Burwood North Station Excavation - Hydrogeological Design Report

SMWSTCTP-AFJ-BWT-GE-RPT-000008 Revision 00

Sydney Metro West – Central Tunnelling Package

DOCUMENT APPROVAL

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REVISION HISTORY

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CONTENTS

1. INTRODUCTION

This report provides hydrogeological advice for the design of Burwood North Station excavations, and provides the following:

- Updated hydrogeological conceptual model(s) of the site
- Updated groundwater modelling to estimate the potential groundwater inflows to the station excavations, and associated groundwater level drawdown
- Assessment of potential groundwater-related impacts and risks for Burwood North Station excavations
- The following additional assessments are included in this report since the previous Stage 2 design report:
- Two-dimensional modelling to estimate the potential groundwater inflows to the station excavations, and associated groundwater level drawdown
- Review of the potential influence of the M4 East infrastructure on groundwater levels based on more recent groundwater monitoring data.

2. PARTICULAR SPECIFICATIONS AND MINISTERS' CONDITIONS OF APPROVAL

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

4.1.7 Groundwater control

(a) The Tunnelling contractor must comply with the following for the drainage of assets:

(v) Crossover Caverns - undrained [SM-W-CTP-PS-1015]

(vi) Station Caverns and Station Adits - undrained [SM-W-CTP-PS-1019]

(vii) Station Excavations - drained [SM-W-CTP-PS-1020]

(viii) Shaft Excavations - drained [SM-W-CTP-PS-1021]

(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.

(d) The Tunnelling Contractor must design undrained structures without the inclusion of a groundwater pressure relief system. [SM-W-CTP-PS-1027]

4.1.8 Groundwater Seepage

(a) The Tunnelling Contractor must prevent groundwater seepage into or through the station structure from the cavern and adits structures, including at connection joints. [SM-W-CTP-PS-1029]

(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]

(c) The Tunnelling Contractor must not use any permanent dewatering systems for lowering of groundwater levels. [SM-W-CTP-PS-1031]

(d) The Tunnelling Contractor must ensure that there are no adverse impacts from groundwater chemistry on the integrity or operation of the Works for the relevant Design Life. [SM-W-CTP-PS-1032]

(e) The Tunnelling Contractor must prevent groundwater seepage through the Cross Passages, Cross Passages with Sump, Niches, Station Adits and Station Caverns concrete linings. [SM-W-CTP-PS-1033]

(f) The Tunnelling Contractor must ensure groundwater seepage through the Running Tunnels, Nozzle Enlargements and Crossover Caverns concrete linings does not exceed: [SM-W-CTP-PS-1035]

(i). an average of 2.0ml per hour per m² of the concrete lining intrados surfaces for any section between high points of the Running Tunnel, measured in the sump; and [SM-W-CTP -PS-1036]

h) The groundwater seepage within each Station Excavation and each Shaft Excavation must not exceed: [SM-W-CTP-PS-1039]

(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. Burwood North Station Excavation: 114,000 litres; [SM-W-CTP-PS-1046]

D. Burwood North Station Shaft Excavation: 16,000 litres; [SM-W-CTP-PS-2629]

Design groundwater levels have been developed considering:

4.1 Civil and Structural

4.1.3 Design Loading

4.1.3.1 General

(a) The Tunnelling Contractor must apply design loading cases for all civil and structural works, including load factors for the ultimate, fatigue and serviceability limit states in accordance with the following standards where applicable, in the following order of precedence: [SM-W-CTP-PS-702]

(i) AS 5100 Bridge Design Series [SM-W-CTP-PS-703]

(ii) 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]

(iii) AS 4678 Earth - retaining structures for ground loadings, for free-standing retaining walls; and [SM-W-CTP-PS-705]

(iv) AS 1657 Fixed Platforms, walkways, stairways and ladders - Design, Construction and installation. [SM-W-CTP-PS-706]

(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]

(e) The Tunnelling Contractor must design all underground structures to resist foreseeable differences in groundwater table level between opposite sides of the completed underground structures for the applicable Design Life. [SM-W-CTP-PS-711]

(f) The Tunnelling Contractor must use a minimum difference in groundwater level table of 5m. This exceptional or temporary load case is considered 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]

(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]

4.1.3.6 Ground Pressure and Hydrostatic Pressure Loading

(a) The Tunnelling Contractor must assume the following design action resulting from hydrostatic pressure of water acting on surfaces below ground level (Fgw) for all underground structures including Project Works and Temporary Works: [SM-W-CTP-PS-909]

(i) the water level is at ground level; or [SM-W-CTP-PS-910]

where information is available, the ground water level with an annual probability of exceedance of 1 in 100. [SM-W-CTP-PS-911]

4.3 Tunnels, Caverns and Adits

4.3.1 Design Loadings

(a) The Tunnelling Contractor must design the Works to accommodate the following loadings: [SM-W-CTP-PS-1375]

(xiv) long term variations in groundwater levels; [SM-W-CTP-PS-1389]

Ministers' Conditions of Approval Relevant to this Report

Condition D122: The Proponent must submit a revised Groundwater Modelling Report in association with Stage 1 of the CSSI to the Planning Secretary for information before bulk excavation at the relevant construction location. The Groundwater Modelling Report must include:

(a) for each construction site where excavation will be undertaken, cumulative (additive) impacts from nearby developments, parallel transport projects and nearby excavation associated with the CSSI;
(b) predicted incidental groundwater take (dewatering) including cumulative project effects;
(c) potential impacts for all latter stages of the CSSI or detail and demonstrate why these later stages of the CSSI will not have lasting impacts to the groundwater system, ongoing groundwater incidental take and groundwater level drawdown effects;
(d) actions required after Stage 1 to minimise the risk of inflows (including in the event latter stages of the CSSI are delayed or do not progress) and a strategy for accounting for any water taken beyond the life of the operation of the CSSI;
(e) saltwater intrusion modelling analysis, from estuarine and saline groundwater in shale, into The Bays metro station site and other relevant metro station sties; and
(f) a schematic of the conceptual hydrogeological model.

3. DESIGN INFORMATION

Burwood North Station comprises a station box excavation, with a crossover cavern to the immediate west, and a pedestrian access adit that crosses Parramatta Road to the south of the station box.

4. HYDROGEOLOGICAL CONCEPTUAL MODEL

4.1 GEOLOGY

4.1.1 HYDROGEOLOGICAL UNITS

The location of Burwood North Station, along with the location of boreholes and inferred geological structure, is illustrated in Figure 4-1.

The geological section of Burwood North Station is illustrated in the long sections shown in the Geotechnical Interpretive Report, including a section through the adit and shaft that lie to the south of the station box.

The geological units at the station include:

- Fill
- Residual soils
- Ashfield Shale
- Mittagong Formation
- Hawkesbury Sandstone
- Dyke.

The station box will be excavated through all of the above units, while the crossover cavern will be excavated in the Mittagong Formation and Hawkesbury Sandstone only, and the adit and shaft will be excavated into Ashfield Shale only.

The fill and residual soils are relatively thin, having a typical combined thickness of up to about two metres.

As shown in the long sections shown in the Geotechnical Interpretive Report, a dyke is inferred to cross the far eastern end of the station box, and another dyke is inferred to cross the far western end of the cavern.

If 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 sandstone 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. It is possible that more weathered zones of sandstone would exhibit relatively higher permeability than the surrounding rock or metamorphosed zones.

An inferred (potential) fault zone with approximately north-south orientation lies in close proximity to the western end of the station box. It is possible that rock in the vicinity of the inferred fault zone is of higher permeability than the adjacent rock.

4.1.2 PALAEOCHANNEL

A palaeochannel is interpreted to lie to the east of the station site, surrounding St Lukes Canal. Figure 4-2 shows the interpreted location of the palaeochannel. As shown in the Geotechnical Interpretive Report, the maximum thickness of the alluvium is interpreted to be approximately 4 m.

There is limited data available within the palaeochannel at this location, with boreholes drilled close to the alignment recording only fill and residual soil overlying the underlying Ashfield Shale.

As noted in the Geotechnical Interpretive Report, the borehole logs classify the soils in this area as thicker residual soil (in comparison to shallow residual soil outside of the palaeochannel). This may be the result of misclassification of the alluvium within the palaeochannel as residual soil, as the presence of thick residual soils in this area is uncharacteristic when reviewed against the surface and rock topography.

The assumed alluvial extent has therefore been based on interpretation of top of rock (Ashfield Shale) and a conservative approach, assuming that, as is common, residual materials were eroded during palaeochannel formation and alluvium sits directly upon rock. Properties have been inferred based on boreholes which encountered alluvium within the adjacent palaeochannel to the east, which is also part of the larger Hen and Chicken Bay Palaeochannel network, and likely saw similar depositional environments and materials.

The alluvium beneath the Cintra Park Decline facility appears to have been removed and replaced by fill, most likely as part of the previous development, as the presence of fill (and absence of alluvium) is recorded in boreholes conducted prior to the M4 East development.

FIGURE 4-1: LOCATION OF BURWOOD NORTH STATION

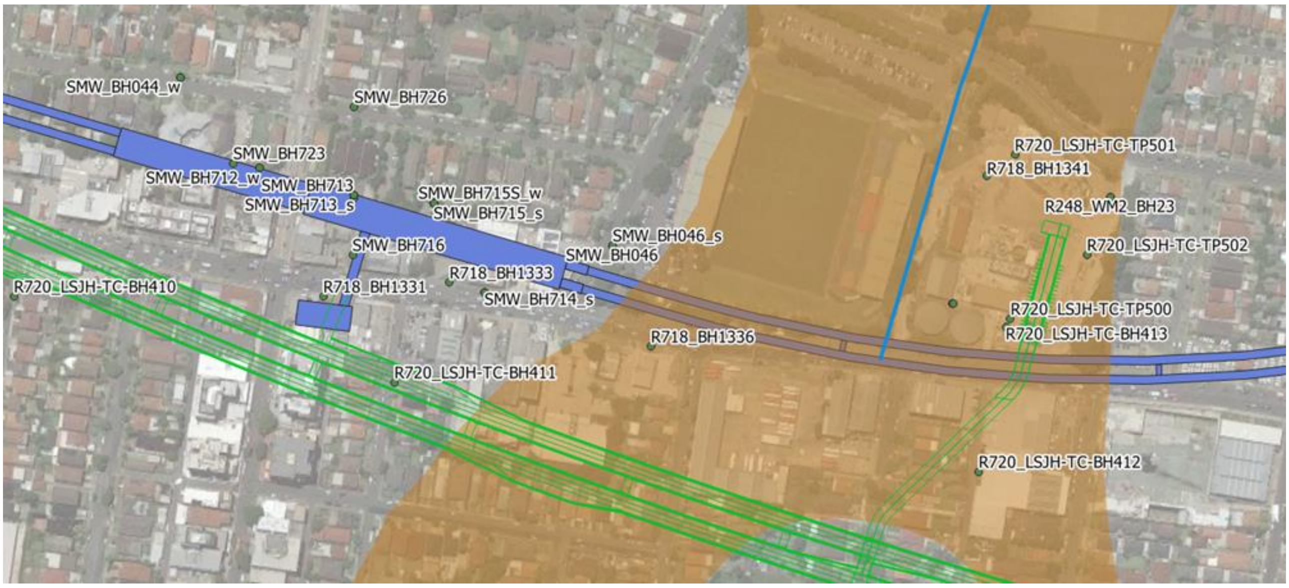


FIGURE 4-2: LOCATION OF PALEOCHANNEL NEAR BURWOOD NORTH STATION (ALSO SHOWING GEOTECHNICAL BOREHOLES LOCATIONS AND M4 EAST INFRASTRUCTURE IN GREEN)

4.2 HYDROGEOLOGICAL UNITS

Based on the above discussion, three hydrogeological units are considered to occur at Burwood North Station including:

- Alluvium (to the east of the station box only)

- Ashfield Shale
- Mittagong Formation and Hawkesbury Sandstone.

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

The Mittagong Formation and Hawkesbury Sandstone is likely to act as a combined aquifer unit.

Groundwater flow in these units 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 lower permeabilities.

4.3 GROUNDWATER LEVELS

4.3.1 OBSERVED GROUNDWATER LEVELS

Figure 4-3 shows groundwater monitoring locations in the vicinity of the Burwood North Station site.

FIGURE 4-3: GROUNDWATER MONITORING LOCATIONS AT BURWOOD NORTH STATION

Table 2 lists piezometer construction details, and Table 3 lists recorded groundwater levels. Table 3 and notes the material in which the groundwater lies for the piezometers.

Monitored groundwater levels at Burwood North station range between approximately 17 m AHD and 1 m AHD, and between about 2 m and 11 m below ground surface.

In addition to the piezometers listed in Table 1, two piezometers were installed for the M4 East project (CSJH, 2016), LSJH-TC-BH411-S and LSJH-TC-BH411-D, located about 2 km south of the station box and screened in Ashfield Shale and Hawkesbury Sandstone, respectively. Reported groundwater levels for these piezometers are both 5.5 m below ground surface. These levels are consistent with the monitoring at the station site.

The groundwater level lies within the Ashfield Shale in all piezometers except SMW_BH712w, and on average lies about 4 m below the top of extremely weathered Ashfield Shale (note that this is based on a particular interpretation of the top of Ashfield Shale).

Based on the available information, there does not appear to be a perched water table within the fill/residual soils.

The deeper groundwater levels observed in piezometers that are partially or fully screened across the Hawkesbury Sandstone suggest hydraulic separation between the Ashfield Shale and Hawkesbury Sandstone.

Figure 4-4 explores this more fully, plotting groundwater pressure head against the elevation of the base of the piezometer screen for all piezometers listed in Table 2 (except SMW_BH046 which has unreliable groundwater level values reported, with reported groundwater levels inconsistent with monitoring well construction details). The pressure heads are therefore representative of the groundwater pressure head across the screen interval of the geological unit but represented at the elevation of the base of the screen interval. The figure shows the maximum and minimum hydrostatic profiles that would exist for these piezometers as well as the pressure head in all piezometers, including coupled (shallow and deep) piezometers. In all cases, the trend in pressure profile is close to a hydrostatic trend, suggesting the potential for a hydraulically connected groundwater system across the residual soils/Ashfield Shale and Hawkesbury Sandstone. The responses in groundwater levels in shallow piezometers and corresponding dampened responses in groundwater levels in deeper piezometers (see Annexure A) further suggests a hydraulic connection, although this connection appears to reduce slightly in the deeper sandstone. In the absence of other (hydraulic testing) data, it is considered that there is not a perched water table in the existing system and that there is hydraulic connection between the geological units.

There is no groundwater monitoring data available in the vicinity of the paleochannel to the east of the station box. Groundwater levels in the alluvium are unknown, and it is not known whether the alluvium is a significant water-bearing hydrogeological unit.

