North Strathfield Station - Hydrogeological Assessment Report

SMWSTCTP-AFJ-NST-GE-RPT-000001 Revision 00

Sydney Metro West – Central Tunnelling Package

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EXECUTIVE SUMMARY

JTJV has been engaged by Acciona Construction Australia Pty Ltd and Ferrovial Construction (Australia) Pty Ltd Joint Venture to undertake a review of available information to inform the Detailed Design of the key alignment site(s) (stations, shaft and tunnels) and adjacent areas of the Sydney Metro West – Tunnel and Excavation – Central Package.

North Strathfield Station involves excavation for a station box. The station box will be excavated through fill, residual soils and Ashfield Shale units.

JTJV has prepared this technical report specific to the North Strathfield Station box excavation.

The North Strathfield Station box excavation 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 negligible hydrostatic pressure on the retaining walls.

The critical design load case scenario for the geotechnical/structural design of the western retaining wall is the flooding scenario. Design groundwater pressures are nominated for the flood scenario in Annexure C of this report (this is also considered in the geotechnical/structural design).

At the station, the current water table is interpreted to occur at approximately 15 m AHD and the groundwater system from the water table down to the station excavation base level is interpreted to be reasonably well connected.

North Strathfield Rail Underpass project infrastructure lie in the vicinity of North Strathfield Station, including a 148 m-long rail underpass completed in June 2015. The North Strathfield Rail Underpass project infrastructure is interpreted to not be currently drawing down groundwater levels applicable to the North Strathfield Station excavation.

Numerical groundwater modelling was undertaken to estimate the potential groundwater inflows to the station excavation, and associated groundwater level drawdown. Predicted inflows to the station box excavation are up to approximately 36 m³/d (0.42 L/s). The long-term inflows to the station box excavation are predicted to be approximately 13 m³/d (0.15 L/s). The modelling approach considers instantaneous excavations (excavations are “wished-in-place”). Given that the actual excavation will be progressive, the estimated groundwater inflows (the peak inflow and the timing of peak inflow) may therefore differ to those reported here.

The model predicted inflows to the station box are within the inflow limits as specified in the Particular Specification (condition SM-W-CTP-PS-2882). Condition SM-W-CTP-PS-1040 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. Should local features be encountered that exceed the inflow limit, localised grouting of the features will be required.

Two inferred (potential) faults with approximately north-north-east orientation are anticipated within the North Strathfield Station box excavation, with a third inferred fault passing through the southeastern nozzle, immediately adjacent to the southeastern corner of the station box excavation. It is possible that rock in the vicinity of the inferred fault zones is of higher permeability than the adjacent rock. If enhanced permeability occurs in the vicinity of the faults, based on bulk permeability calculations, groundwater inflows could be significantly higher than the inflow predicted by the model. Under these circumstances, the groundwater inflow rate would exceed Particular Specification SM-W-CTP-PS-1040 and SM-W-CTP-PS-2882. Therefore, localised grouting during excavation would be required to limit groundwater inflows to the Particular Specification criteria. Additionally, if hydraulic conductivity values are elevated in other not-yet identified zones, then groundwater inflows may be potentially higher. 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 predicted drawdown due to CTP excavation works is similar, but slightly smaller in extent, to the drawdown predicted in the EIS.

Alluvium is mapped about 400 m to the southwest of the station. The maximum modelled drawdown within the alluvium was approximately 0.75 m.

The following comments are made in relation to groundwater level drawdown and groundwater inflows:

- Groundwater users (groundwater supply bores registered with WaterNSW) are unlikely to be affected by drawdown
- There is a low risk to groundwater dependent ecosystems located to the northeast of the station
- Potential acid sulfate soils were not identified within the predicted extent of groundwater level drawdown. However, disturbed soils are present to the west of the modelled 2 m groundwater level drawdown extent. It is possible that construction excavation induced drawdown could impact potential acid sulfate soils in that area, if present.
- Groundwater in the vicinity of the North Strathfield Station was slightly acidic to slightly alkaline and there were instances where manganese concentrations exceed Human Health Criteria and Recreational and Aesthetic criteria at some locations, and instances where some metals (nickel, zinc, manganese, arsenic, copper and aluminum) and ammonia (as N) exceeded ANZG (2018) guideline trigger levels for 95% Protection of freshwater aquatic ecosystems. Additionally, groundwater electrical conductivity values generally exceeded the ANZECC (2000) guideline trigger level of 2,200 µS/cm (lowland rivers) for 95% Protection of freshwater aquatic ecosystems.
- There are potential contamination risks associated with locations beyond the site. The North Strathfield Station excavation is anticipated to act as a groundwater sink. It is therefore possible that contaminated groundwater at distance from the site will be drawn towards the excavation. Based on average linear flow velocity calculations, for the 10 year design life, this distance from the site is likely to be less than approximately 100 m. However, it could be greater if (a)water bearing feature(s) with relatively high permeability is present. The risk of contaminated groundwater at distance from the site is unknown.
- Treatment of groundwater seepage to the excavations prior to disposal will likely be required, depending on the disposal options proposed
- Saline intrusion from the coastal aquifers near the Parramatta River is considered to be a low risk.

1. INTRODUCTION

1.1 OBJECTIVES AND SCOPE

The objective of this report is to provide hydrogeological advice for the design of the North Strathfield 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 North Strathfield 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.
- Documentation of revised groundwater modelling that has occurred since Stage 1
- 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 North Strathfield 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 10m², at any and all locations within the sides and bases of the shafts and excavations, except for The Bays Station Excavation where groundwater seepage must not exceed 50,000 litres in any 24-hour period, measured over any square with an area of 10m², at any and all locations within the sides and bases of the excavation; and [SM-W-CTP-PS-1040]

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

A. North Strathfield Station Excavation: 92,000 litres; [SM-W-CTP-PS-2882]

3. HYDROGEOLOGICAL CONCEPTUAL SITE MODEL

3.1 GEOLOGY

The station box will be excavated through the fill, residual soils and Ashfield Shale units. The fill and residual soil hydrogeological units are relatively thin (typical combined thickness of up to about 4 m) and conceptualised to be generally unsaturated or intermittently saturated. Approximately 2 m of highly to moderately weathered Ashfield Shale underlies the residual soils. The station box has an excavation floor level of about -2.5 m AHD and the top of the Mittagong Formation ranges from about 7.0 m to 12.5 m below the base of the station box excavation, with the formation typically about 2.5 m to 4.0 m thick. Thus, the Ashfield Shale hydrogeological unit is considered most relevant to assessment of groundwater levels, inflows, quality and drawdown at the site.

Two inferred faults pass through the central portion of the station excavation, with a third inferred fault passing through the southern nozzle, coming within very close proximity to the southeastern station excavation corner. Additionally, there are two other inferred faults of the same orientation located 60 m and 90 m south of the station. The inferred faults are shown in Figure 3-1.

An inferred (potential) dyke with an approximately north-northeast orientation is projected to exist approximately 100 m from the southeast corner of the station box (Figure 3-1).

If present, the dyke is 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 potential influence of the inferred faulting/joint swarm zones and dyke on hydrogeological unit permeability is discussed in Section 3.3.

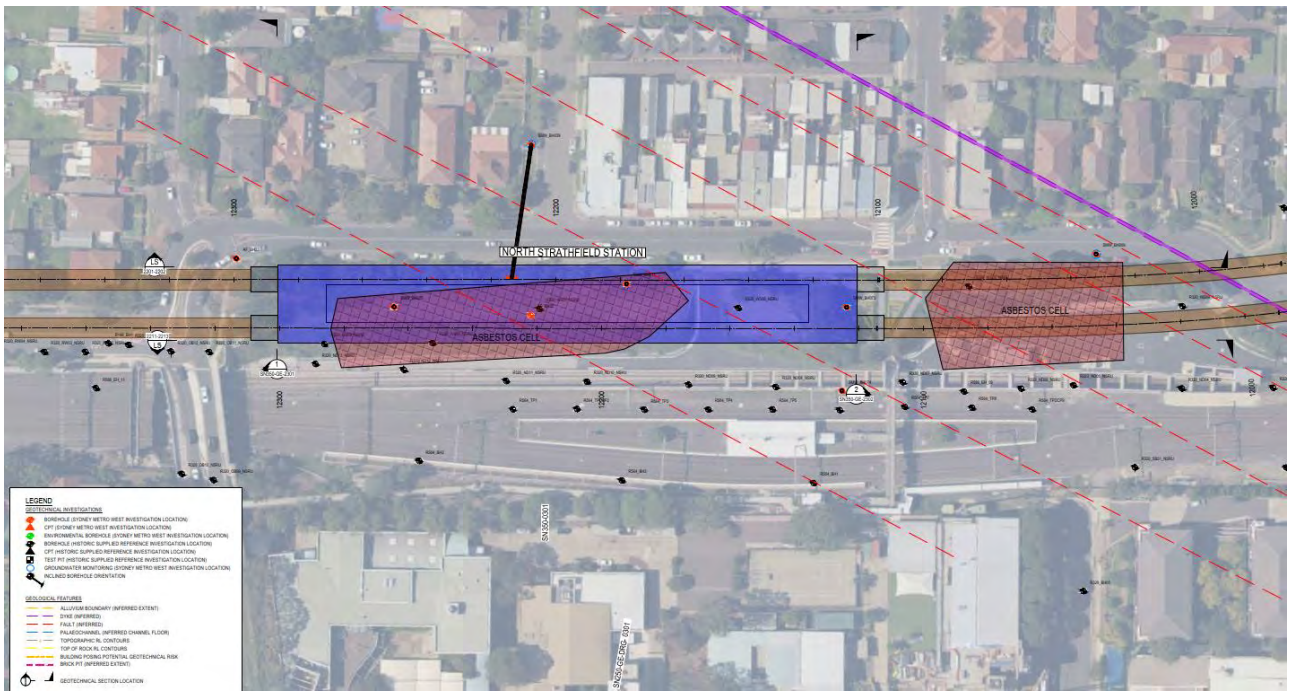


FIGURE 3-1: PLAN VIEW OF NORTH STRATHFIELD STATION, SHOWING INTERPRETED GEOLOGICAL FEATURES

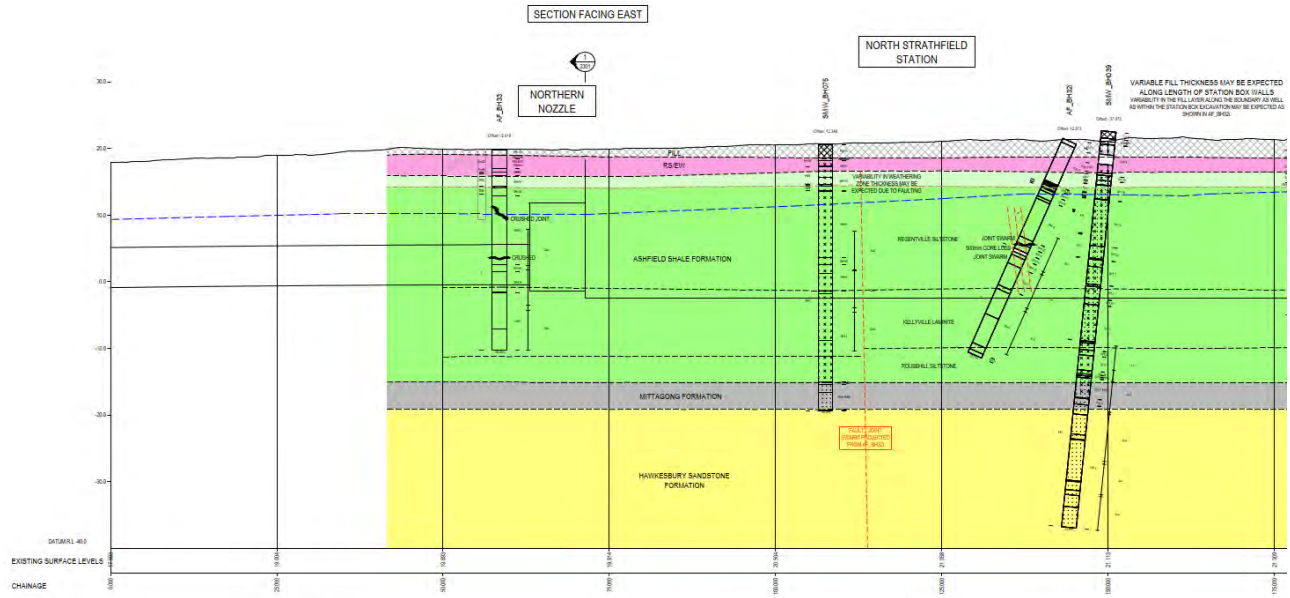


FIGURE 3-2: GEOLOGICAL LONG SECTION (LS 2201-2202) OF NORTH STRATHFIELD STATION

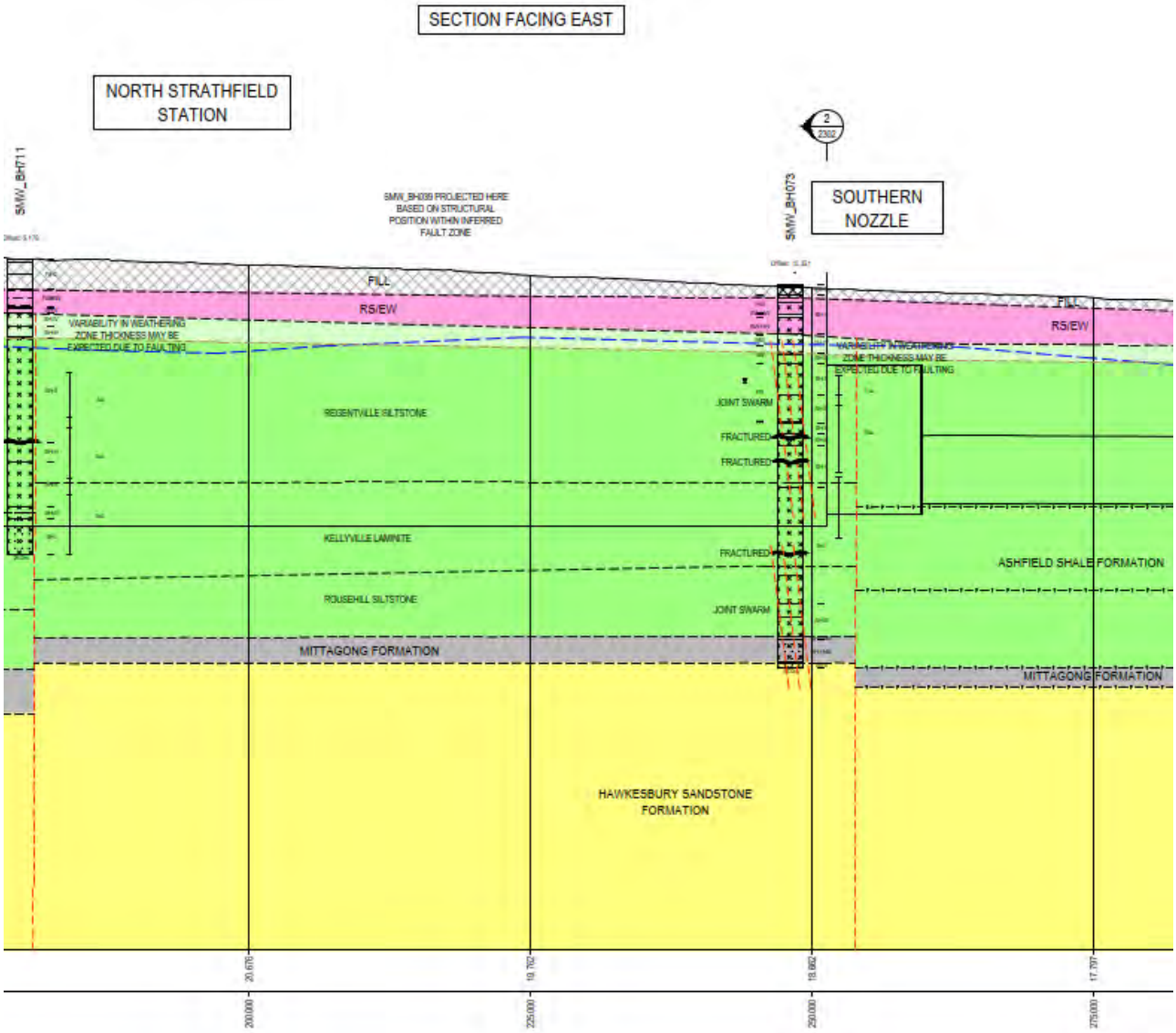


FIGURE 3-2 (CONTINUED): GEOLOGICAL LONG SECTION (LS 2201-2202) OF NORTH STRATHFIELD STATION

3.2 GROUNDWATER LEVELS

3.2.1 OBSERVED GROUNDWATER LEVELS

There is currently a total of 11 SMW and AFJV groundwater monitoring locations (Figure 3-3) in the vicinity of the North Strathfield Station, with an additional seven reference site groundwater monitoring locations (denoted as R320 series locations in Figure 3-3). Two of the SMW locations are vibrating wire piezometers, SMW_BH036_v (single sensor) and SMW_BH039_v (two sensors), with the remaining locations being standpipe piezometers.

FIGURE 3-3: GROUNDWATER MONITORING LOCATIONS NEAR NORTH STRATHFIELD STATION, AND NORTH STRATHFIELD RAIL UNDERPASS ALIGNMENT

As outlined in Section 3.1, the Ashfield Shale hydrogeological unit is considered most relevant to assessment of groundwater levels, inflows, quality and drawdown at the North Strathfield Station.

Due to its relatively low permeability, the Ashfield Shale may act as a distinct aquifer from the underlying Mittagong Formation and Hawkesbury Sandstone.

Ashfield Shale generally has relatively low permeability and groundwater flow in this unit is typically controlled by secondary features such as fractures, joints, shears and bedding planes, resulting in the unit effectively acting as a fractured rock aquifer. Areas where the unit is more fractured tend to yield greater permeabilities while more competent sections typically yield lower permeabilities.

Table 3-1 lists piezometer construction details for groundwater monitoring locations in the vicinity of the North Strathfield Station site and Table 3-2 lists recorded groundwater levels. Table 3-3 notes the geological unit in which the groundwater level lies for the piezometers.

As shown in Table 3-3, the groundwater level lies within Ashfield Shale at all monitoring locations except R320_ND06_NSRU, where it lies within residual soil (clay), only about 0.2 m above the top

of Ashfield Shale. The monitoring period duration is only 35 days at location R320_ND06_NSRU. Therefore, it is unknown whether the groundwater level at this location resides in the residual soil on a temporary basis, or alternatively, on a long-term basis.

Monitored groundwater levels in the vicinity of North Strathfield station range between approximately -13 m AHD and 25 m AHD, and between about 2 m and 36 m below ground surface. Thus, there is considerable variation in observed groundwater levels.

To explore the considerable variation in observed groundwater levels, observed typical groundwater levels are plotted against the filter pack mid-point level (or VWP sensor level) in Figure 3-4 and pressure head is plotted against the filter pack mid-point level (or VWP sensor level) in Figure 3-5.

It is noted that, for the purpose of creating Figure 3-4 and Figure 3-5, groundwater levels of -13.4 mAHD and 7.8 mAHD were adopted for SMW_BH009_w and SMW_BH036_v, respectively. At location SMW_BH009_w the groundwater level is increasing throughout the monitoring period, although possibly stabilising at the end of the monitoring period. The increasing level at SMW_BH009_w is attributed to post-installation equilibration. The adopted value of -13.4 mAHD for SMW_BH009_w represents the maximum observed groundwater level. Location SMW_BH036_v groundwater level rises and then falls, although possibly stabilising at the end of the monitoring period. The cause of the instability at SMW_BH036_v is not known. The adopted value of 7.75 mAHD for SMW_BH036_v represents an inferred post-stabilisation groundwater level.

The considerable variation in observed groundwater levels is interpreted to occur due to the following:

- Groundwater levels are highest at SMW_BH035_s and are interpreted to be associated with a perched groundwater system present at this location. SMW_BH035_s is considered an outlier. SMW_BH035_s is also located relatively far away from the station.
- The group of bores denoted as Group A in Figure 3-4 and Figure 3-5 monitor the Ashfield Shale. Except for AF_BH33_w, all these monitoring locations have a filter pack mid-point level located above the approximate base level of the station excavation and these bores are therefore most relevant to assessment of groundwater conditions applicable to the station. The Group A pressure head trend is generally close to hydrostatic. However, it is noted that the monitoring locations are distributed over a somewhat broad area and additionally there is only one clustered location, SMW_BH711_s and SMW_BH711_w, where both bores are monitoring the Ashfield Shale. At this location the typical observed groundwater level is 15 mAHD and 8.6 mAHD at SMW_BH711_s and SMW_BH711_w, respectively.

The range in typical groundwater level for the Group A bores is approximately 8 m. The variation is attributed to:

- Variation in monitoring location position (maximum separation distance between two bores is approximately 360 m)
- The Ashfield Shale likely having somewhat stratified groundwater systems with very low vertical hydraulic conductivity. Thus, the range in monitoring interval level at different monitoring locations is manifesting in groundwater level variation.
- The group of bores denoted as Group B in Figure 34 and Figure 35 either partially or fully monitor the Mittagong Formation or Hawkesbury Sandstone. The relatively low groundwater levels observed at these bores compared to the Group A bores is attributed to a degree of hydraulic disconnection between the Ashfield Shale and the underlying Mittagong Formation and/or Hawkesbury Sandstone.

Monitoring location SMW_BH711_s is considered to have groundwater level observations that are representative of the water table level at the station. The observed typical groundwater level at SMW_BH711_s is 15 mAHD and is similar to the typical groundwater level observed at SMW_BH009_s (0.2 m lower), SMW_BH039_v (sensor 1) (0.4 m lower), and fairly similar to those at R320_ND08_NSRU (2.6m lower) and R320_ND10_NSRU (2.2 m lower).

In conclusion, at the station, the current water table is interpreted to occur at approximately 15 mAHD and the groundwater system from the water table down to the station excavation base level is interpreted to be reasonably well connected. A degree of hydraulic disconnection between the Ashfield Shale and the underlying Mittagong Formation and/or Hawkesbury Sandstone is interpreted.

TABLE 3-1: SUMMARY OF GROUNDWATER MONITORING PIEZOMETERS AT NORTH STRATHFIELD STATION

Bore ID	Easting (56 MGA94)	Northing (56 MGA94)	Ground Surface Elevation (m AHD)	Effective Screen Depth Top (m bgl)	Effective Screen Depth Bottom (m bgl)	Unit(s)	Monitoring Period
SMW_BH009_s	323220	6251758	18.6	1	5	Gravelly clay and siltstone	Aug 18 to Aug 19
SMW_BH009_w	323220	6251759	18.5	37.5	40.5	Sandstone	Aug 18 to Aug 19
SMW_BH035_s	323361	6251851	26.6	1.7	3.2	Siltstone	Aug 18 to Aug 19
SMW_BH035_w	323362	6251851	26.7	33.5	45.5	Siltstone and sandstone	Sep 18 to Aug 19
SMW_BH036_v	323375	6251745	27	28.59*	-	Siltstone and sandstone	Jul 18 to Aug 19
SMW_BH038_w	323008	6251870	9.9	26	32	Siltstone and sandstone	May 18 to May 19
SMW_BH039_v (sensor 1)	323201	6251939	22.6	19.05*	-	Interlaminated siltstone and sandstone	May 18 to Aug 19
SMW_BH039_v (sensor 2)	323201	6251939	22.6	37.35*	-	Siltstone and sandstone	May 18 to Aug 19
SMW_BH073_w	323182	6251830	18.9	10.2	13.2	Siltstone	Jun 20 to Sep 20
SMW_BH711_s	323170	6251894	21.2	4	7	Clay, siltstone	Mar 21 to Jun 21
SMW_BH711_w	323169	6251895	21.2	11.2	14.2	Siltstone	Mar 21 to Jun 21
AF_BH33_w	323138	6252011	19.9	21.2	30.2	Siltstone	Dec 21 to Jan 22
R320_ND02_NSRU	323207	6251664	14.2	12.1	15	Siltstone	Jul 11 to Aug 11
R320_ND04_NSRU	323189	6251723	14.5	8.9	11.9	Siltstone	Jul 11 to Aug 11
R320_ND06_NSRU	323174	6251771	14.9	9	12	Siltstone	Jul 11 to Aug 11

R320_ND08_NSRU	323151	6251844	14.9	4	8	Siltstone	Jul 11 to Aug 11
R320_ND10_NSRU	323135	6251900	14.8	5	8	Siltstone	Jul 11 to Aug 11
R320_T09_NSRU	323187	6251654	13.6	9	12	Siltstone	Jul 11 to Aug 11
R320_T14_NSRU	323175	6251685	14	9	12	Siltstone	Jul 11 to Aug 11
Notes: * VWP sensor vertical depth below ground level							

TABLE 3-2: SUMMARY OF GROUNDWATER LEVELS AT NORTH STRATHFIELD STATION

Bore ID	Ground Surface Elevation (m AHD)	Unit(s)	Monitoring Period	Groundwater Elevation (m AHD)		Groundwater Depth (m bgl)	
				ApproxTypical	Maximum	Approx Typical	Shallowest
SMW_BH009_s	18.6	Clay and ASH	Aug 18 to Aug 19	14.8	15	3.9	3.7
SMW_BH009_w	18.5	HAW	Aug 18 to Aug 19	*	-13.4 *	*	31.9
SMW_BH035_s	26.6	ASH	Aug 18 to Aug 19	24.3	24.8	2.3	1.9
SMW_BH035_w	26.7	ASH/MIT	Sep 18 to Aug 19	-8.9	-8.7	35.6	35.4
SMW_BH036_v	27	ASH	Jul 18 to Aug 19	7.75**	10.3	19.2**	16.7
SMW_BH038_w	9.9	MIT and HAW	May 18 to May 19	-3.5	-3.4	13.4	13.3
SMW_BH039_v (sensor 1)	22.6	ASH	May 18 to Aug 19	14.6	15	8	7.6
SMW_BH039_v (sensor 2)	22.6	MIT	May 18 to Aug 19	-7	-6.6	29.6	29.2
SMW_BH073_w	18.9	ASH	Jun 20 to Sep 20	8.2	8.3	10.7	10.6
SMW_BH711_s	21.2	Clay and ASH	Mar 21 to Jun 21	15	15.2	6.2	6
SMW_BH711_w	21.2	ASH	Mar 21 to Jun 21	8.6	9.9	12.7	11.4
AF_BH33_w	19.9	ASH	Dec 21 to Jan 22	12.4	12.5	7.6	7.4
R320_ND02_NSRU	14.2	ASH	Jul 11 to Aug 11	10.9	11	3.3	3.1
R320_ND04_NSRU	14.5	ASH	Jul 11 to Aug 11	10.9	11.1	3.6	3.5

R320_ND06_NSRU	14.9	ASH	Jul 11 to Aug 11	12	12	2.9	2.9
R320_ND08_NSRU	14.9	ASH	Jul 11 to Aug 11	12.4	12.5	2.5	2.4
R320_ND10_NSRU	14.8	ASH	Jul 11 to Aug 11	12.8	13	2	1.8
R320_T09_NSRU	13.6	ASH	Jul 11 to Aug 11	10.1	10.2	3.5	3.3
R320_T14_NSRU	14	ASH	Jul 11 to Aug 11	9.6	9.7	4.4	4.3

Notes: ASH means Ashfield Shale, MIT means Mittagong Formation, HAW means Hawkesbury Sandstone, ND means no/insufficient data

Vibrating wire piezometers (VWPs) are identified with suffix "v", all others are standpipe piezometers (s and w being shallow and deep).

*General increasing trend over the monitoring period. Only maximum level shown.

**Levels are not stable, equilibrium not established. Inferred post-stabilisation groundwater level shown.

TABLE 3-3: SUMMARY OF GROUNDWATER LEVEL/DEPTHS AND STRATIGRAPHIC LOCATION OF GROUNDWATER LEVEL AT NORTH STRATHFIELD STATION

Bore ID	Effective Screened Unit(s)	Typical Groundwater Level (m AHD)	Typical Groundwater Level (m bgl)	Stratigraphic Location of Groundwater Level
SMW_BH009_s	Clay and ASH	14.8	3.9	1m into ASH
SMW_BH009_w	HAW	*	*	*
SMW_BH035_s	ASH	24.3	2.3	2 m into ASH
SMW_BH035_w	ASH/MIT	-8.9	35.6	35 m into ASH
SMW_BH036_v	ASH	7.8 **	19.2 **	4 m into ASH
SMW_BH038_w	MITT and HAW	-3.5	13.4	10 m into ASH
SMW_BH039_v (sensor 1)	ASH	14.6	8	1 m into ASH
SMW_BH039_v (sensor 2)	MIT	-7	29.6	24 m into ASH
SMW_BH073_w	ASH	8.2	10.7	6 m into ASH
SMW_BH711_s	Clay and ASH	15	6.2	1 m into ASH
SMW_BH711_w	ASH	8.6	12.7	5 m into ASH
AF_BH33_w	ASH	12.4	7.6	4 m into ASH
R320_ND02_NSRU	ASH	10.9	3.3	3 m into ASH
R320_ND04_NSRU	ASH	10.9	3.6	3 m into ASH
R320_ND06_NSRU	ASH	12	2.9	RS, 0.22 m above top of ASH
R320_ND08_NSRU	ASH	12.4	2.5	2 m into ASH
R320_ND10_NSRU	ASH	12.8	2	1 m into ASH
R320_T09_NSRU	ASH	10.1	3.5	2 m into ASH

R320_T14_NSRU	ASH	9.6	4.4	4 m into ASH

Notes: ASH means Ashfield Shale, MIT means Mittagong Formation, HAW means Hawkesbury Sandstone. RS means residual soil. * Increasing groundwater level trend over monitoring period. ** Levels are not stable, equilibrium not established. Inferred post-stabilisation groundwater level shown.

