILTSTONE TEST INTERVALS BY DEPTH CATEGORY

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.

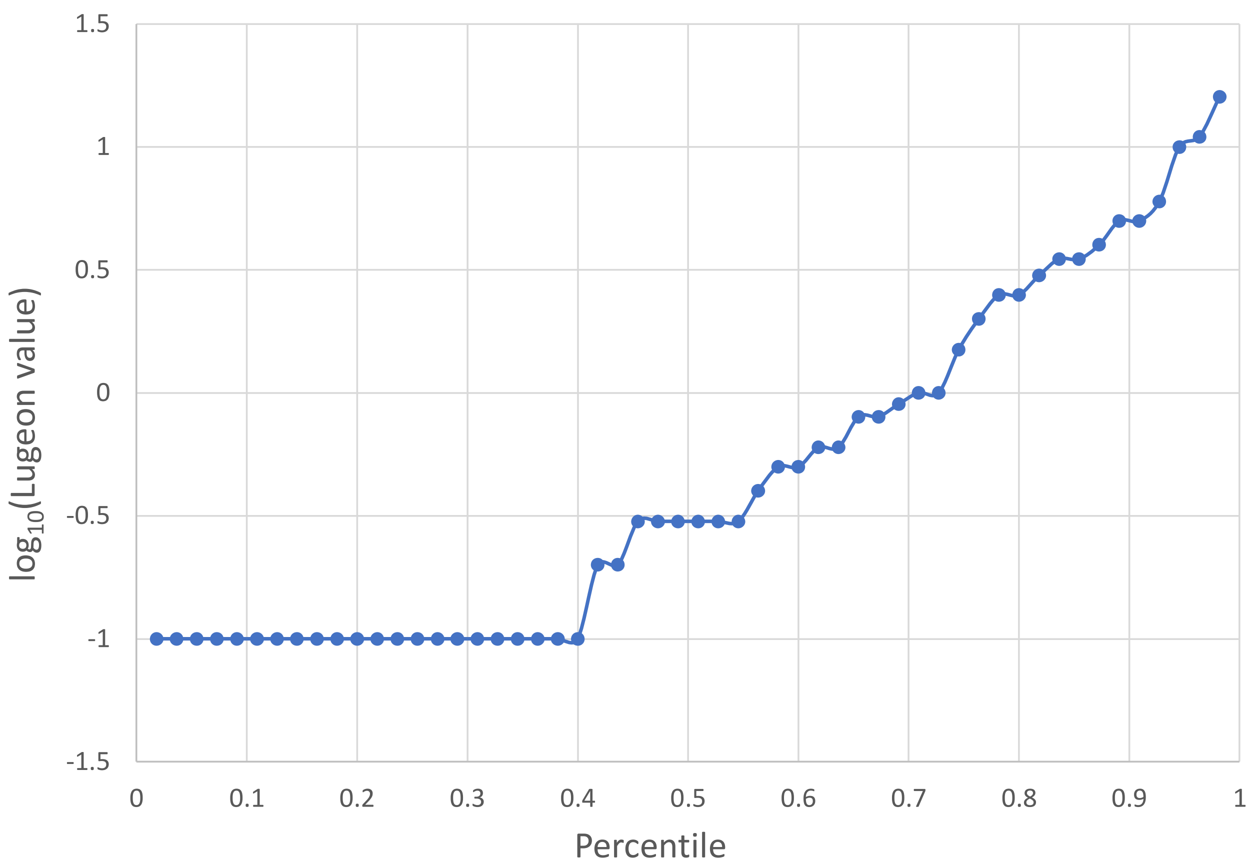


FIGURE 3-3: CUMULATIVE DISTRIBUTION OF LUGEON VALUES FOR SILTSTONE TEST INTERVALS

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.

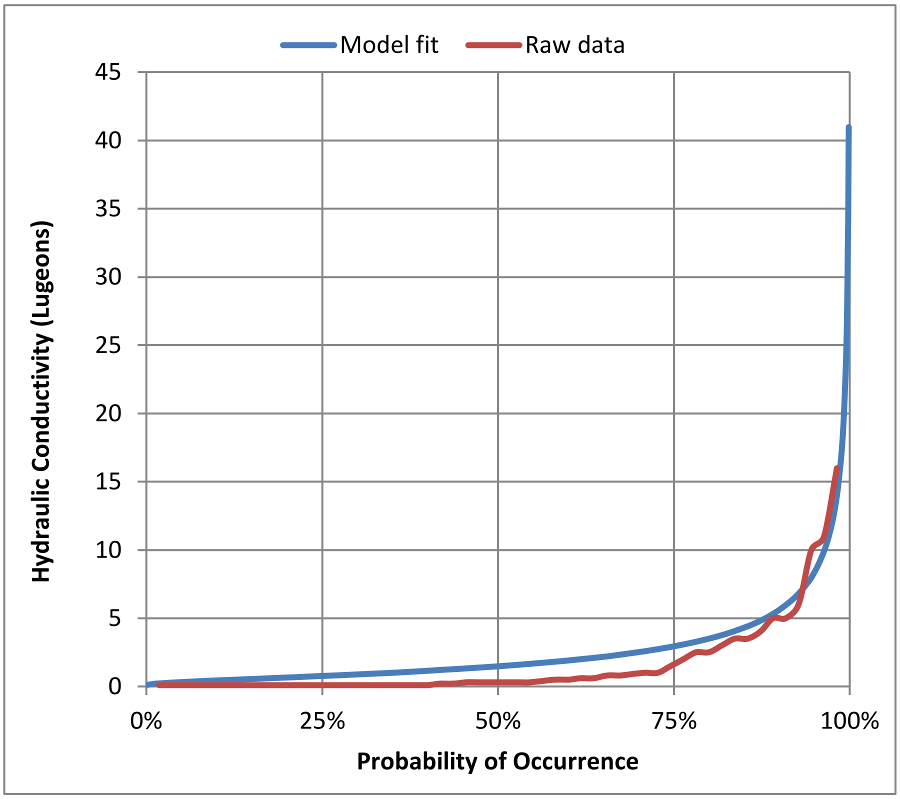


FIGURE 3-4: CUMULATIVE DISTRIBUTION OF LUGEON VALUES FOR SILTSTONE TEST INTERVALS AND NORMAL DISTRIBUTION MODEL FIT TO DATA