TABLE 4-1: SUMMARY OF GROUNDWATER MONITORING PIEZOMETERS AT BURWOOD NORTH STATION

Bore ID	Ground Surface Elevation (m AHD)	Effective Screen Depth Top (m)	Effective Screen Depth Bottom (m)	Unit(s)	Monitoring Period
SMW_BH044	22.7	20.5	36.5	HAW	Sep 18 to May 19
SMW_BH046	6.5	1.1	3.3	Clay	May 18 to Aug 19
SMW_BH046s	6.5	4.0	9.5	MITT and HAW	Unknown
SMW_BH712w	18.8	1.2	13.0	Clay and ASH	Feb 21 to Jun 21
SMW_BH713s	14.6	1.6	7.0	Clay and ASH	Feb 21 to Jun 21
SMW_BH713w	14.6	11.5	17.9	ASH	Feb 21 to Jun 21
SMW_BH714s	8.2	1.6	7.2	ASH	Feb 21 to Jun 21
SMW_BH714w	8.2	8.1	17.5	ASH and HAW	Feb 21 to Jun 21
SMW_BH715s	12.0	1.6	7.0	Clay and ASH	Feb 21 to Jun 21
SMW_BH715w	12.1	5.0	11.0	ASH	Feb 21 to Jun 21

R718_BH1331	15.0	4.0	7.0	ASH	June 15 (single dip)
R718_BH1333	11.0	5.0	8.0	ASH	June 15 (single dip)
R718_BH1336	Unknown	5.0	8.0	Clay	June 15 (single dip)
AF_BH28	13.4	11.0	20.3	ASH and MITT	Dec 21 to Jan 22

TABLE 4-2: SUMMARY OF GROUNDWATER LEVELS AT BURWOOD NORTH STATION

Bore ID	Ground Surface Elevation (m AHD)	Unit(s)	Monitoring Period	Groundwater Elevation (m AHD)		Groundwater Depth (m bgl)	
				Approx. Typical	Maximum	Approx. Typical	Shallowest
SMW_BH044	22.7	HAW	Sep 18 to May 19	14.0	14.0	8.7	8.7
SMW_BH046	6.5	Clay	May 18 to Aug 19	-4.0*	-3.7*	10.5*	10.2*
SMW_BH046s	6.5	MITT and HAW	May 18 to Aug 19 (assumed)	<2.5 (dry)	<2.5 (dry)	>9.0 (dry)	>9.0 (dry)
SMW_BH712w	18.8	Clay and ASH	Feb 21 to Jun 21	16.9	17.6	1.9	1.2
SMW_BH713s	14.6	Clay and ASH	Feb 21 to Jun 21	9.4	9.6	5.2	5.0
SMW_BH713w	14.6	ASH	Feb 21 to Jun 21	8.6	8.7	6.0	5.9
SMW_BH714s	8.2	ASH	Feb 21 to Jun 21	3.4	3.7	4.8	4.5
SMW_BH714w	8.2	ASH and HAW	Feb 21 to Jun 21	-1.0	-0.4	9.2	8.6
SMW_BH715s	12.0	Clay and ASH	Feb 21 to Jun 21	9.6	9.9	2.4	2.1
SMW_BH715w	12.1	ASH	Feb 21 to Jun 21	9.5	9.7	2.6	2.4
R718_BH1331	15	ASH		11.7	ND	3.3	ND
R718_BH1333	11	ASH		6.8	ND	4.3	ND

Bore ID	Ground Surface Elevation (m AHD)	Unit(s)	Monitoring Period	Groundwater Elevation (m AHD)		Groundwater Depth (m bgl)	
				Approx. Typical	Maximum	Approx. Typical	Shallowest
R718_BH1336	Unknown	Clay	June 15 (single dip)	Unknown	ND	2.9	ND
AF_BH28	13.4	ASH, MITT	Dec 21 to Jan 22	Recovering. 0 (estimated)	ND	Recovering. 13.4 (estimated)	ND

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

*Reported values are inconsistent with monitoring well construction details and cannot be relied upon

TABLE 4-3: SUMMARY OF GROUNDWATER LEVEL DEPTHS AT BURWOOD NORTH STATION

Bore ID	Effective Screened Unit(s)	Typical Groundwater Level (m AHD)	Typical Groundwater Level (m bgl)	Stratigraphic Location of Groundwater Level
SMW_BH712w	Clay and ASH	16.9	1.9	Near base of clay
SMW_BH713s	Clay and ASH	9.4	5.2	3 m into ASH
SMW_BH715s	Clay and ASH	9.6	2.4	0.5 m into ASH
SMW_BH715w	ASH	9.5	2.6	1 m into ASH
SMW_BH713w	ASH	8.6	6	4 m into ASH
SMW_BH714s	ASH	3.4	4.8	3 m into ASH
SMW_BH714w	ASH and HAW	-1	9.2	7 m into ASH
SMW_BH046s	MIT and HAW	<-3	>9.5	In/below clay

				(borehole only intersects clay)
SMW_BH044	HAW	14	8.7	8 m into ASH
AF_BH28	ASH, MIT	0 (estimated)	13.4 (estimated)	10 m into ASH

Notes: ASH means Ashfield Shale, MIT means Mittagong Formation, HAW means Hawkesbury Sandstone

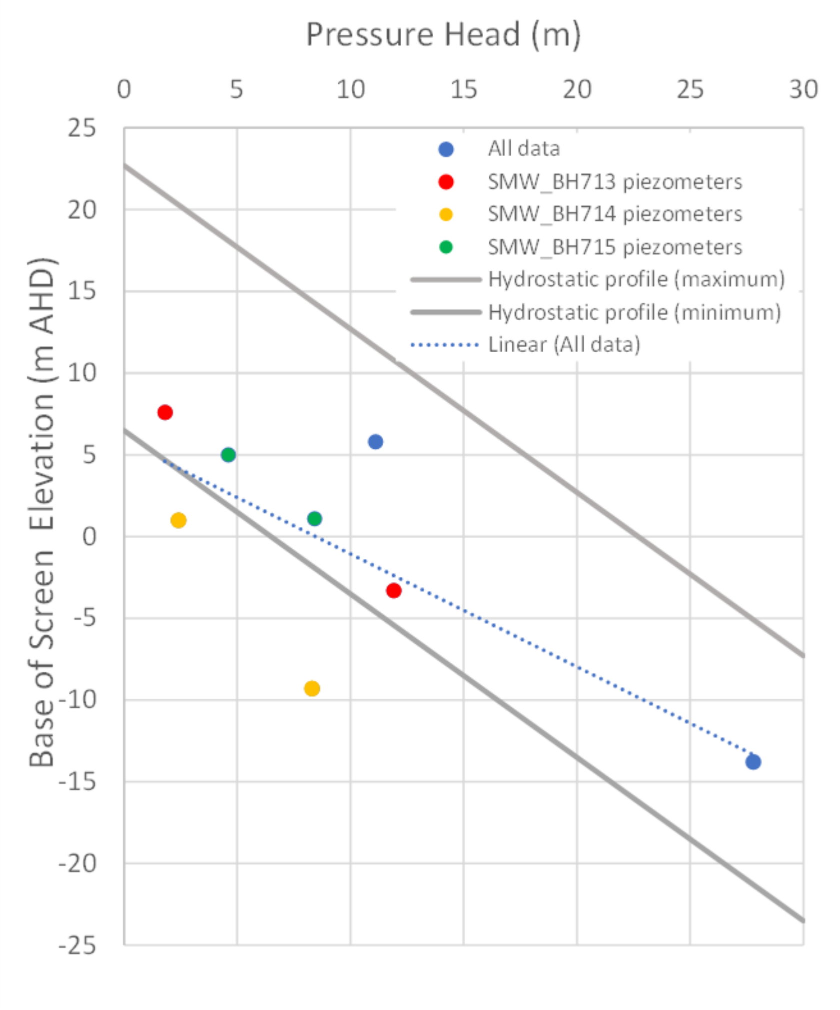


FIGURE 4-4: GROUNDWATER PRESSURE HEAD IN PIEZOMETERS AT BURWOOD NORTH STATION

4.3.2 RAINFALL AND DROUGHT

The cumulative mean monthly rainfall deviation since the year 2000 is shown in Figure 4-5 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.

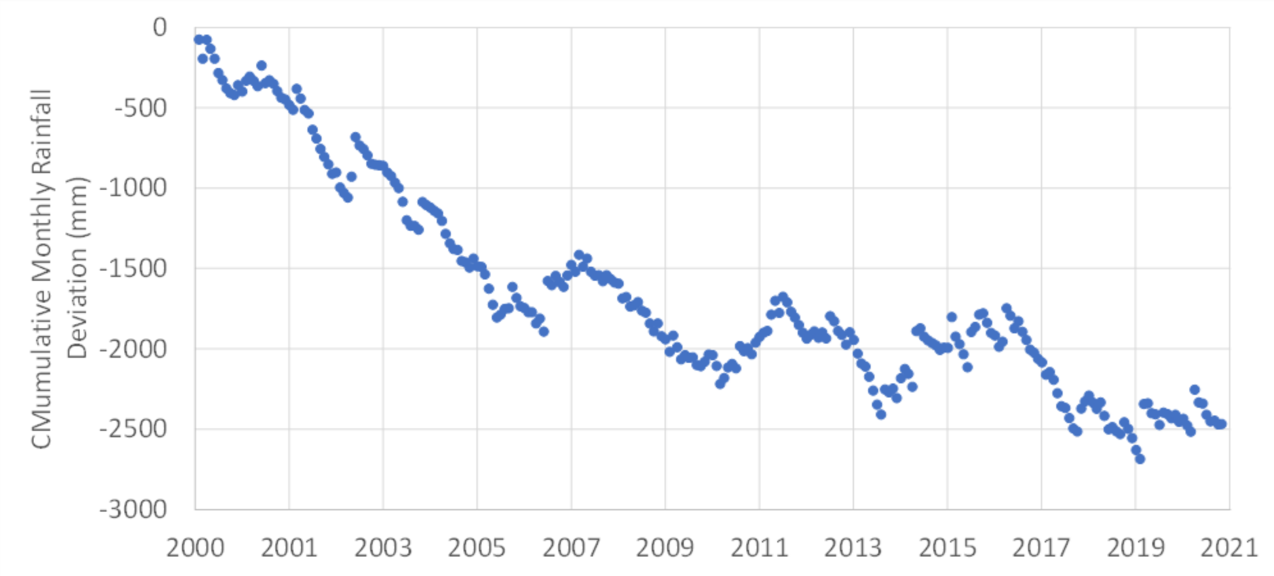


FIGURE 4-5: CUMULATIVE DEVIATION FROM MEAN MONTHLY RAINFALL AT CONCORD GOLF CLUB (BUREAU OF METEOROLOGY STATION 66013)

4.3.3 WETCONNEX M4 EAST

The WestConnex New M4 infrastructure (also referred to as M4 East) lie in the vicinity of Burwood North Station, including the M4 East tunnels, Substation No. 5, and the Cintra decline. These structures are shown in Figure 4-6.

Excavation of these infrastructure was complete in December 2018 and the M4 road tunnels were opened to traffic in July 2019.

FIGURE 4-6: LOCATION OF BURWOOD NORTH STATION BOX, CAVERN AND SHAFT (IN BLUE); AND M4 EAST TUNNELS, SUBSTATION NO. 5 AND CINTRA DECLINE (IN GREEN)

Groundwater level monitoring data for the M4 East project, or in the vicinity of these structures, have not been made available to JTJV. Responses from Sydney Metro to JTJV's/AFJV's Requests for Information on M4 East groundwater monitoring data remain outstanding. The existing status of groundwater levels in the vicinity of these structures is therefore unknown.

Available data is limited to the single (discrete) groundwater level measurements in M4 East piezometers R718_BH1331, R718_BH1333 and R718_BH1336; the approximately three-month continuous monitoring in SMW piezometers at the site; and one AFJV piezometer (AF_BH28) at the location of the shaft.

The only M4 East piezometer in close proximity to a SMW piezometer is R718_BH1333. Piezometers SMW_BH714s and R718_BH1333 are both screened across shallow Ashfield Shale, and over a similar depth horizon (although SMW_BH714s is shallower). Available data show a groundwater level in these piezometers of 6.8 m AHD in 2015 at R718_BH1333, and 3.4 m AHD in 2021 at SMW_BH714s. The difference in groundwater level of 3.4 m between these two piezometers (screened in the same geological unit) is greater than the short-term groundwater level variation observed. It is possible that groundwater levels in the vicinity of these piezometer may have fallen between 2015 and 2021 due to drawdown induced by the M4 East tunnels, as construction and opening of the M4 East tunnels took place between 2015 and 2021. However, as shown in Figure 4-5, this is also a period of below-average rainfall, and it is possible that decline in groundwater levels may (partially or wholly) be due to climatic conditions rather than the M4 East infrastructure.

Additional (and ideally continuous) monitoring data is required between 2015 and 2020 to assess the influence of M4 East tunnels on groundwater levels in the vicinity of Burwood North Station.

The drained M4 East tunnels lie to the immediate south of the station box, and run approximately parallel to the CTP tunnel alignment, as shown in Figure 4-6.

CSJH (2016) predicted groundwater level drawdown due to the M4 East drained structures prior to construction of the M4 East. The modelled long-term groundwater level drawdown associated with the M4 East drained structures is illustrated in Figure 4-7 below. This indicates that predicted groundwater level drawdown associated with the M4 East drained structures ranges between 15 m and 30 m at the Burwood North Station site.

The extent to which the tunnel has actually drained the rock is unknown.

The modelling indicates that groundwater pressures at the tunnel crown would be less than 10 kPa within about one year of tunnel drainage. Based on this, it is likely that the potential drawdown due to the tunnels (which opened for operation in July 2019) has already been fully realised. This is further supported by the fact that the groundwater levels monitored (from late 2018) at Burwood North Station for Sydney Metro West (see Annexure A) do not show declines in groundwater level over time. However, this cannot be confirmed without the additional monitoring data noted above.

Substation No. 5 forms part of WestConnex M4 East project. It is a drained mined tunnel that connects with the WestConnex mainline tunnel M110. Substation No. 5 is approximately 17 meters below the Burwood North Southern Entry Shaft and partially below the Burwood North Pedestrian Adit, as shown in Figure 4-8 and Figure 4-9.

Groundwater level monitoring in piezometer AF_BH28 from December 2021 to January 2022 (see Annexure A and tables above), located within the shaft footprint, shows groundwater levels significantly lower than those in surrounding piezometers screened across similar depth horizons in the Ashfield Shale. This suggests that groundwater levels at this location have been drawdown due to the M4 East infrastructure (Substation and potentially the tunnels).

Figure 4-9 shows the interpreted groundwater level between the station box and M4 East infrastructure in the vicinity of the pedestrian adit, based on monitoring from piezometers both prior to M4 East construction and in more recent years. Assuming a pre-M4 East watertable as shown in the figure, the drawdown induced by the M4 East tunnels/Substation No. 5 would be up to 12 m at the location of the M4 East tunnels, and up to 9 m at the location of the proposed Burwood North Station shaft. This drawdown reduces to negligible some 130 m from the M4 East tunnels/Substation No. 5 (hydraulic gradient is approximately 0.09 between the station box and the M4 East tunnels/Substation No. 5).

The drawdown predicted by CSJH (2016) at the location of the shaft is approximately 30 m. This suggests that either the model developed by CSJH (2016) overestimates drawdown due to the M4 East structures significantly, or the full drawdown is yet to be realised.

The Cintra decline is a drained open trough ramp structure which converts to a single span mined decline tunnel. It lies between approximately 230 m and 340 m east of the station box, between Luke Avenue and the Wests Tigers Rugby League Club west of Lucas Road. Its location is shown in Figure 4-6.

It is expected that the Cintra decline has caused groundwater level drawdown in its vicinity. There are no monitoring data available in the vicinity of the decline to confirm this.

Available monitoring data are insufficient to assess whether groundwater level drawdown due to the M4 East infrastructure is ongoing, or has reached approximately steady state (i.e., approximately maximum drawdown expected to occur due to M4 East infrastructure).

The groundwater level drawdown due to the M4 East drained structures that was predicted by CSJH (2016) is indicated to be greater than any possible drawdown that has been experienced in the vicinity of the station site based on available monitoring data.

However, given that the M4 East excavations were complete in December 2018, it is considered likely that M4 East infrastructure has already caused most of the drawdown that they are likely to induce.

In the absence of confirmatory data, it has been assumed for the purposes of this assessment that the M4 East infrastructure have already induced the maximum drawdown they are likely to.