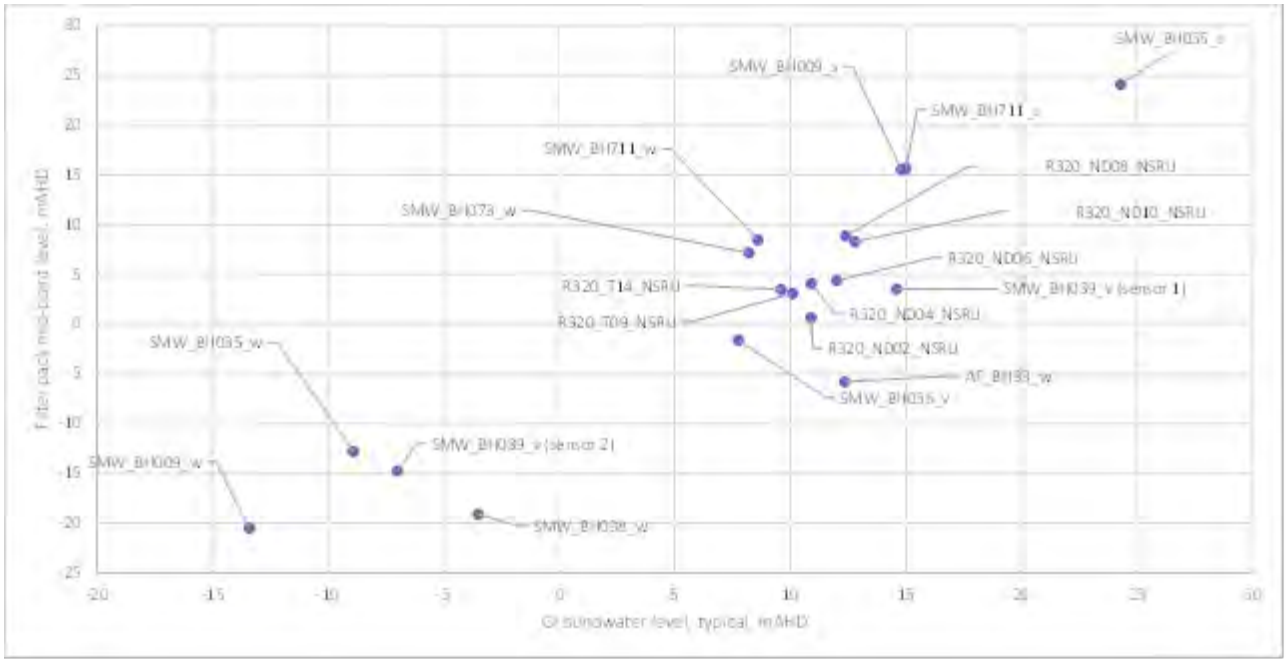


FIGURE 3-4: TYPICAL GROUNDWATER LEVEL VS FILTER PACK MID-POINT LEVEL, OR VWP LEVEL, AT NORTH STRATHFIELD SITE

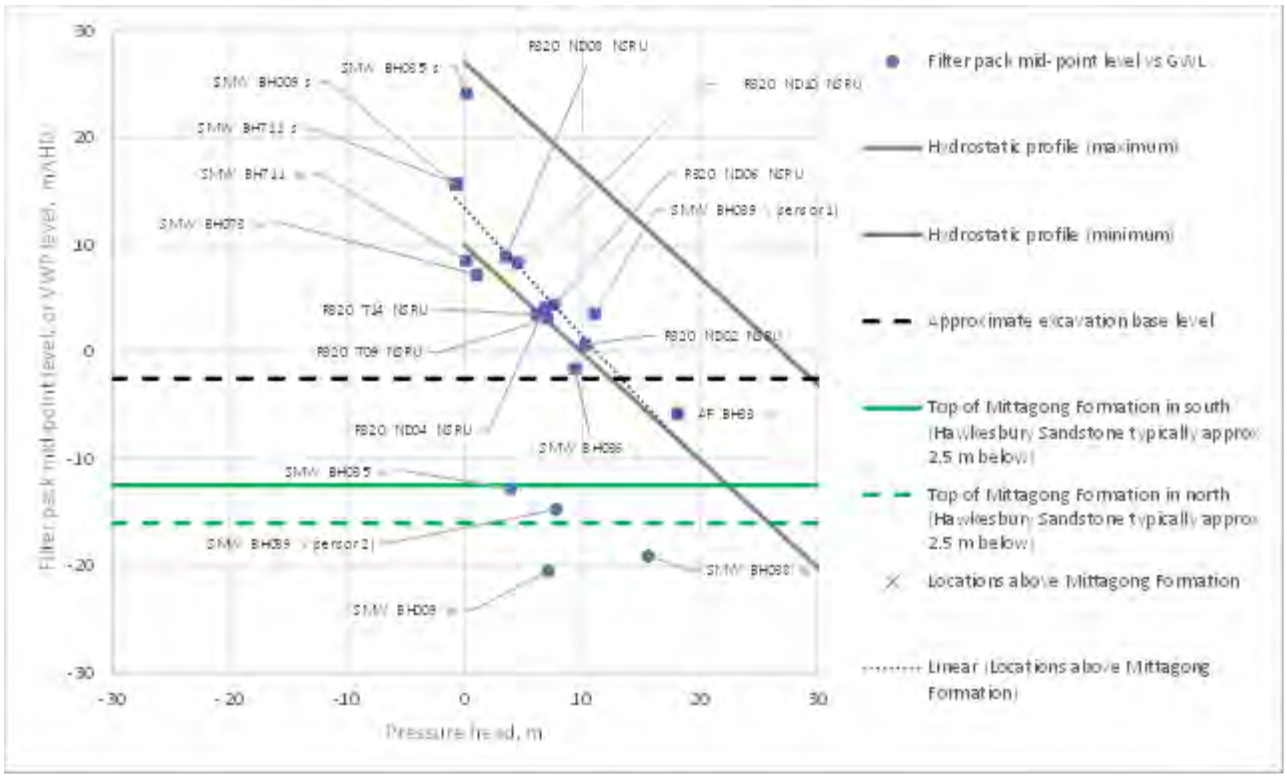


FIGURE 3-5: GROUNDWATER PRESSURE HEAD (CALCULATED FROM FILTER PACK MID-POINT, OR VWP LEVEL) IN PIEZOMETERS AT NORTH STRATHFIELD SITE, AND MINIMUM AND MAXIMUM

HYDROSTATIC PROFILES, INDICATIVE TOP OF MITTAGONG FORMATION LEVEL AND STATION EXCAVATION BASE LEVEL

3.2.2 RAINFALL AND DROUGHT

The cumulative mean monthly rainfall deviation since the year 2000 is shown in Figure 3-6 for rainfall recorded at the nearest Bureau of Meteorology station, at Concord Gold Club (Station 66013). The downward trend reflects a continuing period of below average rainfall, suggesting that drier conditions have prevailed over the last two decades (drought period). However, the period between 2018 and 2021 has not shown a net downward trend. This suggests that groundwater levels in more recent years are unlikely to be low in response to drought conditions, although groundwater levels may have fallen in the locality due to below-average rainfall in the period between 2015 and 2017.

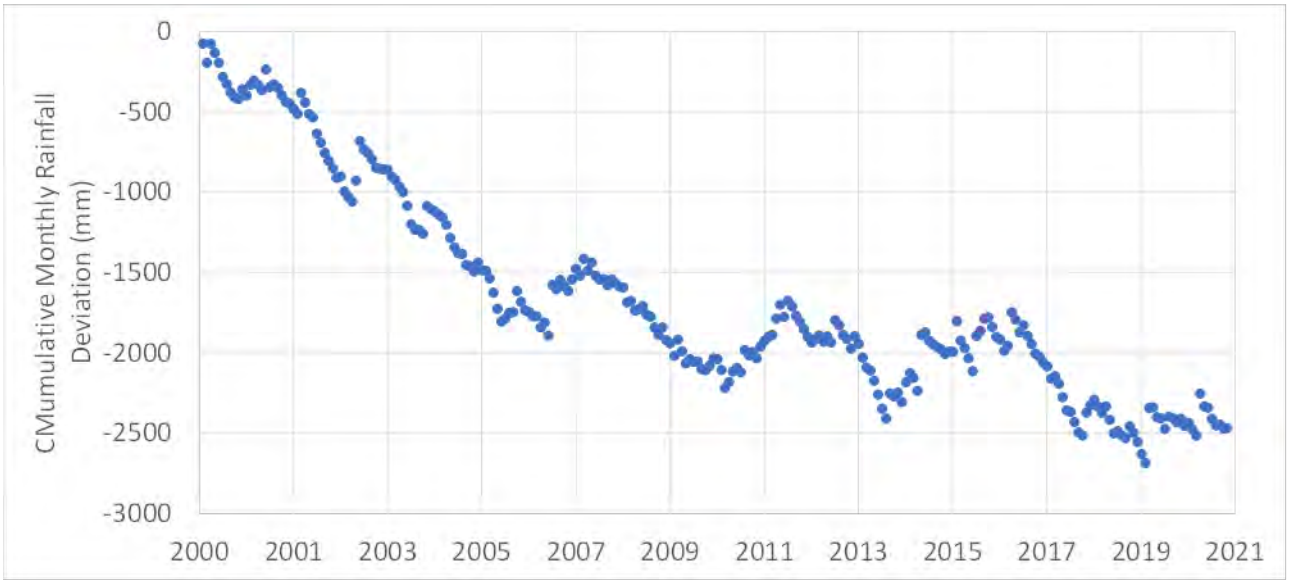


FIGURE 3-6: CUMULATIVE DEVIATION FROM MEAN MONTHLY RAINFALL AT CONCORD GOLF CLUB (BUREAU OF METEOROLOGY STATION 66013)

3.2.3 NORTH STRATHFIELD RAIL UNDERPASS PROJECT

The North Strathfield Rail Underpass project infrastructure (Figure 3-3) lie in the vicinity of North Strathfield Station, including a 148 m-long rail underpass completed in June 2015.

Groundwater data is available from the North Strathfield Rail Underpass project boreholes, with groundwater level data recorded from July 2011 to August 2011 for 17 monitoring bores in the area, seven of which are relatively close to the site. Packer test data is available from the project North Strathfield Rail Underpass Geotechnical Interpretive Report (SKM and Parsons Brinckerhoff, 2013). The existing status of groundwater levels is unknown.

Average groundwater levels for the seven closest North Strathfield Rail Underpass bores to the site ranged from 9.6 m AHD (4.4 m bgl) to 12.8 m AHD (2 m bgl) in 2011 monitoring. Groundwater piezometer monitoring depths range from 8 mBGL to 15 mBGL within Ashfield Shale.

The rail level reaches an approximate depth of 11 mBGL (4 m AHD).

The North Strathfield Rail Underpass project infrastructure is interpreted to not be currently drawing down groundwater levels applicable to the North Strathfield Station excavation. This interpretation is made based on the data as displayed in Figure 3-4 and Figure 3-5 in Section 3.2. This data shows that observed groundwater levels for locations with the filter pack mid-point level above the top of the Mittagong Formation are generally higher at SMW piezometers compared to North Strathfield Rail Underpass piezometers, whose groundwater levels were recorded prior to the construction of the underpass. The relatively low groundwater levels shown in Figure 3-4 and Figure 3-5 are

attributed to hydraulic disconnection between the Ashfield Shale and the underlying Mittagong Formation and/or Hawkesbury, not due to the North Strathfield Rail Underpass project.

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 North Strathfield 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.

A total of 35 packer tests have been undertaken in seven SMW boreholes, two AFJV bores and four reference boreholes near the site, as listed in Table 3-4. The four reference boreholes were drilled in 2011 for the North Strathfield Rail Underpass project. Of the total 35 packer tests, 21 packer tests are interpreted to have been completed exclusively in Ashfield Shale.

The results of in-situ permeability (packer) tests at North Strathfield Station are summarised in Table 3-4 and the packer test results plotted by depth below ground in Figure 3-7. Figure 3-7 also includes all SMW packer test results outside of the Bays paleo channel, to enable a comparison of the North Strathfield Station results to the broader CTP results.

Packer test results in the vicinity of North Strathfield Station range between less than 0.1 Lugeon (<8.7×10^{-4} m/day) and 16 Lugeons (1.4×10^{1} m/day). The median and arithmetic mean values of all the data at North Strathfield Station are 0.3 Lugeons (2.6×10^{3} m/day) and 1.7 Lugeons (1.5×10^{2} m/day), respectively. With respect to packer tests at North Strathfield Station which are interpreted to have been exclusively completed in Ashfield Shale, the median and arithmetic mean values are 0.4 (3.5×10^{3} m/day) and 2.5 Lugeons (2.2×10^{2} m/day), respectively.

There is a potential trend with depth at North Strathfield Station, with Lugeon value decreasing with depth. However, the correlation is not strong.

The arithmetic mean packer test result at North Strathfield Station of 1.7 Lugeons (1.5×10^{-2} m/day) for all tests and 2.5 Lugeons (2.2×10^{-2} m/day) for tests interpreted to have been exclusively completed in Ashfield Shale is similar to the arithmetic mean and 75th percentile statistic values of 1.6 and 1.8 Lugeons, respectively, for all packer tests completed within siltstone for the whole project (excluding tests completed in palaeochannel at The Bays Station site). The Ashfield Shale North Strathfield Station packer test median value of 0.4 is similar to, but slightly greater 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 North Strathfield 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.

Relatively high Lugeon values were reported for packer tests conducted in borehole SMW_BH073 (included Lugeon values of 11 and 16), located near southern extent of station excavation, and R320_ND04 (Lugeon value of 10), located about 100 m south of the station excavation. The relatively high Lugeon values at SMW_BH073 are interpreted to be associated with the inferred zone of faulting / joint swarms. Similarly, the relatively high value at R320_ND04 is interpreted to be associated with an inferred fault. Additionally, borehole AF_BH32i, drilled at an angle of 66 degrees, intersected the most northern inferred fault. Packer testing within AF_BH32i across the depth zone of the structure, represented by core loss and joint swarms, returned a maximum Lugeon value of 5.

It is possible that rock in the vicinity of the inferred fault zones is of higher permeability than the adjacent rock. However, apart from the tests at SMW_BH073 and R320_ND04 noted above and AF_BH32i, the data does not support this notion. Other tests undertaken near the inferred faulting did not return relatively elevated Lugeon values.

The implications of potential enhanced hydraulic conductivity in the vicinity of the fault zones are discussed in Section 6.4 and 6.5.

It is possible that potential weathered zones of country rock adjacent to the inferred dyke would exhibit relatively higher permeability than the surrounding rock or metamorphosed zones. However, the available data does not support this notion.

TABLE 3-4: SUMMARY OF PACKER TEST RESULTS AT NORTH STRATHFIELD STATION

Bore ID	Depth Top, vertical	Depth Bottom, vertical	Material	Interpreted formation	Result	
	m	m			Lugeons	m/day
SMW_BH009_w	32.5	39.3	Siltstone, interlaminated and interbedded sandstone and siltstone, sandstone	ASH, MIT, HS	0.2	$1.73x10^{-3}$
SMW_BH009_w	24.0	27.2	Interlaminated siltstone and sandstone, siltstone	ASH	0.4	$3.47x10^{-3}$
SMW_BH009_w	26.5	33.4	Siltstone	ASH	0.3	$2.60x10^{-3}$
SMW_BH009_w	38.5	43.4	Sandstone	HS	3	$2.60x10^{-2}$
SMW_BH035_w	37.0	42.9	Siltstone, sandstone	ASH, MIT, HS	0.1	$8.67x10^{-4}$
SMW_BH035_w	41.8	48.0	Sandstone	HS	<0.1	$<8.67x10^{-4}$
SMW_BH035_w	47.0	51.5	Sandstone	HS	<0.1	$<8.67x10^{-4}$
SMW_BH036_v	51.7	57.3	Sandstone	HS	0.1	$8.67x10^{-4}$
SMW_BH036_v	36.0	44.5	Siltstone and sandstone, sandstone	ASH, MIT, HS	0.1	$8.67x10^{-4}$
SMW_BH036_v	45.5	51.9	Sandstone	HS	2	$1.73x10^{-2}$
SMW_BH036_v	32.2	37.1	Siltstone, siltstone and sandstone	ASH	0.1	$8.67x10^{-4}$
SMW_BH038_w	29.5	34.0	Sandstone	HS	0.4	$3.47x10^{-3}$
SMW_BH038_w	23.5	30.0	Siltstone, interlaminated and interbedded siltstone and sandstone, sandstone	ASH, MIT, HS	0.4	$3.47x10^{-3}$
SMW_BH038_w	18.0	24.0	Interlaminated siltstone and sandstone, siltstone	ASH	0.2	$1.73x10^{-3}$
SMW_BH039_v	31.8	37.1	Interlaminated and interbedded siltstone and sandstone, siltstone	ASH	0.1	$8.67x10^{-4}$
SMW_BH039_v	54.1	59.4	Sandstone	HS	0.6	$5.20x10^{-3}$
SMW_BH039_v	49.1	54.4	Predominantly sandstone, very minor siltstone interval (0.15 m thick)	HS	0.2	$1.73x10^{-3}$

Bore ID	Depth Top, vertical	Depth Bottom, vertical	Material	Interpreted formation	Result	
	m	m			Lugeons	m/day
SMW_BH039_v	35.4	42.1	Siltstone, interbedded and interlaminated siltstone and sandstone, sandstone	ASH, MIT	0.2	$1.73x10^{-3}$
SMW_BH039_v	41.7	49.5	Sandstone	HS	0.2	$1.73x10^{-3}$
SMW_BH073_w	17.0	22.5	Siltstone, interlaminated siltstone and sandstone	ASH	0.5	$4.33x10^{-3}$
SMW_BH073_w	8.0	10.7	Siltstone	ASH	11	$9.53x10^{-2}$
SMW_BH073_w	9.7	16.6	Siltstone	ASH	16	$1.39x10^{-1}$
SMW_BH711_w	14.0	20.9	Siltstone	ASH	0.3	$2.60x10^{-3}$
SMW_BH711_w	19.5	26.2	Siltstone, interbedded and interlaminated siltstone and sandstone	ASH	0.1	$8.67x10^{-4}$
SMW_BH711_w	10.0	14.9	Siltstone	ASH	0.8	$6.93x10^{-3}$
AF_BH32i	12.8	18.3	Siltstone	ASH	5	$4.33x10^{-2}$
AF_BH32i	17.8	23.8	Siltstone, interlaminated siltstone and sandstone	ASH	0.6	$5.20x10^{-3}$
AF_BH32i	23.3	30.3	Interlaminated siltstone and sandstone, siltstone	ASH	0.1	$8.67x10^{-4}$
AF_BH33_w	12.0	18.2	Siltstone	ASH	2.5	$2.17x10^{-2}$
AF_BH33_w	17.5	24.2	Siltstone	ASH	0.8	$6.93x10^{-3}$
AF_BH33_w	23.5	30.2	Siltstone	ASH	<0.1	$<8.67x10^{-4}$
R320_T09	7.0	12.0	Siltstone	ASH	2.5	$2.17x10^{-2}$
R320_ND02	12.0	16.2	Siltstone	ASH	0.1	$8.67x10^{-4}$
R320_ND04	8.0	12.0	Siltstone	ASH	10	$8.67x10^{-2}$
R320_ND06	7.0	12.2	Siltstone	ASH	0.1	$8.67x10^{-4}$
Summary statistics						
All packer tests near station (n = 35)						
Median					0.3	$2.60x10^{-3}$
Arithmetic mean					1.7	$1.47x10^{-2}$
Maximum					16.0	$1.39x10^{-1}$
Packer tests completed in Ashfield Shale near station (n = 21)						
Median					0.4	$3.47x10^{-3}$
Arithmetic mean					2.5	$2.13x10^{-2}$
Maximum					16.0	$1.39x10^{-1}$

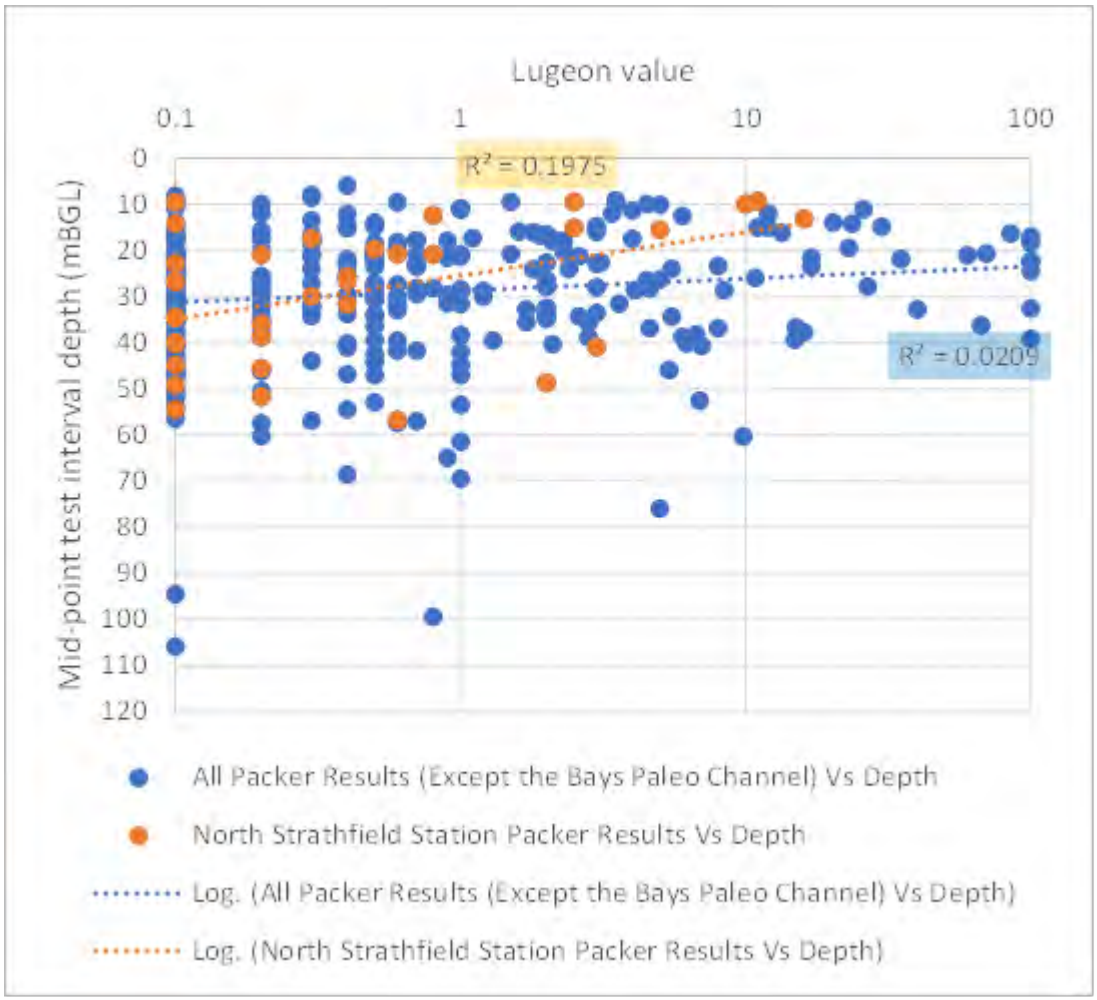


FIGURE 3-7: LUGEON VALUES WITH DEPTH BELOW GROUND SURFACE FOR PACKER TESTS NEAR NORTH STRATHFIELD STATION, AND ALL CTP PACKER TESTS EXCEPT THE BAYS PALEO CHANNEL

Typical ranges and adopted representative hydrogeological parameter values to represent the Ashfield Shale and Hawkesbury Sandstone hydrogeological units for the CTP as a whole (excluding The Bays area) are summarised in Table 3-5. 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.

The adopted representative hydraulic conductivity value for Ashfield Shale of 1.20×10^{-2} m/d (1.4 Lugeons) is similar to, but slightly less than, the arithmetic mean of Ashfield Shale North Strathfield Station packer tests of 2.1×10^{-2} m/day (2.5 Lugeons), and greater than the median of Ashfield Shale North Strathfield Station packer tests of 3.5×10^{-3} m/day (0.4 Lugeons).

TABLE 3-5: 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)	K_v/K_h 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)	0.1 to 1.0	5.00×10^{-6} to 1.00×10^{-5}	0.01 to 0.025

Hydrogeological unit	Typical hydraulic conductivity range (m/day)	K_v/K_h range	Specific storage range (m^{-1})	Specific yield range (-)
	Typical range			
	(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)			
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; 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			
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. DESIGN GROUNDWATER LEVELS

4.1 REQUIREMENTS

Design groundwater levels have been developed considering, and consistent with, the Particular Specifications, as listed in Table 4-1.

TABLE 4-1: PARTICULAR SPECIFICATIONS RELEVANT TO DEVELOPMENT OF DESIGN 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. Civil and structural elements including foundations retaining structures, tunnel portals, tunnel elements, shaft structural elements, and other structural load bearing elements are required to have a design life of 120 years [SM-W-CTP-PS-548]
6. 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]
7. 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]
8. The Tunnelling Contractor must design for the risk of water pressure build-up as a result of blocked drainage. [SM-W-CTP-PS-1030]

4.2 CTP WORK CONDITIONS

The North Strathfield Station box excavation 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, consistent with Particular Specification SM-W-CTP-PS-715.

4.3 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 and the Geotechnical Design Report for more details on this.

4.4 FLOODING

Flooding at North Strathfield Station has the potential to impose groundwater pressures on the station box retaining wall in excess of the general structural/geotechnical load cases. The flooding (accidental load) scenario represents the critical design load scenario for portions of the western retaining wall. See Annexure C for potential groundwater pressures imposed on the western retaining wall due to flooding.

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 and the Geotechnical Design Report for more details on this.

5. GROUNDWATER QUALITY

Groundwater quality at North Strathfield Station is discussed comprehensively in the Contamination Assessment Report. A summary of pH and electrical conductivity (EC) values reported for the purpose durability assessment in the Contamination Assessment Report, and a summary of guideline exceedances reported in the Contamination Assessment Report, is presented in Table 5-1.

Available data indicate that:

- The groundwater pH values reported in the Contamination Assessment Report are within the ANZECC (2000) guideline trigger level range of between 6.5 and 8.5 (lowland rivers) for 95% Protection of freshwater aquatic ecosystems
- The groundwater EC values reported in the Contamination Assessment Report generally exceed the ANZECC (2000) guideline trigger level of 2,200 µS/cm (lowland rivers) for 95% Protection of freshwater aquatic ecosystems
- Groundwater manganese concentrations exceed Human Health Criteria and Recreational and Aesthetic criteria at some locations
- Groundwater exceeded the ANZG (2018) guideline trigger level for 95% Protection of freshwater aquatic ecosystems for some metals (nickel, zinc, manganese, arsenic, copper and aluminum) and ammonia (as N).

Based on the above, groundwater seepage is likely to require dilution or treatment prior to offsite discharge.

TABLE 5-1: SUMMARY OF GROUNDWATER PH AND EC, AND GROUNDWATER QUALITY EXCEEDANCES

Piezometer	Effective Screened Unit(s)	pH	EC (µS/cm)	Exceedances (Human Health Criteria)	Exceedances (Ecological Criteria: 95% Protection - Freshwater)
SMW_BH009_w	HAW	6.97	13,700	0	Ammonia as N, Nickel, Zinc
SMW_BH009_s	Clay and ASH	8.01	880	0	Manganese
SMW_BH035_w	ASH/MIT	6.94	13,700	0	Ammonia as N, Nickel, Zinc
SMW_BH035_s	ASH	-	-	-	-
SMW_BH038_w	MIT/HAW	7.44	8,370	0	Ammonia as N, Arsenic, Copper, Nickel, Zinc
SMW_BH073_w	ASH	6.81	5,500	1 (manganese)*	Manganese, Nickel, Zinc
SMW_BH711_w	ASH	7.02	9,140	0	Copper, Nickel, Zinc
SMW_BH711_s	Clay and ASH	6.62	4,720	0	Copper, Manganese, Nickel, Zinc
AF_BH33	ASH	7.60	1,200	1 (manganese)*	Aluminium, Copper and Zinc

- No results reported for these parameters. *Exceeds Recreational and Aesthetic criteria

6. GROUNDWATER INFLOW AND DRAWDOWN

6.1 OVERVIEW

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

The model was developed using Geoslope's Geostudio SEEP/W, a finite element modelling software package for modelling groundwater flow in porous media.