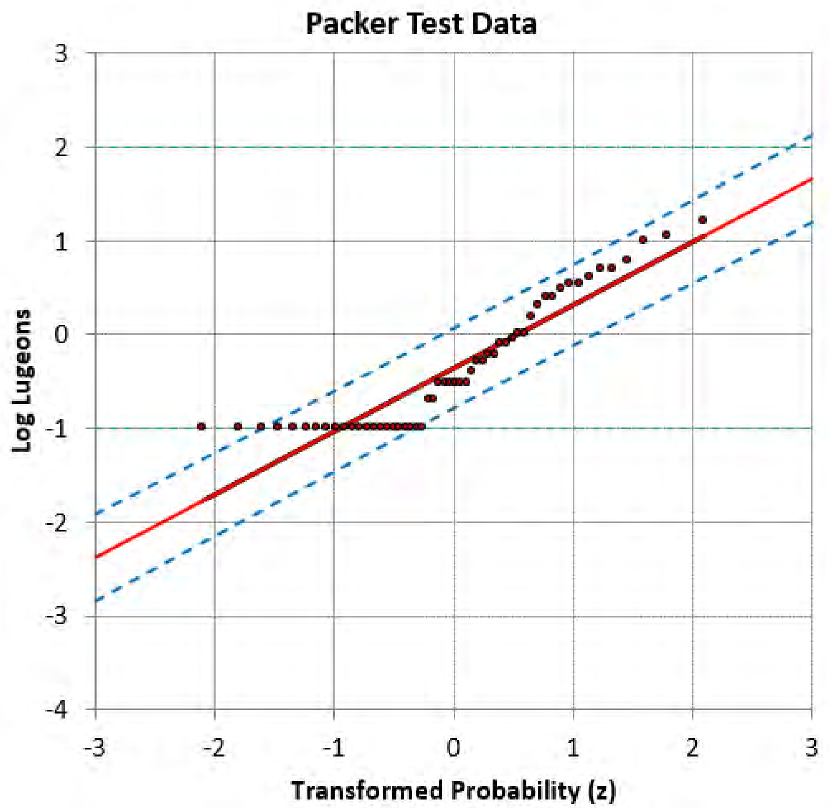


FIGURE 3-5: QUANTILE PLOT OF LUGEON VALUES FOR SILTSTONE TEST INTERVALS

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	Lugeon value	Horizontal hydraulic conductivity, *K* (m/d)
Raw data		
Minimum	0.10	8.64×10^{-4}
5th percentile	0.10	8.64×10^{-4}
10th percentile	0.10	8.64×10^{-4}
25th percentile	0.10	8.64×10^{-4}
Median	0.30	2.59×10^{-3}
Geometric mean	0.44	3.80×10^{-3}
Arithmetic mean	1.61	1.39×10^{-2}
75th percentile	1.39	1.20×10^{-2}
90th percentile	4.70	4.06×10^{-2}
95th percentile	7.40	6.39×10^{-2}
Maximum	16.00	1.38×10^{-1}
Log-normally distributed fit		
Arithmetic mean	1.90	1.64×10^{-2}
K_{3D}	0.70	6.05×10^{-3}
N (number of tests)	54	

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 low Lugeon value (1 to 5 Lugeon), with the rock mass characterised as tight.
- The median, geometric mean and mean value is 0.4 Lugeons, 0.6 Lugeons and 5.9 Lugeons, respectively. The median and geometric mean values are classified as very low Lugeon values (<1 Lugeon), 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 211 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.

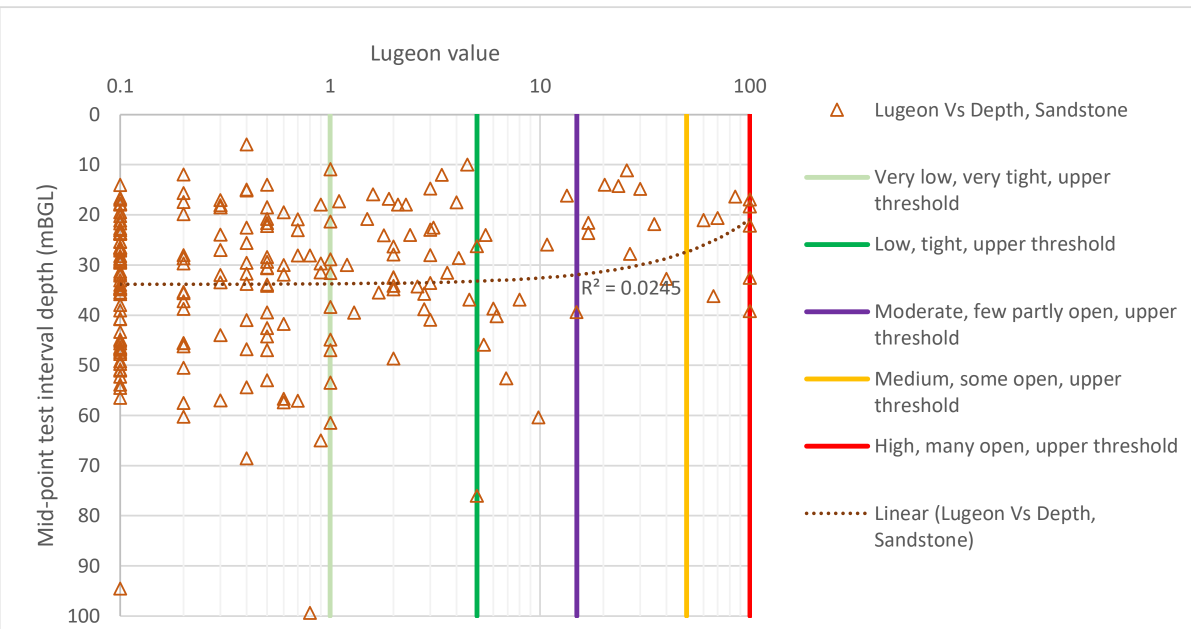


FIGURE 3-6: LUGEON VALUES FOR SANDSTONE TEST INTERVALS, CLASSED ACCORDING TO QUINONES-ROZO (2010)

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	13	3.0	2.4	7.5	2.60×10^{-2}	2.10×10^{-2}	6.54×10^{-2}
15 to <30 m	90	0.5	0.7	8.0	4.33×10^{-3}	6.27×10^{-3}	6.92×10^{-2}
30 to <45 m	65	0.4	0.5	3.8	3.47×10^{-3}	4.28×10^{-3}	3.28×10^{-2}
45 to <60 m	34	0.4	0.5	5.3	3.47×10^{-3}	4.66×10^{-3}	4.59×10^{-2}
60 to 105.9 m (max)	9	0.1	0.1	0.2	8.67×10^{-4}	1.14×10^{-3}	1.35×10^{-3}

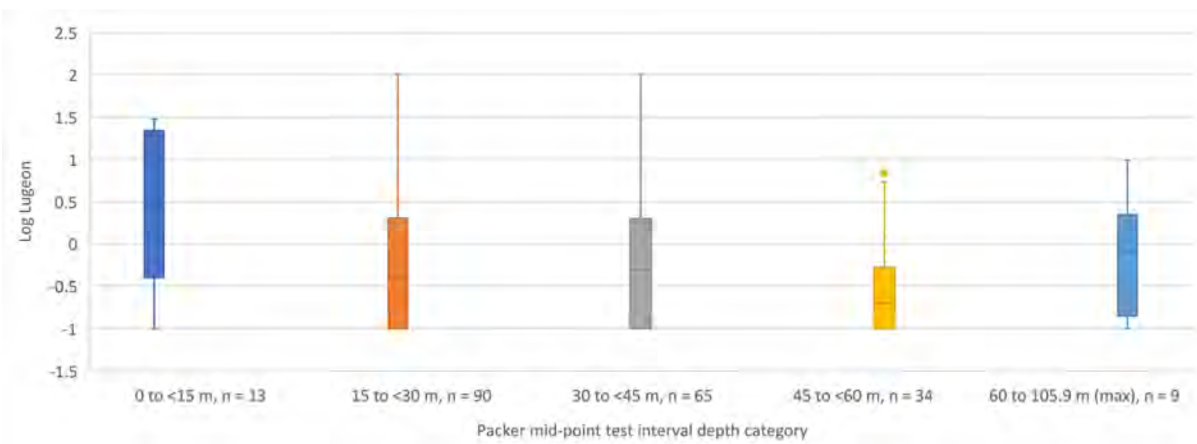


FIGURE 3-7: LOG LUGEON VALUES FOR SANDSTONE TEST INTERVALS BY DEPTH CATEGORY

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.