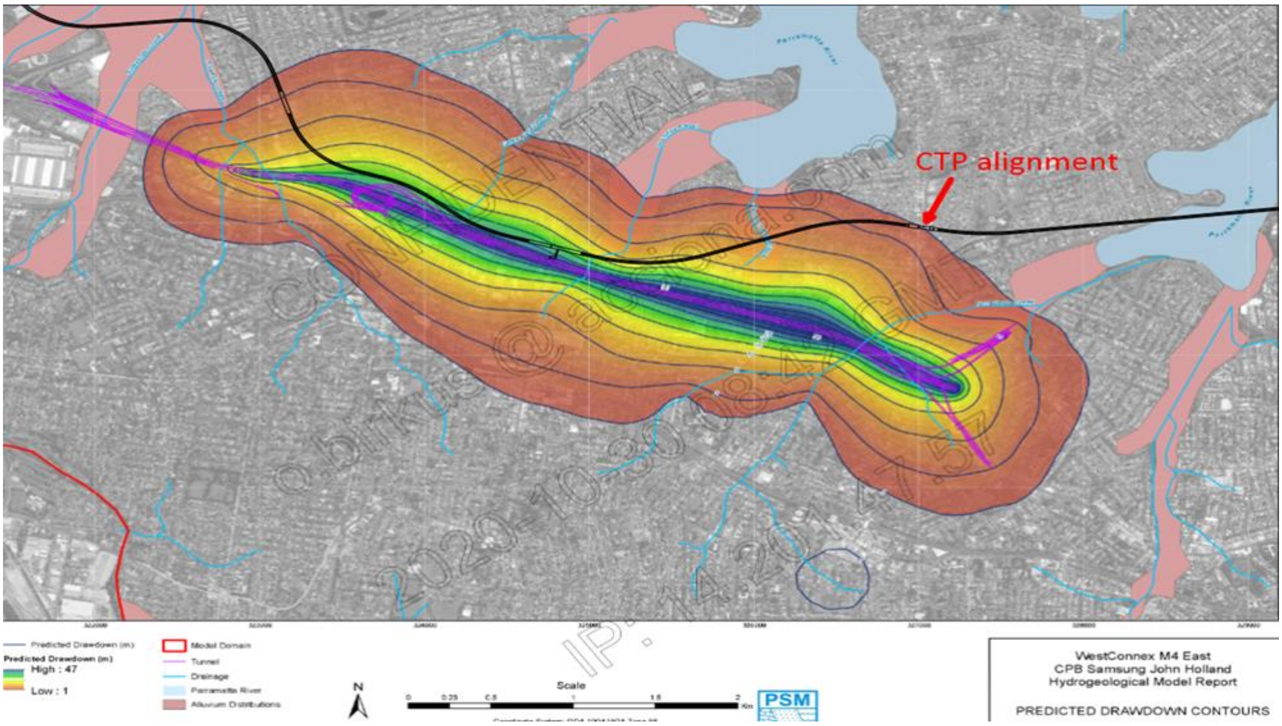


FIGURE 4-7: MODELLED GROUNDWATER DRAWDOWN FROM M4 EAST TUNNEL (FIGURE FROM CSJH, 2016)

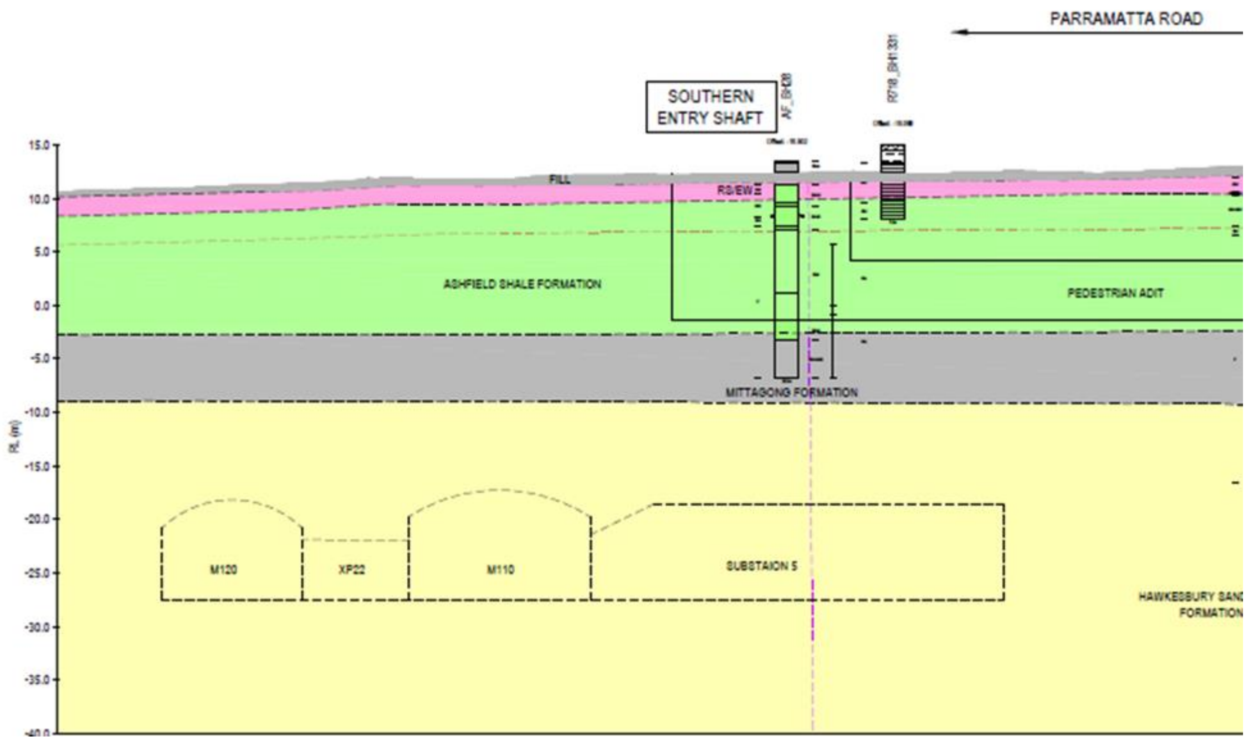


FIGURE 4-8: APPROXIMATE DEPTH OF BURWOOD NORTH STATION SHAFT AND M4 EAST TUNNELS AND SUBSTATION NO. 5

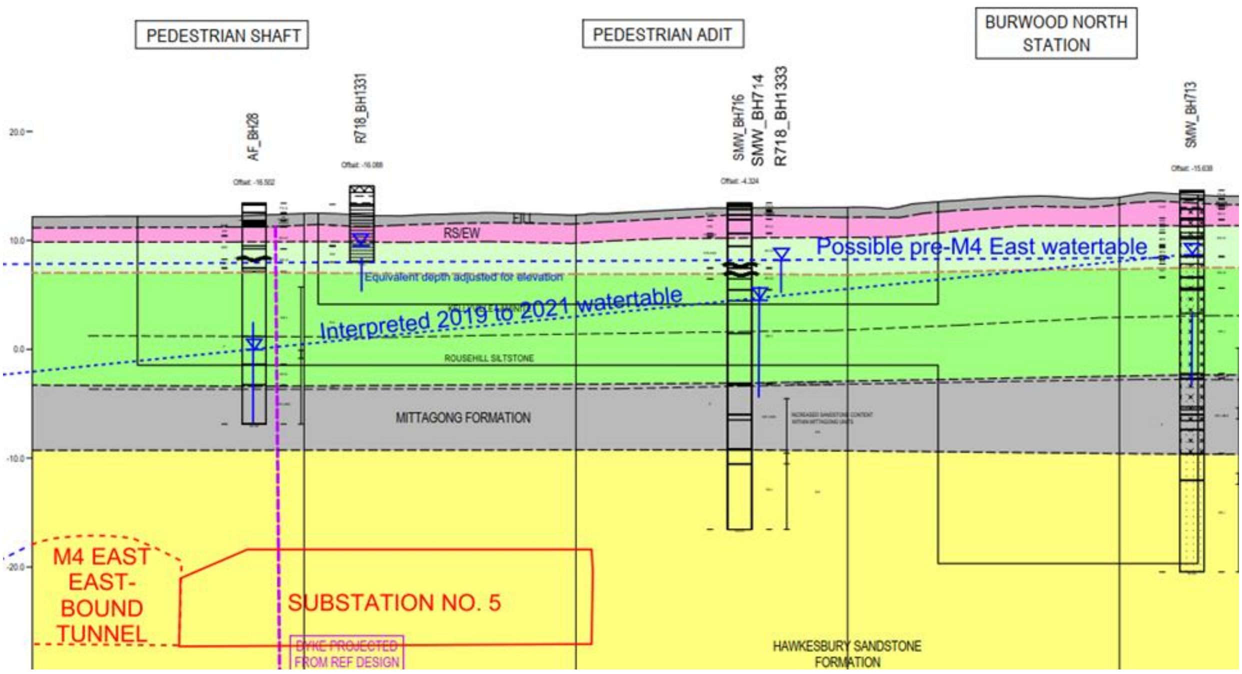


FIGURE 4-9: INTERPRETED GROUNDWATER LEVELS BETWEEN BURWOOD NORTH STATION AND M4 EAST TUNNELS/SUBSTATION NO. 5

4.4 HYDROGEOLOGICAL PARAMETERS

Annexure B provides discussion of hydrogeological units and associated model parameter values adopted for this assessment.

At the Burwood North Station site specifically, packer tests were undertaken in eight boreholes at the site, as listed in Table 4. In addition to the SMW boreholes, an additional borehole (AF_BH28) was drilled by AFJV in November 2021.

The tests are all within the Mittagong Formation/Hawkesbury Sandstone, with the exception of one test in the Ashfield Shale and two tests in the Ashfield Shale and Mittagong Formation.

Figure 4-10 shows the packer test results with depth below ground surface.

Packer test results range between less than 0.1 Lugeon (<8.6×10-4 m/day) and 0.9 Lugeons (7.8×10 3 m/day). The median and average values of all the data are 0.1 and 0.3 Lugeons, respectively. There is no consistent trend with depth, with the exception that maximum Lugeon values appear to progressively reduce below approximately 30 m below ground surface.

These test results and trends at the station site are consistent with the results of tests along the alignment (see Annexure B), and the conditions and parameter values discussed in Annexure B are considered representative of conditions at the site.

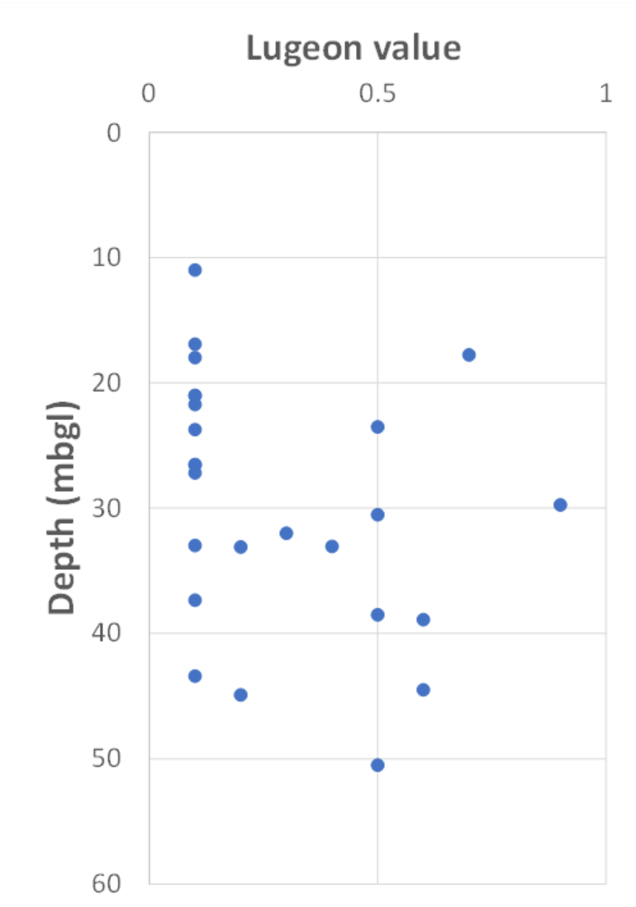


FIGURE 4-10: LUGEON VALUES WITH DEPTH BELOW GROUND SURFACE

TABLE 4-4: SUMMARY OF PERMEABILITY TEST RESULTS AT BURWOOD NORTH STATION SITE

Bore ID	Depth Top	Depth Bottom	Unit(s)	Result	
	m	m		Lugeon value	m/day
SMW_BH044	31	34.93	MIT	<0.1	<8.6×10^{-4}
SMW_BH044	34.7	40	HAW	<0.1	<8.6×10^{-4}
SMW_BH044	39.8	47	HAW	<0.1	<8.6×10^{-4}

Bore ID	Depth Top	Depth Bottom	Unit(s)	Result	
	m	m		Lugeon value	m/day
SMW_BH046	15	20.95	HAW	<0.1	<8.6×10^{-4}
SMW_BH046	20.5	26.92	HAW	<0.1	<8.6×10^{-4}
SMW_BH046	26.5	32.95	HAW	0.9	7.8×10^{-3}
SMW_BH712	29.85	36.35	MIT	0.2	1.7×10^{-3}
SMW_BH712	35.35	42.45	HAW	0.6	5.2×10^{-3}
SMW_BH712	41.35	48.45	HAW	0.2	1.7×10^{-3}
SMW_BH712	47.45	55.5	HAW	<0.1	<8.6×10^{-4}
SMW_BH713	14.5	21	ASH and MIT	0.7	6.1×10^{-3}
SMW_BH713	20	27	MIT and HAW	0.5	4.3×10^{-3}
SMW_BH713	26	35.05	HAW	0.5	4.3×10^{-3}
SMW_BH714	18.7	24.7	HAW	<0.1	<8.6×10^{-4}
SMW_BH714	23.7	30.65	HAW	<0.1	<8.6×10^{-4}
SMW_BH715	18	24	MIT and HAW	<0.1	<8.6×10^{-4}
SMW_BH715	23	30.08	HAW	<0.1	<8.6×10^{-4}
SMW_BH715	29	35	HAW	0.3	2.6×10^{-3}
SMW_BH716	18	24	MIT and HAW	<0.1	<8.6×10^{-4}
SMW_BH716	23	30	HAW	<0.1	<8.6×10^{-4}
SMW_BH723	30	36.1	MIT and HAW	0.4	3.5×10^{-3}
SMW_BH723	35	42	HAW	0.5	4.3×10^{-3}
SMW_BH723	41	48	HAW	0.6	5.2×10^{-3}
SMW_BH723	47	54	HAW	0.5	4.3×10^{-3}
SMW_BH723	53	60	HAW	0.4	3.5×10^{-3}

Notes: ASH means Ashfield Shale, MIT means Mittagong Formation, HAW means Hawkesbury Sandstone

5. DESIGN GROUNDWATER LEVELS

5.1 REQUIREMENTS

Design groundwater levels have been developed considering, and consistent with, the Particular Specifications, as listed in Table 5-1.

TABLE 5-1 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]

5.2 CTP WORK CONDITIONS

The Burwood North 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.

5.3 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).

Annexure C presents the groundwater load condition for the burst water mains scenario in concept, as well as an assessment of the potential groundwater pressures experienced by the retaining wall under the blocked drainage scenario. These load conditions have been adopted in the retaining wall design.

6. GROUNDWATER QUALITY

As noted in the Contamination Assessment Report, sampling and analysis of groundwater quality at Burwood North Station occurred in SMW_BH series piezometers in September 2018 and March 2021. A project-wide Groundwater Monitoring Event (GME) undertaken by Golder/Douglas in May 2021.

There are also historical data from 2014 for a nearby project in the vicinity of the site (three piezometers at reference site R718).

Groundwater was typically of a slightly acidic pH (6.0 to 7.7), with electrical conductivity (EC) recorded between 105 µS/cm and 13,300 µS/cm, and concentrations for various metals and inorganics (nitrate and ammonia) above the default ecological guideline of freshwater 95% species protection (slightly – moderately disturbed) as stipulated in ANZECC/ARMCANZ (2000). PFAS was detected during the groundwater sampling in two piezometers. However, no exceedances of the adopted human health criteria were reported.

Available data indicate that:

- Groundwater in the clay residual soils and/or Ashfield Shale could have a pH that is lower than the ANZG (2000) guideline trigger level of between 6.5 and 8.5 (lowland rivers) for 95% protection of freshwater aquatic ecosystems
- Groundwater in the Ashfield Shale could have a salinity that exceeds the ANZG (2000) guideline trigger level of 2,200 µS/cm (lowland rivers) for 95% protection of freshwater aquatic ecosystems
- Groundwater is likely to exceed the ANZG (2000) guideline trigger level for 95% Protection of freshwater aquatic ecosystems for some metals (cadmium, chromium, copper and zinc).

Based on the above, groundwater seepage is likely to require dilution or treatment prior to discharge to surface waters, if disposal to surface waters is to be considered.

7. GROUNDWATER INFLOW AND DRAWDOWN

7.1 OVERVIEW

Numerical groundwater flow modelling was undertaken to predict potential groundwater inflow rates into the Burwood North Station excavations, and the associated propagation of groundwater level drawdown.

Two-dimensional (cross sectional) models were developed using Geoslope's Geostudio SEEP/W, a finite difference modelling software package for modelling groundwater flow in porous media.

Details of the modelling are covered in Annexure D.

It is understood that the station box will be excavated prior to the shaft and cavern (August 2022 to January 2023). The cavern will be excavated concurrently with the shaft and adit (January 2023 to October 2023). Waterproofing of the adit and cavern will be complete by July 2024. Note that the tunnels and nozzles will be undrained (waterproofed).

7.2 MODEL LAYERS

Two hydrogeological units were represented in the model: Ashfield Shale and Hawkesbury Sandstone. Fill and residual soil units were not included in the model because the water table is generally situated below these units at the station. The Mittagong Formation was not explicitly represented in the model and is instead represented by the Hawkesbury Sandstone unit. This approach was adopted because the Mittagong Formation is relatively thin and has hydrogeological properties characteristically similar to the Hawkesbury Sandstone.