Details of the modelling are covered in Annexure D.

6.2 MODEL LAYERS

Three hydrogeological units are represented in the model: Quaternary Alluvium, Ashfield Shale and Hawkesbury Sandstone. Fill and residual soil units are not included in the model because the water table is generally 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 2.5 m to 4.0 m thick) and is characteristically similar to the Hawkesbury Sandstone in its hydrogeological properties.

The Ashfield Shale and Hawkesbury Sandstone units are continuous throughout the model section, whereas the alluvium is non-continuous and comprises two deposits situated southwest of North Strathfield Station, which are associated with Powells Creek.

The extent of the alluvium is currently unknown. For this reason, two different alluvium profiles were represented in separate versions of the model, a base case scenario, which represented the alluvium extent based on the Sydney 100,000 geological map; and an extended alluvium scenario, which represented the northeastern alluvium deposit associated with Powells Creek to extend to the east of the current Powells Creek concrete lined channel.

6.3 ADOPTED HYDROGEOLOGICAL PARAMETER VALUES FOR MODELLING

Hydrogeological parameter values adopted for the modelling are shown in Table 6-1 and for the Ashfield Shale and Hawkesbury Sandstone hydrogeological units were as per the adopted representative values outlined in Table 3-5, Section 3.3.

Hydrogeological parameter values adopted for the alluvium hydrogeological unit were based on a regional literature review, as documented in the hydrogeological properties annexure, Annexure B.

Due to the lack of borehole data covering the alluvium to the southwest of the station, the alluvium composition is not known. To address this uncertainty, both the base case model and extended alluvium case model represented the alluvium as predominantly sandy, and separately, as predominantly clayey. This is considered to cover the potential range of hydrogeological characteristics that the alluvium material might possess.

TABLE 6-1: HYDROGEOLOGICAL PARAMETER VALUES APPLIED IN MODEL

Parameter	Quaternary alluvium	Ashfield Shale	Hawkesbury Sandstone	Justification
Saturated horizontal hydraulic conductivity (m/d)	1.00 (predominantly sandy) 0.005 (predominantly clayey)	0.012	0.0173	Alluvium based on regional literature review, as documented in hydrogeological properties annexure, Annexure B. Ashfield Shale and Hawkesbury Sandstone 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	N/A	100	100	Applied over North Strathfield Station excavation area to represent free drainage within the excavation that would occur during excavation
Kv/Kh [1]	0.1	0.1	0.1	Based on regional literature review, as documented in hydrogeological properties annexure, Annexure B
Specific yield	0.20 (predominantly sandy) 0.06 (predominantly clayey)	0.02	0.05	Based on regional literature review, as documented in hydrogeological properties annexure, Annexure B
Coefficient of volume compressibility (kPa^{-1})	1.02×10^{-6}	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.

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 36 m³/d. The predicted groundwater inflow rates are similar for the base case alluvium extent and extended alluvium cases, and are similar when the alluvium is represented as predominantly sandy, or alternatively, as predominantly clayey. The highest groundwater inflow rates occur under the extended alluvium case, when the alluvium is represented as predominantly clayey. This is due to relatively higher head in the area of the alluvium and therefore relatively higher hydraulic gradients between the area of the alluvium and station excavation.

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 6-2. 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.

As discussed in Section 3.1 and 3.3, two inferred faults pass through the central portion of the station excavation, with a third inferred fault passing through the southern nozzle, coming within very close proximity to the southeastern station excavation corner. Additionally, there are two other inferred faults of the same orientation located 60 m and 90 m south of the station.

It is possible that rock in the vicinity of the inferred fault zones is of higher permeability than the adjacent rock. However, apart from tests at SMW_BH073, R320_ND04 and AF_BH32i, the packer test data for the site does not support this notion. Other tests undertaken near the inferred faulting did not return relatively elevated Lugeon values.

If three fault zones with enhanced hydraulic conductivity are assumed to pass through the station excavation at a north-northeast orientation and each have an assumed width of 1.5 m (inferred from AF_BH32i), the total area of enhanced hydraulic conductivity relative to the total station excavation area would be approximately 5 percent. Assuming the maximum packer test value of 16 Lugeons (0.9 m/d) for the zone of enhanced hydraulic conductivity, under these circumstances, the bulk hydraulic conductivity in the area of the station excavation could be about 1.6 times higher than the value adopted for modelling. As a result, groundwater inflows under these circumstances could be expected to be approximately 1.6 times higher than modelled and would still be below the inflow rate Particular Specifications. However, if a value of 500 Lugeons (4.3 m/d) is assumed for the zone of enhanced hydraulic conductivity, under these circumstances, the bulk hydraulic conductivity in the area of the station excavation could be up to 20 times higher than the value adopted for modelling, and therefore groundwater inflows could be expected to be about 20 times higher than modelled. Under these circumstances, the inflow rate Particular Specifications would be exceeded and localised grouting during excavation would be required to limit groundwater inflows to the Particular Specification criteria. Additionally, if hydraulic conductivity values are elevated in other not-yet identified zones, then groundwater inflows may be potentially higher. Should water-bearing features be encountered during excavation, groundwater inflows may be higher than estimated, and localised grouting during excavation may be required to limit groundwater inflows to the Particular Specification criteria.

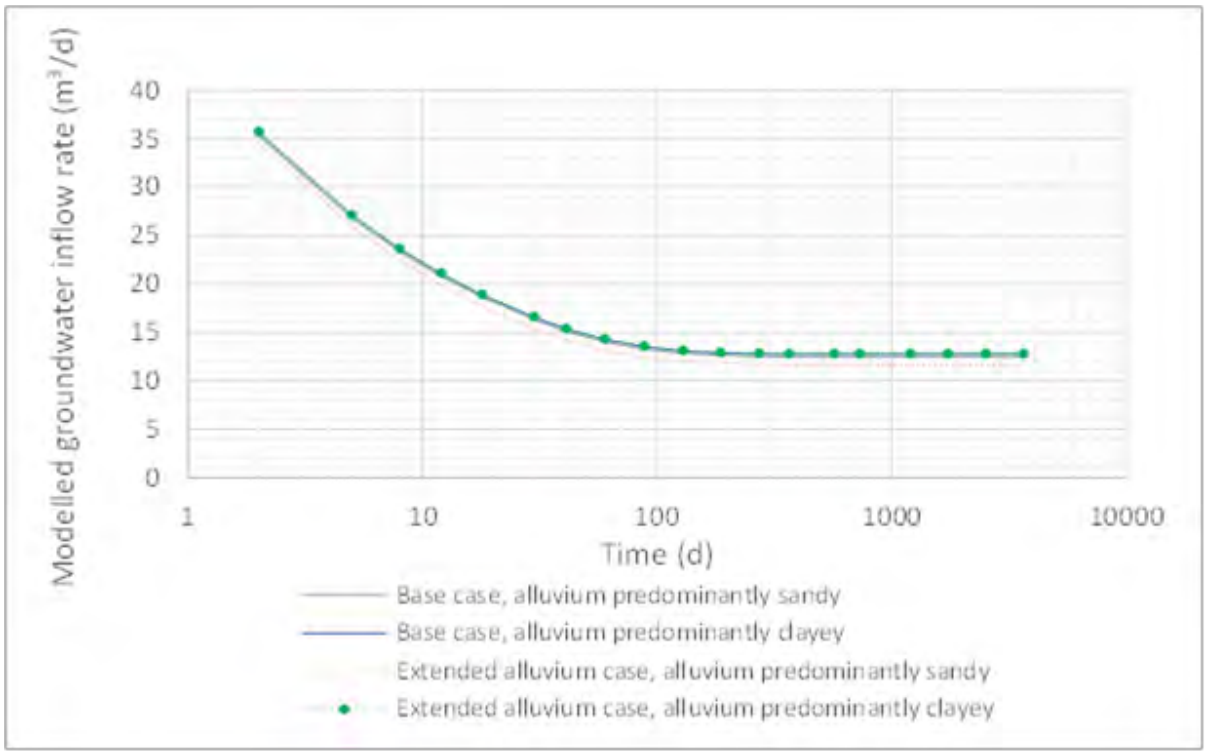


FIGURE 6-1: GROUNDWATER INFLOW RATES CALCULATED BY MODEL

TABLE 6-2: 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 36	92
Any square with an area of 10m², at any and all locations within the sides and bases of the shafts and excavations	Not modelled. Inflows over a given 10 m² area will be dependent on water-bearing features encountered during excavation and will require localised grouting during excavation should inflows exceed criteria	15

6.4.2 CUMULATIVE INFLOW VOLUMES COMPARED TO EIS

The cumulative groundwater inflow volume calculated by the model is compared to the EIS cumulative inflow prediction in Table 6-3. The cumulative inflow calculated by the model is less than the EIS prediction.

TABLE 6-3: 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)
10	34

6.5 GROUNDWATER LEVEL DRAWDOWN

6.5.1 WATER TABLE DRAWDOWN AND COMPARISON TO EIS

Drawdown of the watertable predicted by the base case model is shown in Figure 6-2 and compared to the drawdown predicted in the EIS. the predicted drawdown is similar, but slightly smaller in extent,

to the drawdown predicted in the EIS. the differences are interpreted to be due to different boundary conditions.

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 shown in Figure 6-2.

To the southwest and northeast of the station, respectively, the 2 m drawdown level is about 100 m and 170 m closer to the station than the 2 m drawdown contour predicted in the EIS.

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, dykes 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.

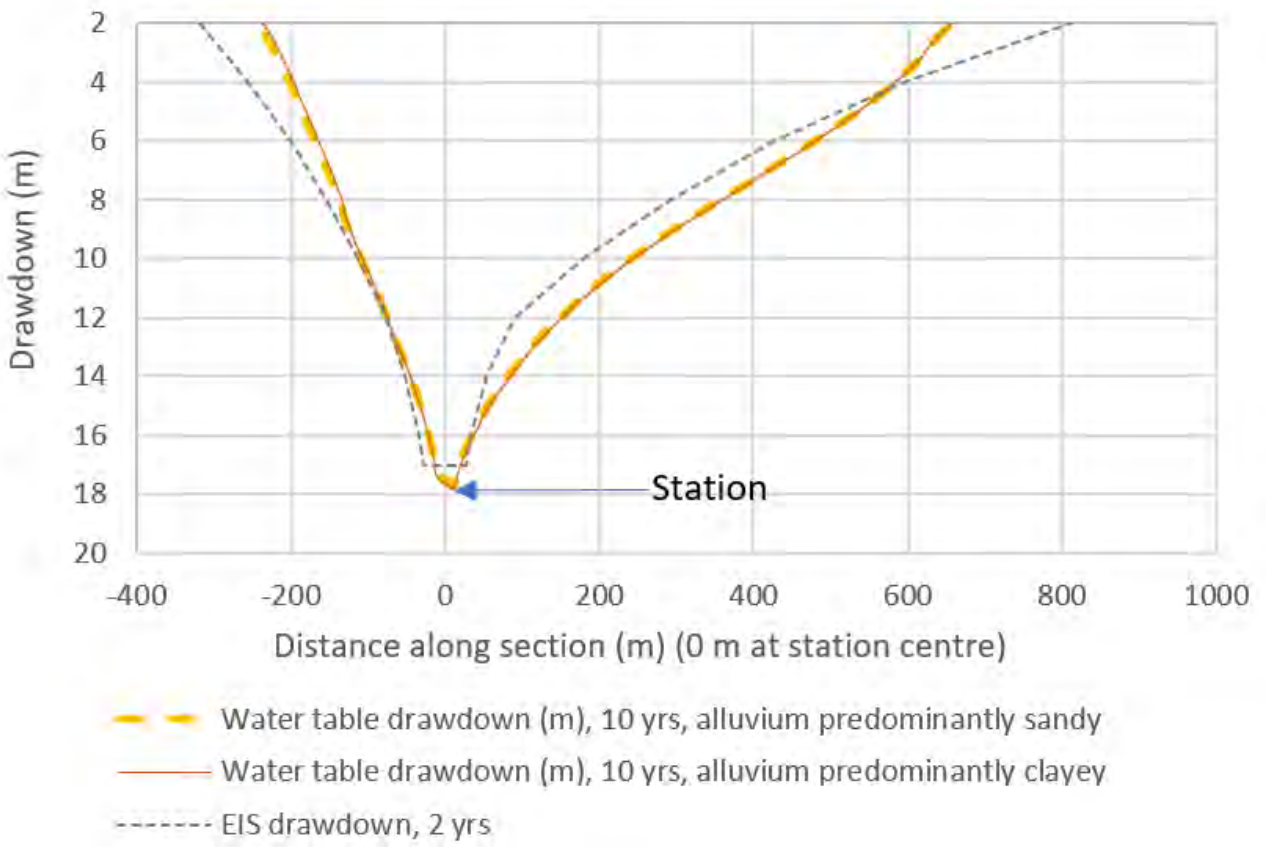


FIGURE 6-2: BASE CASE GROUNDWATER LEVEL DRAWDOWN PREDICTED BY MODEL COMPARED TO THAT IN EIS

6.5.2 WATER TABLE DRAWDOWN IN ALLUVIUM

The predicted “worst case” drawdown is shown in Figure 6-3. The maximum modelled drawdown was approximately 0.88 m and occurred at the eastern extent of the eastern alluvium. The drawdown was negligible in the southwestern portion of the alluvium.

It is noted that drawdown in the alluvium could be greater if the recharge rate is lower than adopted in the model. However, the modelled recharge rate that was applied over the areas of alluvium is considered reasonable. Additionally, supplementary model runs with a uniform recharge rate of one percent resulted in drawdown less than that of the “worst case” scenario described above.

It should be noted that there is significant uncertainty in these predictions as the nature, depth and extend of the alluvium is unknown. Construction phase monitoring of groundwater level drawdown and ground movement in the vicinity of the station will serve to assess the risk of potential impacts due to potential drawdown in the alluvium during construction.

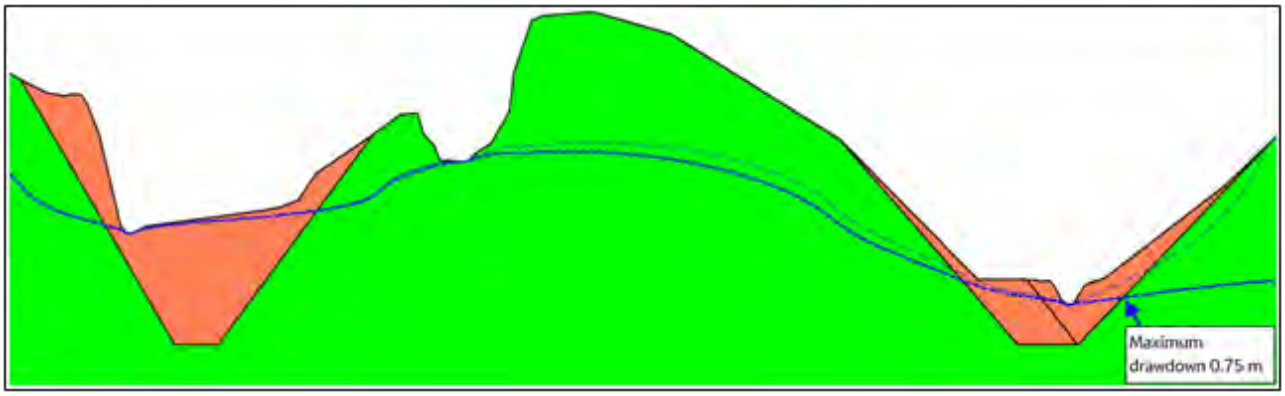


FIGURE 6-3: BASE CASE GROUNDWATER LEVEL DRAWDOWN PREDICTED BY MODEL COMPARED TO THAT IN EIS

7. GROUNDWATER IMPACTS

7.1 GROUNDWATER USERS AND RECEPTORS

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

With respect to existing registered groundwater bores, there are no existing registered bores within the predicted 2 m drawdown contour.

The EIS (Jacobs, 2020) indicates Turpentine Grey Iron Bark open forest groundwater dependent ecosystems (GDEs) are present from about 580 m northeast of the station. Predicted drawdown at this location is up to 4 m in the Ashfield Shale.

This ecosystem system is situated within the Concord Golf Club golfcourse. Thus, the ecosystem currently exists in a highly modified and urbanised setting.

The ecosystem typically grows on shale and the rootzone is likely to lie within residual clay soils of the shale and/or the shale itself (where the shale is shallow). The Ashfield Shale typically has relatively low permeability. Station-induced groundwater level drawdown within the shale is considered unlikely to impact this ecosystem.

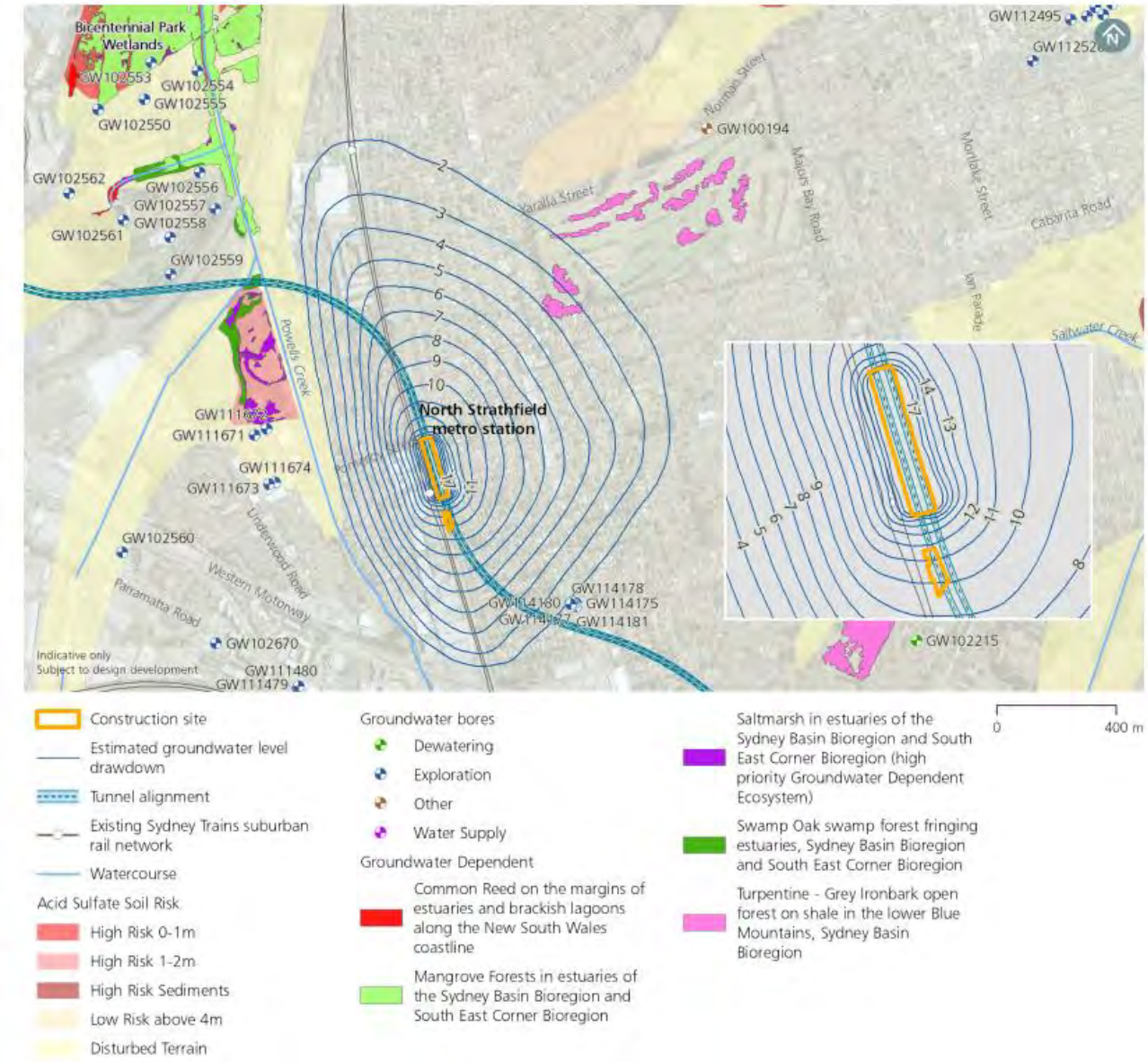


FIGURE 7-1: GROUNDWATER RECEPTORS NEAR NORTH STRATHFIELD STATION, AND DRAWDOWN PREDICTED IN THE EIS (JACOBS, 2020)

7.2 ACID SULFATE SOILS

The Contamination Assessment Report indicates that, based on the site's elevation, underlying geology, available mapping and laboratory testing, there is a low potential for the occurrence of actual or potential acid sulfate soils in the vicinity of the station.

Potential acid sulfate soils were not identified within the predicted extent of groundwater level drawdown. However, disturbed soils are present to the west of the modelled 2 m groundwater level drawdown extent. It is possible that construction excavation induced drawdown could impact potential acid sulfate soils in that area, if present.

7.3 SETTLEMENT

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

7.4 CONTAMINATION

As noted in the Contamination Assessment Report, and Section 5, groundwater in the vicinity of the North Strathfield Station was slightly acidic to slightly alkaline and there were instances where manganese concentrations exceed Human Health Criteria and Recreational and Aesthetic criteria

at some locations, and instances where some metals (nickel, zinc, manganese, arsenic, copper and aluminum) and ammonia (as N) exceeded ANZG (2018) guideline trigger level for 95% Protection of freshwater aquatic ecosystems.

There are potential contamination risks associated with locations beyond the site. The North Strathfield Station excavation is anticipated to act as a groundwater sink. It is therefore possible that contaminated groundwater at distance from the site will be drawn towards the excavation. Based on average linear flow velocity calculations, for the 10 year design life, this distance from the site is likely to be less than approximately 100 m. However, it could be greater if (a)water bearing feature(s) with relatively high permeability is present. The risk of contaminated groundwater at distance from the site is unknown.

7.4.1 GROUNDWATER SEEPAGE TO STATION BOX EXCAVATION

Based on the Contamination Assessment Report, groundwater seepage to the excavation 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.

7.4.2 SALINE INTRUSION

Considering the predicted extent of groundwater level drawdown, and given that the nearest saline water body (Mason Park Wetlands is greater than 400 m from the North Strathfield Station, the risk of saline water migrating to the excavation over the 10 year design life is considered low.

The hydraulic gradient induced by the station box excavation is considered insufficient to cause migration of saline water within the Ashfield Shale to reach the station site over the 10 year design life.

8. CONSTRUCTION PHASE MONITORING

Table 8-1 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 8-1: SUMMARY OF RECOMMENDED CONSTRUCTION PHASE MONITORING LOCATIONS AND PREDICTED DRAWDOWN

Location I.D.	Existing / proposed monitoring location	Easting (56 MGA94)	Northing (56 MGA94)	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 water table drawdown, two years after excavation commenced (m)
SMW_BH009_s	Existing	323220	6251758	18.6	1 to 5	Clay and ASH	75 m south	14.8	1.2	12.5 [a]
SMW_BH009_w	Existing	323220	6251759	18.5	37.5 to 40.5	HAW	75 m south	-13.4**	8.6	12.5 [a]
SMW_BH035_s	Existing	323361	6251851	26.6	1.7 to 3.2	ASH	170 m east	24.3	0.9	8.1 [a]
SMW_BH035_w	Existing	323362	6251851	26.7	33.5 to 45.5	ASH/MIT	170 m east	-8.9	9.9	8.1 [a]
SMW_BH036_v *	Existing	323375	6251745	27	28.59*	ASH	200 m southeast	7.75***	9.3	6.9 [a]
SMW_BH038_w	Existing	323008	6251870	9.9	26 to 32	MIT and HAW	145 m west	-3.5	18.6	6.6
SMW_BH039_v (sensor 1) *	Existing	323201	6251939	22.6	19.05	ASH	35 m east	14.6	11.1	15.4
SMW_BH039_v (sensor 2) *	Existing	323201	6251939	22.6	37.35	ASH	40 m east	-7	7.8	15.4
AF_BH33_w	Existing	323138	6252011	19.9	21.2 to 30.2	ASH	8 m north	12.4	22.7	16.7 [a]

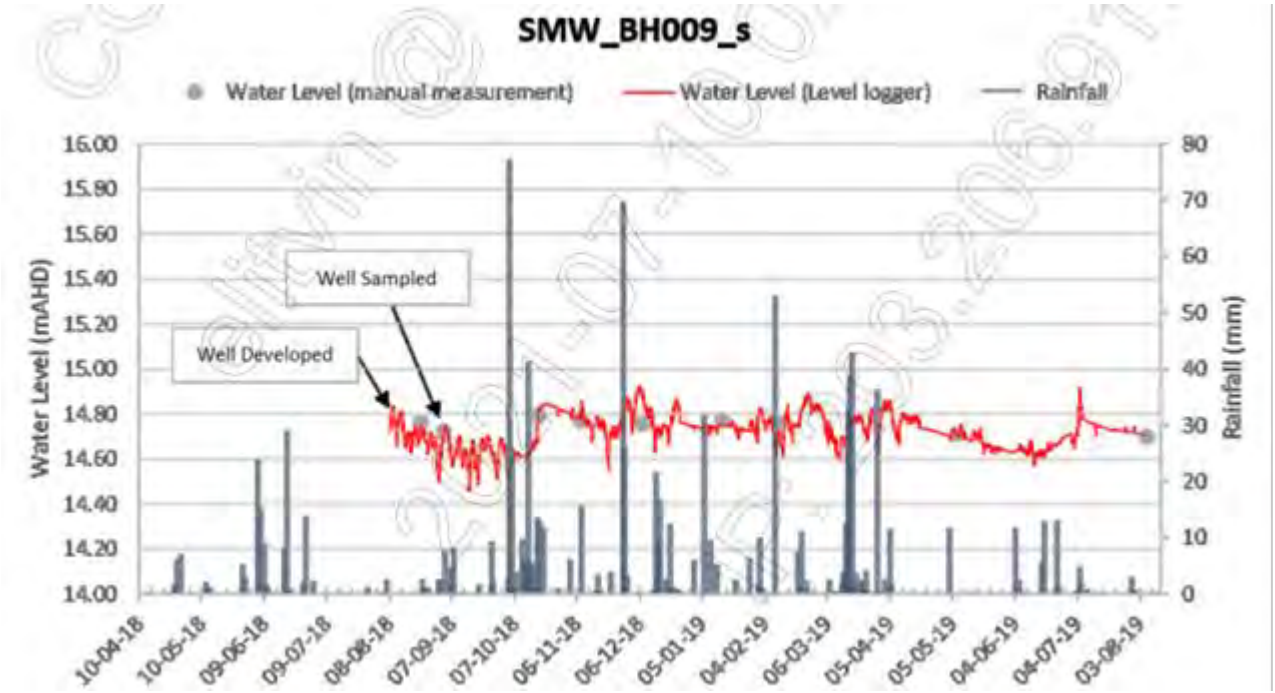
Notes: [a] Monitoring piezometer located perpendicular to model section. Average of drawdown to northeast and southwest of station adopted. * Denotes VWP. ** General increasing trend over the monitoring period. Maximum level adopted as representative level. *** Equilibrium in groundwater level not established. Representative level estimated based on hydrograph. ASH means Ashfield Shale, MIT means Mittagong Formation and HAW means Hawkesbury Sandstone.