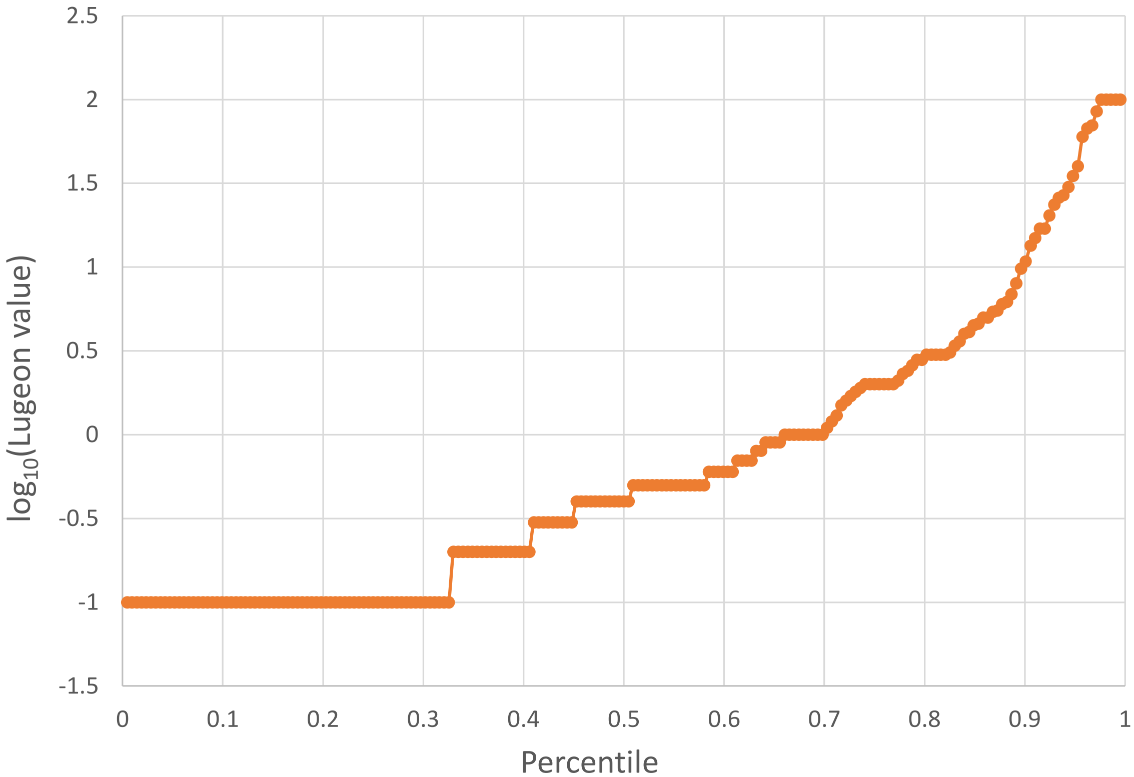


FIGURE 3-8: CUMULATIVE DISTRIBUTION OF LUGEON VALUES FOR SANDSTONE TEST INTERVALS

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.

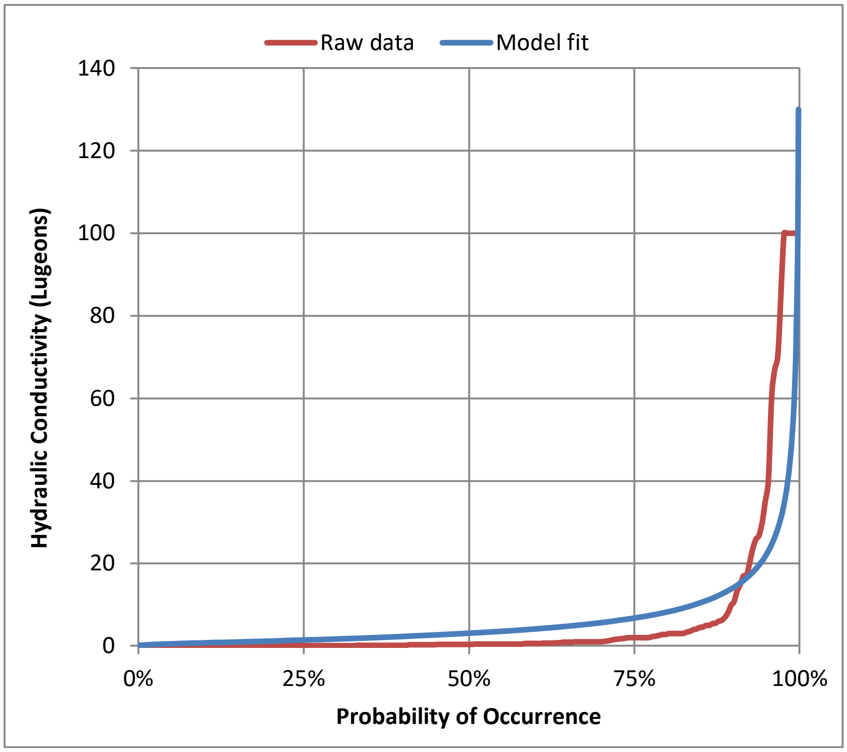


FIGURE 3-9: CUMULATIVE DISTRIBUTION OF LUGEON VALUES FOR SANDSTONE TEST INTERVALS AND NORMAL DISTRIBUTION MODEL FIT TO DATA

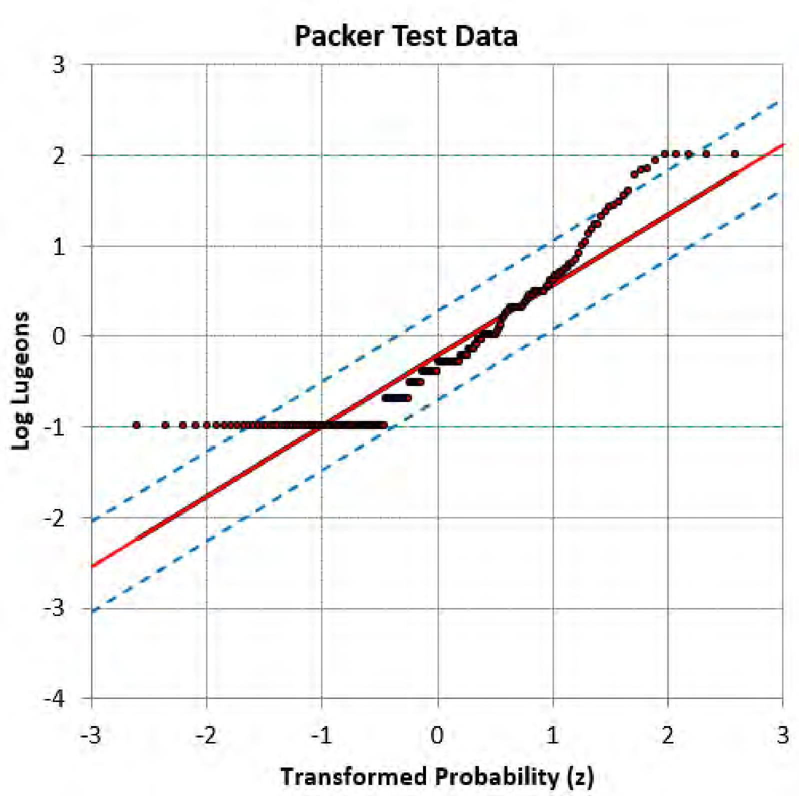


FIGURE 3-10: QUANTILE PLOT OF LUGEON VALUES FOR SANDSTONE TEST INTERVALS

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	8.64×10^{-4}
5th percentile	0.10	8.64×10^{-4}
10th percentile	0.10	8.64×10^{-4}
25th percentile	0.10	8.64×10^{-4}
Median	0.40	3.46×10^{-3}
Geometric mean	0.61	5.27×10^{-3}
Arithmetic mean	5.90	5.10×10^{-2}
75th percentile	2.00	1.73×10^{-2}
90th percentile	9.80	8.47×10^{-2}
95th percentile	32.50	2.81×10^{-1}
Maximum	100.00	8.64×10^{-1}
Log-normally distributed fit		
Arithmetic mean	3.10	2.68×10^{-2}
K_{3D}	1.00	8.64×10^{-3}
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	3.80×10^{-3} to 1.20×10^{-2} (0.4 to 1.4 Lugeons) (geometric mean to 75th percentile) (Log-normally distributed arithmetic mean is 1.64×10^{-2} = 1.9 Lugeons; K_{3D} value is 6.05×10^{-3} m/d = 0.7 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	5.27×10^{-3} to 1.73×10^{-2} (0.6 to 2.0 Lugeons) (geometric mean to 75th percentile) (Log-normally distributed arithmetic mean is 2.65×10^{-2} m/d = 3.1 Lugeons; K_{3D} value is 9.06×10^{-3} m/d = 1.0 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: 1.21×10^{-2} (1.4 Lugeons; 75th percentile) Likely: 6.05×10^{-3} m/d (0.7 Lugeons; K_{3D} value)	0.1	5.00×10^{-6}	0.02
Mittagong Formation and Hawkesbury Sandstone	Conservative: 1.73×10^{-2} (2.0 Lugeons; 75th percentile) Likely: 8.64×10^{-3} m/d (1.0 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.