7.3 ADOPTED HYDROGEOLOGICAL PARAMETER VALUES FOR MODELLING

Hydrogeological parameter values adopted for the modelling were as per the adopted representative values outlined in Annexure D and Section 4.4.

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

7.4 GROUNDWATER INFLOWS

7.4.1 INFLOW RATES

Predicted groundwater inflow rates to the station box and cavern excavations are shown in Figure 7-1. Predicted inflows to the station box excavation are up to approximately 0.85 L/s (70 m^3/d) and to the cavern are up to 0.55 L/s (50 m^3/d). The long-term inflows are predicted to be approximately 0.4 L/s (35 m^3/d) to the station box excavation and 0.3 L/s (25 m^3/d) to the cavern excavation for the adopted hydrogeological conditions.

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.

Because the station box excavation underdrains the shaft, and the shaft is much smaller in footprint than the station box, inflows to the shaft and pedestrian adit are expected to be minor to negligible relative to those to the station box.

Table 6 summarises the predicted inflows to the excavations and the inflow limits as specified in the Particular Specification.

The Particular Specification requires that groundwater inflows to the Burwood North Station excavation do not exceed 114,000 litres in a 24 hour period (1.32 L/s) and inflows to the shaft excavation do not exceed 16,000 litres in a 24 hour period (0.19 L/s). Based on the modelling undertaken, unmitigated groundwater inflows are likely to meet this requirement.

Particular Specification 4.1.8(h)(ii) states that groundwater seepage must not exceed 15,000 litres in any 24-hour period, measured over any square with an area of 10 m^2. Inflows over any given 10 m² area of excavation face will depend on the water-bearing features encountered during excavation. Should local features be encountered that exceed the inflow limit, localised grouting of the features will be required.

Note that these inflows consider the presence of the existing Westconnex drained structures to the south (the M4 East tunnels and Substation No. 5) and assume that these existing structures have already incurred the maximum groundwater level drawdown that would occur due to these structures in the vicinity of the station. In the case that drawdown due to Westconnex drained structures is ongoing (continues from late 2021 to the time when the station excavation is undertaken), it is possible that inflows to the CTP excavations may be lower than the rates predicted above (because the groundwater heads will potentially be lower than was assumed for the inflows reported above).

It is possible that localised higher permeability is associated with possible geological features that may intersect the station excavations. As shown in the Geotechnical Interpretive Report, the Reference Design infer the presence of a potential fault zone at approximately the interface of the cavern and the station box excavations; and the presence of two potential dykes, one intersecting the western end of the cavern, and one intersecting the eastern end of the station box; and a joint swarm has been identified within the Ashfield Shale some 10 metres above the cavern crown.

Available packer test data does not indicate that the rock is of high permeability. However, it is possible that these features may act as conduits for groundwater flow. In such case, groundwater inflows to the station excavations may be greater than those predicted here.

There is a possibility that the hydraulic conductivity of rock may be relatively high in the vicinity of fault zones, dykes, joint swarms, or in other unidentified geological features. Should this be the case, groundwater inflows to the excavations may be higher than those predicted here.

Where groundwater inflows are greater than predicted here, or in the vicinity of significant water-bearing geological features, localised grouting of the rock may be required during excavation.

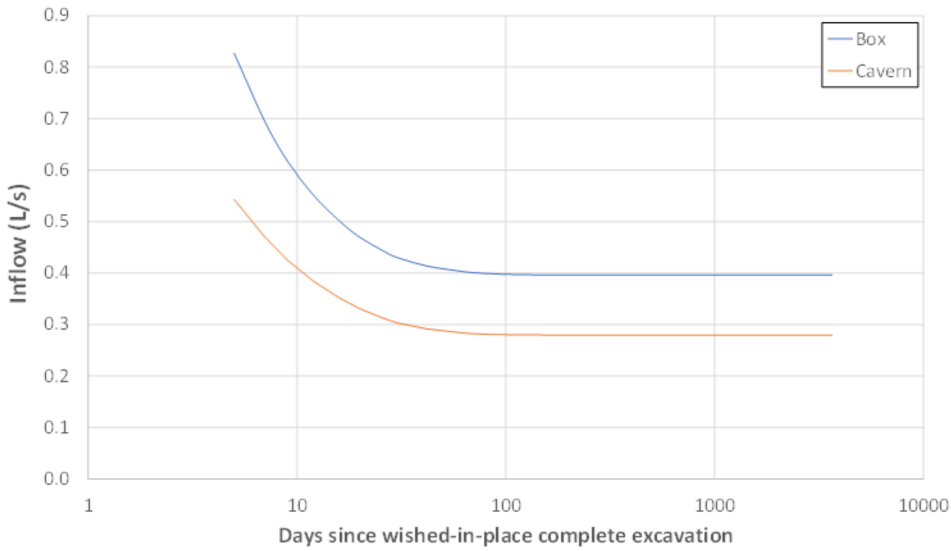


FIGURE 7-1: PREDICTED GROUNDWATER INFLOW RATES

TABLE 7-1: SUMMARY OF GROUNDWATER INFLOWS ESTIMATED BY MODELLING AND INFLOW LIMITS IN THE PARTICULAR SPECIFICATION

Particular Specification Clause	Element	Maximum allowable inflow rate nominated in the Particular Specification (m^3/d)	Predicted groundwater inflow rate (m^3/d)
4.1.8(h)(ii)(C)	Station box excavation	114	Up to 70
4.1.8(h)(ii)(D)	Shaft excavation	16	Negligible to minor (less than 16)
4.1.8(h)(i)	Any square with an area of 10 m^2, at any and all locations within the sides and bases of the shafts and excavations	15 over any 10 m^2 area of excavation face	Not modelled. Inflows over any given 10 m^2 area will be dependent on water-bearing features encountered during excavation and will require localised grouting during excavation should inflows exceed criteria

7.4.2 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 7. The cumulative inflow calculated by the model is less than the inflow predicted in the EIS (Jacobs, 2020).

7.5 GROUNDWATER LEVEL DRAWDOWN

As noted above, it is not known whether groundwater level drawdown associated with the M4 infrastructure is at (near) steady state or ongoing. For this reason, two different models were used to predict the potential drawdown associated with the station excavations.

The modelled initial watertable (considered representative of the interpreted groundwater levels in early 2022 – refer to Figure 4-9) and the predicted long-term watertable (two years after wished in place excavation) surfaces are shown in Figure 7-2 and Figure 7-3.

Figure 7-2 shows the predicted long-term watertable assuming that the M4 East infrastructure is at steady state prior to the CTP works (i.e., no further drawdown occurs due to the M4 East infrastructure), and Figure 7-3 shows the predicted long-term watertable assuming that drawdown due to the M4 East infrastructure is ongoing during CTP works.

The predicted long-term watertable at 10 years after excavation is the same as the predicted long-term watertable at two years after excavation (i.e., near-steady state conditions have been reached after two years).

Figure 7-4 shows the predicted drawdown of the watertable (based on the initial and long-term groundwater levels shown in Figure 7-2 and Figure 7-3) for the (i) the case that the M4 East infrastructure is at steady state prior to the CTP works, (ii) the case that drawdown due to the M4 East infrastructure is ongoing during CTP works, and (iii) reported in the EIS.

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

The predicted drawdown due to CTP excavation works is less than that predicted in the EIS for the case that the M4 East infrastructure is at steady state prior to the CTP works. However, for the case that drawdown due to the M4 East infrastructure is ongoing during CTP works, the predicted drawdown is lesser in extent than the drawdown predicted by the EIS but exceeds the drawdown magnitude predicted by the EIS.

This is because the drawdown predicted quantitatively in the EIS is for the Sydney Metro project works only. Cumulative impacts due to other projects (such as M4 East) are discussed in the EIS but are not assessed quantitatively.

As noted in Section 4.1.2 a palaeochannel is interpreted to lie to the east of the station site, surrounding St Lukes Canal. The nature, extent and depth of the alluvium is unconfirmed. The presence and level of groundwater within the alluvium is also unknown.

Groundwater modelling (see Annexure D) indicates that the alluvium could be partially desaturated across much of its extent due to the CTP excavation works. The predicted watertable before CTP excavation works and two to ten years after CTP excavation works is shown in Figure 7-5. This assessment is moderately conservative because it:

- Considers low rainfall recharge conditions
- Assumes that the alluvium contains groundwater, whereas it is considered likely that the presence of groundwater within the alluvium is ephemeral
- Assumes that the Cintra Decline and M4 East tunnels have not already caused drawdown in the alluvium, which is likely
- Assumes that the watertable is relatively shallow in the alluvium.

The watertable drawdown predicted by this moderately conservative modelling has been used to assess potential ground movement/settlement in the vicinity of the alluvium, which shows that ground movement criteria are not exceeded under these conditions.

It should be noted that there is significant uncertainty in the predicted groundwater level drawdown in the alluvium. The nature, extent and depth of the alluvium is unknown. Numerous assumptions have been made with regard to hydrogeological conditions. In the case that actual conditions vary from those adopted in this modelling, groundwater levels and drawdown may be significantly different to those predicted here.

Further, there is a possibility that unidentified water-bearing geological features may be present in the vicinity of the station excavations, and these may have higher permeability than the hydrogeological conditions adopted in this assessment. Should this be the case, groundwater level drawdown could propagate further from the station compared to the drawdown predicted in this assessment.

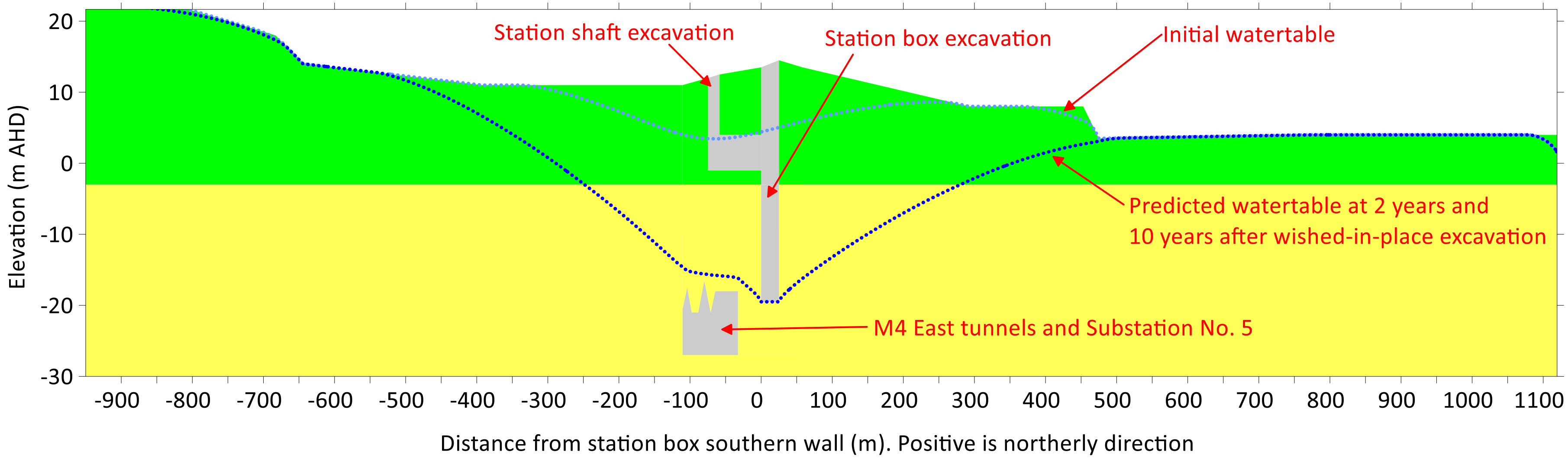


FIGURE 7-2: INTERPRETED PRE-CTP-WORKS EXCAVATION (INITIAL) WATERTABLE AND PREDICTED WATERTABLE AFTER (2 AND 10 YEARS SINCE) EXCAVATION, WITH SCENARIO IN WHICH DRAWDOWN AT M4 EAST INFRASTRUCTURE REACHED STEADY STATE PRIOR TO 2022

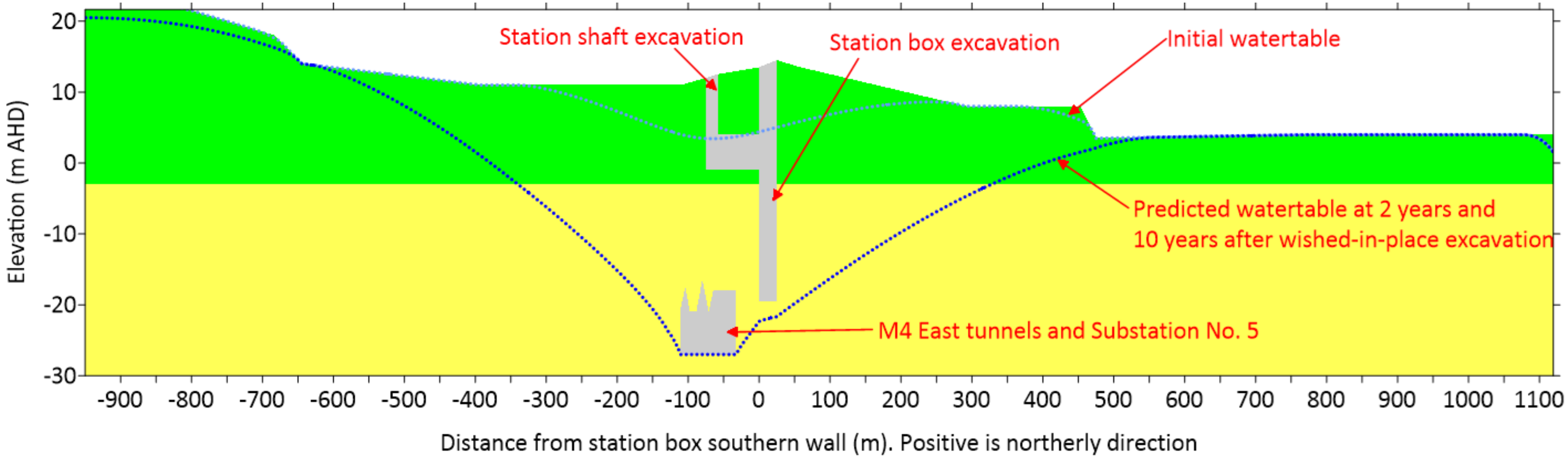


FIGURE 7-3: INTERPRETED PRE-CTP-WORKS EXCAVATION (INITIAL) WATERTABLE AND PREDICTED WATERTABLE AFTER (2 AND 10 YEARS SINCE) EXCAVATION, WITH SCENARIO IN WHICH DRAWDOWN AT M4 EAST INFRASTRUCTURE IS ONGOING IN 2022