FIGURE 8-1: CONSTRUCTION PHASE GROUNDWATER LEVEL MONITORING LOCATIONS

REFERENCES

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

ANNEXURE A: HYDROGRAPHS



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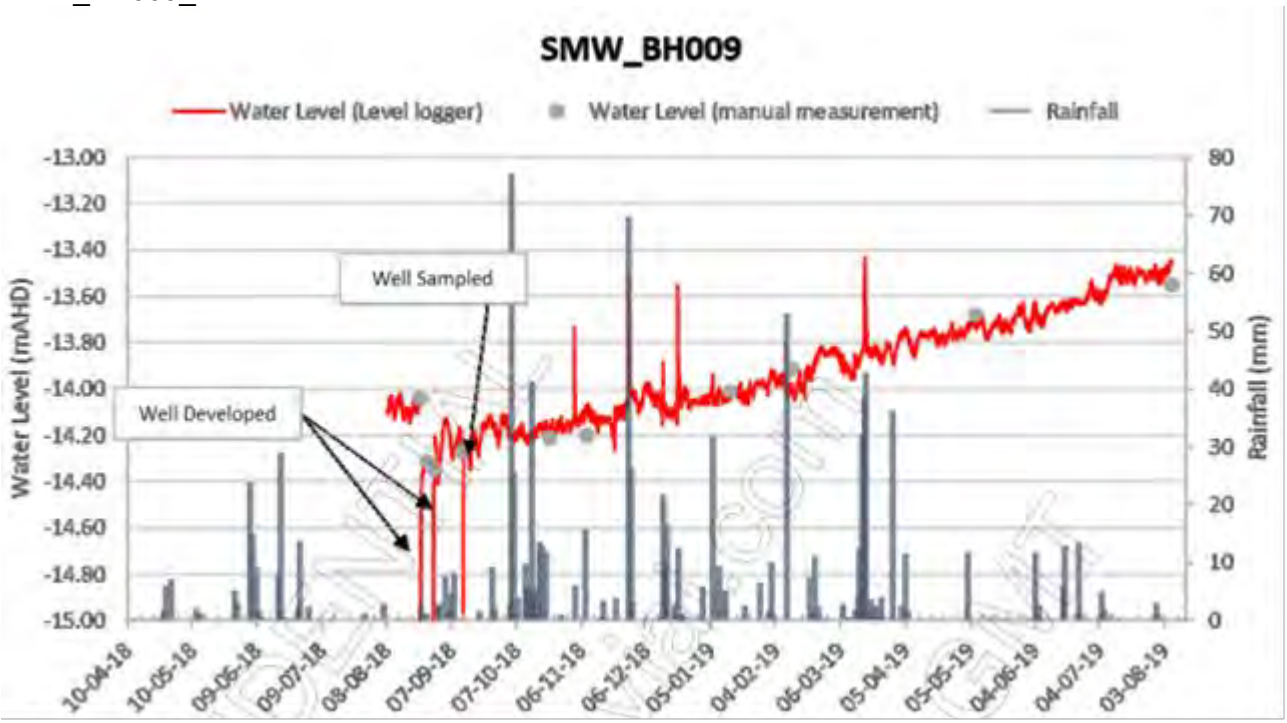

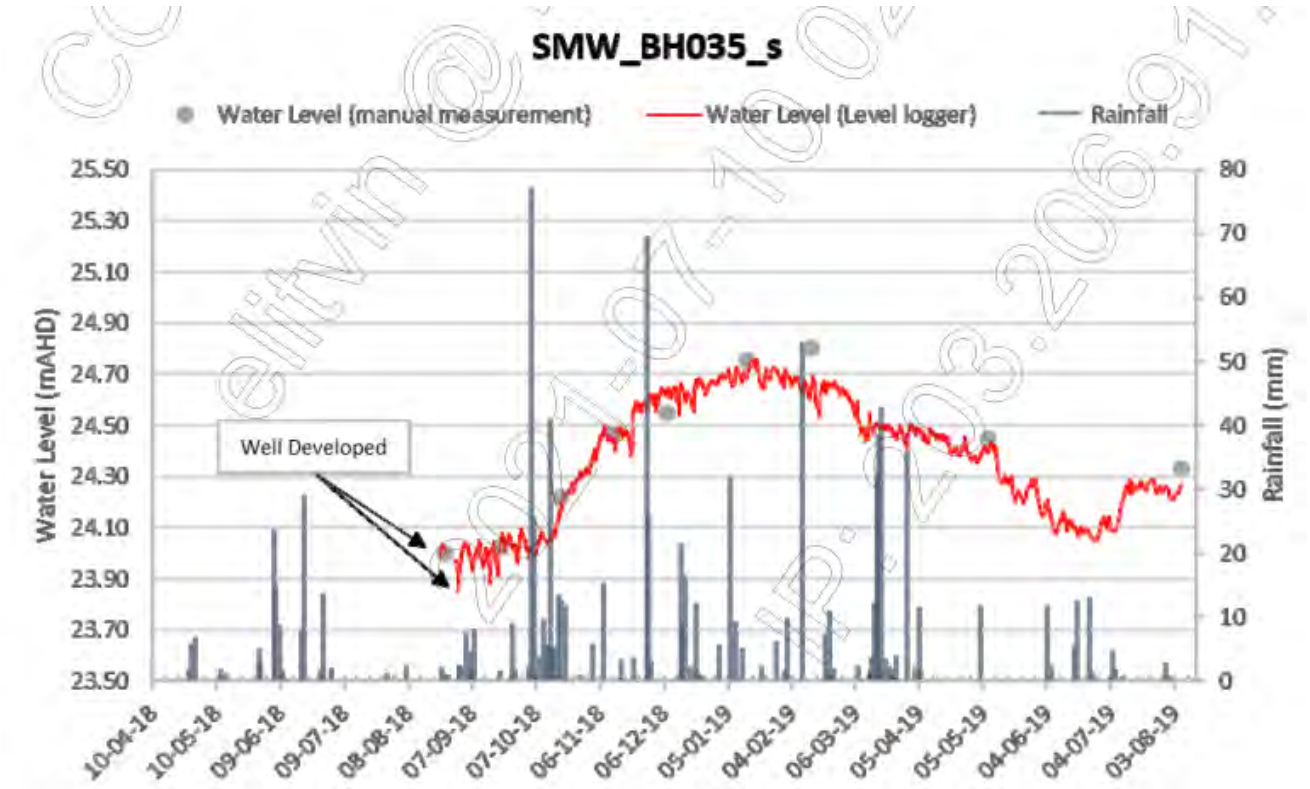

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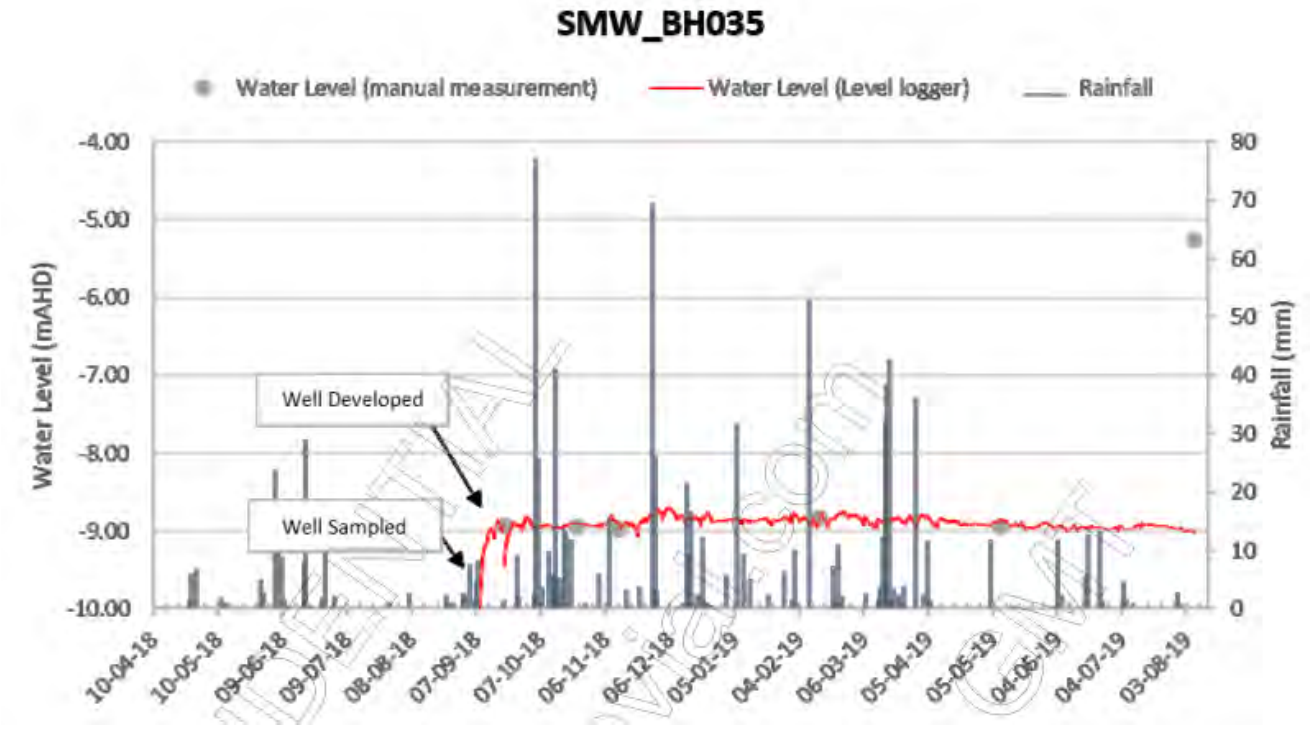

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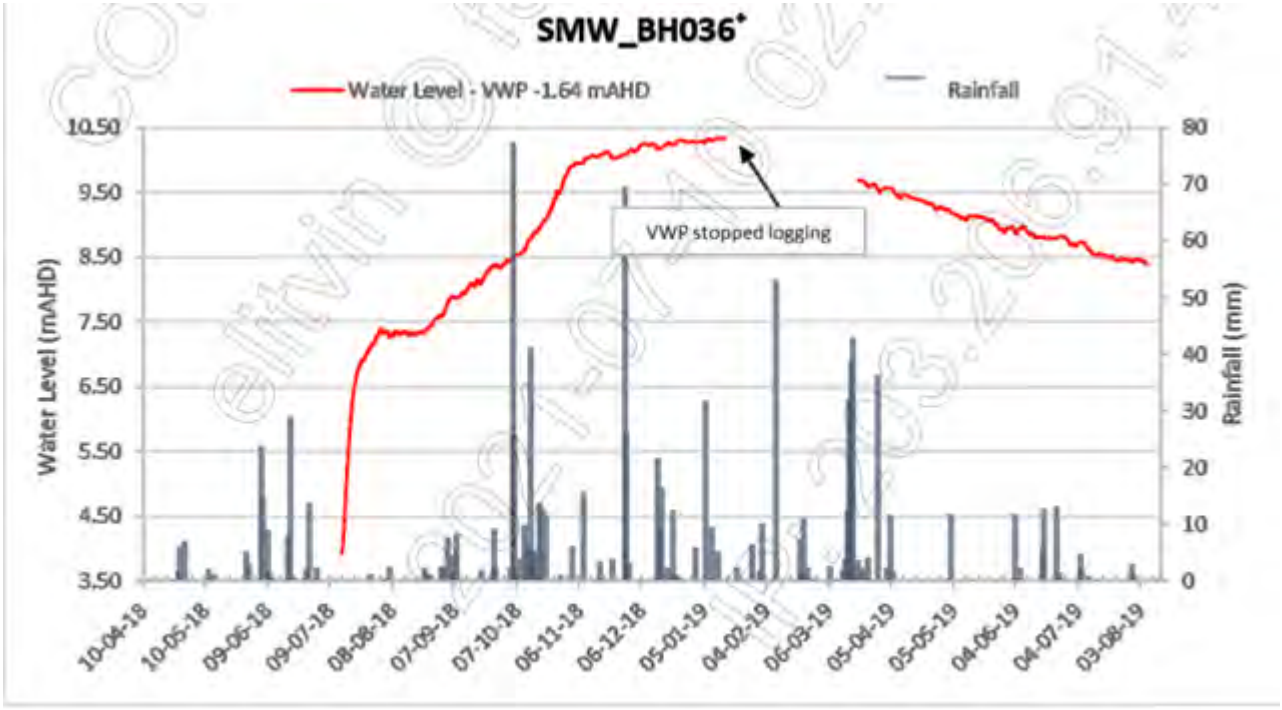

F_0 and T_0 values used for the Pore Pressure Calculation are taken from the VWP calibration sheet.

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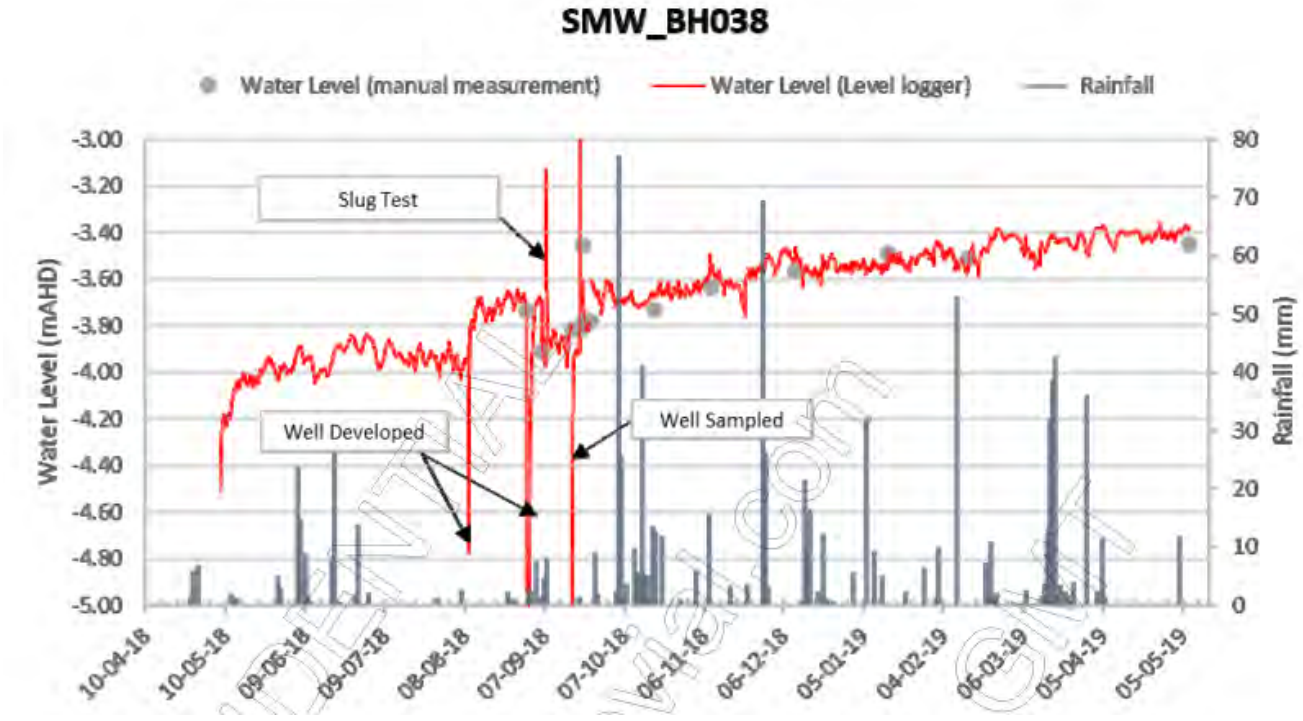

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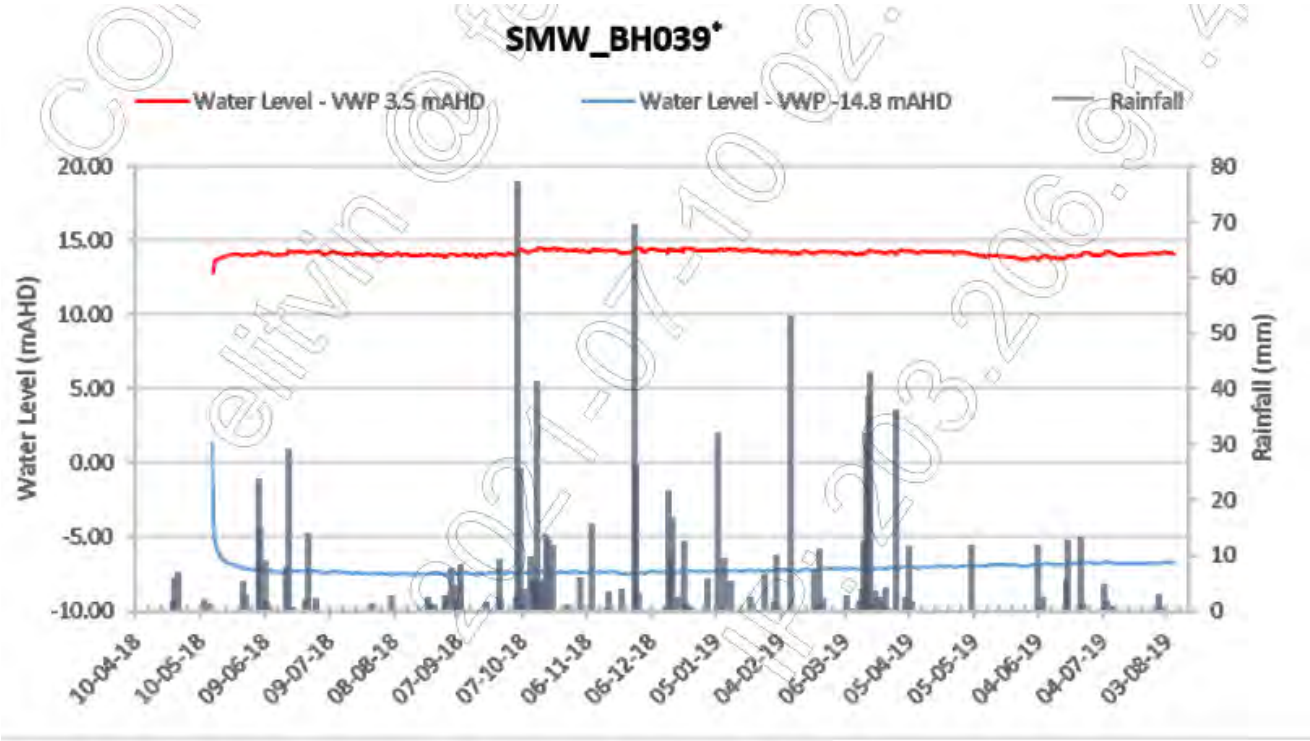

F_0 and T_0 values used for the Pore Pressure Calculation are taken from the VWP calibration sheet.

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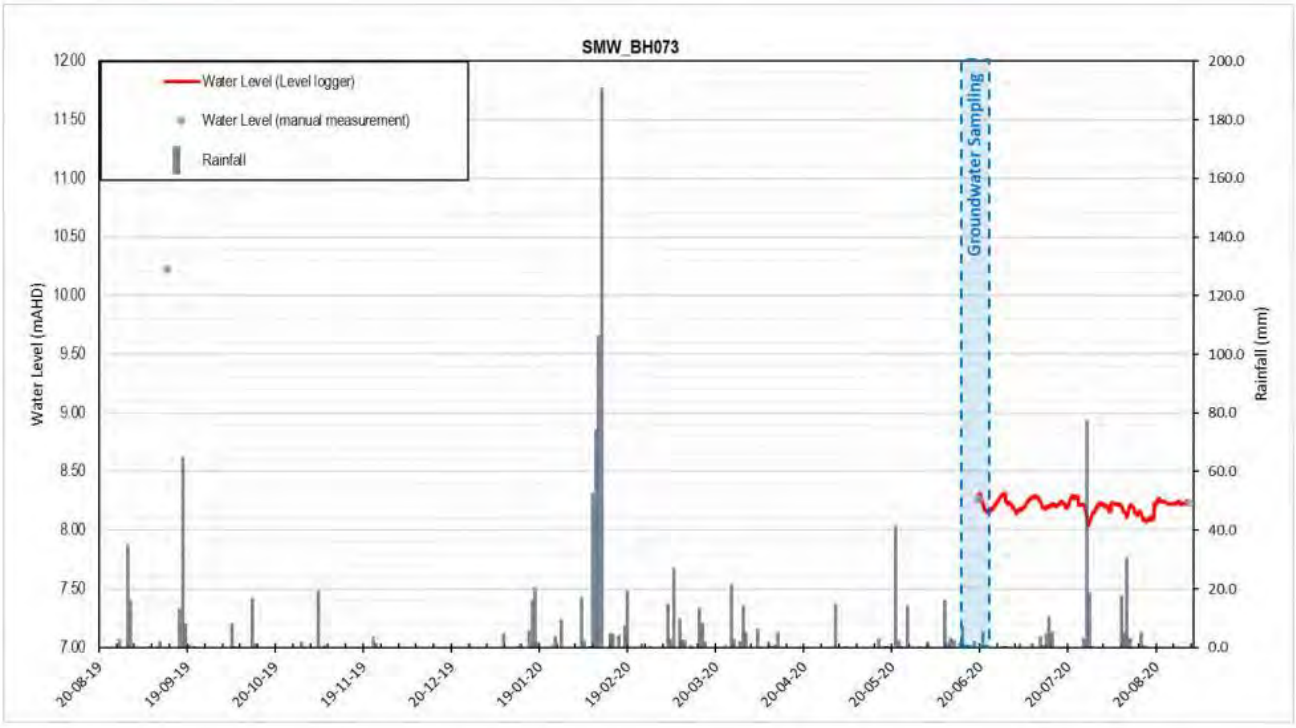

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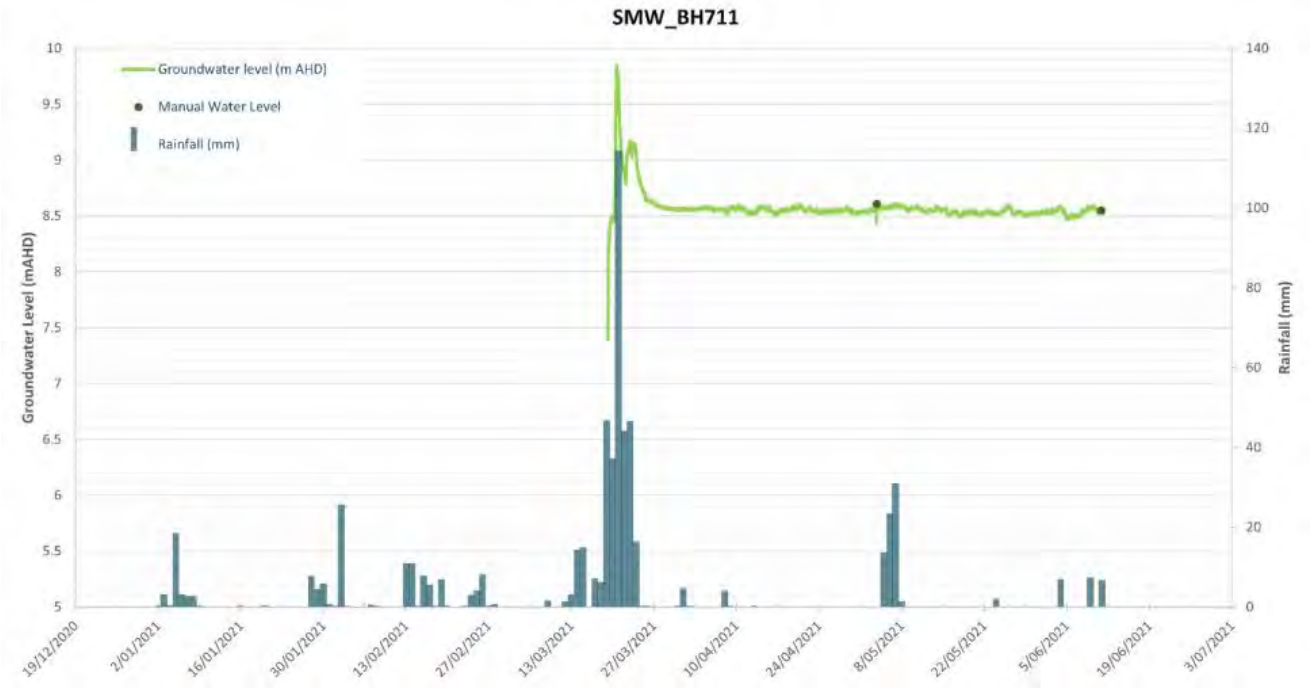
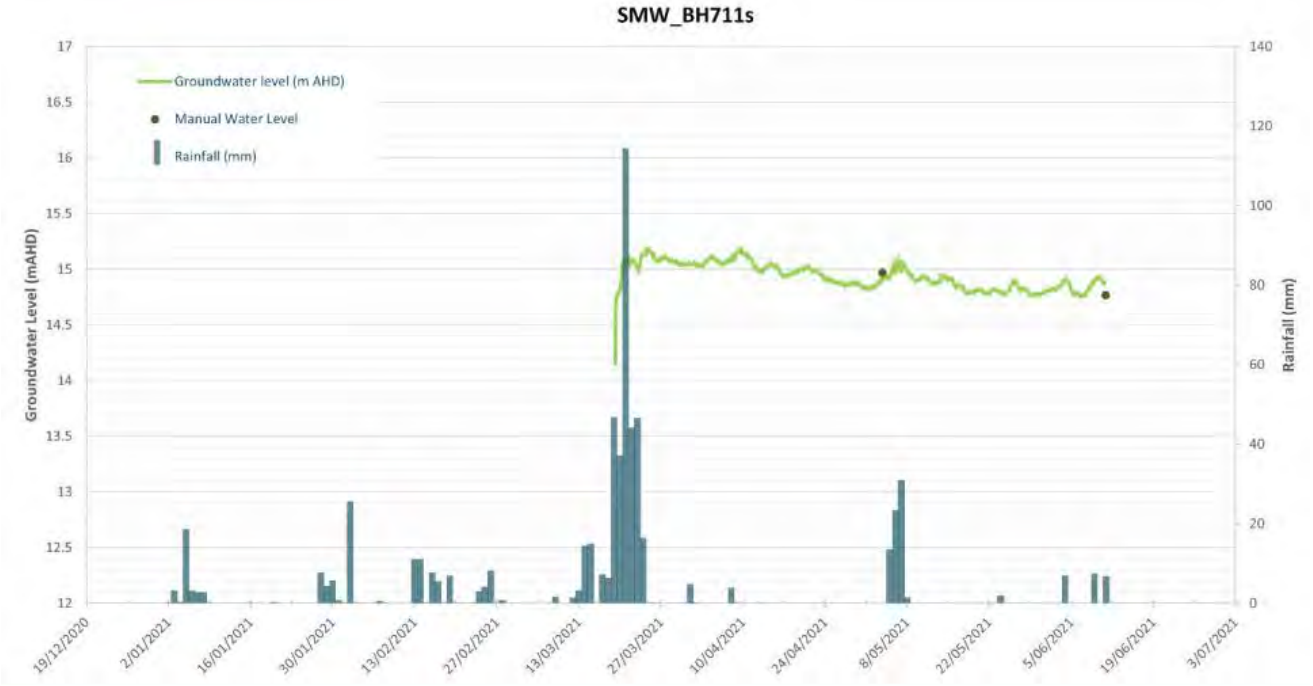

SMW_BH711
Groundwater level (m AHD)
Manual Water Level
Rainfall (mm)
Groundwater Level (mAHD)
Rainfall (mm)
10
9.5
9
8.5
8
7.5
7
6.5
6
5.5
5
140
120
100
80
60
40
20
0
19/12/2020
2/01/2021
16/01/2021
30/01/2021
13/02/2021
27/02/2021
13/03/2021
27/03/2021
10/04/2021
24/04/2021
8/05/2021
22/05/2021
5/06/2021
19/06/2021
3/07/2021

SMW_BH711_s:


SMW_BH711s
Groundwater level (m AHD)
Manual Water Level
Rainfall (mm)
Groundwater Level (mAHD)
Rainfall (mm)
17
16.5
16
15.5
15
14.5
14
13.5
13
12.5
12
140
120
100
80
60
40
20
0
19/12/2020
2/01/2021
16/01/2021
30/01/2021
13/02/2021
27/02/2021
13/03/2021
27/03/2021
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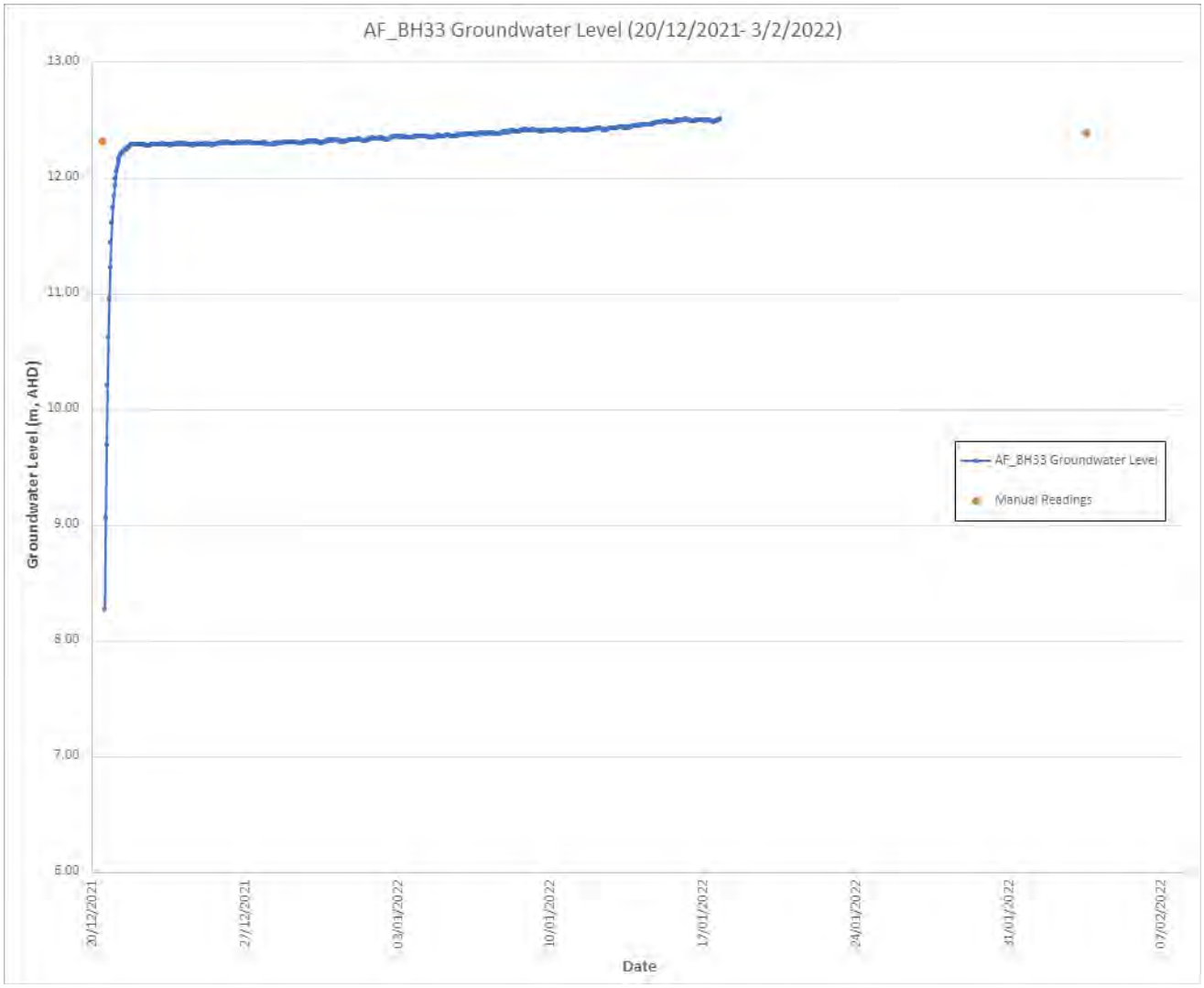
3/07/2021



Note: datalogger malfunctioning from 17/01/2021 to 03/02/2022. Data logger to be replaced.