5. References

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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,SN300,SN350,SN400-ST-RPT-Generic Appendix G Annexure XX
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.

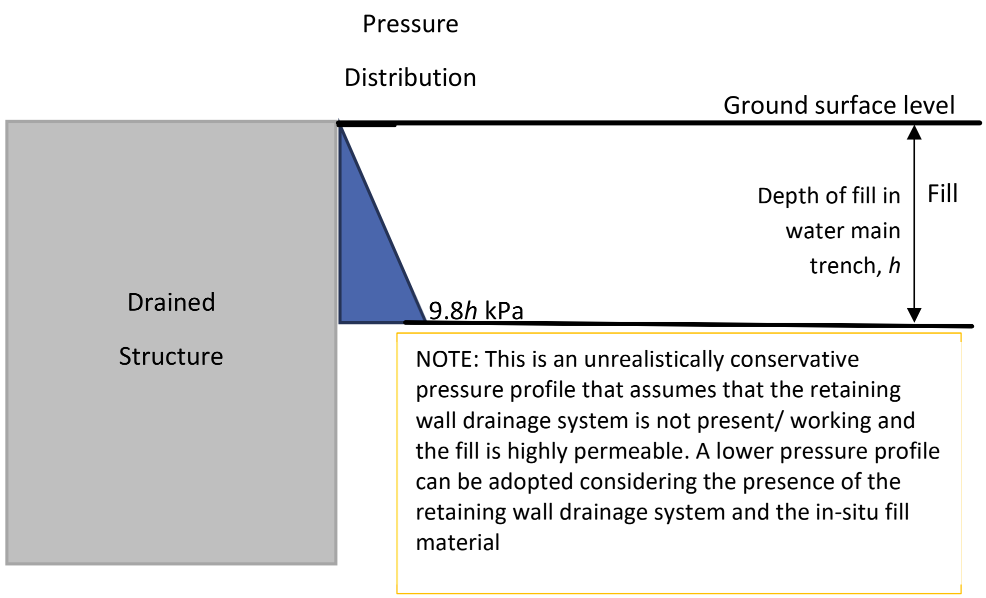

FIGURE 1: EXCEPTIONAL GROUNDWATER PRESSURE CONDITION FOR BURST WATER MAIN

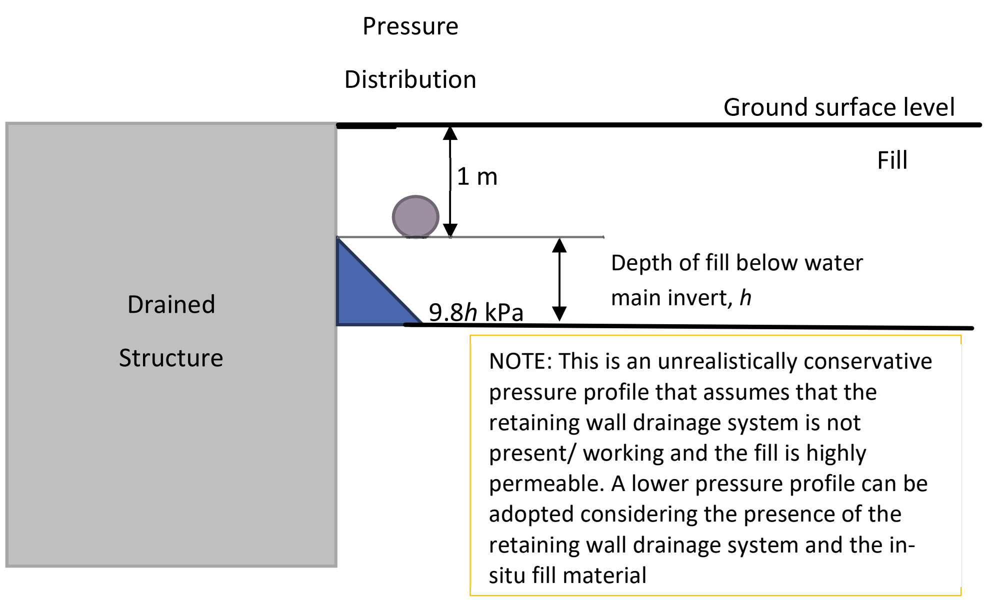

FIGURE 2: EXCEPTIONAL GROUNDWATER PRESSURE CONDITION FOR BURST WATER MAIN CONSIDERING PIPE INVERT LEVEL

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.