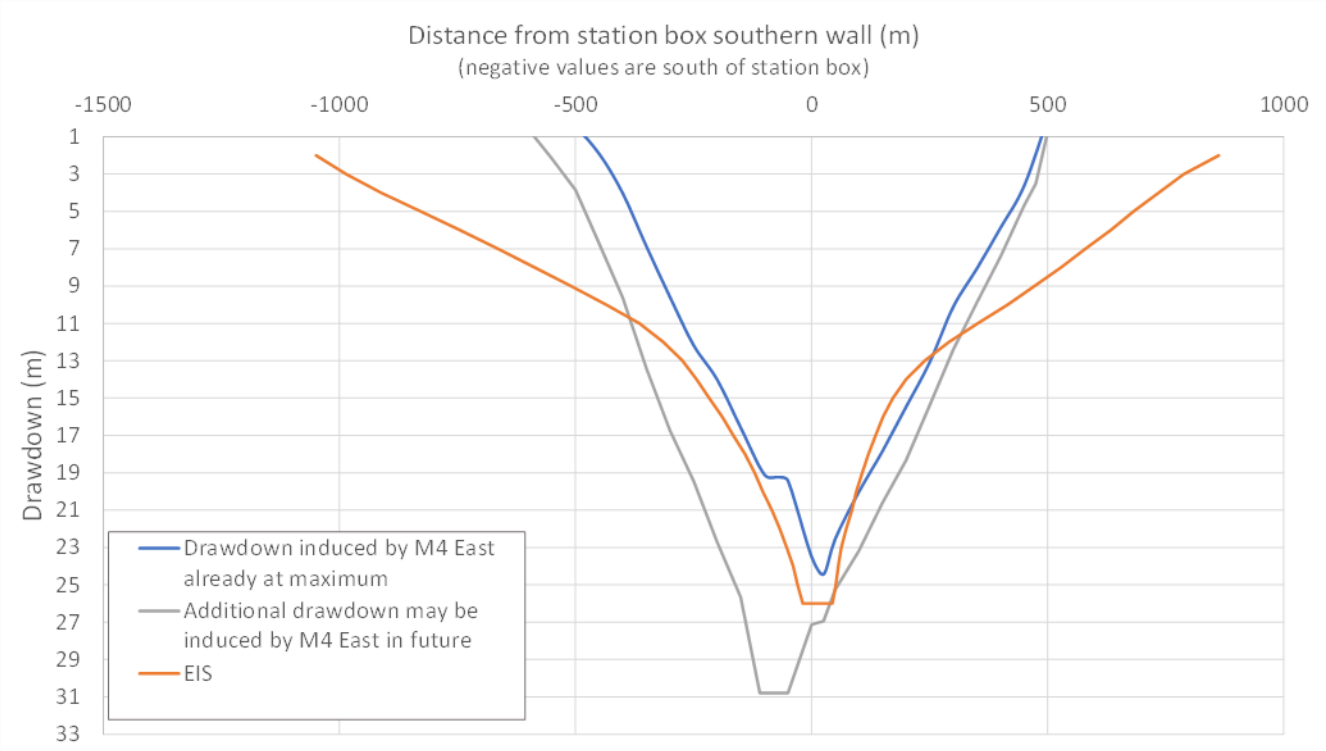


FIGURE 7-4: PREDICTED DRAWDOWN OF THE WATERTABLE AND DRAWDOWN PREDICTED IN THE EIS

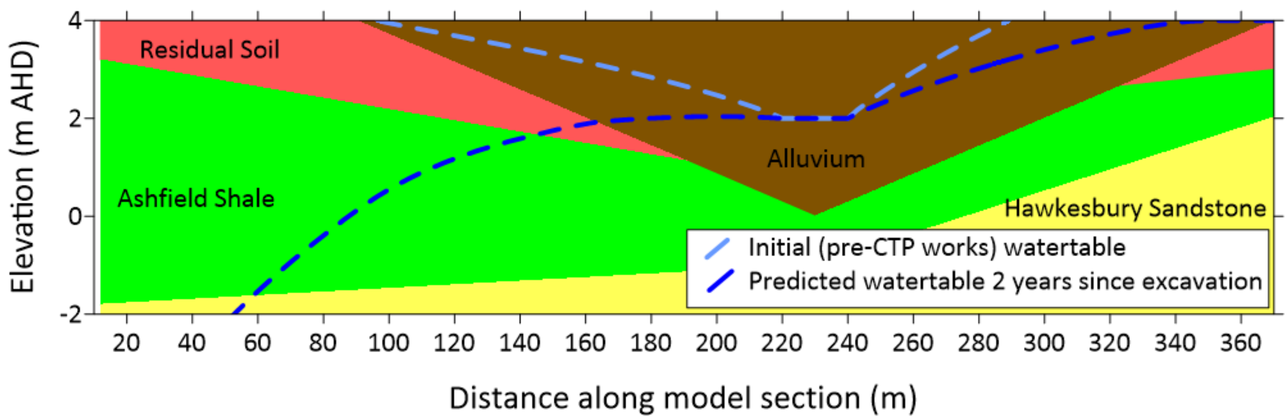


FIGURE 7-5: INTERPRETED PRE-CTP-WORKS EXCAVATION (INITIAL) WATERTABLE AND PREDICTED WATERTABLE AFTER (2 AND 10 YEARS SINCE) EXCAVATION IN ALLUVIUM TO THE EAST OF THE STATION

8. GROUNDWATER-RELATED IMPACTS

8.1 GROUNDWATER USERS AND RECEPTORS

The EIS identifies a potential groundwater dependent ecosystem (terrestrial vegetation) approximately 450 m north-west of Burwood North metro station (Turpentine – Grey Ironbark open forest on shale in the lower Blue Mountains, Sydney Basin Bioregion at Queen Elizabeth Park) (Jacobs, 2020). This ecosystem is shown in Figure 8-1, which has been reproduced from the EIS.

As noted in the EIS, this ecosystem grows on Wianamatta Shale and the rootzone is likely to lie within residual clay soils of the shale and/or the shale itself (where shallow). These geological units are likely to be of relatively low permeability, with a potential perched watertable present (which may be temporary) upon which these groundwater dependent ecosystems may intermittently rely. The groundwater level drawdown in the rock, induced by station excavation, is not likely to cause direct groundwater level drawdown in a potentially ephemeral perched aquifer that potentially lies within the shallow clay or shale. Based on this, the likelihood of this ecosystem being impacted by the groundwater level drawdown associated with CTP works is considered to be low.

It should also be noted that the modelled long-term groundwater level drawdown associated with the M4 East tunnels, as illustrated in Figure 4-7 (CSJH, 2016), is significant in the vicinity of this ecosystem. It is possible that, if this ecosystem has been impacted due to groundwater level drawdown, the cause may be the construction and operation of the M4 East tunnels.

Twelve WaterNSW-registered bores were identified within the predicted extent of groundwater level drawdown. WaterNSW reports the purpose of these as monitoring bores; with the exception of one bore, which is reported as a dewatering bore. As such, the viability of groundwater supply bores (WaterNSW-registered groundwater bore users) is not expected to be impacted by the project.

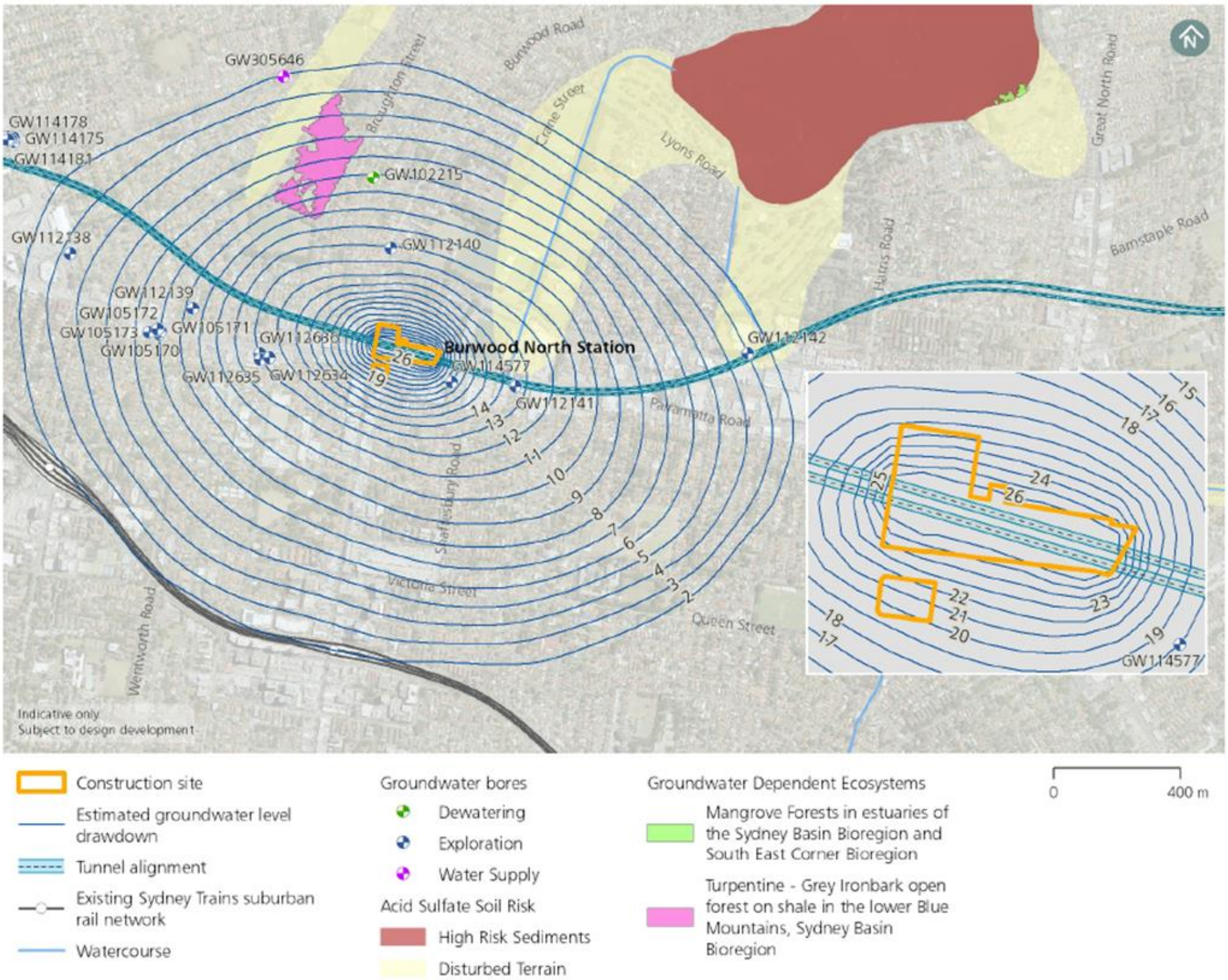


FIGURE 8-1: GROUNDWATER RECEPTORS NEAR BURWOOD NORTH STATION AND PREDICTED GROUNDWATER LEVEL DRAWDOWN FROM THE EIS (JACOBS, 2020)

8.2 ACID SULFATE SOILS

As noted in the Contamination Assessment Report, potential acid sulfate soils (PASS) have been identified approximately 65 m east of the station box. It is possible that (P)ASS are associated with the alluvial palaeochannel to the east of the site. However, available data shows mixed results and there is therefore uncertainty regarding the presence of (P)ASS within the area.

As noted in the Contamination Assessment Report, based on the Burwood Local Environmental Plan 2012 and Canada Bay Local Environmental Plan 2013, Class 2 acid sulfate soils land is located approximately 250 m north-east of the station box. Available data for these soils indicate that they have the potential to be classified as Acid Sulfate Soils (ASS). A review of the previous site investigation results suggests that in general the likelihood of ASS occurrence throughout this Class 2 acid sulfate soil land is low, and the ASS is likely limited to localised areas. However, no comprehensive ASS investigation has been undertaken (or was available for review) to confirm presence and extent.

As noted in the Contamination Assessment Report, Detailed Site Investigations are proposed to further assess the risks associated with (P)ASS.

Potential impacts to (P)ASS associated with groundwater level drawdown would be addressed though development of a specific Acid Sulfate Soils Management Plan (ASSMP).

FIGURE 8-2: PEROXIDE OXIDISABLE SULFUR EXCEEDING 0.03% S (IN RED) AND BELOW 0.03% S (IN YELLOW) AND PREDICTED WATERTABLE DRAWDOWN (METRES) FOR THE MITIGATED BASE CASE SCENARIO (DECEMBER 2024)

8.3 SETTLEMENT

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

The potential watertable drawdown in the alluvium to the east of the station (see Section 7.5 above), and the drawdown in the rock around the station, have been used to assess potential ground settlement in the vicinity of the station.

Based on a moderately conservative assessment (see Section 7.5), the alluvium could be partially desaturated across much of its extent due to the CTP excavation works. This drawdown been used to assess potential ground movement/settlement in the vicinity of the alluvium, which shows that ground movement criteria are not exceeded under these conditions.

8.4 CONTAMINATION

As noted in the Contamination Assessment Report, groundwater exceedances above the 95% species protection for freshwater were identified across most groundwater samples collected between 2015 and 2021. The contaminants included organics (nitrate and ammonia) and heavy metals.

Groundwater seepage to the CTP excavations is therefore likely to require dilution or treatment prior to discharge to surface waters. Evaporation of seepage (during drier periods) from within the station box excavation may reduce the total inflows required to be discharged.

These risks are expected to be manageable through the implementation of appropriate management measures outlined in a Construction Environmental Management Plan.

There are potential contamination risks associated with locations beyond the site. The Burwood North Station excavations will act as groundwater sinks. It is therefore possible that contaminated groundwater at distance from the site will be drawn towards the excavation. The risk of contaminated groundwater at distance from the site is unknown.

As noted in the Contamination Assessment Report, given the multiple commercial activities along Parramatta Road that are in proximity to the Burwood North Station site, it is possible that groundwater beyond the site footprint is contaminated, and this contaminated groundwater could be drawn into the station box excavation. It is possible that contaminated groundwater (if present) may already have migrated from distant sites towards the M4 East tunnels.

It is understood that further investigation of commercial activities surrounding the Burwood North Station development would be undertaken as part of the detailed environmental site investigation.

8.5 SALINE INTRUSION

As noted in the Contamination Assessment Report, groundwater exceedances above the 95% species protection for freshwater were identified across most groundwater samples collected between 2015 and 2021. The contaminants included organics (nitrate and ammonia) and heavy metals.

Groundwater seepage to the CTP excavations is therefore likely to require dilution or treatment prior to discharge to surface waters. Evaporation of seepage (during drier periods) from within the station box excavation may reduce the total inflows required to be discharged.

These risks are expected to be manageable through the implementation of appropriate management measures outlined in a Construction Environmental Management Plan.

There are potential contamination risks associated with locations beyond the site. The Burwood North Station excavations will act as groundwater sinks. It is therefore possible that contaminated groundwater at distance from the site will be drawn towards the excavation. The risk of contaminated groundwater at distance from the site is unknown.

As noted in the Contamination Assessment Report, given the multiple commercial activities along Parramatta Road that are in proximity to the Burwood North Station site, it is possible that groundwater beyond the site footprint is contaminated, and this contaminated groundwater could be drawn into the station box excavation. It is possible that contaminated groundwater (if present) may already have migrated from distant sites towards the M4 East tunnels.

It is understood that further investigation of commercial activities surrounding the Burwood North Station development would be undertaken as part of the detailed environmental site investigation.

8.6 ST LUKES CANAL

St Lukes Canal is a lined channel located approximately 240 m east of the station box, as shown in Figure 4-2.

It is understood that City of Canada Bay Council source water from St Lukes Canal for irrigation.

The predicted groundwater level drawdown due to CTP works extends across the alluvium at which St Lukes Canal is located.

It is not known whether groundwater contributes to canal stream flows significantly. In the case that groundwater in the alluvium around the canal contributes to stream flows significantly, it is possible that the CTP works could reduce the groundwater baseflow contribution to stream flows. However, it is possible that the M4 East tunnels and Cintra decline have already caused groundwater level drawdown in the vicinity of the canal, reducing groundwater contributions to canal streamflow. It is also possible that groundwater baseflow to the canal is ephemeral, with contributions to streamflow following periods of significant rainfall, but negligible contributions during drier periods. In such a case, it is unlikely that the CTP works would have any significant impact on groundwater baseflow contributions to canal streamflows.

However, hydrogeological conditions in the vicinity of the canal, including the nature, extent, depth, and presence of groundwater in the alluvium, are unknown. Given these unknowns, to address the risk of CTP works impacting canal streamflows, groundwater level monitoring in the vicinity of St Lukes Canal, and monitoring (gauging) of streamflows in the canal would be required both before, during and following the CTP excavation works. Groundwater monitoring piezometers are nominated in Table 9 1 to address this.

8.7 CUMULATIVE IMPACTS

The M4 East infrastructure have the potential to cause (cumulative) additional drawdown to the CTP works alone.

The groundwater level drawdown due to the M4 East drained structures that was predicted by CSJH (2016) is indicated to be significant. Monitoring data are insufficient to assess the influence of the M4 East infrastructure on groundwater levels with confidence. Greater confidence may be obtained if groundwater level monitoring data for the M4 East project is made available to JTJV.

As noted in Section 4.3.3, based on limited available data, the interpreted groundwater level drawdown between the station box and M4 East infrastructure is significantly lower than the drawdown predicted by CSJH (2016). This suggests that either the model developed by CSJH (2016) overestimates drawdown due to the M4 East structures significantly, or the full drawdown is yet to be realised.

If the full influence of the M4 East infrastructure is yet to be realised in the vicinity of the Burwood North Station site, then it is possible that drawdown greater than predicted (see Section 7.5) could occur. In such as case, potential impacts associated with ground movement/settlement, contaminant migration and activation of acid sulfate soils may exceed the impacts as assessed in this report. However, that the M4 East excavations were complete in December 2018, it is considered likely that M4 East infrastructure has already caused most of the drawdown that they are likely to induce.