ANNEXURE B: HYDROGEOLOGICAL UNITS AND PARAMETER VALUE

Annexure B: Hydrogeological units and parameter values

Document History and Status

Revision	Date	Description	Author	Checked	Reviewed	Approved
A	28/02/2022	Draft Report	[redacted]	[redacted]	[redacted]	[redacted]
B	8/03/2022	Draft Report	[redacted]	[redacted]	[redacted]	[redacted]

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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

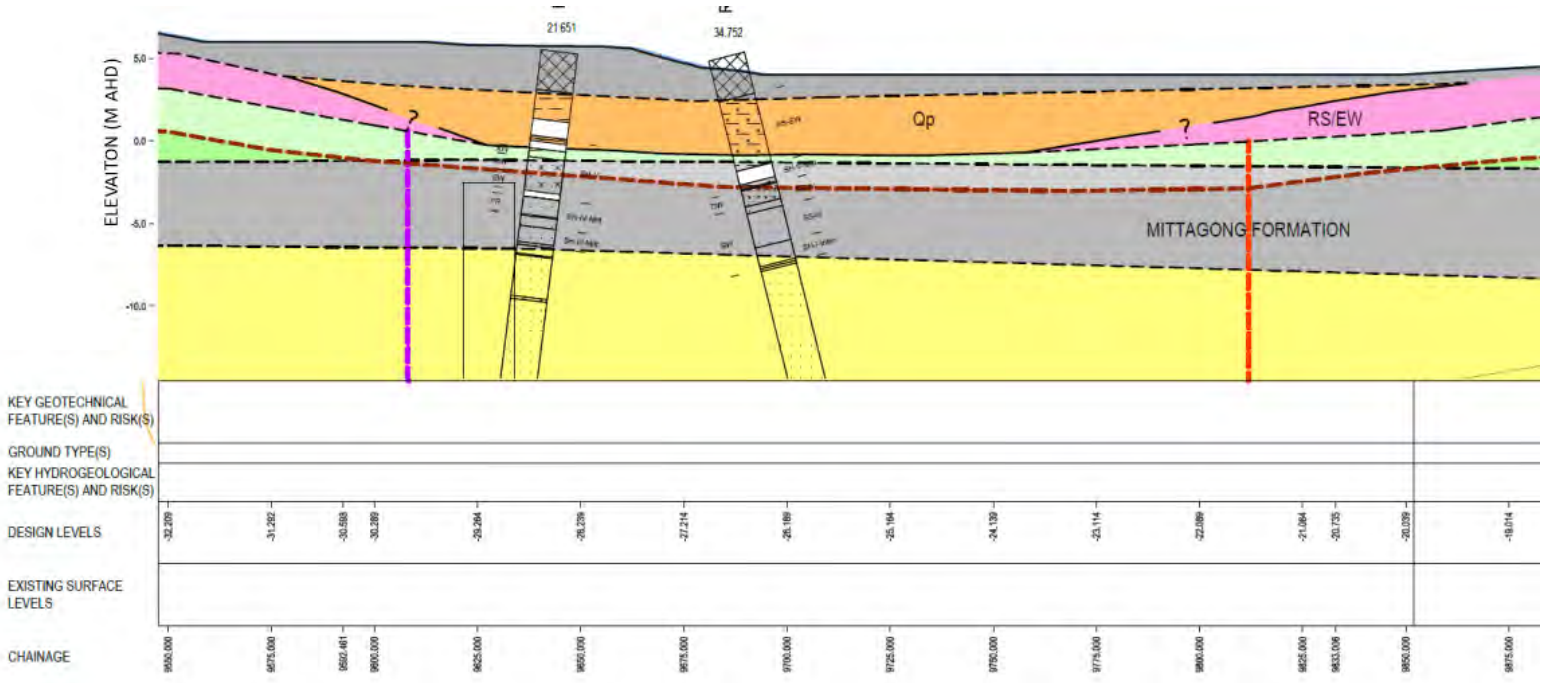

FIGURE 2-2: LOCATION OF ALLUVIUM AT BURWOOD NORTH STATION IN SECTION

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.

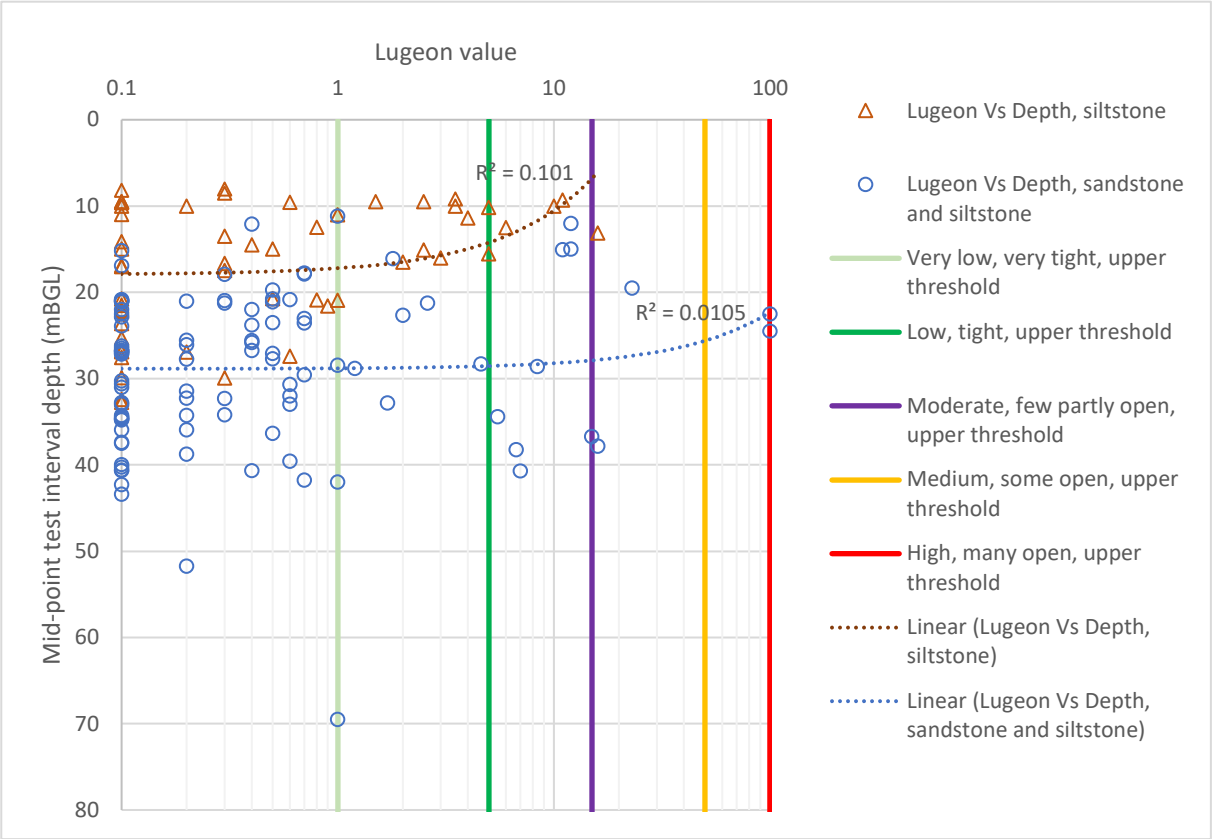

FIGURE 3-1: LUGEON VALUES FOR SILTSTONE TEST INTERVALS, AND SANDSTONE AND SILTSTONE TEST INTERVALS, CLASSED ACCORDING TO QUINONES-ROZO (2010)

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}

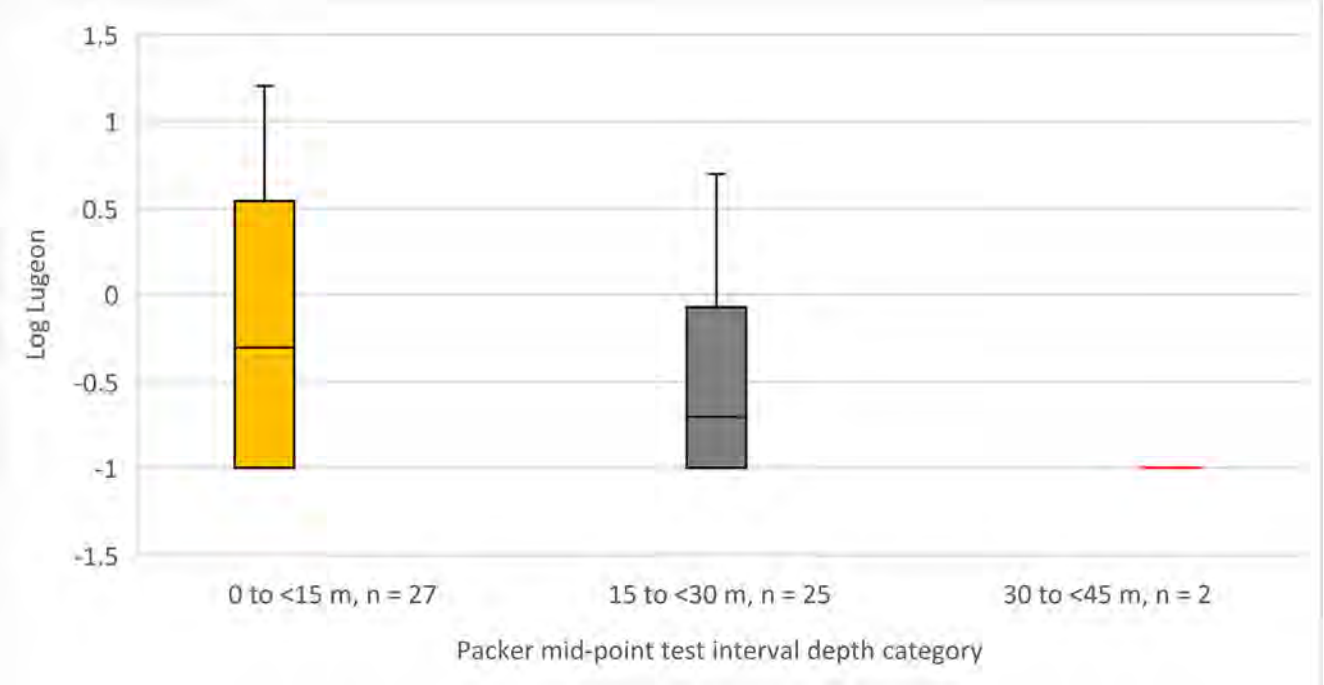

FIGURE 3-2: LOG LUGEON VALUES FOR SILTSTONE TEST INTERVALS BY DEPTH CATEGORY

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.

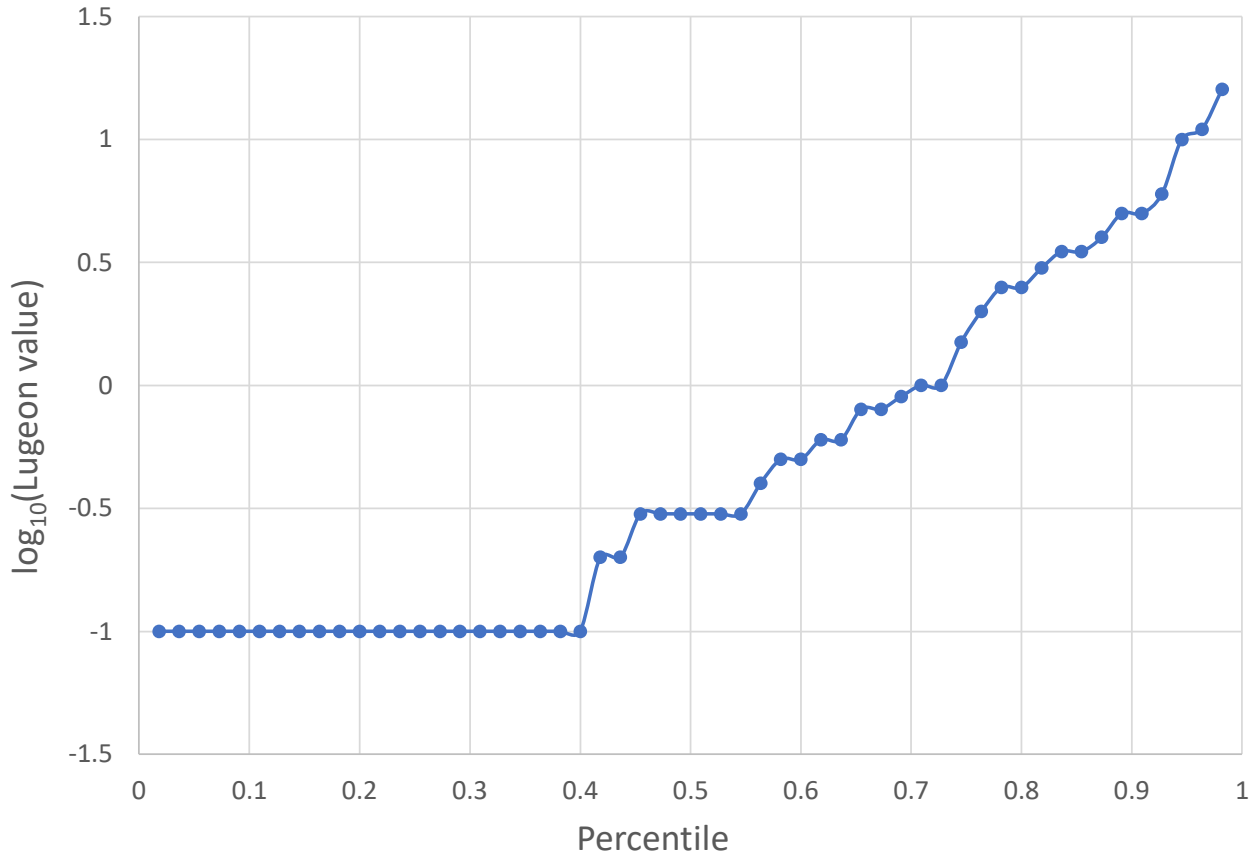

FIGURE 3-3: CUMULATIVE DISTRIBUTION OF LUGEON VALUES FOR SILTSTONE TEST INTERVALS

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.

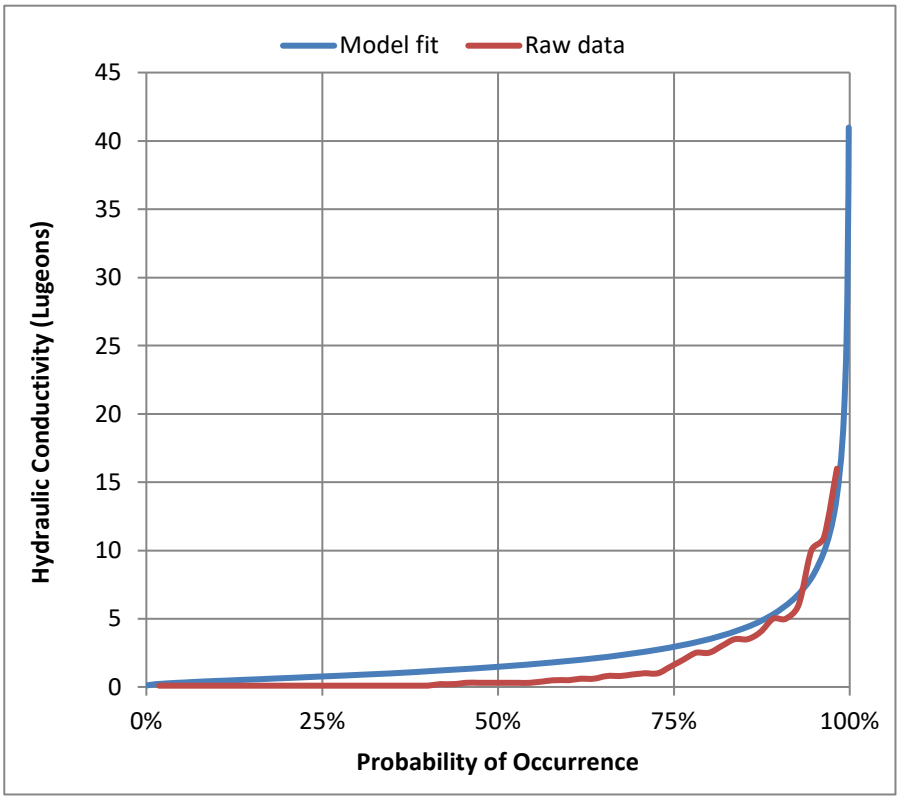

FIGURE 3-4: CUMULATIVE DISTRIBUTION OF LUGEON VALUES FOR SILTSTONE TEST INTERVALS AND NORMAL DISTRIBUTION MODEL FIT TO DATA

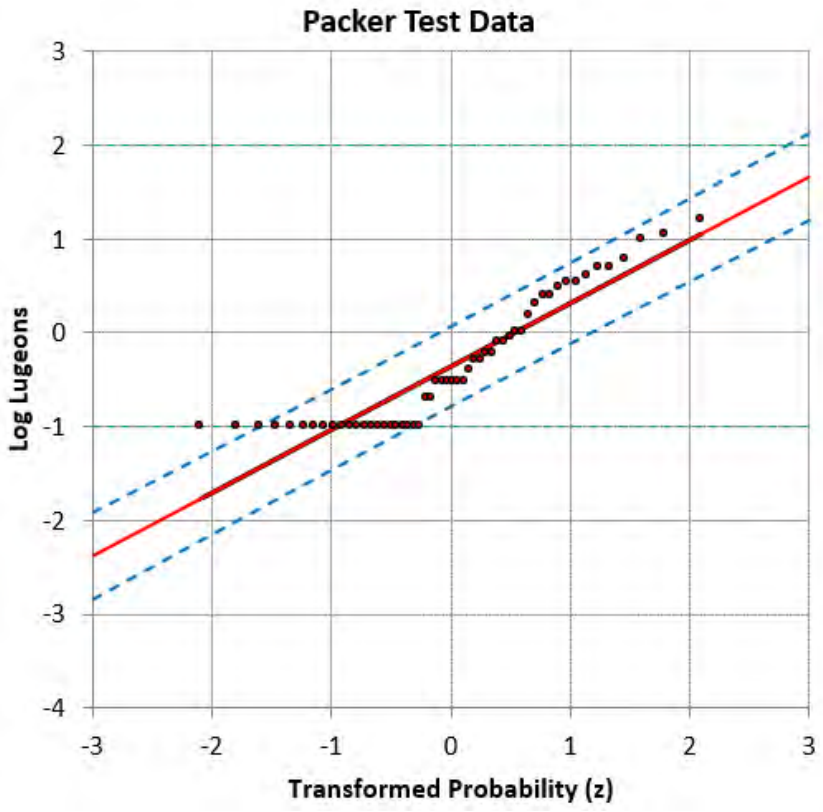

FIGURE 3-5: QUANTILE PLOT OF LUGEON VALUES FOR SILTSTONE TEST INTERVALS

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.

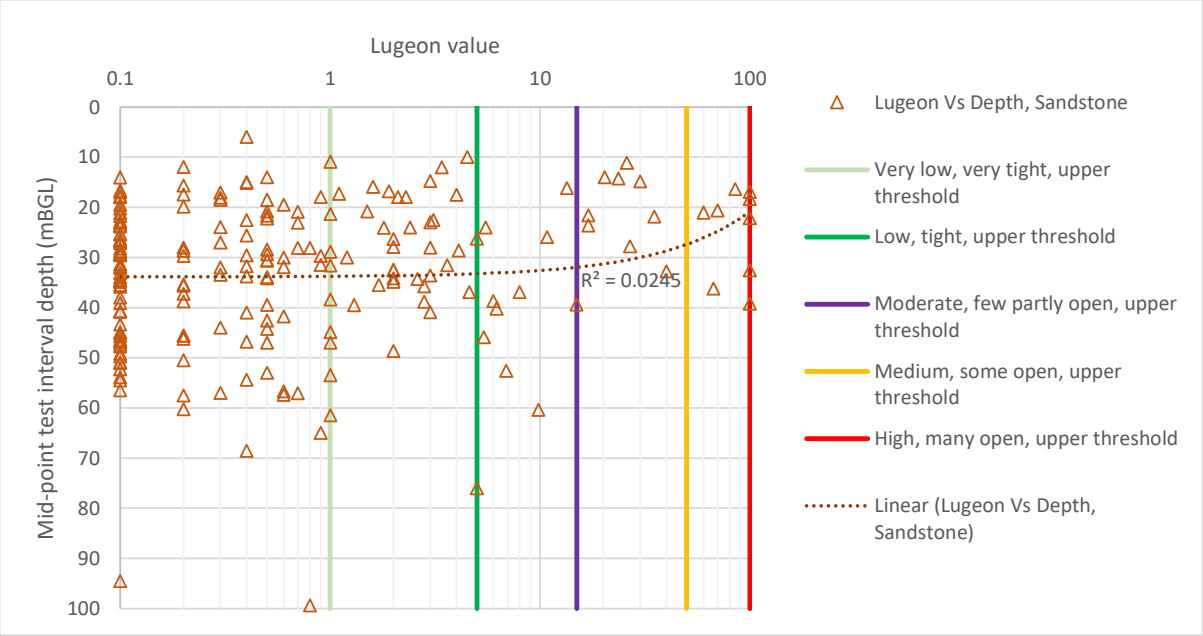

FIGURE 3-6: LUGEON VALUES FOR SANDSTONE TEST INTERVALS, CLASSED ACCORDING TO QUINONES-ROZO (2010)

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}

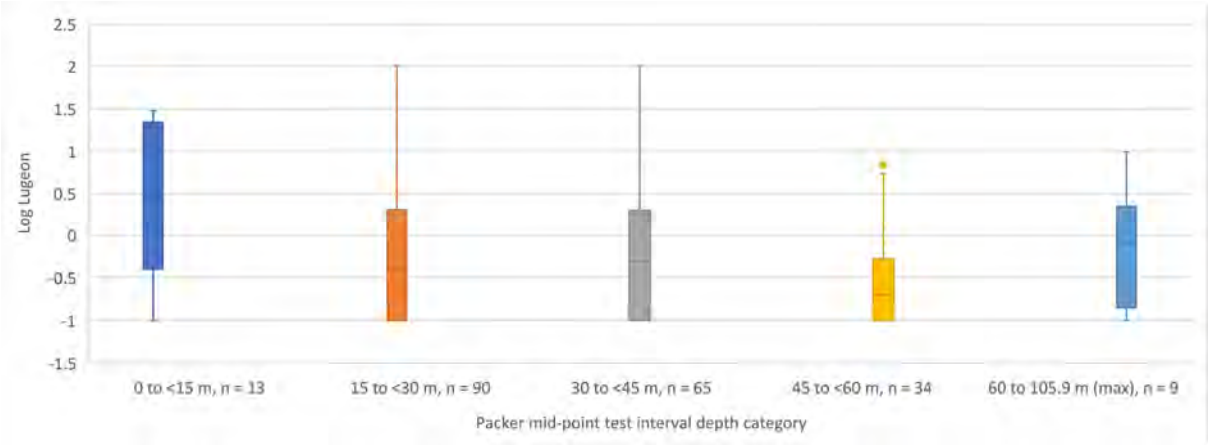

FIGURE 3-7: LOG LUGEON VALUES FOR SANDSTONE TEST INTERVALS BY DEPTH CATEGORY

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.

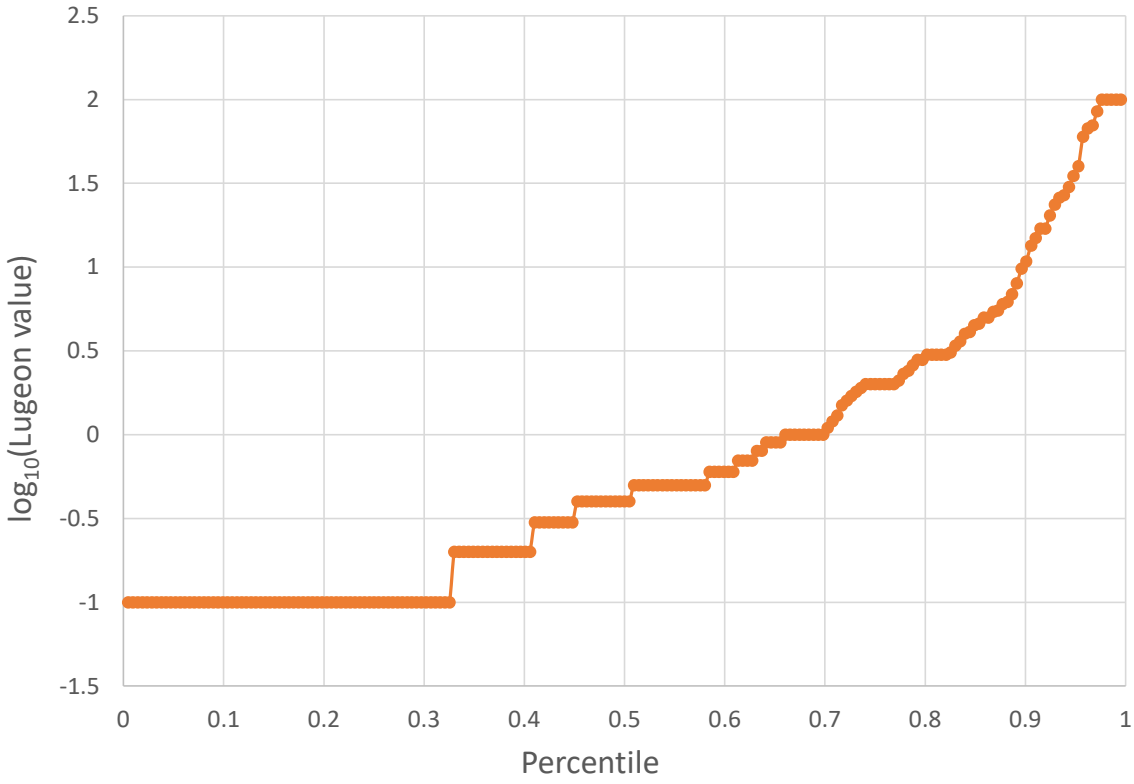

FIGURE 3-8: CUMULATIVE DISTRIBUTION OF LUGEON VALUES FOR SANDSTONE TEST INTERVALS

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.