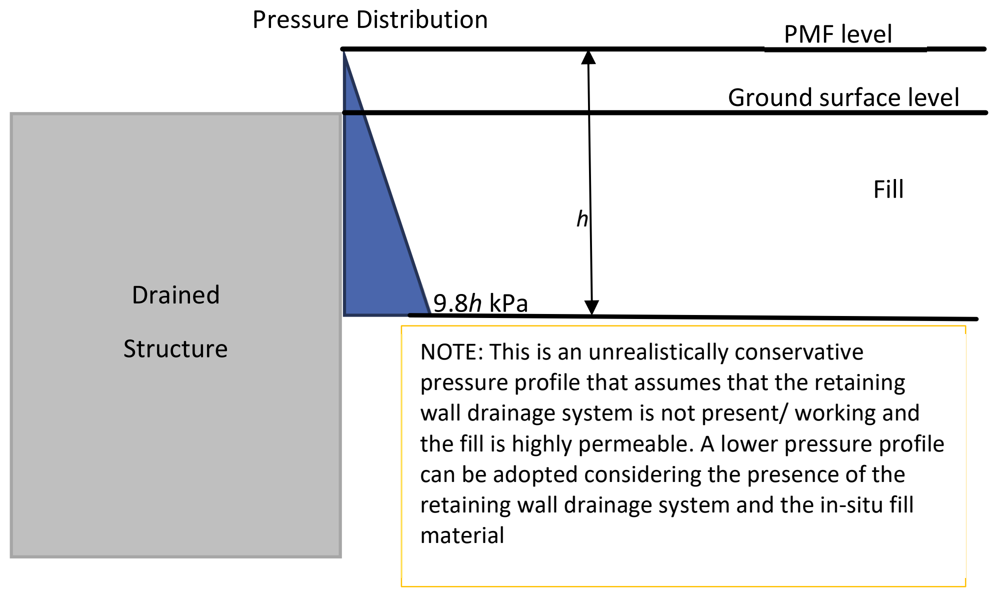

FIGURE 3: EXCEPTIONAL GROUNDWATER PRESSURE CONDITION FOR FLOOD SCENARIO

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

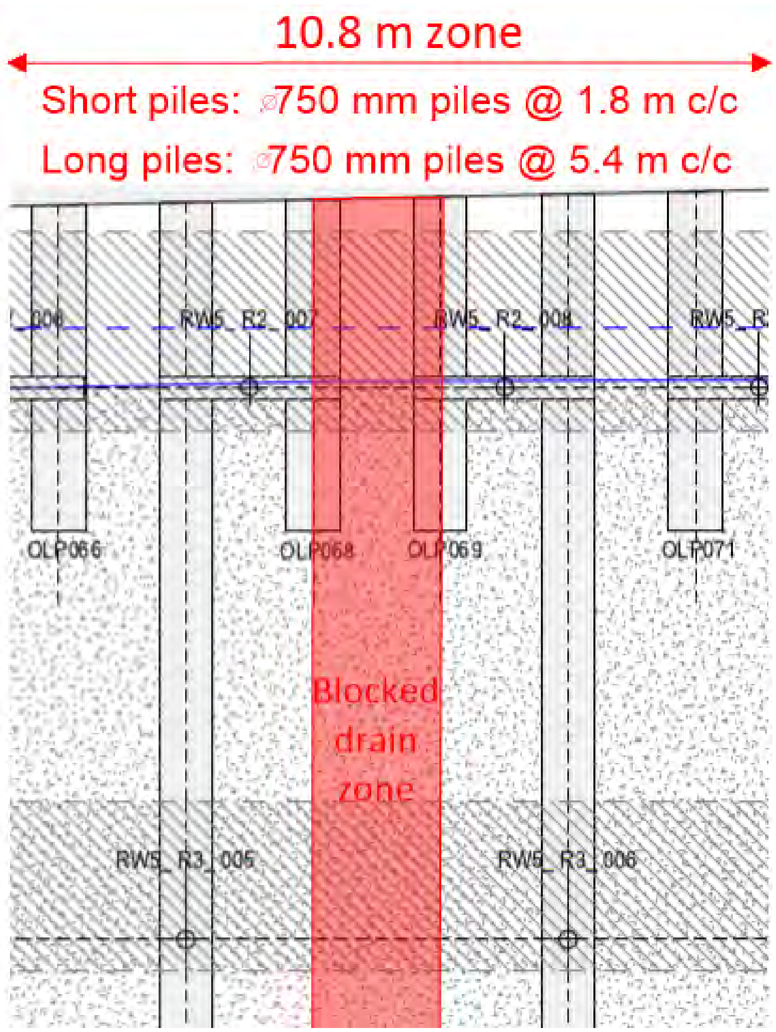

FIGURE 4: TYPICAL PILE LAYOUT AND BLOCKED DRAINAGE ZONE

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

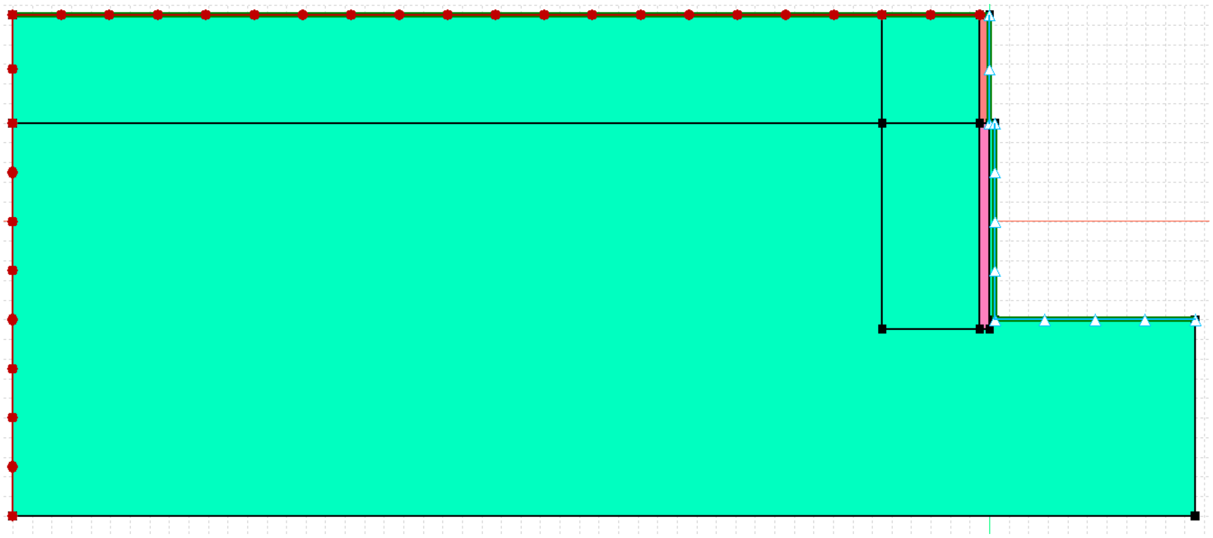

FIGURE 5: MODEL DOMAIN

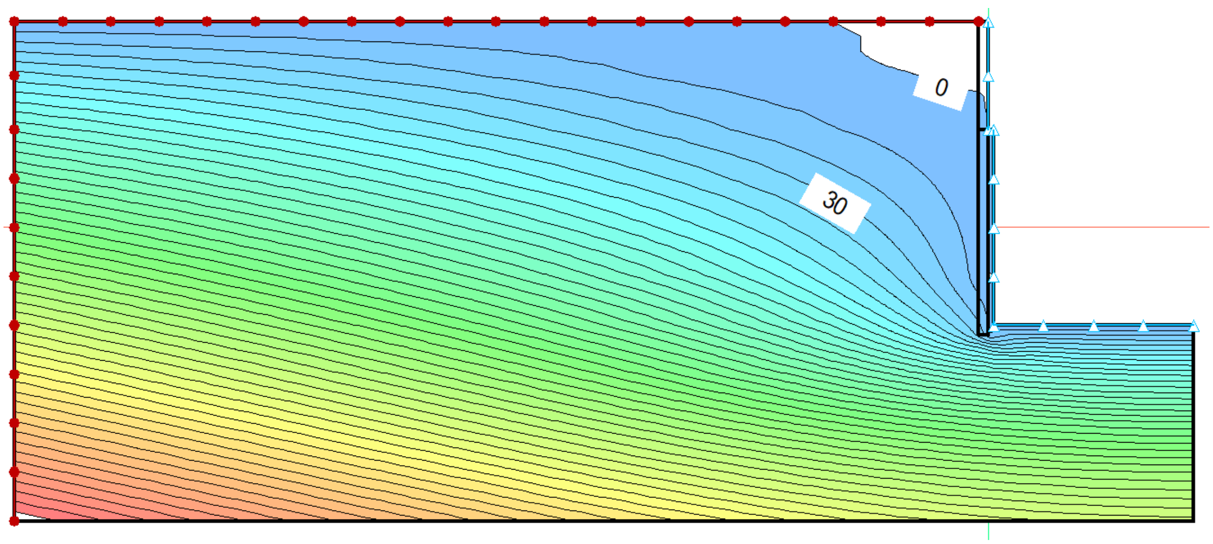

FIGURE 6: EXAMPLE MODEL OUTPUT

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.