As such, the potential impacts due to CTP works are likely to be less than those predicted here (which are minor to negligible), because drawdown in excess of that predicted due to CTP works is likely to have already occurred due to the M4 East infrastructure.

9. CONSTRUCTION PHASE MONITORING

Figure 9-1 shows the groundwater level monitoring locations during construction phase. Table 9 1 lists the details of the monitoring piezometers. Table 9 2 lists the existing groundwater levels based on available monitoring data, and the predicted watertable drawdown from which trigger thresholds are developed for the Instrumentation and Monitoring.

The locations have been selected based on consideration of predicted groundwater level drawdown, the locations of assets sensitive to ground settlement, potential groundwater quality-related issues, and monitoring of potential cumulative groundwater level drawdown due to CTP works and the M4 East project. Proposed locations have not checked for access / approval / services conflicts.

It is assumed that the existing piezometers listed are accessible and in suitable working order. Note that Sydney Metro has not confirmed which piezometers are operable/decommissioned/destroyed (with the exception of the SMW_BH700 series). In the event that the existing piezometers listed are inaccessible or destroyed, alternative monitoring locations may be required.

It should also be noted that existing piezometers and open boreholes that would be intersected by the TBM could act as a conduit for groundwater flow into the tunnel excavation. Existing piezometers, open boreholes, or any other in-ground structure that could act as a conduit for groundwater flow into the tunnel excavation, should be grouted prior to TBM mining.

To address the potential risk of CTP works impacting St Lukes Canal streamflows, groundwater level monitoring in the vicinity of St Lukes Canal, and monitoring (gauging) of streamflows in the canal would be required both before, during and following the CTP excavation works. Groundwater monitoring piezometers are nominated in Table 7 to address this.

Note that pre-excavation groundwater level monitoring will be required at new monitoring locations to obtain baseline data.

TABLE 9-1: SUMMARY OF RECOMMENDED CONSTRUCTION PHASE MONITORING LOCATIONS

Location ID	Purpose	Piezometer / VWP	Existing/proposed monitoring location	Easting	Northing	Ground surface (m AHD)	Total borehole depth (m bgl)	Depth to monitoring horizon (top) (mbgl)	Depth to monitoring horizon (bottom) (mbgl)	Screened geology
R718_BH1326	Potential check dewatering consistent with expectation	Piezometer	Existing	324447	6250779	Unknown	Unknown	Unknown	Unknown	ASH
R718_BH1331	Potential check dewatering consistent with expectation	Piezometer	Existing	324785	6250750	15	Unknown	Unknown	Unknown	ASH
R718_BH1333	Potential check dewatering consistent with expectation	Piezometer	Existing	324876	6250760	11	Unknown	Unknown	Unknown	ASH
R718_BH1336	Potential check dewatering consistent with expectation	Piezometer	Existing	325021	6250714	Unknown	Unknown	Unknown	Unknown	RS (clay)
SMW_BH044_w	Check dewatering consistent with expectation	Piezometer	Existing	324682	6250908	22.67	47	20.5	36.5	HAW
SMW_BH046_w	Check dewatering consistent with expectation	Piezometer	Existing	324993	6250787	6.47	32.95	4	16	MIT and HAW
SMW_BH714_s	Check dewatering consistent with expectation	Piezometer	Existing	324901	6250753	8.23	7.2	1.6	7.2	ASH
SMW_BH714_w	Check dewatering consistent with expectation	Piezometer	Existing	324901	6250753	8.23	30.65	8.08	17.5	ASH and HAW
SMW_BH715_s	Check dewatering consistent with expectation	Piezometer	Existing	324865	6250817	12.09	7	1.6	7	RS (clay) and ASH

Location ID	Purpose	Piezometer / VWP	Existing/proposed monitoring location	Easting	Northing	Ground surface (m AHD)	Total borehole depth (m bgl)	Depth to monitoring horizon (top) (mbgl)	Depth to monitoring horizon (bottom) (mbgl)	Screened geology
SMW_BH715_w	Check dewatering consistent with expectation	Piezometer	Existing	324865	6250817	12.09	35	5	10.5	ASH
I1	Interface with M4 East Substation No. 5	Piezometer	Proposed	324777	6250723	~13	30	25	30	ASH and HAW
SLCM1	Impact on St Lukes Canal	Piezometer	Proposed	325191	6250748	~4 (TBC)	~5 (TBC)	~1 (TBC)	~5 (TBC)	Alluvium
SLCM2	Impact on St Lukes Canal	Piezometer	Proposed	325236	6250890	~4 (TBC)	~5 (TBC)	~1 (TBC)	~5 (TBC)	Alluvium

Notes: Dry means piezometer is expected to be dry (no groundwater present) when maximum drawdown is realised. RS means residual soil, ASH means Ashfield Shale, MIT mean Mittagong Formation, HAW means Hawkesbury Sandstone. bgl is below ground level

TABLE 9-2: SUMMARY OF RECOMMENDED CONSTRUCTION PHASE MONITORING LOCATIONS AND PREDICTED DRAWDOWN

Location ID	Typical pre-construction groundwater level (m bgl)	Deepest pre-construction groundwater level (m bgl)	Maximum predicted drawdown groundwater level (m AHD)	Maximum predicted drawdown (m bgl)	Comment
R718_BH1326	7.4	Insufficient data	-7	Insufficient data	May exceed predicted level if drawdown due to WestConnex M4 East tunnels / Substation No. 5 / Cintra Decline is ongoing from end of 2021
R718_BH1331	3.3	Insufficient data	-16	27.7	
R718_BH1333	4.3	Insufficient data	-18	24.7	
R718_BH1336	2.9	Insufficient data	Dry	Insufficient data	

Location ID	Typical pre-construction groundwater level (m bgl)	Deepest pre-construction groundwater level (m bgl)	Maximum predicted drawndown groundwater level (m AHD)	Maximum predicted drawdown (m bgl)	Comment
SMW_BH044_w	8.7	9.7	-17	31.0	
SMW_BH046_w	10.6	11.3	-19	14.9	
SMW_BH714_s	4.8	5.2	Dry	Dry	
SMW_BH714_w	9.2	9.3	-18	17.0	
SMW_BH715_s	2.5	2.8	Dry	Dry	
SMW_BH715_w	2.6	2.7	-20	29.5	
I1	No data	No data	-20	Require baseline data	
SLCM1	No data	No data	Up to 1	Require baseline data	
SLCM2	No data	No data	Up to 1	Require baseline data	

Notes: Dry means piezometer is expected to be dry (no groundwater present) when maximum drawdown is realised. RS means residual soil, ASH means Ashfield Shale, MIT mean Mittagong Formation, HAW means Hawkesbury Sandstone. bgl is below ground level

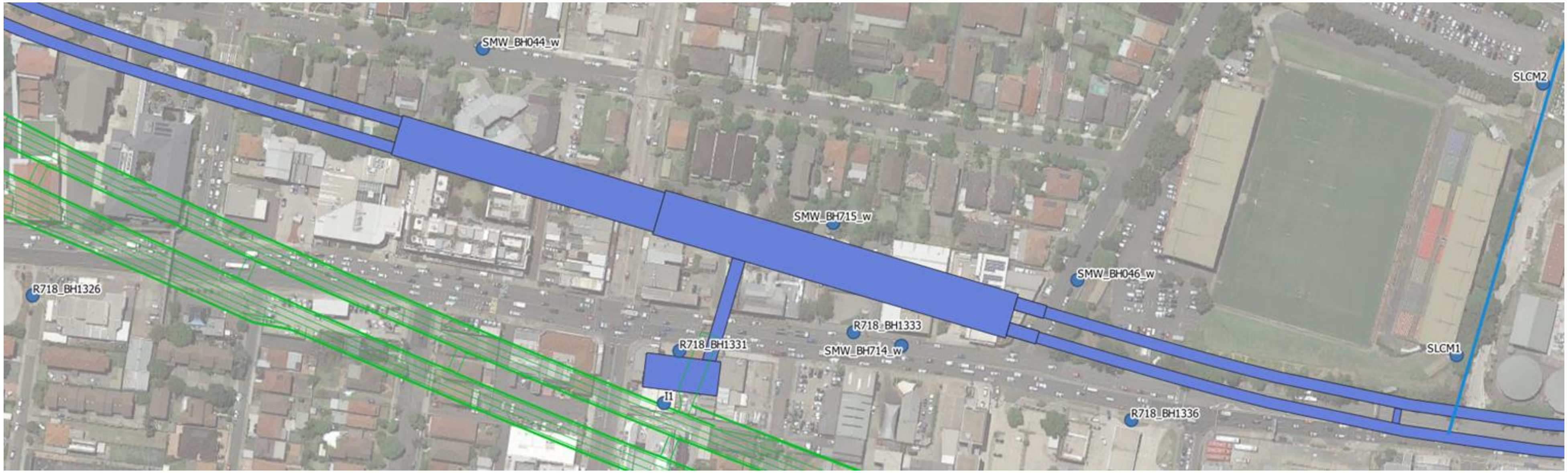


FIGURE 9-1: CONSTRUCTION PHASE GROUNDWATER LEVEL MONITORING LOCATIONS

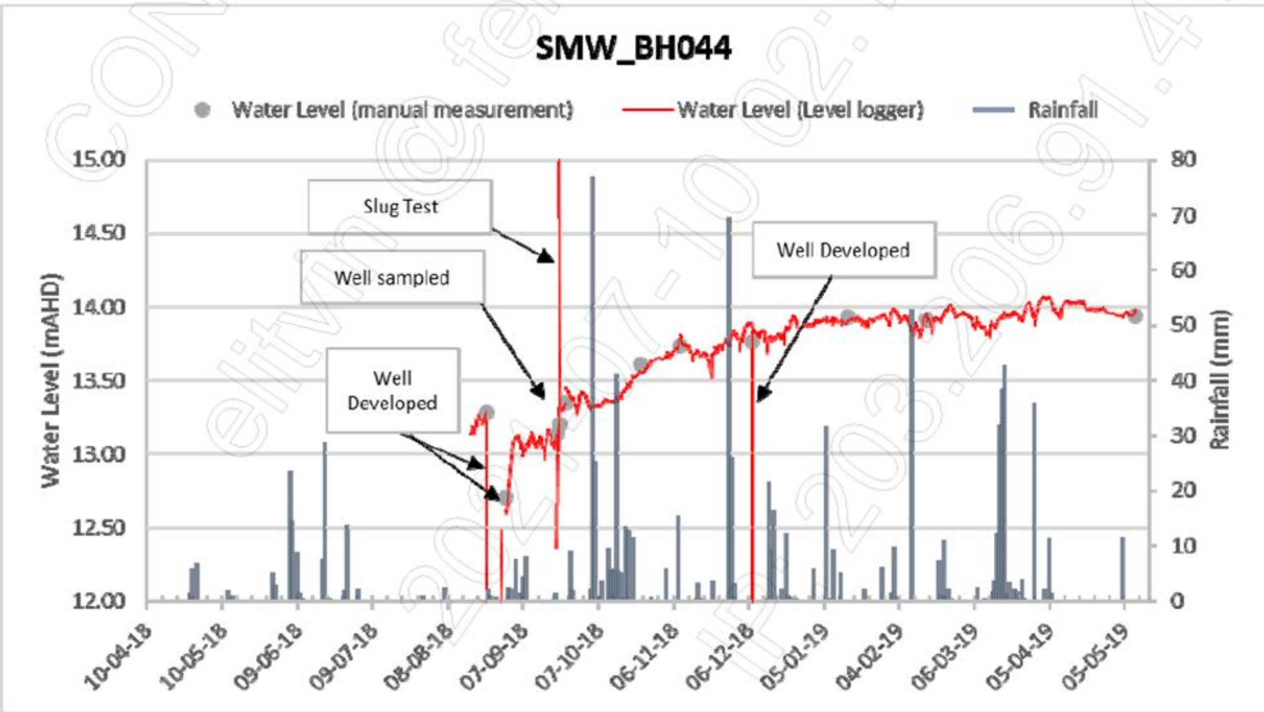
REFERENCES

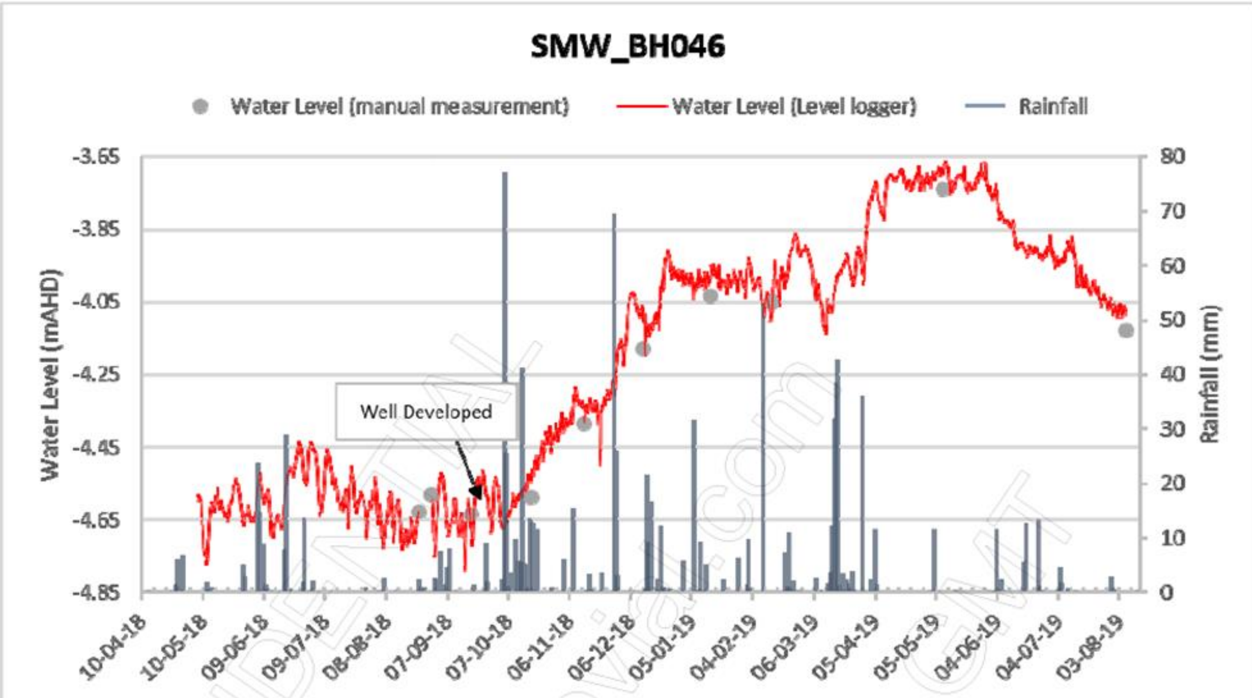
ANZG (2000), Australian and New Zealand Environment and Conservation Council (ANZECC) & Agriculture and Resource Management Council of Australia and New Zealand (ARMCANZ) water quality guidelines Australian and New Zealand Governments and Australian state and territory governments, Canberra ACT, Australia. Available at www.waterquality.gov.au/anz-guidelines.