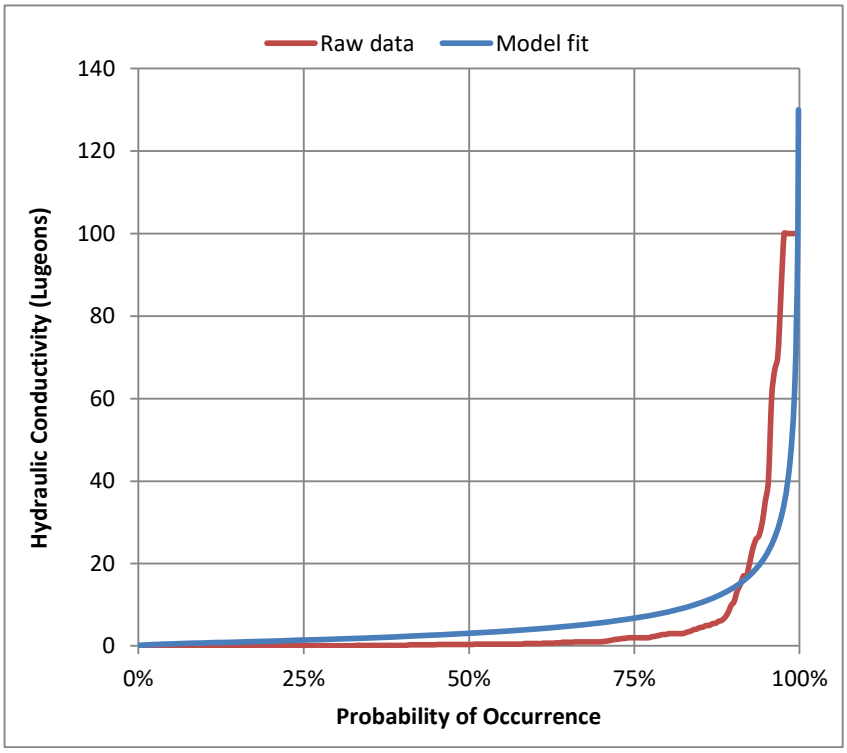

FIGURE 3-9: CUMULATIVE DISTRIBUTION OF LUGEON VALUES FOR SANDSTONE TEST INTERVALS AND NORMAL DISTRIBUTION MODEL FIT TO DATA

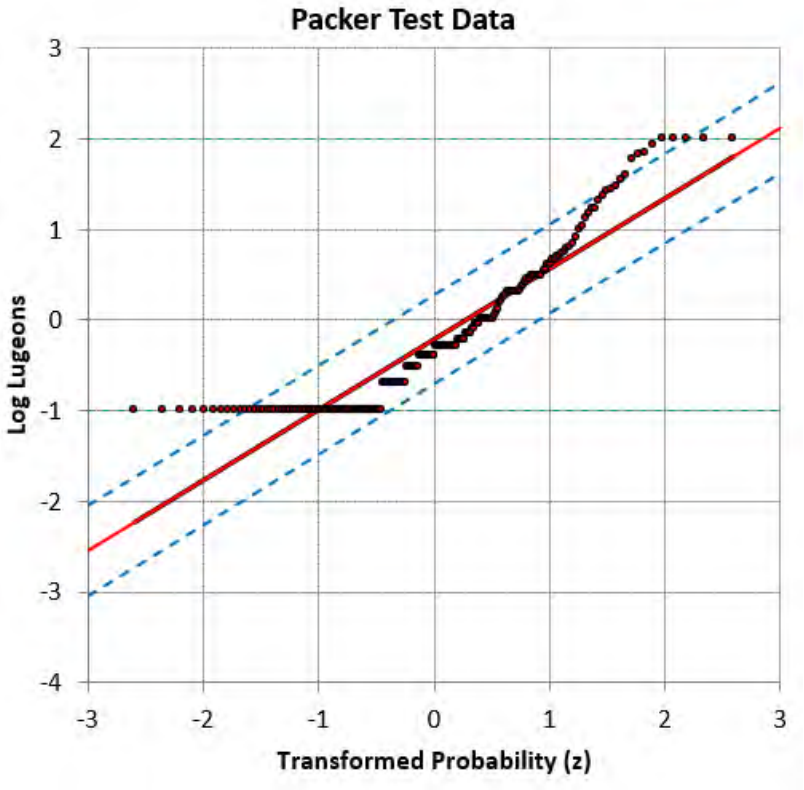

FIGURE 3-10: QUANTILE PLOT OF LUGEON VALUES FOR SANDSTONE TEST INTERVALS

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.

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

Jacobs TYPSA
Jacobs Typsa Joint Venture

Technical Memo

To	[redacted]	Date	21 June 2022
Copies	[redacted]	Document ID	SMWSTCTP-AFJ-NST-SN350-ST-RPT-003050[C.1] Appendix G Annexure C
From	[redacted]	Revision	D
Subject	Design groundwater loads for station soil retaining walls – accidental load cases – burst water main, flooding, blocked drainage - North Strathfield Station		

1. Introduction

This memorandum provides hydrogeological advice in support of the accidental load scenarios for geotechnical and structural design of the North Strathfield 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.

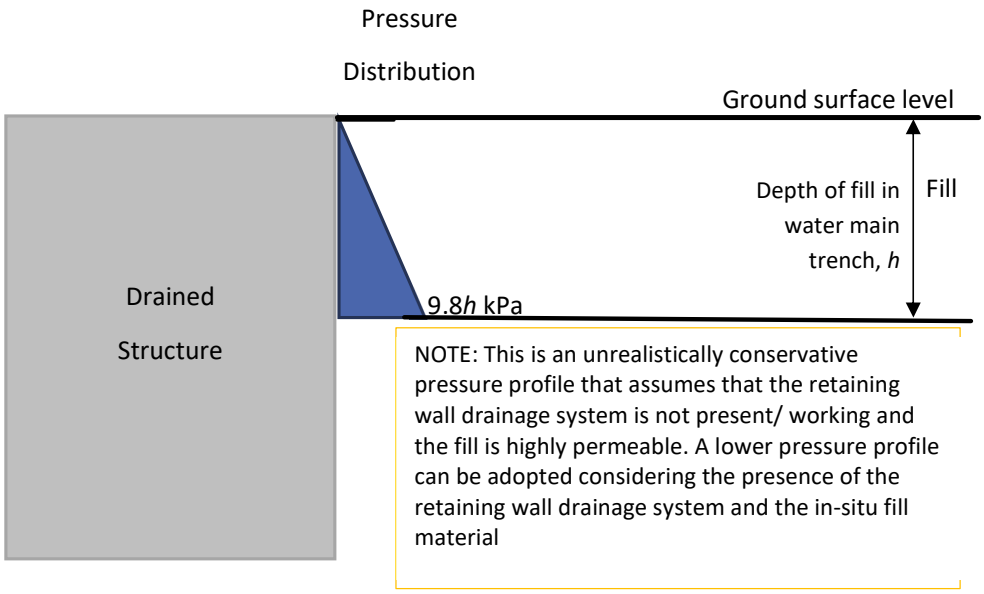

FIGURE 1: EXCEPTIONAL GROUNDWATER PRESSURE CONDITION FOR BURST WATER MAIN

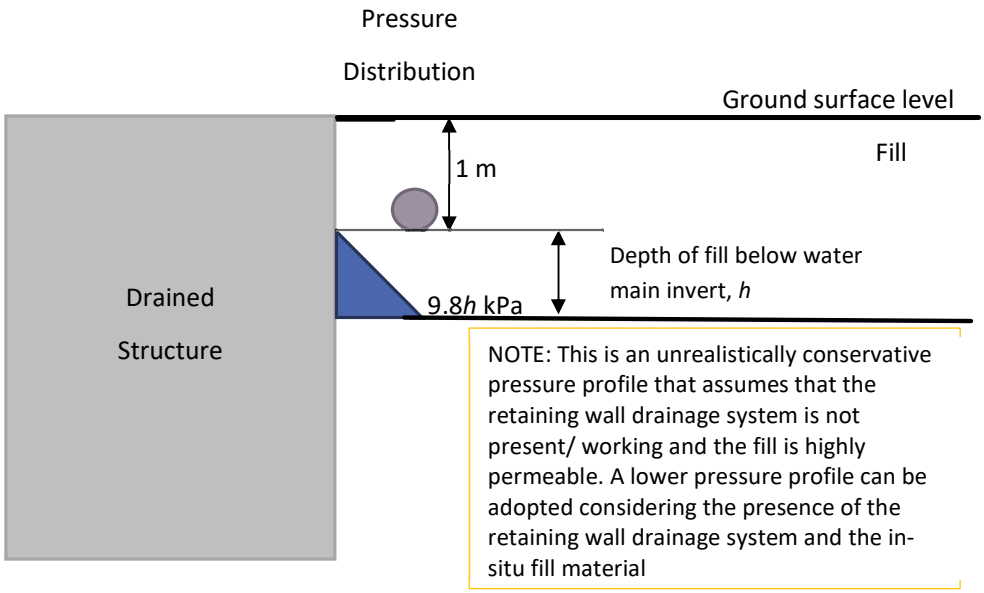

FIGURE 2: EXCEPTIONAL GROUNDWATER PRESSURE CONDITION FOR BURST WATER MAIN CONSIDERING PIPE INVERT LEVEL

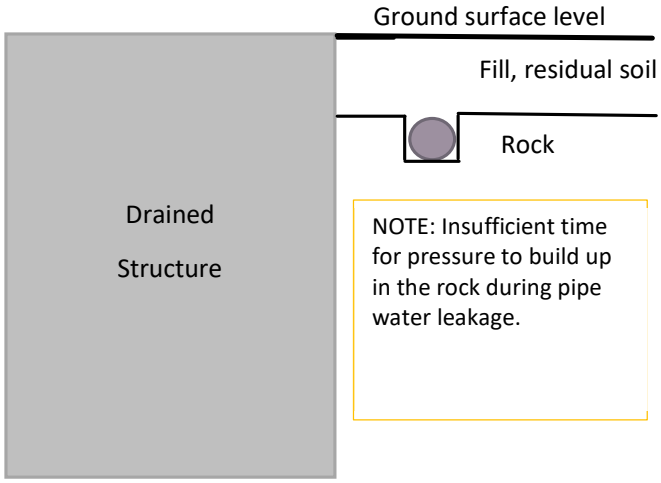

FIGURE 3: EXCEPTIONAL GROUNDWATER PRESSURE CONDITION FOR BURST WATER MAIN CONSIDERING PIPE INVERT LEVEL

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.

5.1. Retaining wall design

Geotechnical and structural analysis identified that the exceptional load case governs the design along must of the western retaining wall of the station box.

The retaining wall at this location comprises a solider pile wall with alternating piles of two 900 mm-diameter short piles spaced at 1.8 m centres and 900 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 10.

5.2. 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
- A flood event consistent with the Probable Maximum Flood (PMF) event that has the greatest flood water depth and duration along the western wall, as assessed by flood modelling. The flood water elevation profile against time for this event is shown in Figure 5. The flood waters

are applied over a lateral zone consistent with the results of the flood modelling (extending about 5 m west of the retaining wall).
The flood modelling considers the presence of blocked stormwater drainage. The dive structure to the west is present, and flood waters can overtop the dive wall and enter the dive structure. A temporary flood protection barrier between flood waters and the CTP excavation retaining wall is assumed to be present immediately adjacent to the retaining wall.

- A western model constant head boundary condition representing Powells Creek

It is understood that a temporary flood protection barrier will be placed adjacent to the CTP works excavation. This structure may prevent the ingress of flood waters into and CTP works excavation. However, if flood waters were to exceed the height of these walls, the flood waters would flow into the dive and/or the CTP works excavation.

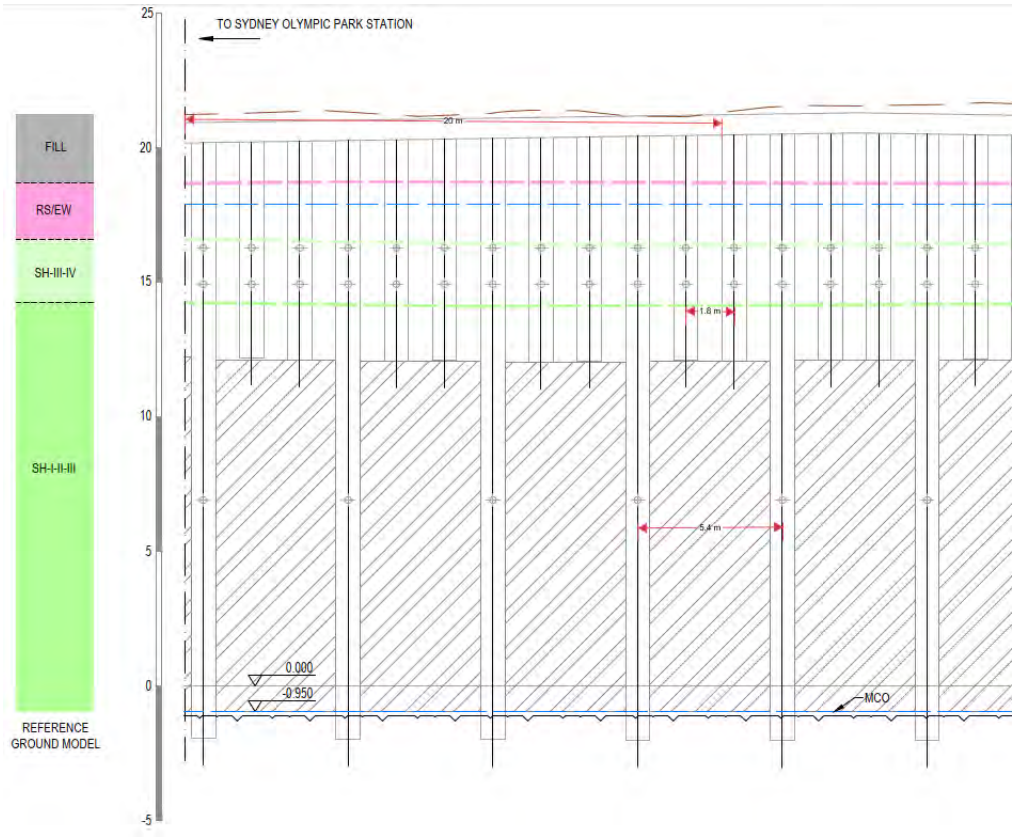

FIGURE 4: PILE LAYOUT AT WESTERN RETAINING WALL WHERE EXCEPTIONAL LOAD CASE GOVERNS

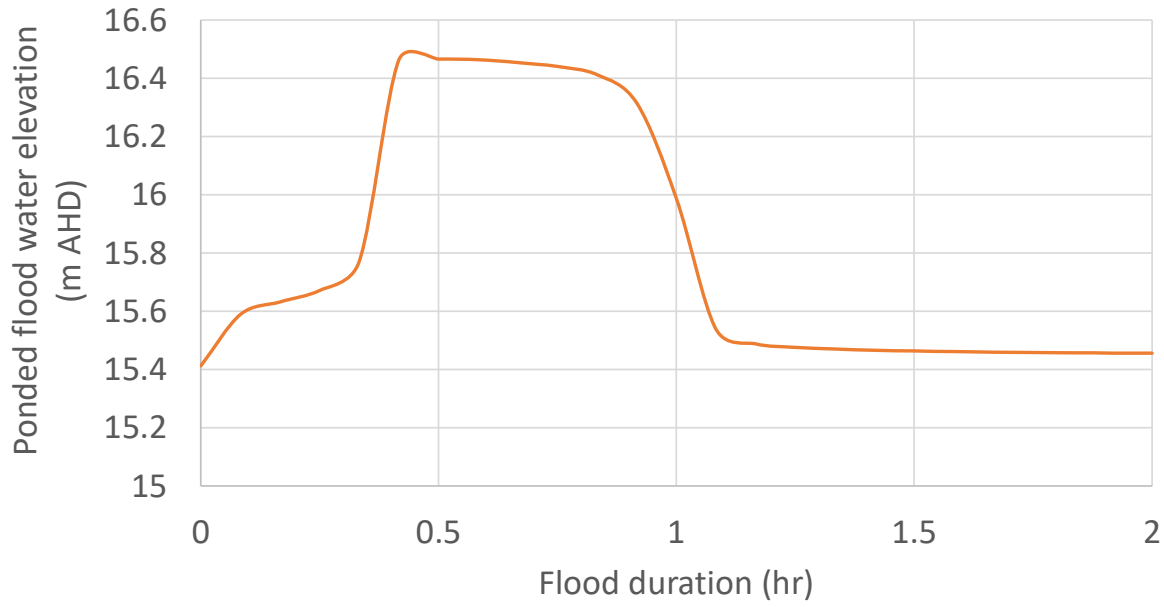

FIGURE 5: CRITICAL FLOOD EVENT PROFILE (BASED ON FLOOD EVENT AT CRITICAL LOCATION – FLOOD MODELLING LOCATION H162)

5.3. Ground profile and model parameter values

The adopted ground profile is based on the geotechnical interpretation, comprising up to 3 m of fill, overlying 2 m of residual soils, overlying Ashfield Shale.

Note that this is a conservative profile with respect to likely groundwater pressures experienced during a flood event, because the fill is assumed to be relatively thick. Data are scarce at the wall location and the geotechnical interpretation has therefore adopted a relatively thick fill profile. In practice, the thickness of fill may be much less than 3 m, leading to lower groundwater pressures on the retaining wall than those predicted here.

Adopted hydrogeological parameter values are provided in Table 2.

The model domain 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 (-)
Fill	1	1	5×10^{-6}	0.05
Residual soil and Ashfield Shale	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
Zone where short piles and soil/rock present	1.3×10^{-3}	0.1	5×10^{-6}	0.015
Zone where long piles and soil/rock present	2.2×10^{-3}	0.1	5×10^{-6}	0.018

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

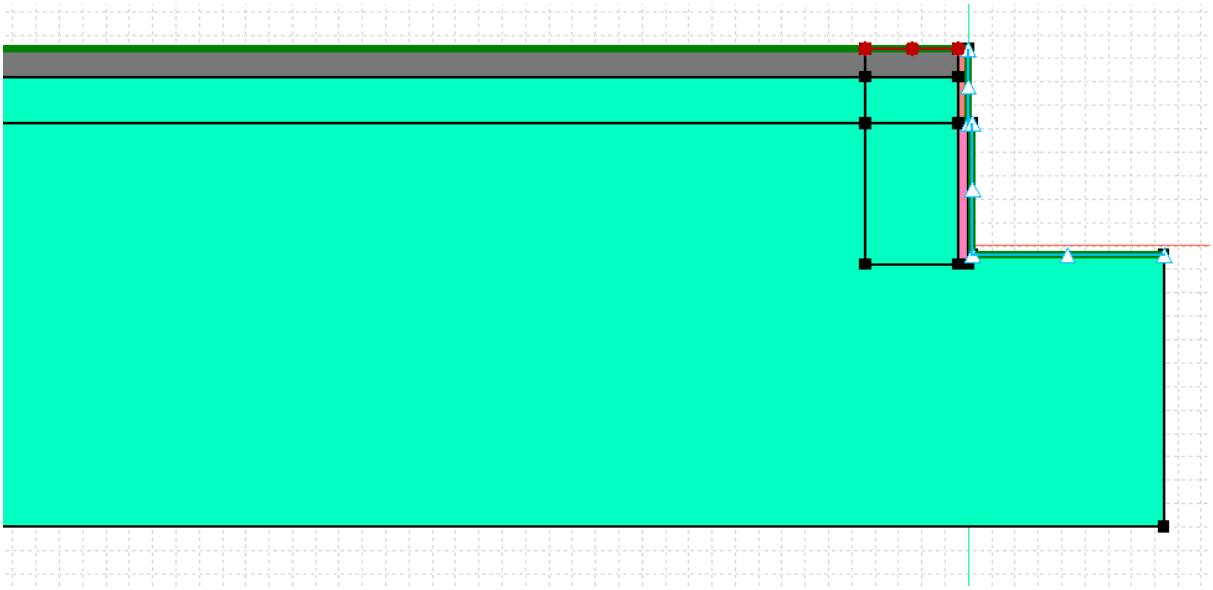

FIGURE 6: MODEL DOMAIN (EXTENDS TO POWELLS CREEK TO THE WEST)

5.1. Modelling results

An example model output (showing pore water pressure in kPa) is shown in Figure 7.

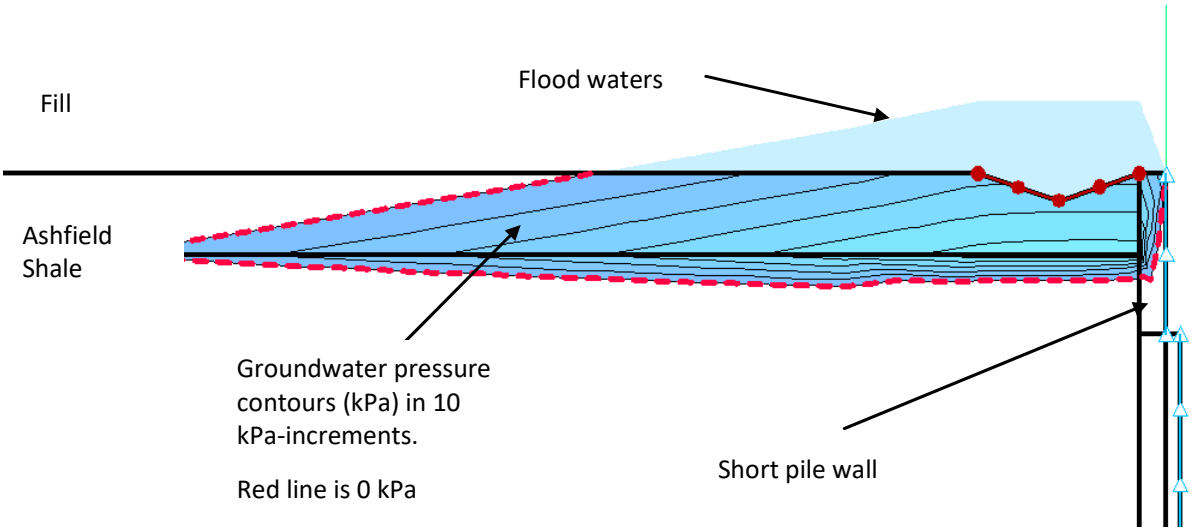

FIGURE 7: EXAMPLE MODEL OUTPUT (NOT TO SCALE)

The predicted maximum groundwater pressures on the rear of the retaining wall (blue line) for the flood scenario are shown in Figure 14 for the critical location (flood model location H162). The maximum pressure coincides with the time at which the flood waters are at their maximum depth (with the first hour of the flood event). This maximum pressure represents the PMF flood level (16.5 m AHD) exerting pressure to the base of the assumed 3 m depth of fill (12.4 m AHD), i.e., a pressure of 40 kPa. During the flood event, groundwater seeps from the fill into the underlying residual soil; causing minor additional pressure to be present over an additional 0.5 metre depth in the residual soil underlying the fill. Based on sensitivity analysis modelling (results not shown), it's possible that the depth could increase to up to 1 metre (extreme cases and durations). Pressures across the deeper horizon (long piles), in the rock, are not significant.

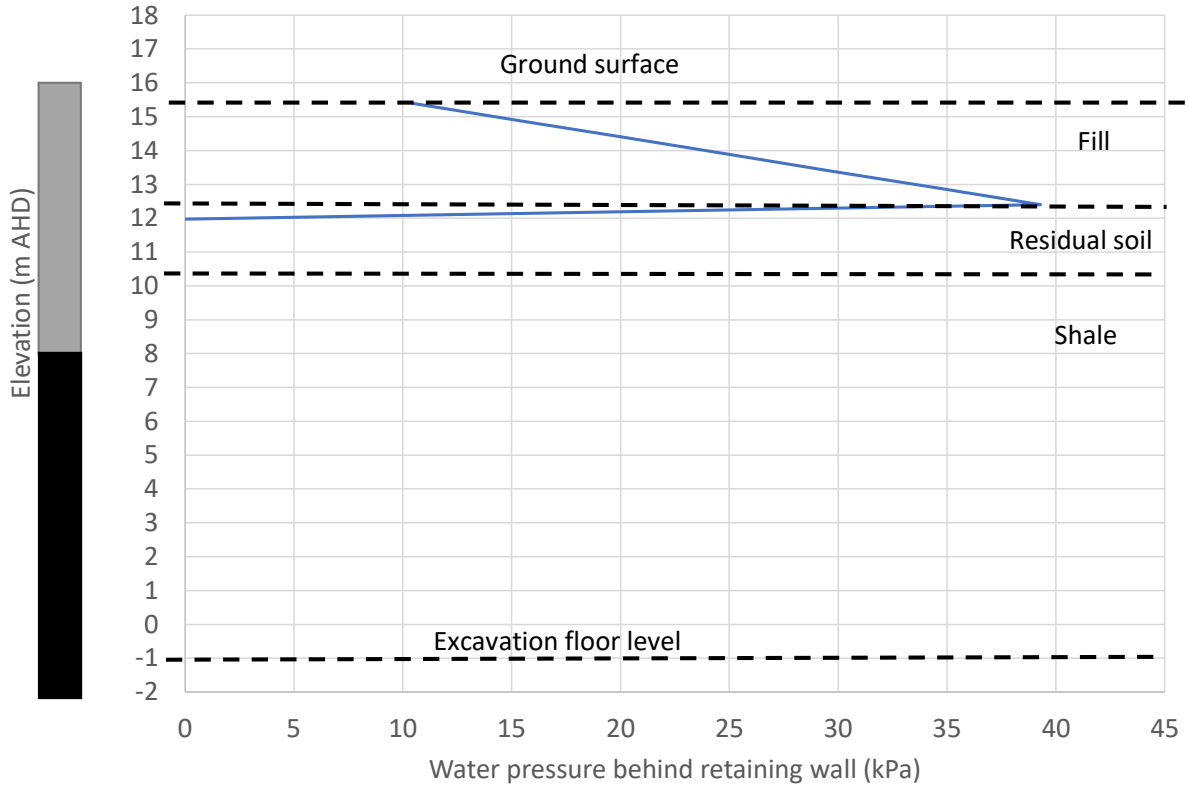

FIGURE 8: MODEL RESULTS – GROUNDWATER PRESSURE PROFILE ALONG PILED WALL (BLUE LINE)

5.1. Design implications

As discussed above, the modelling results indicate that the retaining wall is likely to experience water pressures equal to the height of the PMF flood event, increasing to a maximum pressure at the base of the fill material, and the reducing to zero approximately one metre in the residual soils/rock below the base of the fill.

Figure 15 shows a simplified pressure profile for the (short pile) retaining wall for the general case, based on the modelling results. This is the groundwater pressure, for the conditions modelled, likely to be experienced by the western retaining wall during the PMF event. Note that, if the CTP works temporary flood barrier or the top of the dive structure wall are below the PMF level, then flood waters will enter the CTP works excavation or the dive structure; reducing the flood level to the lowest of either the PMF Level, top of CTP works temporary flood barrier, or top of dive structure wall.

If additional barriers are placed upstream of water that is predicted to flood between the CTP flood barrier and the dive structure, the ponding flood water may be reduced/designed-out and the hydrostatic loads presented here would not apply.

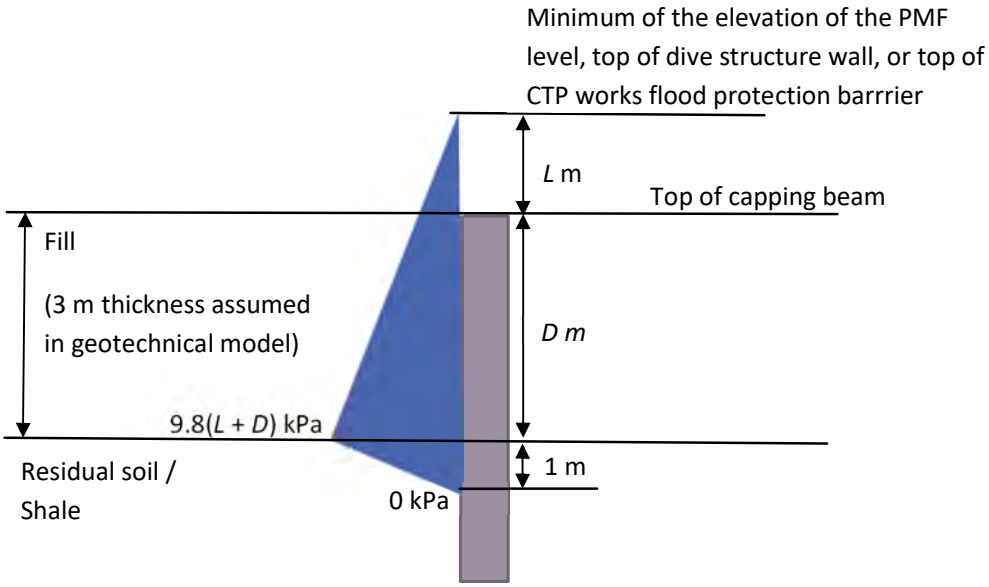

FIGURE 9: GROUNDWATER PRESSURE PROFILE ON SHORT PILES BASED ON MODELLING RESULTS

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 10.