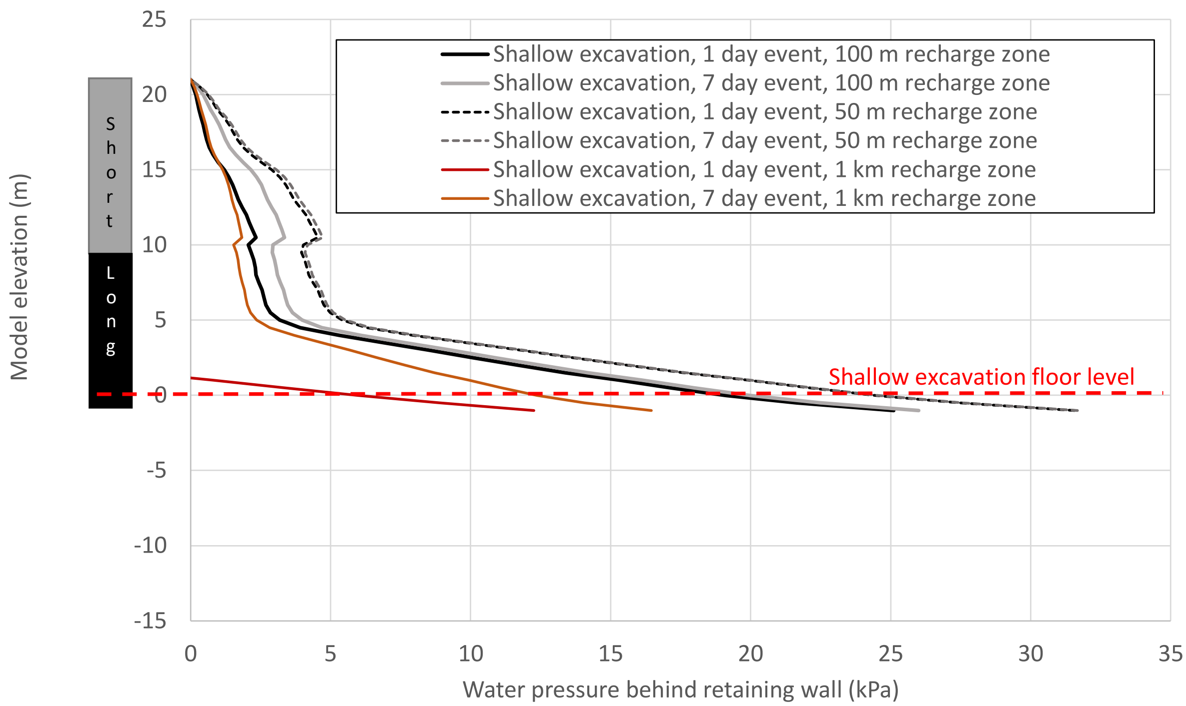

FIGURE 7: MODEL RESULTS – GROUNDWATER PRESSURE PROFILE ALONG PILED WALL – SHALLOW EXCAVATION

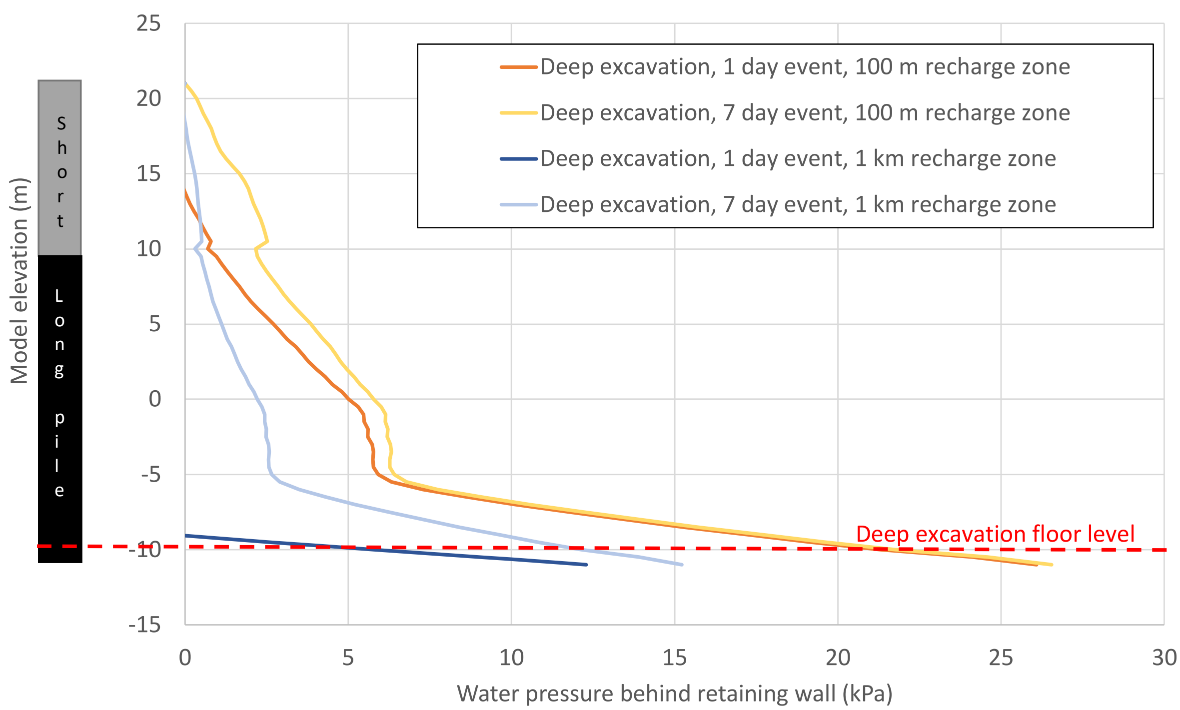

FIGURE 8: MODEL RESULTS – GROUNDWATER PRESSURE PROFILE ALONG PILED WALL – DEEP EXCAVATION

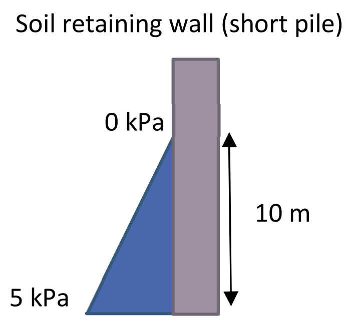

FIGURE 9: PRESSURE PROFILE DIAGRAM BASED ON MODEL RESULTS

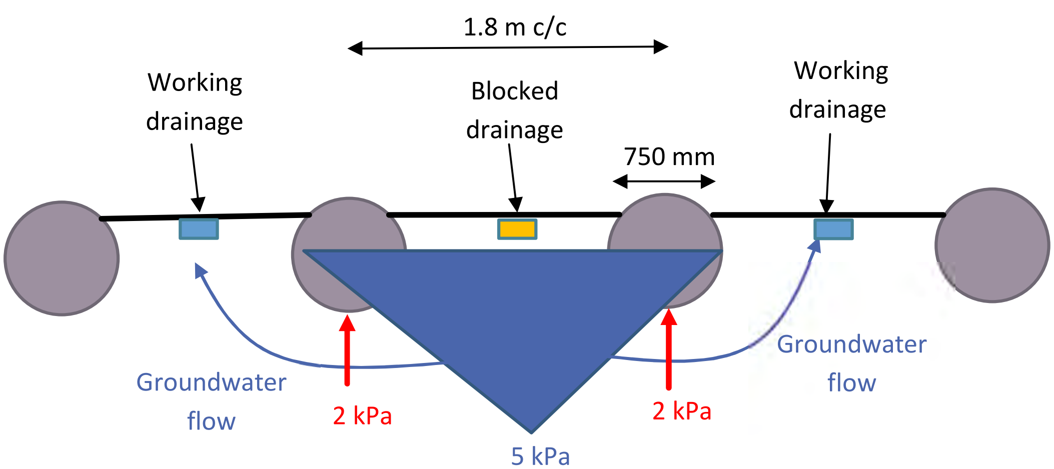

FIGURE 10: PRESSURE PROFILE DIAGRAM (IN PLAN VIEW)

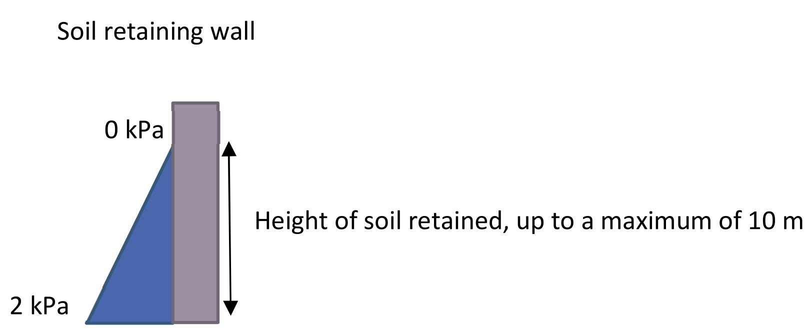

FIGURE 11: PRESSURE PROFILE TO ADOPT IN DESIGN OF SOIL RETAINING WALLS FOR EXCEPTIONAL LOAD CONDITION (GROUNDWATER) REPRESENTING BLOCKED DRAINAGE

Annexure D. Groundwater modelling

Jacobs TYPSA
Jacobs Typsa Joint Venture

Technical Memo

To	Ali Mohiti	Date 8 March 2022
Copies		Document ID SMWSTCTP-AFJ-OLP-SN400-ST-RPT-003060 Appendix G Annexure D
From	Ben Rose	Revision A
Subject	Sydney Olympic Park 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 Olympic Park 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 are instead covered in the main respective Stage 3 Olympic Park 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 Olympic Park Station design. The modelling objectives were to:

- Predict groundwater inflow rates to the Olympic Park Station excavation.
- 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 difference 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 southwest to northeast through Olympic Park Station and extended to appropriate boundaries. The model was calibrated to existing representative groundwater levels at Olympic Park Station in steady state by adjusting the rainfall recharge rate. Upon achieving suitable calibration, a transient model was developed, which incorporated boundary conditions to simulate groundwater drainage associated with the station excavation. This boundary condition enabled prediction of groundwater inflow rates into the station excavation and estimation of groundwater level drawdown (by comparison to existing groundwater level conditions as calculated by the steady state calibration model).

The cross section model was established to be 1 m thick. Thus, groundwater inflow rates were calculated by multiplying the station excavation length with the modelled groundwater inflow rate.