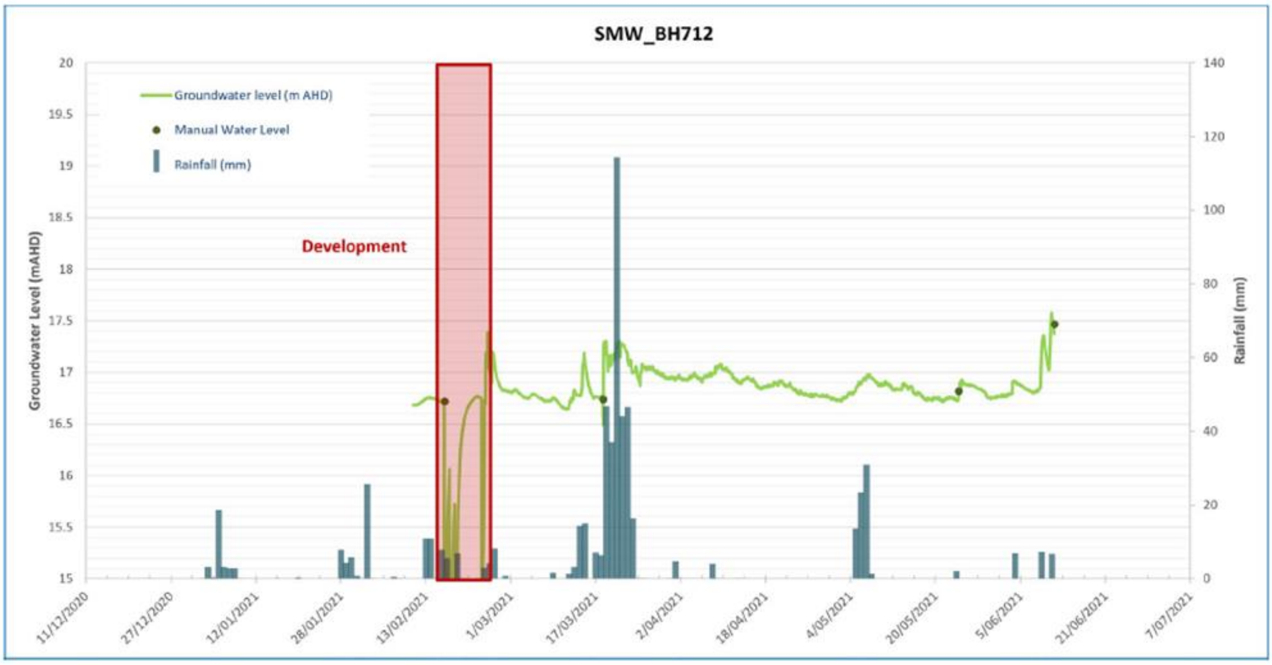
CBP Samsung John Holland Joint Venture (CSJH) (2016), M4 East Design and Construct, Technical Report – Hydrogeological Model Report – Final, 8 September 2016.

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

ANNEXURE A: GROUNDWATER HYDROGRAPHS






SMW_BH712
Groundwater level (m AHD)
Manual Water Level
Rainfall (mm)
Development
Groundwater Level (mAHD)
Rainfall (mm)
20
19.5
19
18.5
18
17.5
17
16.5
16
15.5
15
140
120
100
80
60
40
20
0
11/12/2020
27/12/2020
12/01/2021
28/01/2021
13/02/2021
1/03/2021
17/03/2021
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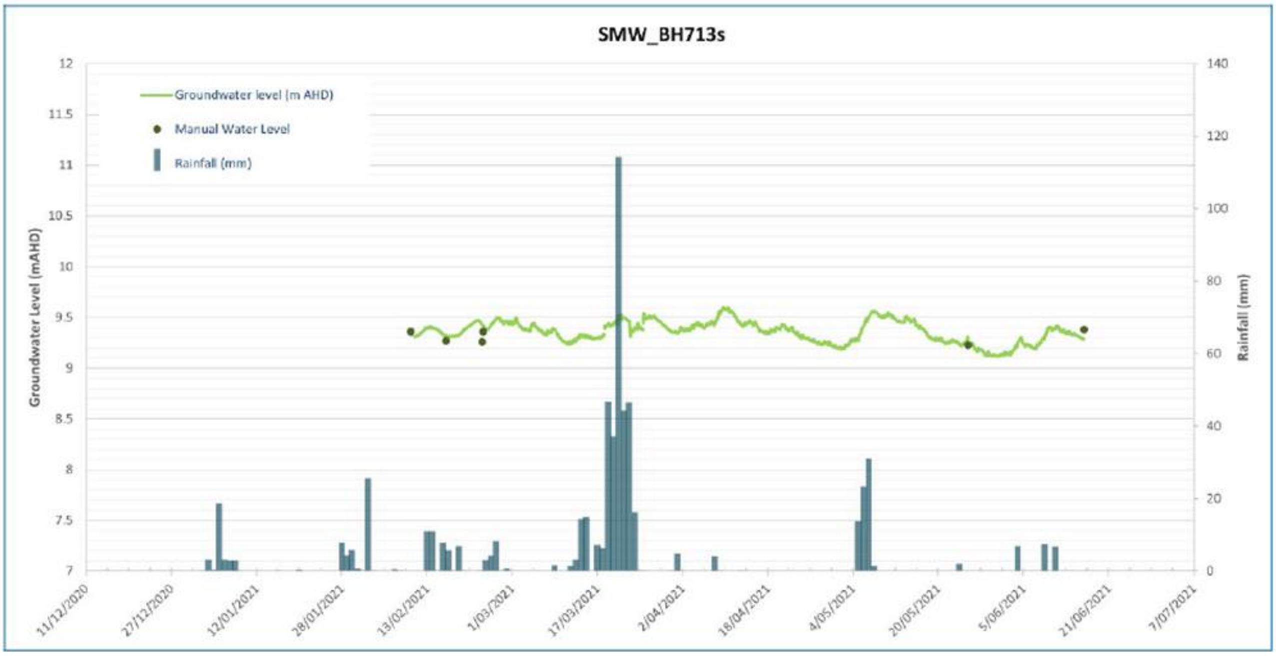
21/06/2021
7/07/2021


SMW_BH713s
Groundwater level (m AHD)
Manual Water Level
Rainfall (mm)
Groundwater Level (mAHD)
Rainfall (mm)
12
11.5
11
10.5
10
9.5
9
8.5
8
7.5
7
140
120
100
80
60
40
20
0
11/12/2020
27/12/2020
12/01/2021
28/01/2021
13/02/2021
1/03/2021
17/03/2021
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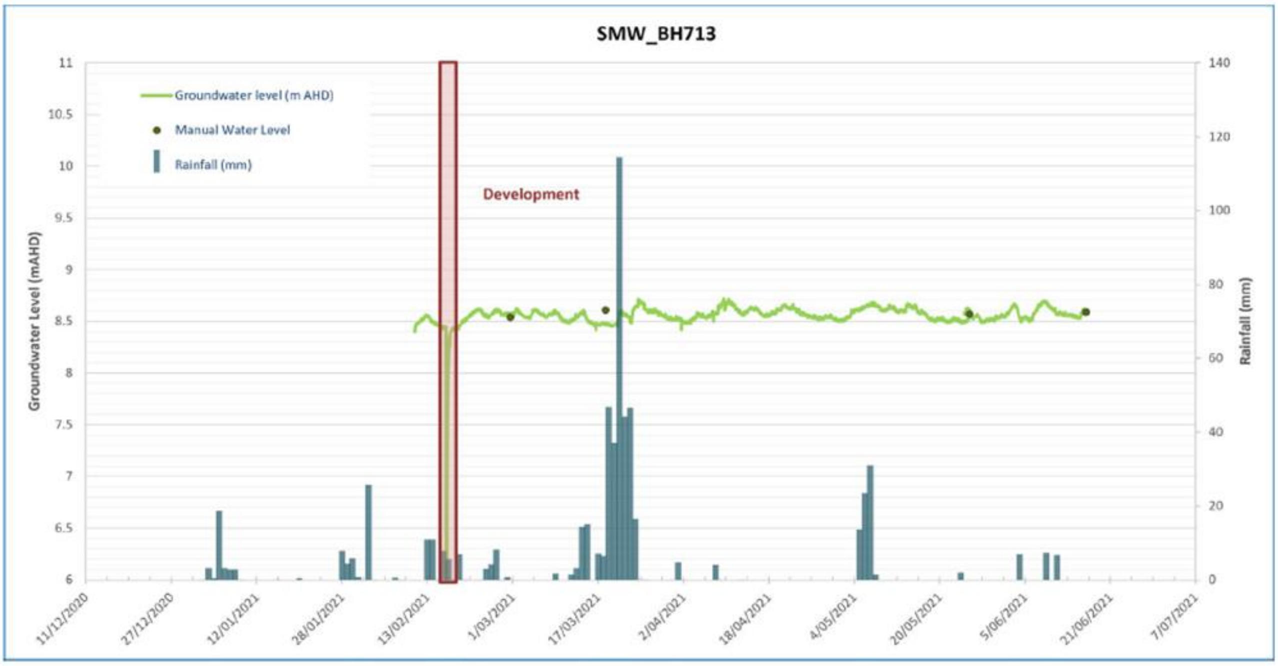
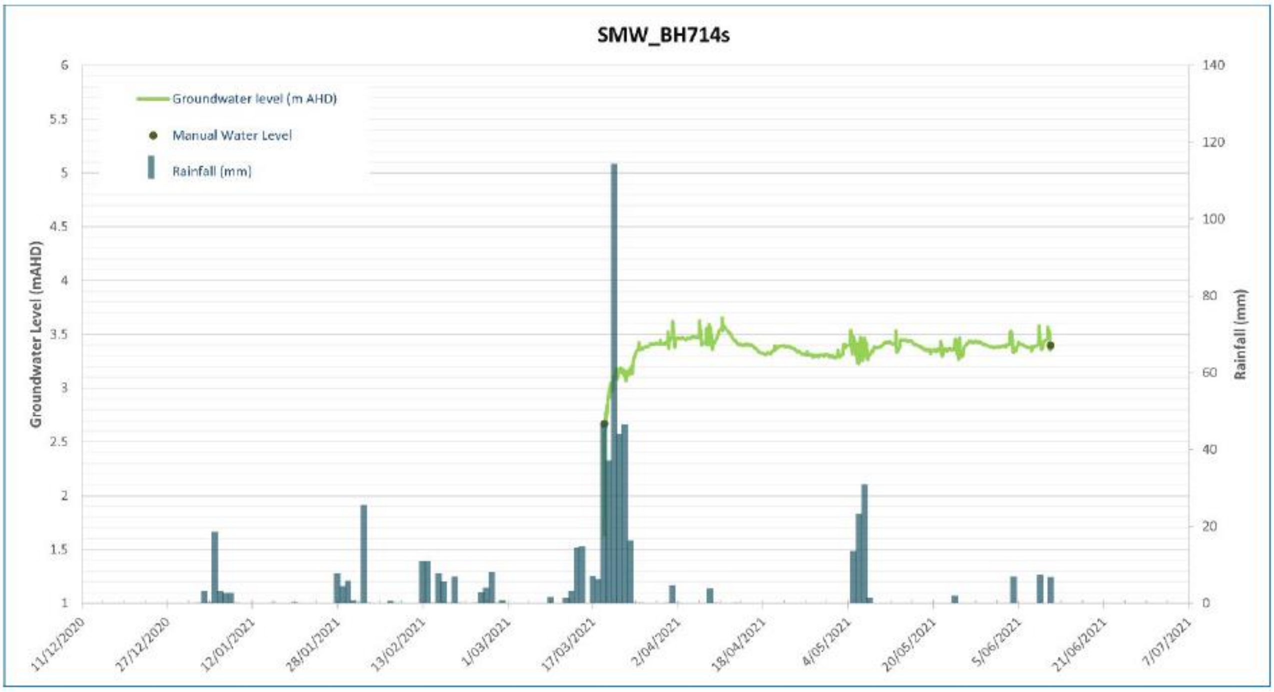
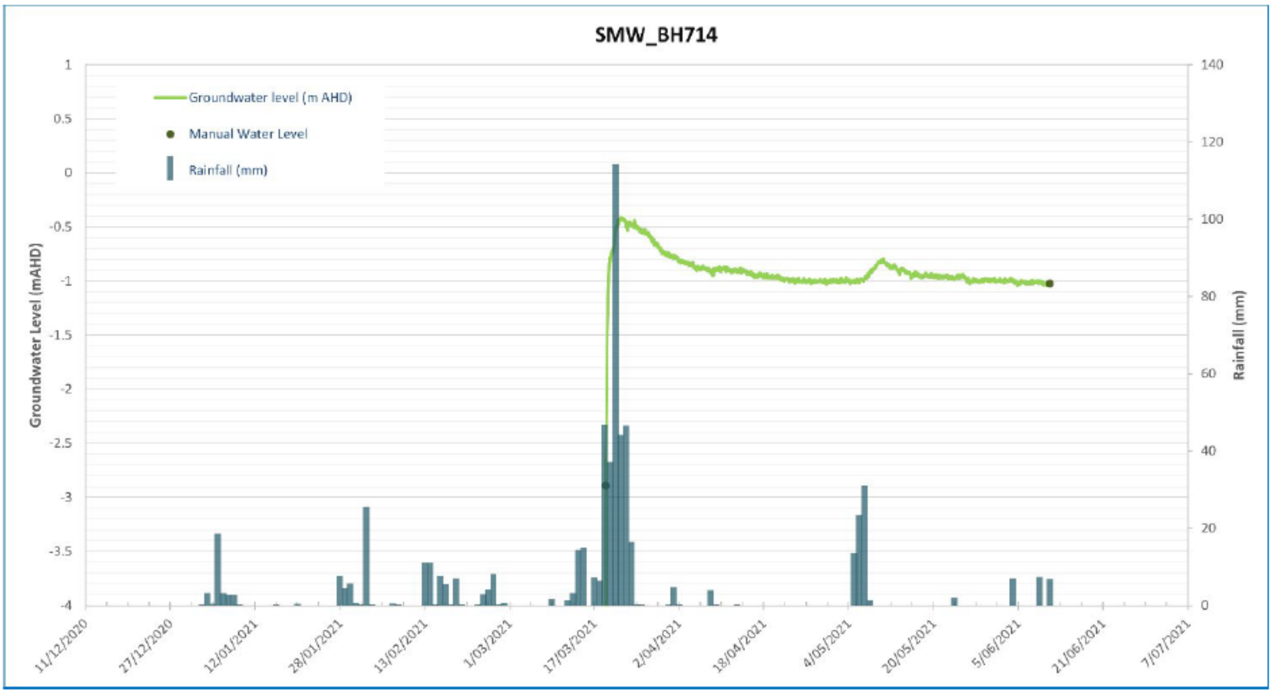
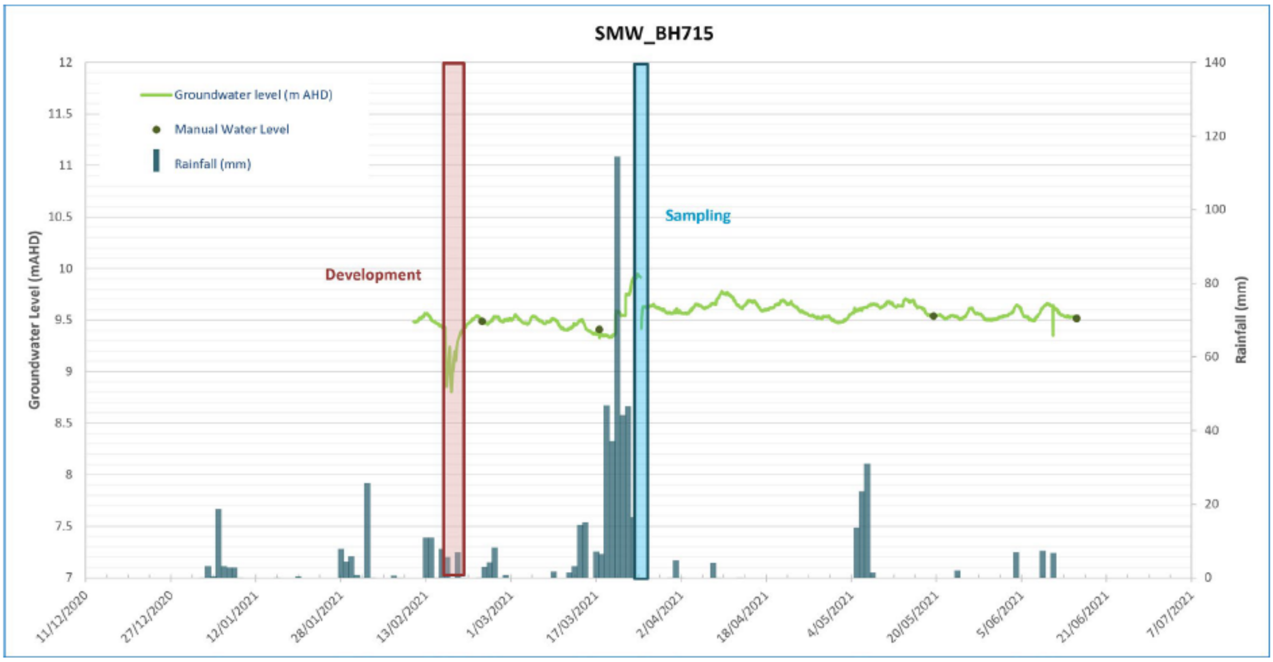
21/06/2021
7/07/2021


SMW_BH713
Groundwater level (m AHD)
Manual Water Level
Rainfall (mm)
Development
Groundwater Level (mAHD)
Rainfall (mm)