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
- Based on available data (borehole logging material descriptions), the fill is likely to be relatively permeable
- The equivalent length of retaining wall that is modelled by this equivalent net permeability is shown in Figure 10
- 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 10

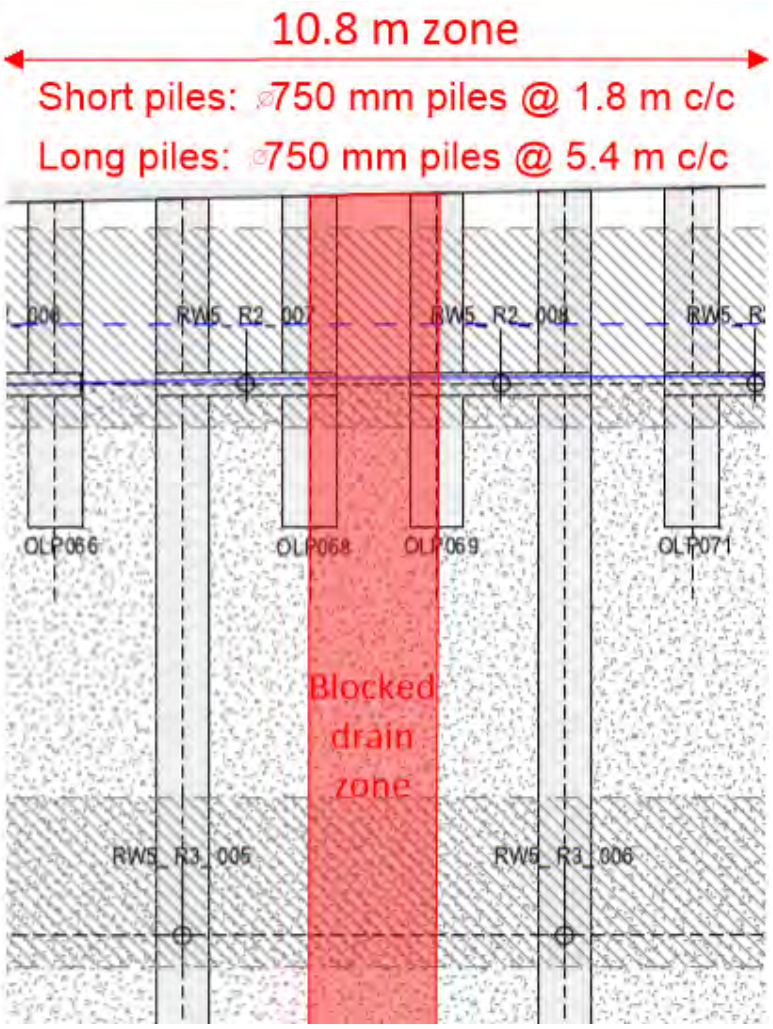

FIGURE 10: TYPICAL PILE LAYOUT AND BLOCKED DRAINAGE ZONE

6.3. Model parameter values

Adopted hydrogeological parameter values are provided in Table 2.

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 11 and an example model output (showing pore water pressure in kPa) is shown in Figure 12.

TABLE 2 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

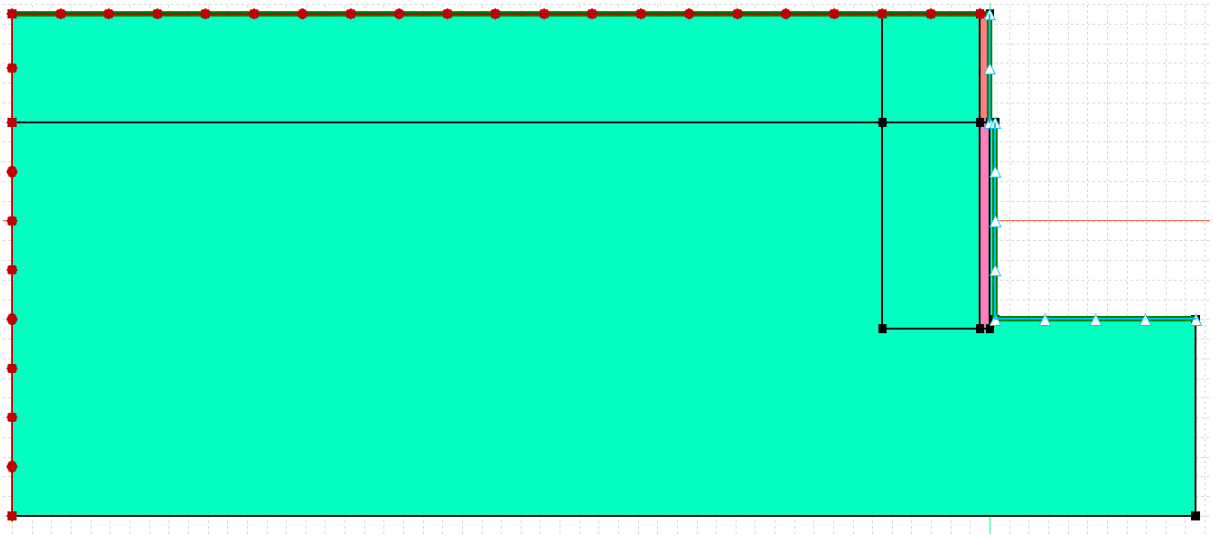

FIGURE 11: MODEL DOMAIN

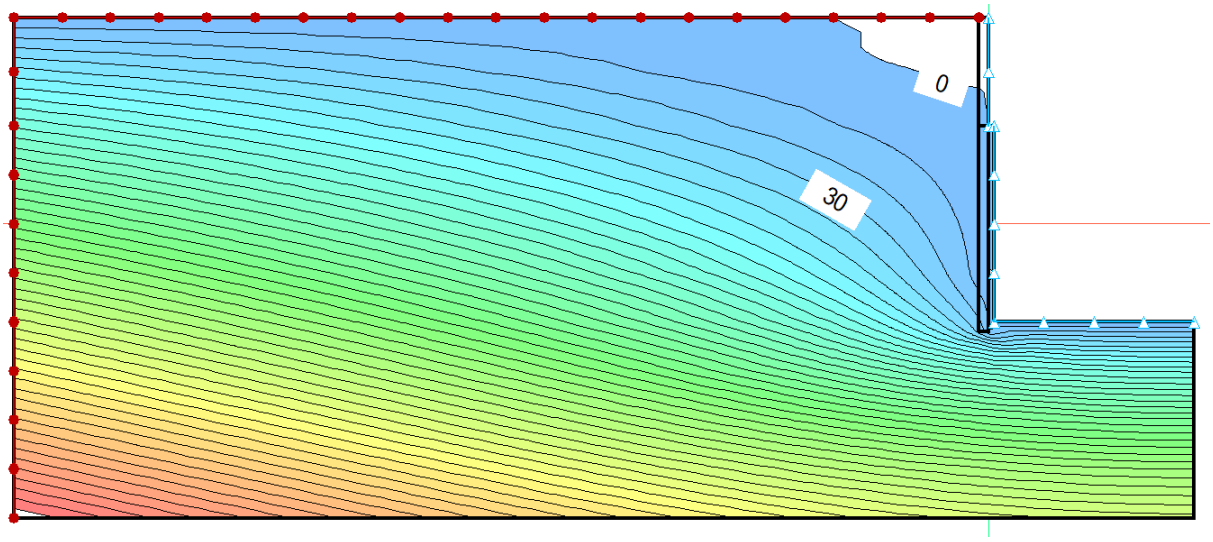

FIGURE 12: EXAMPLE MODEL OUTPUT

6.4. Modelling results

Figure 13 and Figure 14 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 15 shows a simplified pressure profile for the soil retaining wall.

Because the modelling is two-dimensional, the results shown in Figure 15 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 16.

Based on this, the pressure experienced by a pile adjacent to the blocked drainage zone is shown in Figure 17.

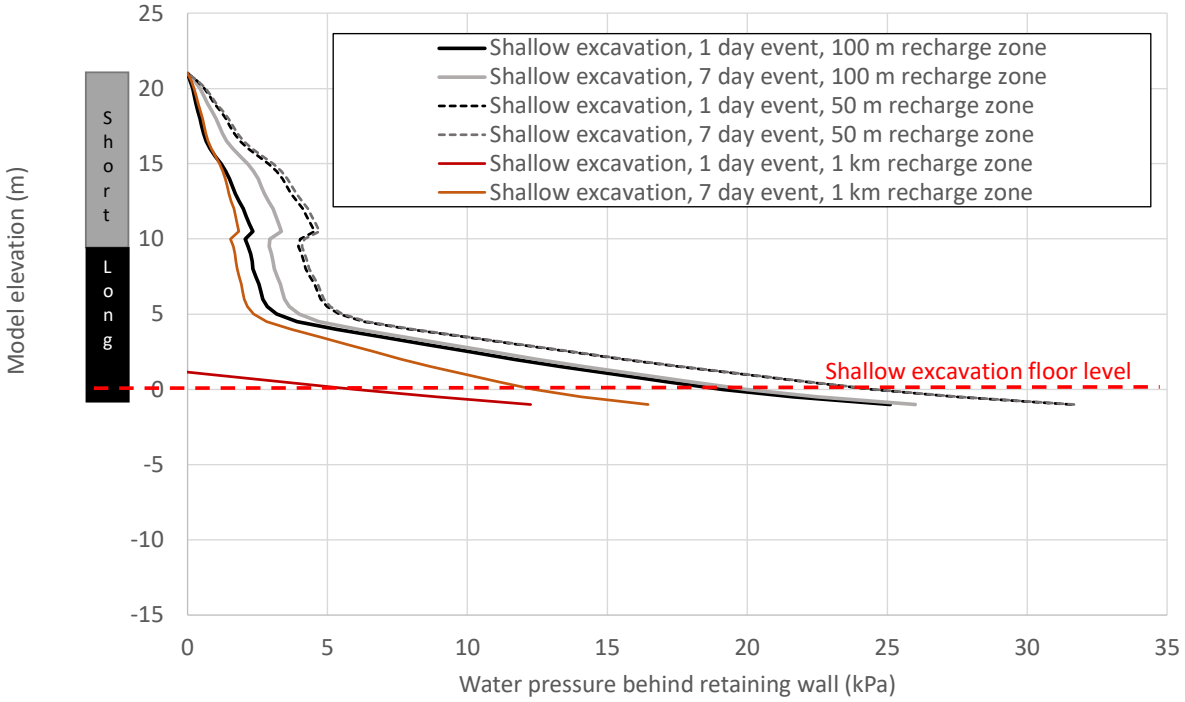

FIGURE 13: MODEL RESULTS – GROUNDWATER PRESSURE PROFILE ALONG PILED WALL – SHALLOW EXCAVATION

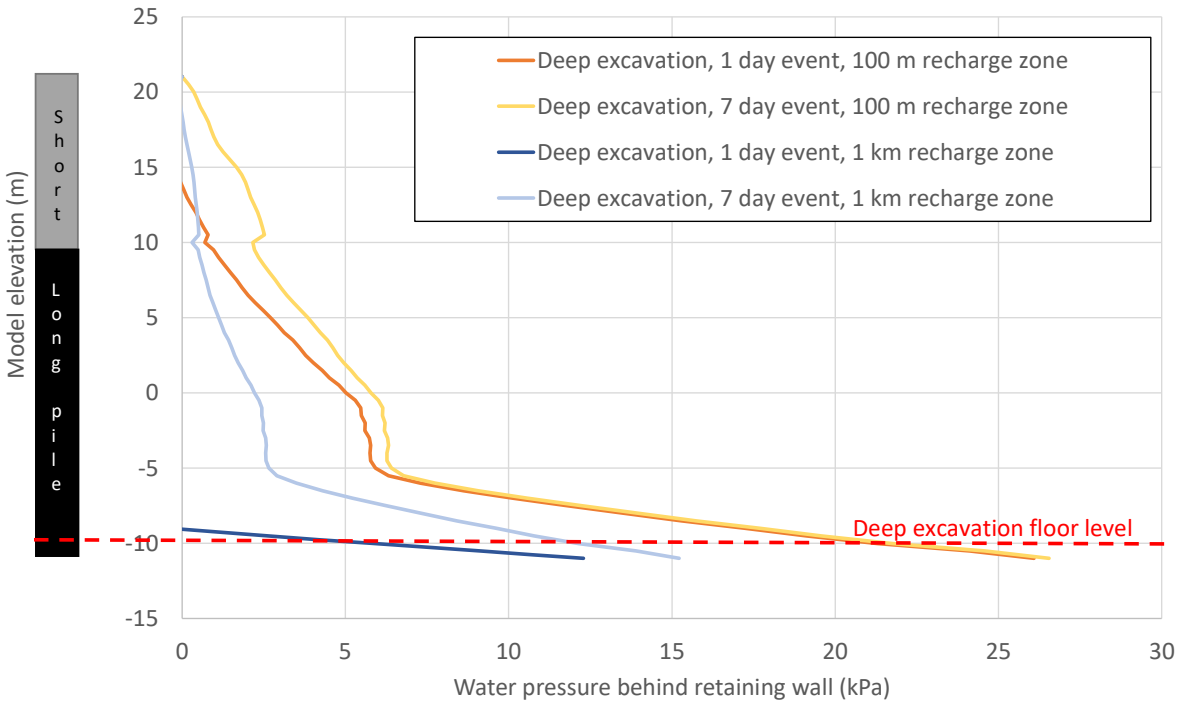

FIGURE 14: MODEL RESULTS – GROUNDWATER PRESSURE PROFILE ALONG PILED WALL – DEEP EXCAVATION

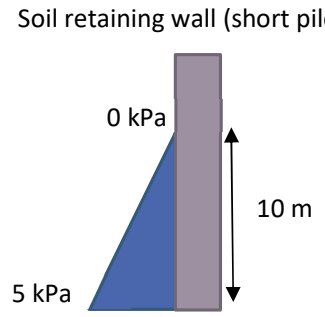

FIGURE 15: PRESSURE PROFILE DIAGRAM BASED ON MODEL RESULTS

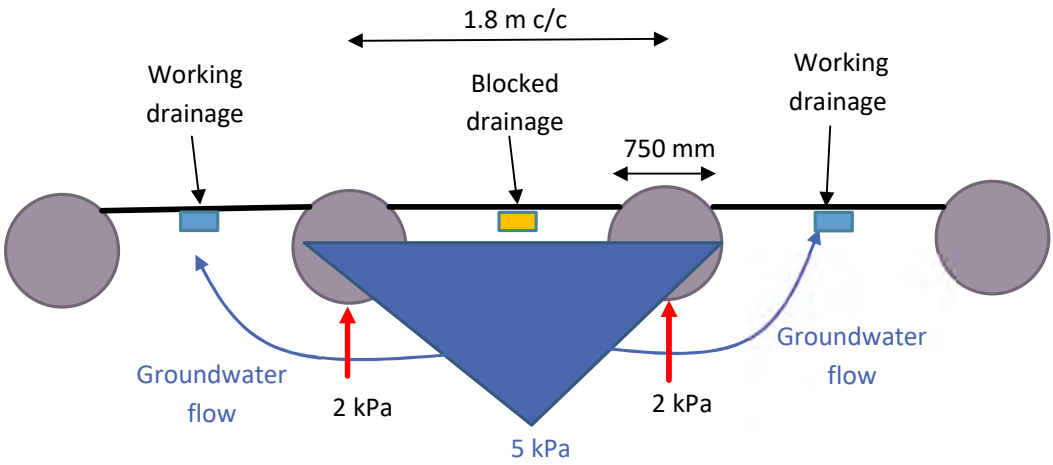

FIGURE 16: PRESSURE PROFILE DIAGRAM (IN PLAN VIEW)

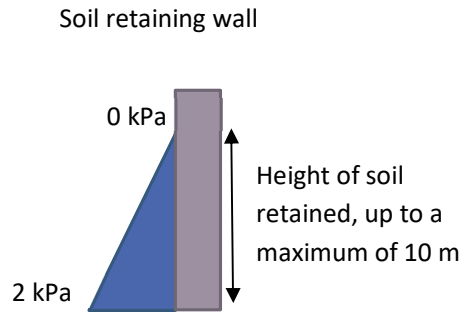

FIGURE 17: PRESSURE PROFILE TO ADOPT IN DESIGN OF SOIL RETAINING WALLS FOR EXCEPTIONAL LOAD CONDITION (GROUNDWATER) REPRESENTING BLOCKED DRAINAGE

ANNEXURE D: GROUNDWATER MODELLING

Jacobs TYPSA
Jacobs Typsa Joint Venture

Technical Memo

To	[redacted]	Date 8 April 2022
Copies		Document ID SMWSTCTP-AFJ-NST-SN350-ST-RPT-003050[B] Appendix G Appendix G Annexure D
From	[redacted]	Revision A
Subject	North Strathfield Station Groundwater Modelling – Stage 2 – Annexure D	

1. Introduction

The objective of this memorandum is to summarise groundwater modelling undertaken in support of the Stage 2 North Strathfield 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 2 North Strathfield 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 2 North Strathfield Station design. The modelling objectives were to:

- Predict groundwater inflow rates to the North Strathfield 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 element modelling package for modelling groundwater flow in porous media.

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

2.3. Modelling method summary

A 2D cross section model was developed approximately southwest to northeast through North Strathfield Station and extended to appropriate boundaries. The model was calibrated to existing representative groundwater levels at North Strathfield 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 a ridge near Rodio Street, Lidcombe in the southwest, to Hen and Chicken Bay of the Paramatta River 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.

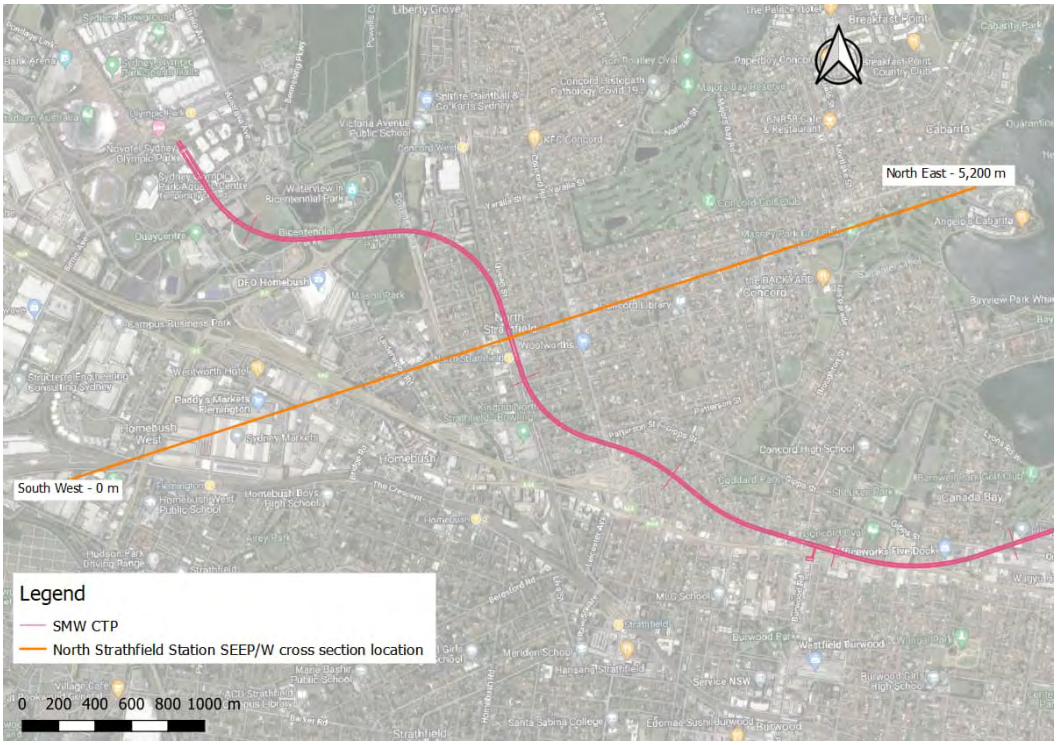

FIGURE 1 NORTH STRATHFIELD STATION SEEP/W CROSS SECTION LOCATION

2.4.2. Model layers

Three hydrogeological units are represented in the model: Quaternary Alluvium, Ashfield Shale and Hawkesbury Sandstone. Fill and residual soil units are not included in the model because the water table is generally 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 2.5 m to 4.0 m thick) and is characteristically similar to the Hawkesbury Sandstone in its hydrogeological properties.

The extent of the alluvium is currently unknown. For this reason, two different alluvium profiles were represented in separate versions of the model: a base case scenario, which represented the alluvium extent based on the Sydney 100,000 geological map; and an extended alluvium scenario, which represented the alluvium deposit associated with Powells Creek extending to the east of the current Powells Creek concrete lined channel. This approach was taken due to a lack of borehole data for the area of the alluvium to the southwest of the station. In both alluvium cases, the ground profile inferred in the CTP geological long section, in the location where Powells Creek is crossed by the CTP (about 730 m north of SEEP/W section), was considered to infer alluvium thickness.

The Ashfield Shale layer is represented from ground surface level, or beneath the alluvium, where the alluvium is present, to a uniform level of -15 mAHD along the entire section and is based on the level of the Ashfield Shale/Mittagong Formation interface which is applicable for the majority of the station excavation. The Hawkesbury Sandstone/ Mittagong Formation model layer occurs beneath the Ashfield Shale layer and its base is represented at a level of -65 mAHD. This base level is 62.5 m below the base of the station excavation (-2.5 mAHD) and therefore provides sufficient model thickness to enable interaction of the station excavations with the underlying groundwater system.

The base case model layers and boundary conditions are shown in Figure 2. The extended alluvium case model layers are shown in Figure 3, which shows how the alluvium is extended to the east of the current Powells Creek concrete lined channel under the extended alluvium extent case.

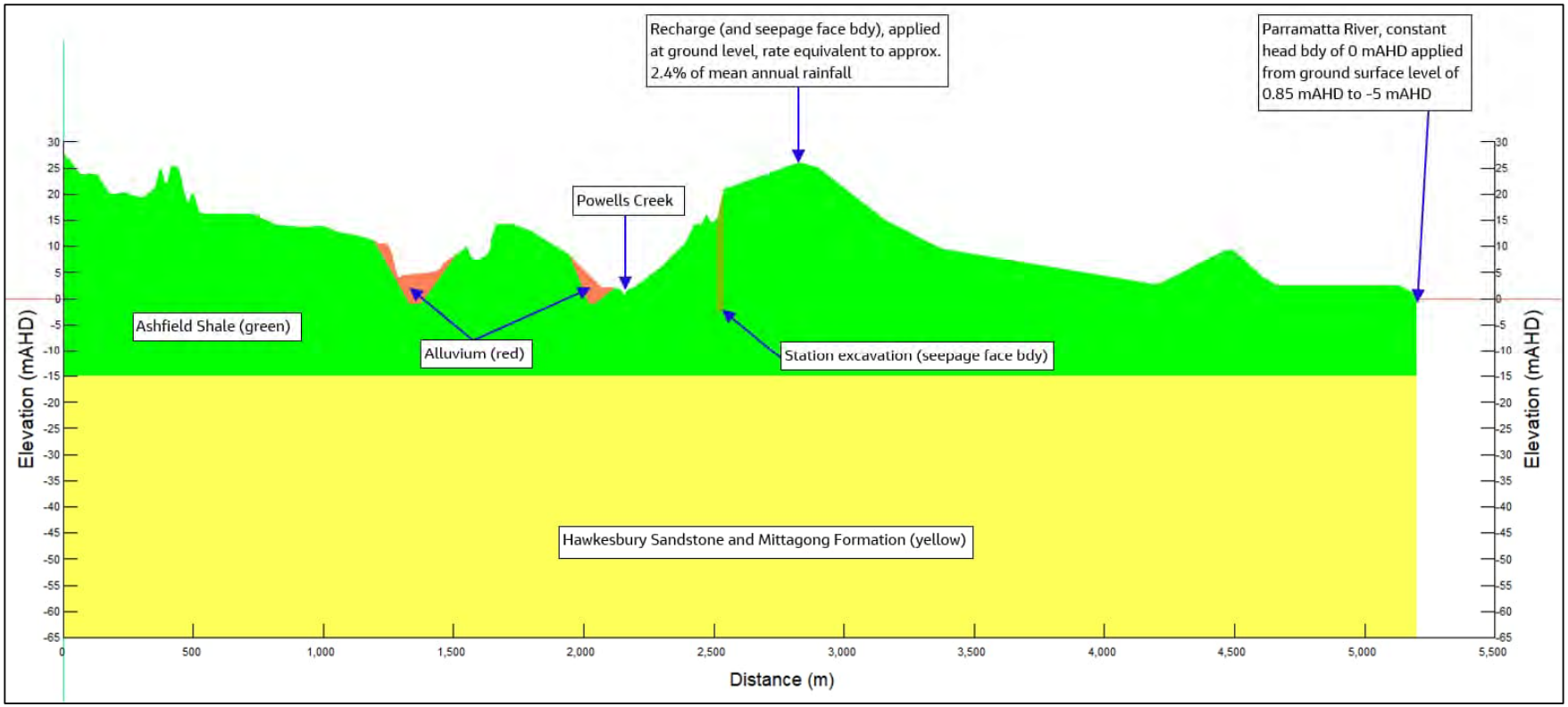

FIGURE 2 NORTH STRATHFIELD STATION SEEP/W MODEL SET UP. BASE CASE ALLUVIUM PROFILE SHOWN. NOTE VERTICAL EXAGGERATION OF 20:1

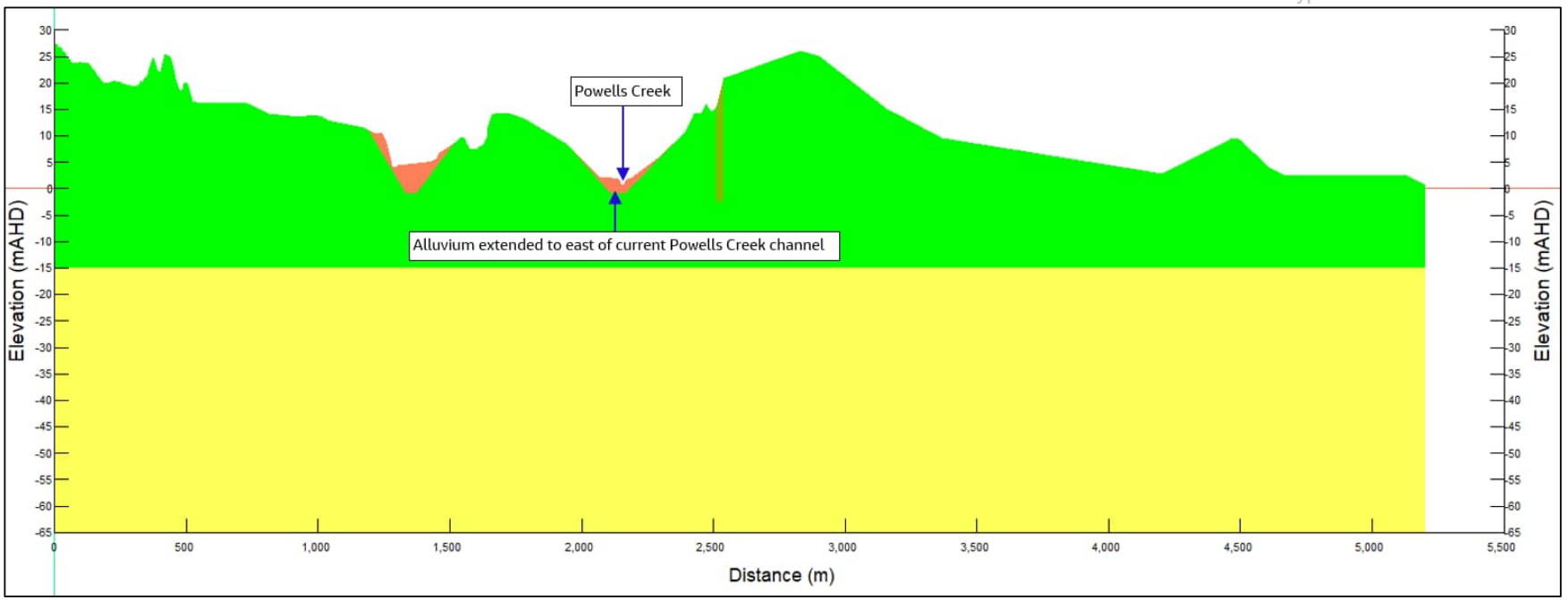

FIGURE 3 NORTH STRATHFIELD STATION SEEP/W MODEL. EXTENDED ALLUVIUM CASE. NOTE VERTICAL EXAGGERATION OF 20:1

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, typically 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 2 Hydrogeological Assessment Report).

Due to the lack of borehole data covering the alluvium to the southwest of the station, the alluvium composition is not known. To address this uncertainty, both the base case model and extended alluvium case model represented the alluvium as predominantly sandy, and separately, as predominantly clayey.

TABLE 1 HYDROGEOLOGICAL PARAMETER VALUES APPLIED IN MODEL

Parameter	Quaternary alluvium	Ashfield Shale	Hawkesbury Sandstone	Justification
Saturated horizontal hydraulic conductivity (m/d)	1.00 (predominantly sandy) 0.005 (predominantly clayey)	0.012	0.0173	Alluvium based on regional literature review, as documented in hydrogeological properties annexure, Annexure B. Ashfield Shale and Hawkesbury Sandstone 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	N/A	100	100	Applied over North Strathfield Station excavation area to represent free drainage within the excavation that would occur during excavation
Ratio of vertical to horizontal hydraulic conductivity	0.1	0.1	0.1	Based on regional literature review, as documented in hydrogeological properties annexure, Annexure B
Specific yield	0.20 (predominantly sandy) 0.06 (predominantly clayey)	0.02	0.05	Based on regional literature review, as documented in hydrogeological properties annexure, Annexure B
Coefficient of volume compressibility (kPa^{-1})	1.02×10^{-6}	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

2.4.3. Mesh resolution

A mesh resolution of 5 m was applied globally.

2.4.4. Boundary conditions

Boundary conditions are shown in Figure 2 and included:

- External constant head – applied at a level of 0 mAHD, from ground level (0.85 mAHD) to a level of -5 mAHD, at northeastern extent of model, to represent Hen and Chicken Bay of the Parramatta River.
- Recharge applied at a rate equivalent to 2.4% of mean annual rainfall over the whole section. This recharge rate was arrived at during model calibration by matching modelled groundwater levels to existing conditions.
- A seepage face boundary was applied over the whole section to represent potential evapotranspiration and discharge at areas of relatively low elevation.
- Internal potential seepage face applied around station excavation. 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 constant head boundary was 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 approximately 15 mAHD at the centre of the station. The water table level target was achieved and the calibrated water table level for the base case scenario with the alluvium represented as predominantly sandy, and alternatively, as predominantly clayey, is shown in Figure 4 and Figure 5, respectively.