To account for potential groundwater inflows to the station excavation faces perpendicular to the cross section, a multiplier of 1.1 was applied to the net inflow to the station excavation. 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 is shown in Figure 1. The cross section extends from an unnamed drainage line near Phillips Park in the southwest to Bennelong Pond, which is connected to Powells Creek, in the northeast.

This cross section was selected to provide reasonable representation of distant boundary conditions and because it dissects the approximate centre of the station excavation, perpendicular to the longest sides of the rectangular excavation.

At the station site, the ground profiles reported in the Geotechnical Interpretive Report were considered.

FIGURE 1 OLYMPIC PARK STATION SEEP/W CROSS SECTION LOCATION

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 at the station (e.g. about 5 m thick) and is characteristically similar to the Hawkesbury Sandstone in its hydrogeological properties.

The Ashfield Shale layer is represented from ground surface level to a uniform level of -23 mAHD along the entire section and is based on the level of the Ashfield Shale/Mittagong Formation interface at the approximate centre of the station. The Hawkesbury Sandstone/ Mittagong Formation model layer occurs beneath the Ashfield Shale layer and its base is represented at a level of -50 mAHD. This base level is 45 m below the base of the station excavation (-5 mAHD) and therefore provides sufficient model thickness to enable interaction of the station excavations with the underlying groundwater system.

The model layers and boundary conditions are shown in Figure 2.

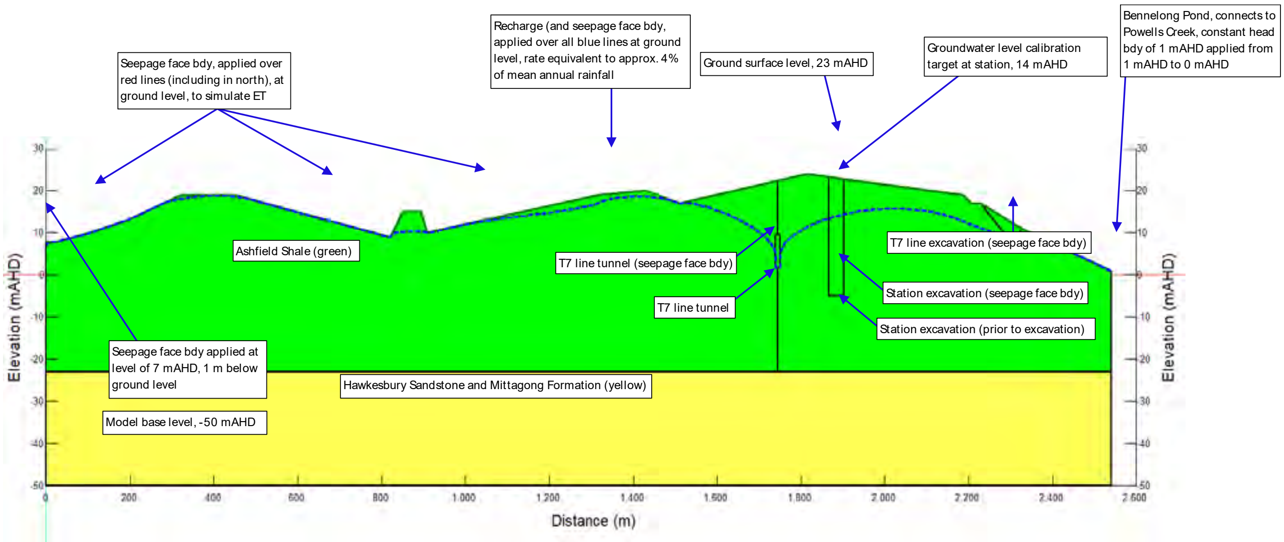

FIGURE 2 OLYMPIC PARK STATION SEEP/W MODEL SET UP. NOTE VERTICAL EXAGGERATION OF 10:1

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 MODEL

Parameter	Ashfield Shale	Hawkesbury Sandstone	Justification
Saturated horizontal hydraulic conductivity (m/d)	0.012	0.0173	Equivalent to 75th percentile of CTP packer testing for siltstone and sandstone intervals, respectively, as documented in hydrogeological properties annexure, Annexure B
Saturated hydraulic conductivity (m/d) applied over excavation	100	100	Applied over Sydney Olympic Park Station 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 on 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

Except for the T7 line tunnel, located southwest of the station, a mesh resolution of 5 m was applied. The T7 line tunnel was assigned a mesh resolution of 1 m. The applied mesh is shown in Figure 2.

2.4.4. Boundary conditions

Boundary conditions are shown in Figure 2 and included:

- External constant head – applied at a level of 1 mAHD, from ground level (1 mAHD) to a level of 1 m below ground level, at northeastern extent of model, to represent Bennnelong Pond, which connects to Powells Creek.
- External potential seepage face – applied at a level of 7 mAHD, 1 m below ground level, at southwestern extent of model, to represent potential discharge.
  A potential seepage face was also applied at ground level in 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 rainfall over the whole section, except where ET simulated. This recharge rate was arrived at during model calibration by matching modelled groundwater levels to existing conditions. The rate is slightly high, which is likely due to the particular combination of boundary conditions at this location – lower recharge values did not provide a suitable match to observed groundwater levels
- Internal potential seepage face applied around station excavation, T7 line tunnel southeast of station and at base of T7 line excavation northeast of station. This boundary condition simulates dewatering due to the excavations.
- No flow boundaries applied at base of model, and at southwestern and northeastern extents of model, except where the seepage face and constant head boundaries were applied.

2.4.5. Approach

The model calibration to existing groundwater levels was solved in steady state mode. A transient model was developed and used the solved 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 model was the internal seepage face boundaries applied around the station excavation, 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

The model was calibrated by adjusting the recharge rate to achieve the targeted existing representative water table level of 14 mAHD at the centre of the station. The watertable level target was achieved and the calibrated watertable level is shown in Figure 3.

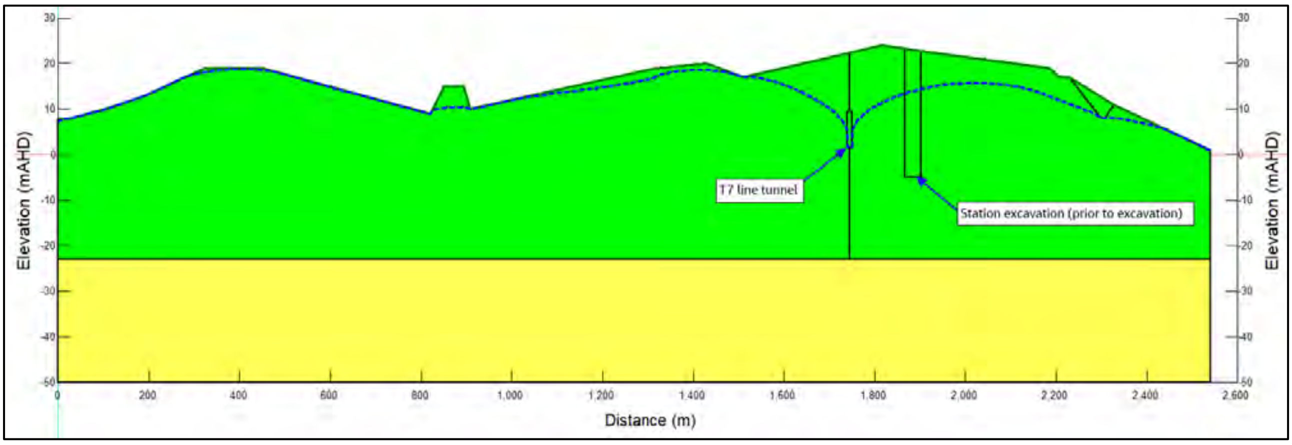

FIGURE 3 CALIBRATED WATERTABLE LEVEL (BLUE DASHED LINE). NOTE VERTICAL EXAGGERATION OF 10:1

2.5.2. Groundwater inflows

Model-predicted groundwater inflow rates to the station excavation are shown in Figure 4 and were up to 55 m^3/d.