The calibrated water table levels are generally similar for the base case scenario with the alluvium represented as predominantly sandy, and alternatively, as predominantly clayey. Noteworthy differences are as follows:

- For the base case scenario where the alluvium is represented as predominantly sandy, the water table at the station is approximately 14.5 mAHD, compared to approximately 15.2 mAHD for the predominantly clayey scenario.
- For the base case scenario where the alluvium is represented as predominantly sandy, the water table within the alluvium is deeper compared to the predominantly clayey scenario. The difference is up to approximately 4 m.

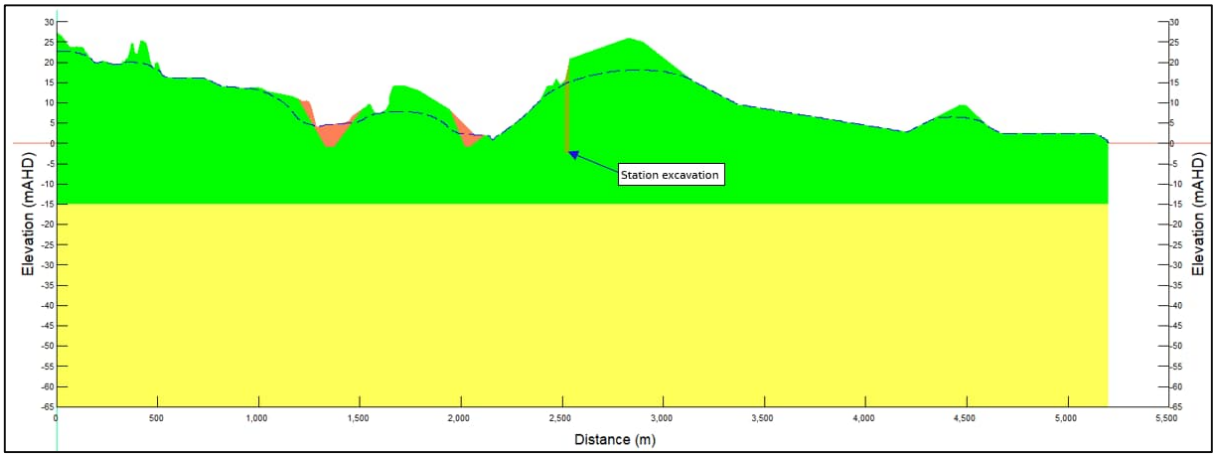

FIGURE 4 CALIBRATED WATER TABLE LEVEL (BLUE DASHED LINE) FOR BASE CASE, WITH ALLUVIUM REPRESENTED AS PREDOMINANTLY SANDY. NOTE VERTICAL EXAGGERATION OF 20:1

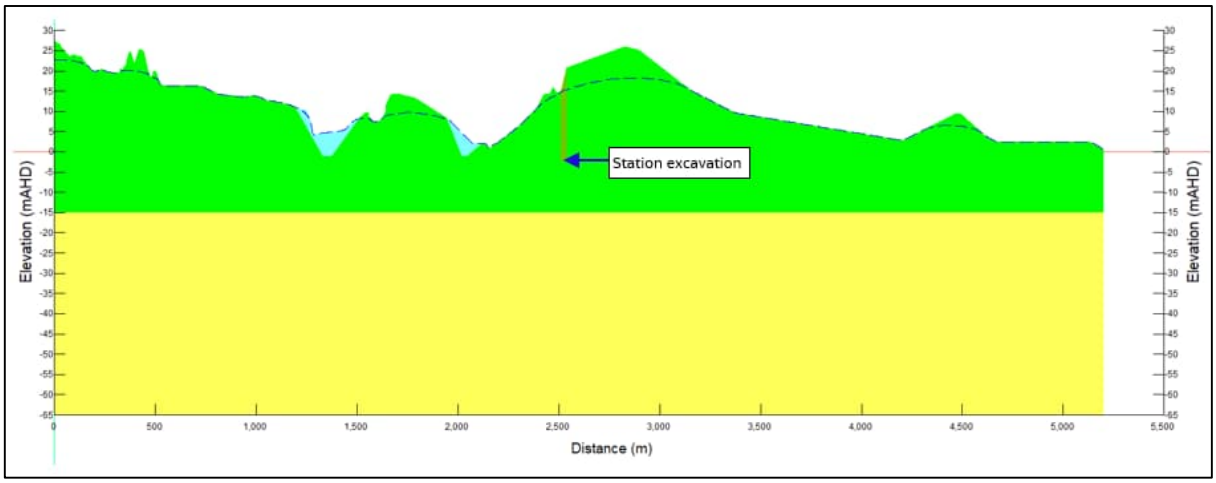

FIGURE 5 CALIBRATED WATER TABLE LEVEL (BLUE DASHED LINE) FOR BASE CASE, WITH ALLUVIUM REPRESENTED AS PREDOMINANTLY CLAYEY. NOTE VERTICAL EXAGGERATION OF 20:1

2.5.2. Groundwater inflows

Model-predicted groundwater inflow rates to the station excavation are shown in Figure 6 and are up to 36 m³/d (0.42 L/s), with a lower steady state inflow rate of approximately 13 m³/d (0.15 L/s). The predicted groundwater inflow rates are similar for the base case alluvium extent and extended alluvium cases, and are similar when the alluvium is represented as predominantly sandy, or alternatively, as predominantly clayey. The highest groundwater inflow rates occur under the extended alluvium case, when the alluvium is represented as predominantly clayey. This is due to relatively higher head in the area of the alluvium and therefore relatively higher hydraulic gradients between the area of the alluvium and station.

As shown in Figure 6, 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.

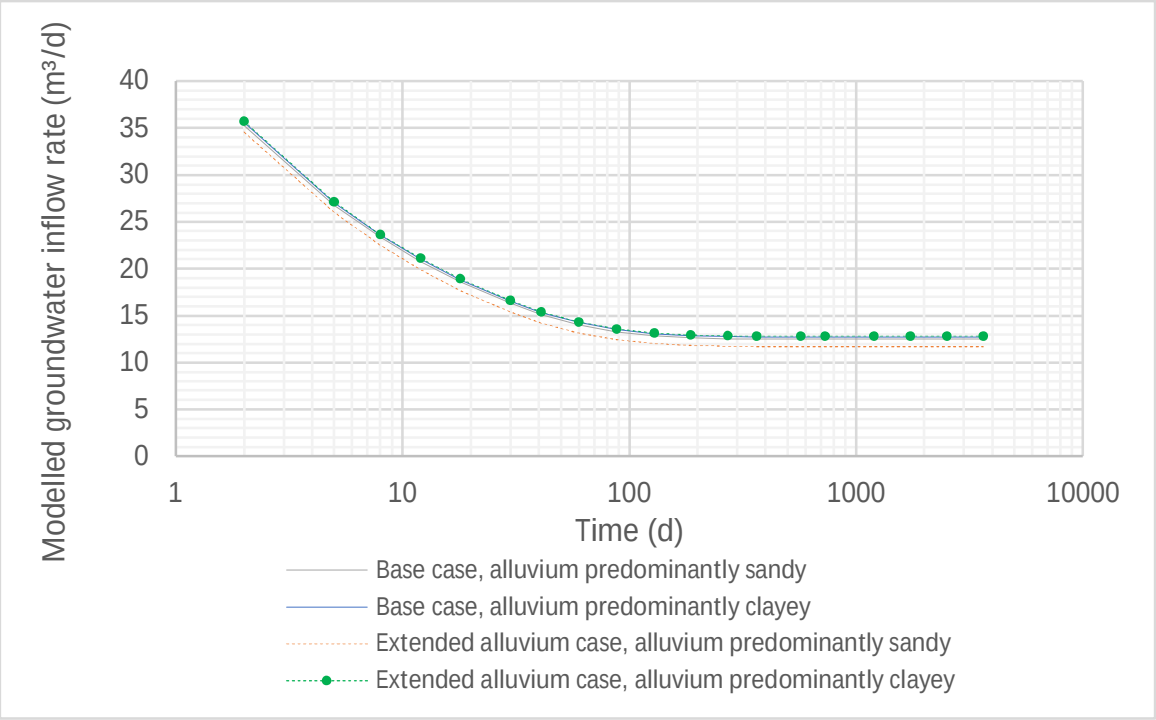

FIGURE 6 GROUNDWATER INFLOW RATES CALCULATED BY MODEL

As discussed in the main body of the North Strathfield Station Hydrogeological Assessment Report, an inferred (potential) fault and shear zone with approximately north-northeast orientation is anticipated within the North Strathfield Station box excavation (Figure 7). Two inferred faults pass through the station box excavation, with a third inferred fault passing through the southern nozzle, coming within very close proximity to the southeastern station excavation corner.

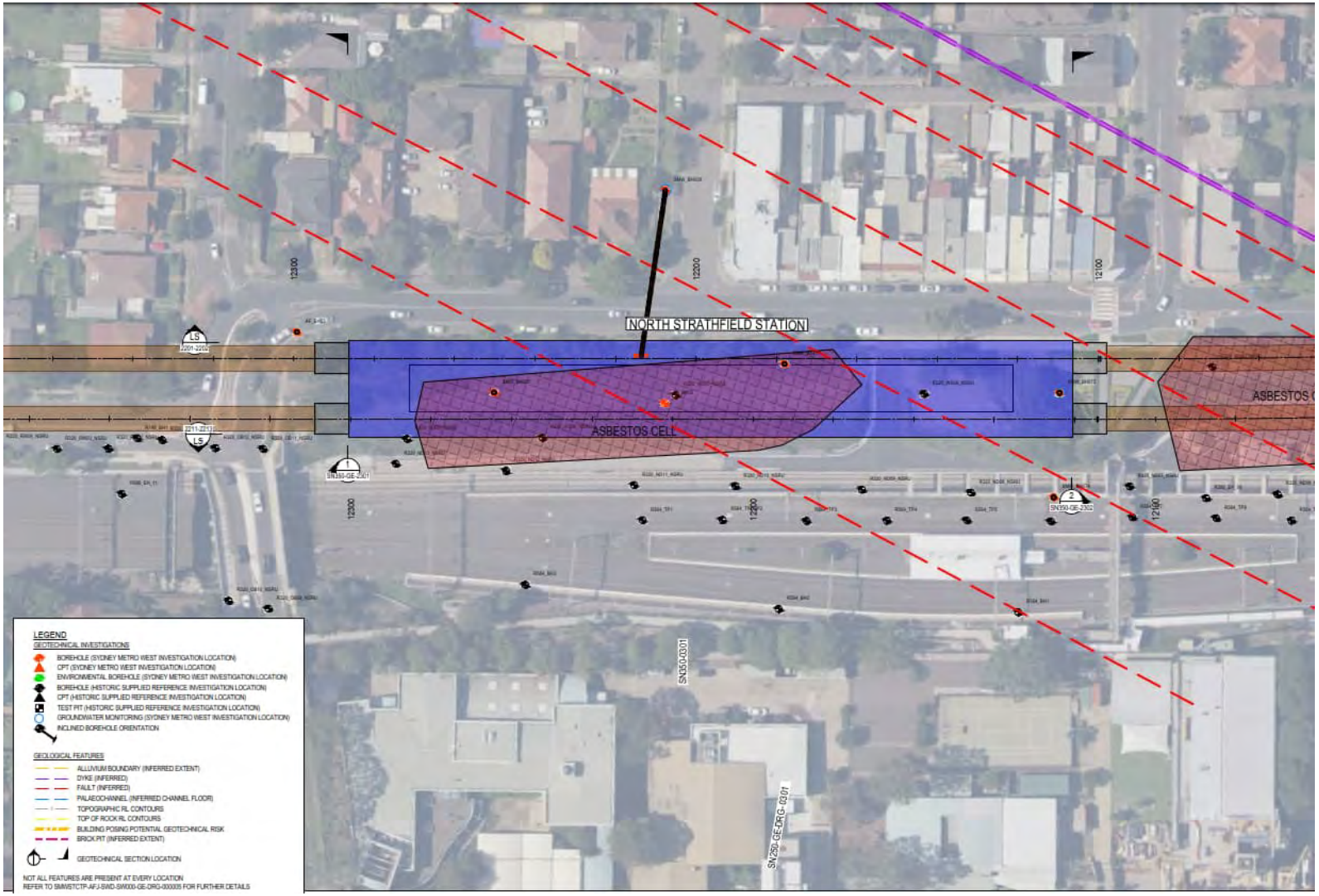

FIGURE 7 GEOLOGICAL FEATURES IN VICINITY OF NORTH STRATHFIELD STATION

Relatively high Lugeon values occurred in boreholes SMW_BH073 (included Lugeon values of 11 and 16), located near southern extent of station excavation, and R320_ND04 (Lugeon value of 10), located about 100 m south of the station excavation. The relatively high Lugeon values at SMW_BH073 are interpreted to be associated with the inferred zone of faulting / joint swarms. Similarly, the relatively high value at R320_ND04 is interpreted to be associated with an inferred fault. Additionally, borehole AF_BH32i, drilled at an angle of 66 degrees, intersected the most northern inferred fault. Packer testing within AF_BH32i across the depth zone of the structure, represented by core loss and joint swarms, returned a maximum Lugeon value of 5.

It is possible that rock in the vicinity of the inferred fault zones is of higher permeability than the adjacent rock. However, apart from the tests at SMW_BH073, R320_ND04 noted above and AF_BH32i, the packer test data for the site does not support this notion. Other tests undertaken near the inferred faulting did not return relatively elevated Lugeon values.

If three fault zones with enhanced hydraulic conductivity are assumed to pass through the station excavation at a north-northeast orientation and each have an assumed width of 1.5 m (inferred from borehole AF_BH32i), the total area of enhanced hydraulic conductivity relative to the total station excavation area would be approximately 5 percent. Assuming the maximum packer test value of 16 Lugeons (0.9 m/d) for the zone of enhanced hydraulic conductivity, under these circumstances, the bulk hydraulic conductivity in the area of the station excavation could be about 1.6 times higher than the value adopted for modelling. As a result, groundwater inflows under these circumstances could be expected to be approximately 1.6 times higher than modelled. Alternatively, if a value of 500 Lugeons (4.3 m/d) is assumed for the zone of enhanced hydraulic conductivity, under these circumstances, the bulk hydraulic conductivity in the area of the station excavation could be about 20 times higher than the value adopted for modelling and therefore groundwater inflows could be expected to be about 20 times higher than modelled. Additionally, if hydraulic conductivity values are elevated in other as-yet unidentified zones, then groundwater inflows may be potentially higher. The potential implications of this are discussed in the main body of the Hydrogeological Assessment Report.

2.5.3. Water table drawdown

Water table drawdown is discussed generally in this section, with water table drawdown specifically in the alluvium discussed in Section 2.5.4.

The modelled water table surfaces for the base case scenario with the alluvium represented as predominantly sandy, and alternatively, as predominantly clayey, is shown in Figure 8 and Figure 9, respectively.

The modelled water table surfaces for the extended alluvium case with the alluvium represented as predominantly sandy, and alternatively, as predominantly clayey, is shown in Figure 10 and Figure 11, respectively.

Drawdown of the water table for the base case scenario with the alluvium represented as predominantly sandy, and alternatively, as predominantly clayey, is shown in Figure 12. In Figure 12, 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). Consequently, drawdown at a time of two years is not shown in Figure 12.

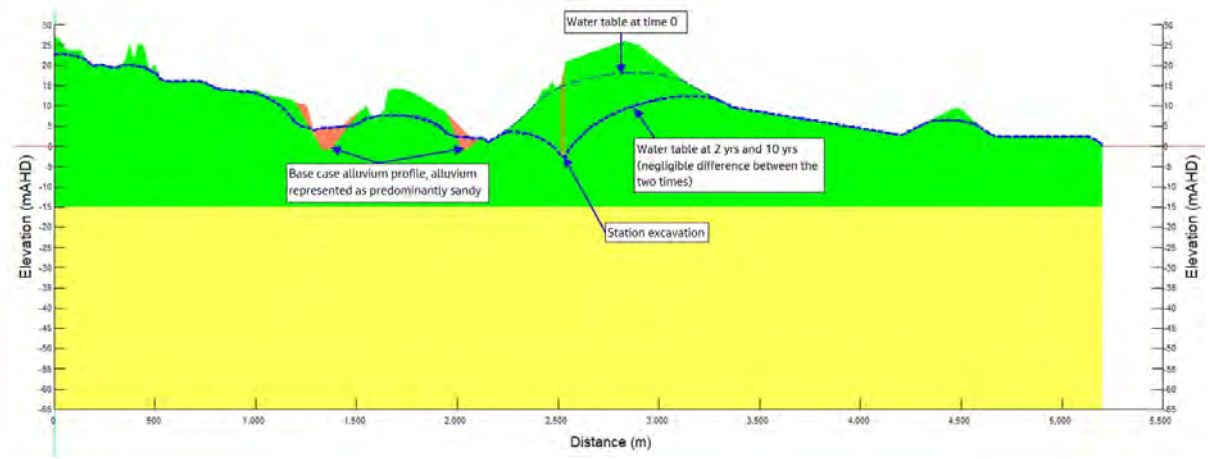

FIGURE 8 MODELLED WATER TABLE LEVELS FOR BASE CASE, WITH ALLUVIUM REPRESENTED AS PREDOMINANTLY SANDY. NOTE VERTICAL EXAGGERATION OF 20:1

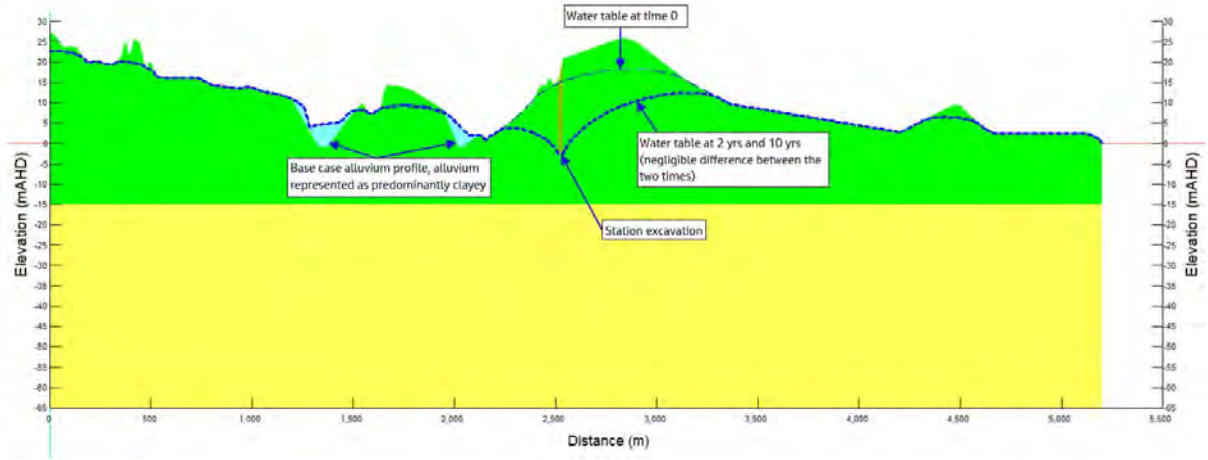

FIGURE 9 MODELLED WATER TABLE LEVELS FOR BASE CASE, WITH ALLUVIUM REPRESENTED AS PREDOMINANTLY CLAYEY. NOTE VERTICAL EXAGGERATION OF 20:1

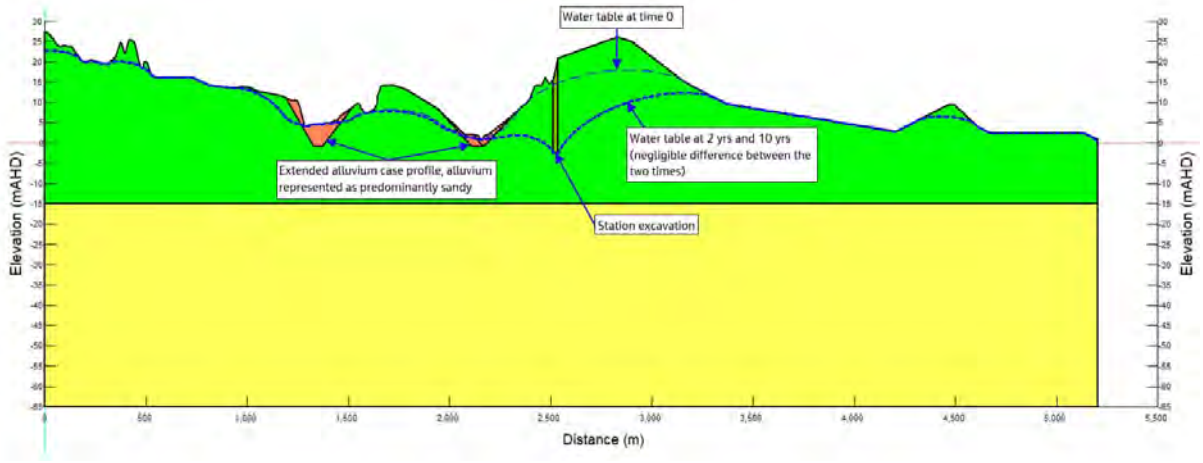

FIGURE 10 MODELLED WATER TABLE LEVELS FOR EXTENDED ALLUVIUM CASE, WITH ALLUVIUM REPRESENTED AS PREDOMINANTLY SANDY. NOTE VERTICAL EXAGGERATION OF 20:1

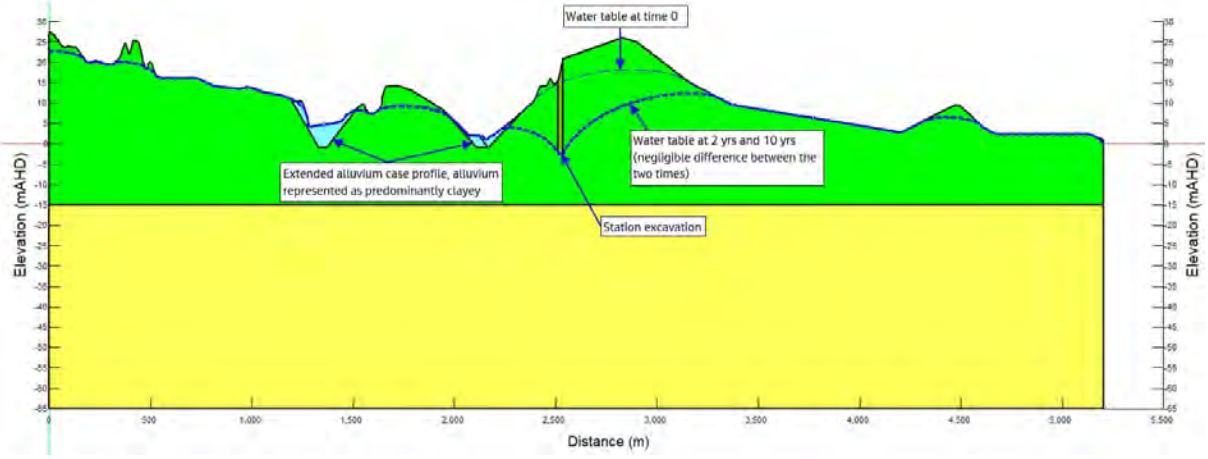

FIGURE 11 MODELLED WATER TABLE LEVELS FOR EXTENDED ALLUVIUM CASE, WITH ALLUVIUM REPRESENTED AS PREDOMINANTLY CLAYEY. NOTE VERTICAL EXAGGERATION OF 20:1

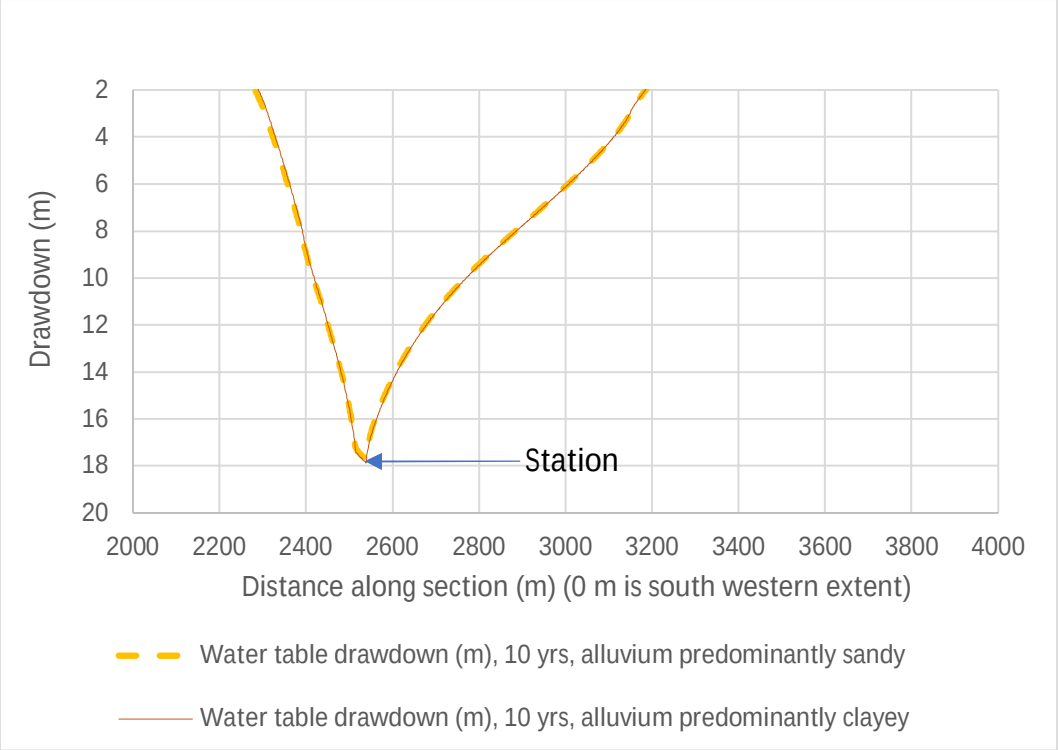

FIGURE 12 MODELLED DRAWDOWN TO WATER TABLE FOR BASE CASE.

2.5.4. Water table drawdown in alluvium

With respect to the base case simulations, modelled drawdown in the alluvium was negligible for scenarios where the alluvium was represented as predominantly sandy or clayey.

The worst case drawdown occurred in the extended alluvium case, with the alluvium represented as predominantly sandy, and is shown in Figure 13. The maximum modelled drawdown was approximately 0.75 m and occurred at the eastern extent of the eastern alluvium. The drawdown was negligible in the southwestern portion of the alluvium.

It is noted that drawdown in the alluvium could be greater if the recharge rate was lower. However, the modelled recharge rate that was applied over the areas of alluvium is considered reasonable. Additionally, supplementary model runs with a uniform recharge rate of one percent resulted in drawdown less than that of the worst case scenario described above.

It is noted that there is significant uncertainty in these predictions as the nature, depth and extend of the alluvium is unknown.

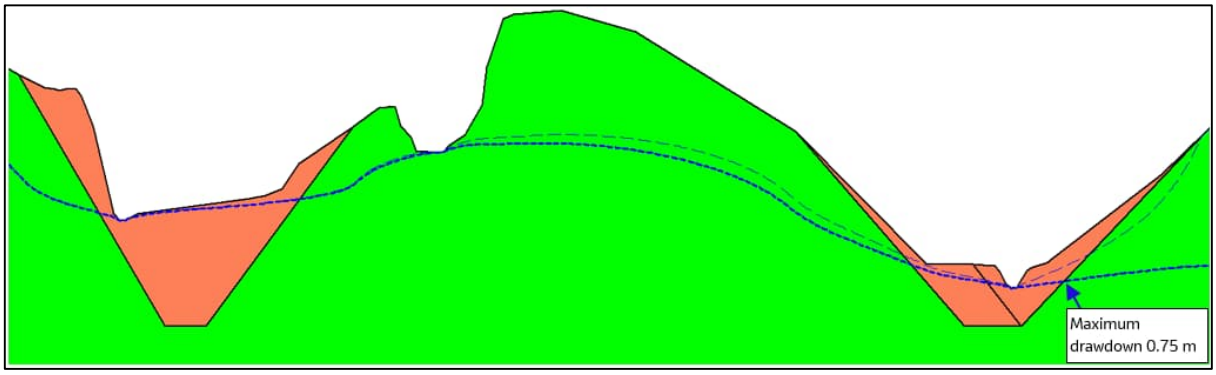

FIGURE 13: MODELLED WATER TABLE LEVELS FOR EXTENDED ALLUVIUM CASE, WITH ALLUVIUM REPRESENTED AS PREDOMINANTLY SANDY, DEMONSTRATING MAXIMUM DRAWDOWN IN ALLUVIUM. NOTE VERTICAL EXAGGERATION OF 20:1