As shown in Figure 4, 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 occurs instantaneously (the excavation is “wished in place”) in the model. 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 Olympic Park Station Hydrogeological Assessment Report, Sydney Olympic Park Station is on the periphery of the Homebush Bay Fault Zone. Geotechnical interpretations indicate some possible faults may be present in the vicinity of the site, likely represented as joint swarms. Despite this, currently available packer testing data does not indicate that the shale is of high permeability. There is a possibility that hydraulic conductivity values may be relatively higher in the vicinity of joint swarms, 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.

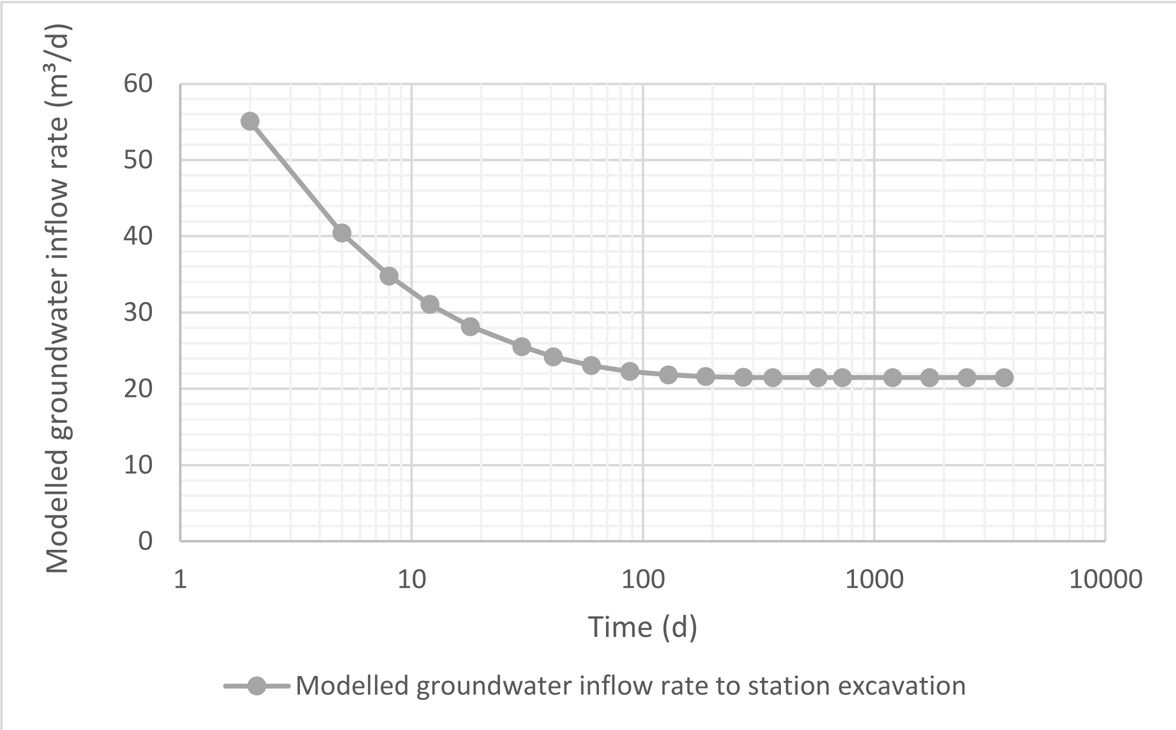

FIGURE 4 GROUNDWATER INFLOW RATES CALCULATED BY MODEL

2.5.3. Watertable drawdown

The modelled watertable surfaces are shown in Figure 5, and drawdown of the watertable is shown in Figure 6. In Figure 6, the distance of 0 m along the section is at the southwestern 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 conditions are reached within two years), which is why drawdown for both output times is not visible in Figure 6.

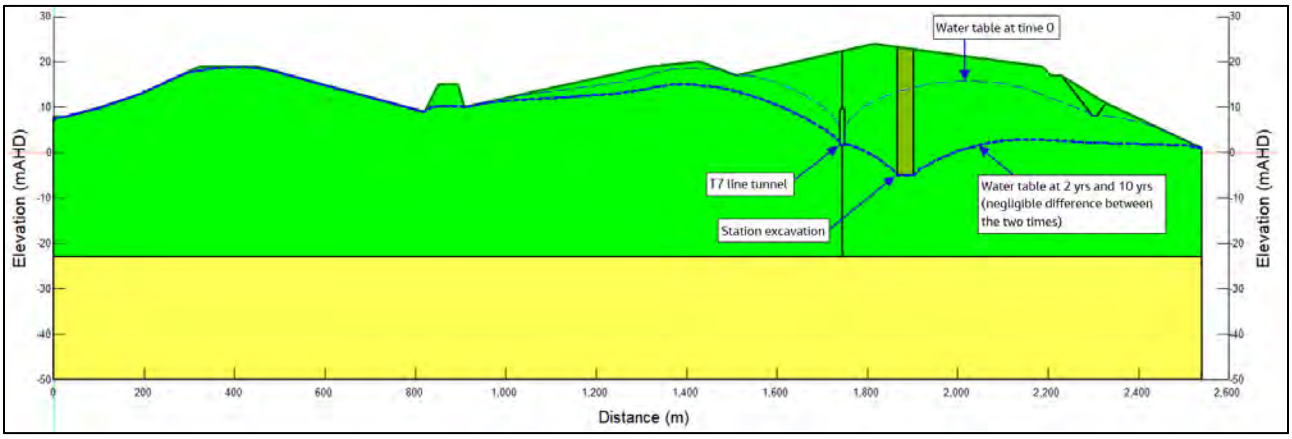

FIGURE 5 MODELLED WATER TABLE LEVELS. NOTE VERTICAL EXAGGERATION OF 10:1

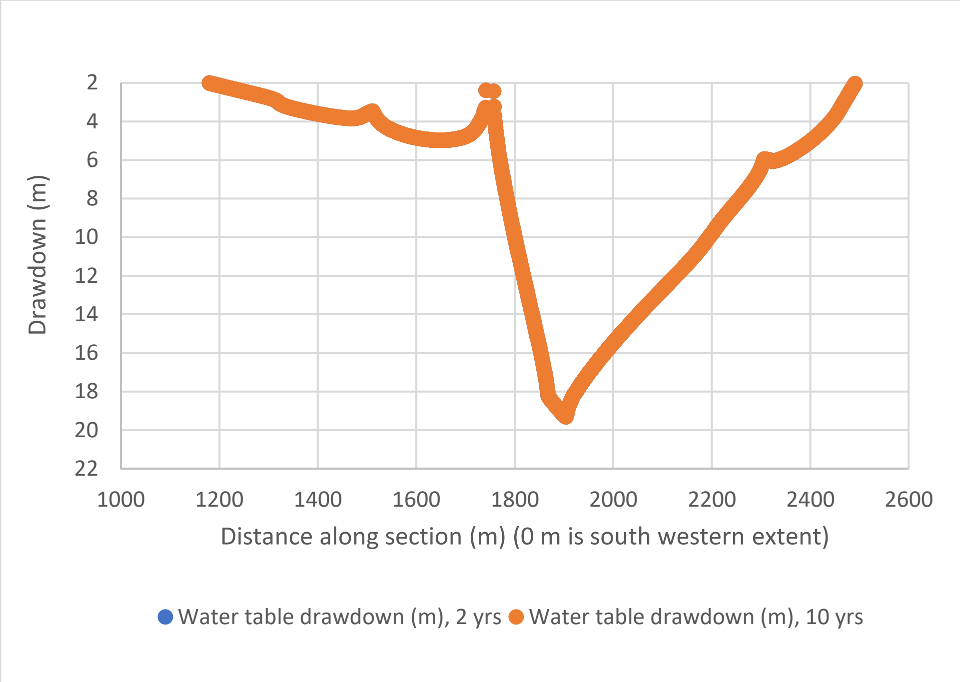

FIGURE 6 MODELLED DRAWDOWN TO WATER TABLE