SMW_BH714s
Groundwater level (m AHD)
Manual Water Level
Rainfall (mm)
Groundwater Level (mAHD)
Rainfall (mm)



BH715s:



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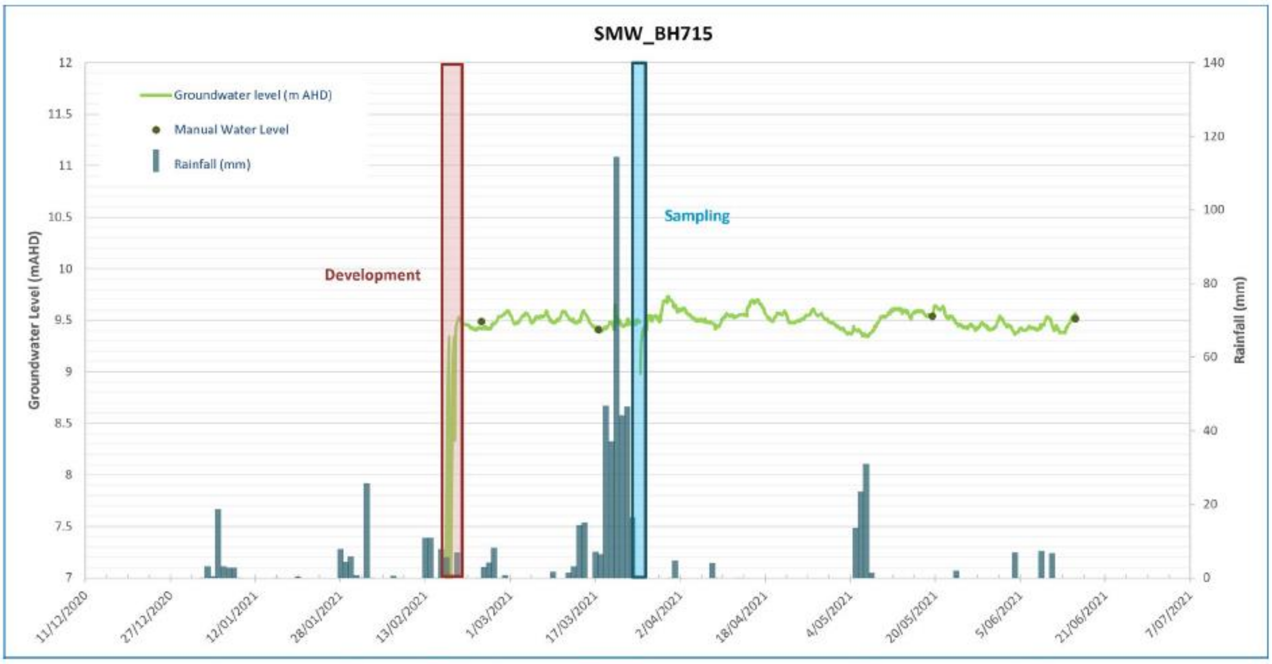
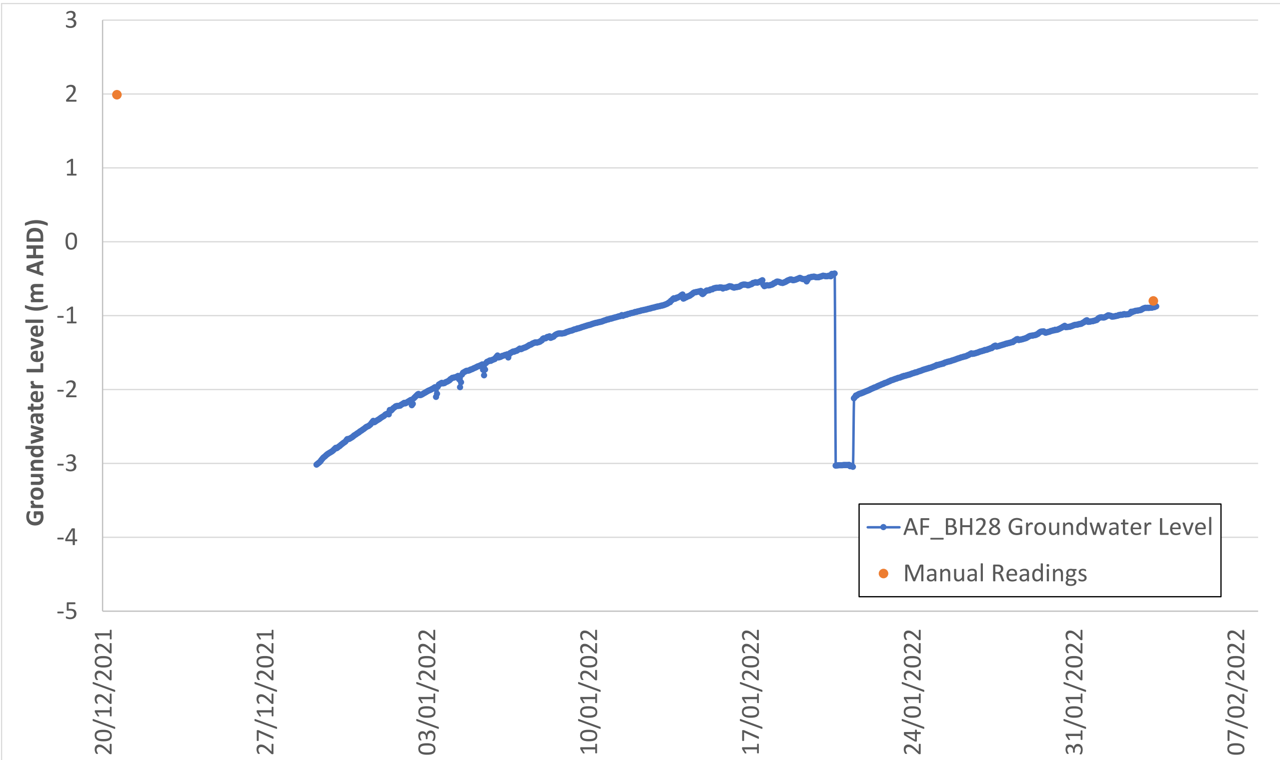

SMW_BH715
Groundwater level (m AHD)
Manual Water Level
Rainfall (mm)
Development
Sampling
Groundwater Level (mAHD)
Rainfall (mm)
12
11.5
11
10.5
10
9.5
9
8.5
8
7.5
7
140
120
100
80
60
40
20
0
11/12/2020
27/12/2020
12/01/2021
28/01/2021
13/02/2021
1/03/2021
17/03/2021
2/04/2021
18/04/2021
4/05/2021
20/05/2021
5/06/2021
21/06/2021
7/07/2021

AF_BH28:


Groundwater Level (m AHD)
3
2
1
0
-1
-2
-3
-4
-5
AF_BH28 Groundwater Level
Manual Readings
20/12/2021
27/12/2021
03/01/2022
10/01/2022
17/01/2022
24/01/2022
31/01/2022
07/02/2022

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	Report	Ben Rose, Ben Rotter	Ben Rotter	Richard Evans	Fernando Lopez Asensio
B	8/03/2022	Report	Ben Rose, Ben Rotter	Ben Rotter	Richard Evans	Fernando Lopez Asensio

Contents

1. Introduction

1.1. Objective and scope

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

1.2. Basis of memorandum

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

The other major projects include:

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

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

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

2. Hydrogeological units

2.1. Overview

There are seven key hydrogeological units applicable to project:

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

Fault zones are also discussed.

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

2.2. Fill

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

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

2.3. Quaternary alluvium

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

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

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

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

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

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

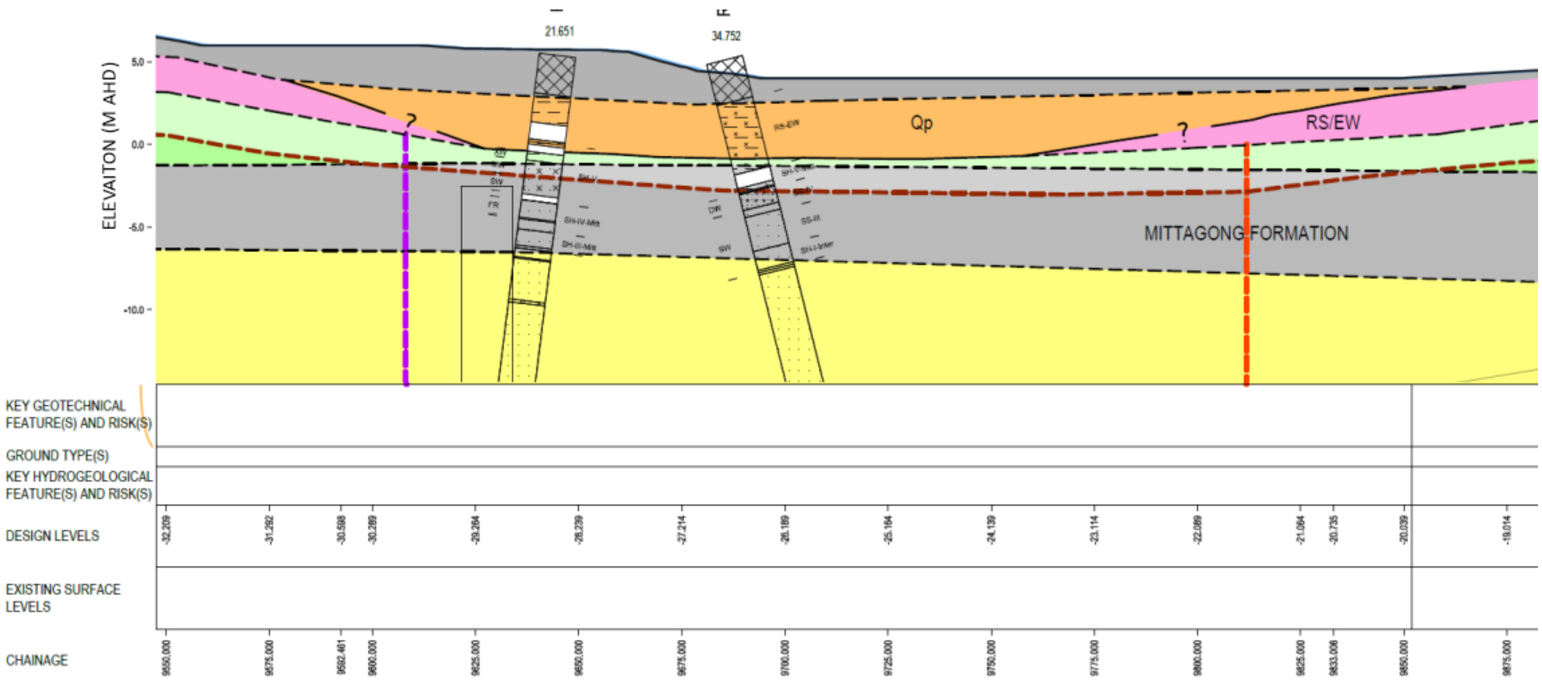


FIGURE 2-2: LOCATION OF ALLUVIUM AT BURWOOD NORTH STATION IN SECTION

2.4. Residual soil

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

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

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

2.5. Ashfield Shale

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

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

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

2.6. Mittagong Formation

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

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

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

2.7. Hawkesbury Sandstone

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

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

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

2.8. Dykes

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

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

2.9. Fault zones

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

3. Hydrogeological testing results and properties

3.1. Hydrogeological test data and literature

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

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

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

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

3.2. Hydrogeological testing results and hydrogeological properties

3.2.1. Fill

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

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

3.2.2. Quaternary alluvium

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

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

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

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

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

3.2.3. Residual soil

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

3.2.4. Ashfield Shale

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

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

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

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

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

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

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

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

- The 75th percentile value for the sandstone and siltstone test intervals is classified as a very low (<1 Lugeon) Lugeon value, with the rock mass characterised as very tight
- The 75th percentile value for the siltstone test intervals is classified as a low Lugeon value (1 to 5 Lugeon), with the rock mass characterised as tight
- For the sandstone and siltstone test intervals, only two out of 88 tests surpassed the medium Lugeon range criteria (15 to 50 Lugeon). These two tests occurred in borehole SMW_BH502 and the recorded result was greater than 100 Lugeons for both tests, which is classified as a very high Lugeon value
- For the siltstone test intervals, only one out of 54 tests surpassed the moderate Lugeon range criteria (5 to 15 Lugeon), the maximum test value of 16 Lugeons

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

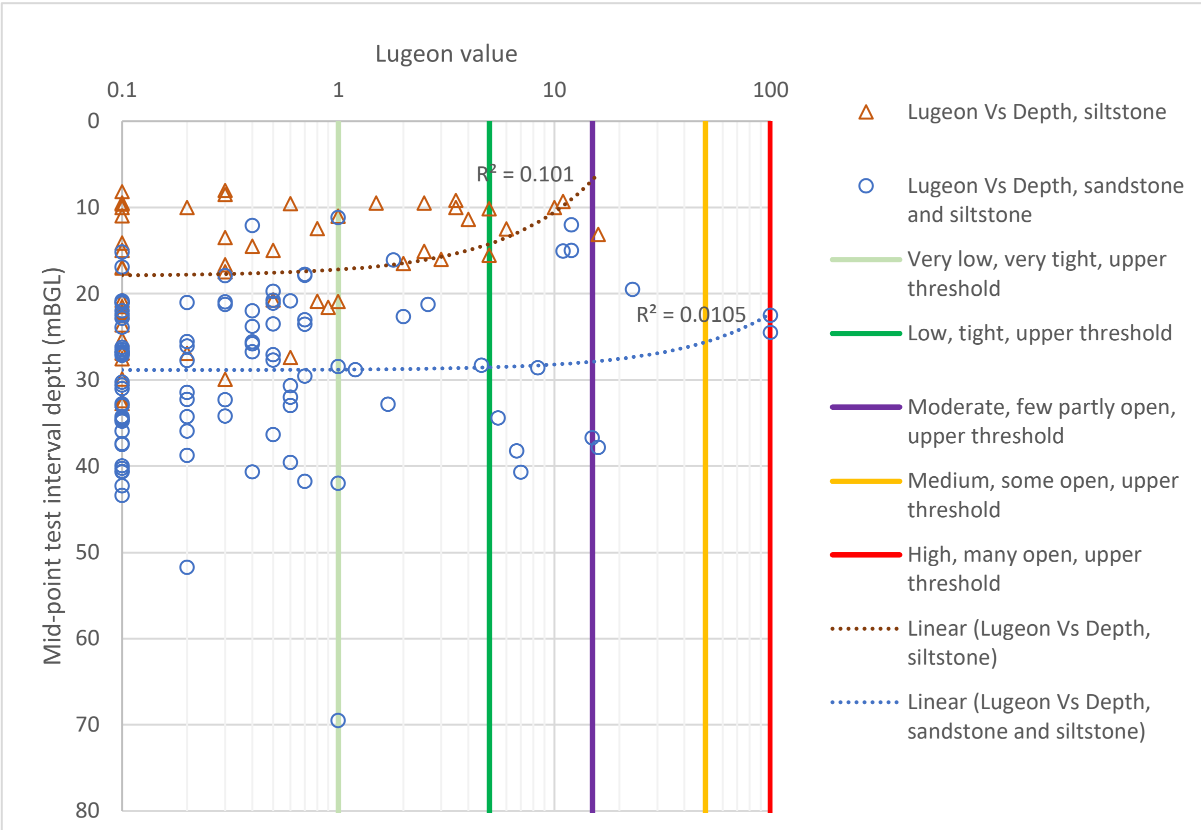


FIGURE 3-1: LUGEON VALUES FOR SILTSTONE TEST INTERVALS, AND SANDSTONE AND SILTSTONE TEST INTERVALS, CLASSED ACCORDING TO QUINONES-ROZO (2010)

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

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

Figure 3-1,

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

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

Packer mid-point test interval depth category	Number of tests	Lugeon value			Horizontal hydraulic conductivity (m/d)		
		Median	Geometric mean	Arithmetic mean	Median	Geometric mean	Arithmetic mean
0 to <15 m	27	0.6	0.8	2.6	5.20×10^{-3}	6.81×10^{-3}	2.28×10^{-2}
15 to <30 m	25	0.1	0.2	0.6	8.67×10^{-4}	1.99×10^{-3}	5.10×10^{-3}
30 to <45 m	2	0.6	0.5	0.6	5.20×10^{-3}	4.50×10^{-3}	5.20×10^{-3}

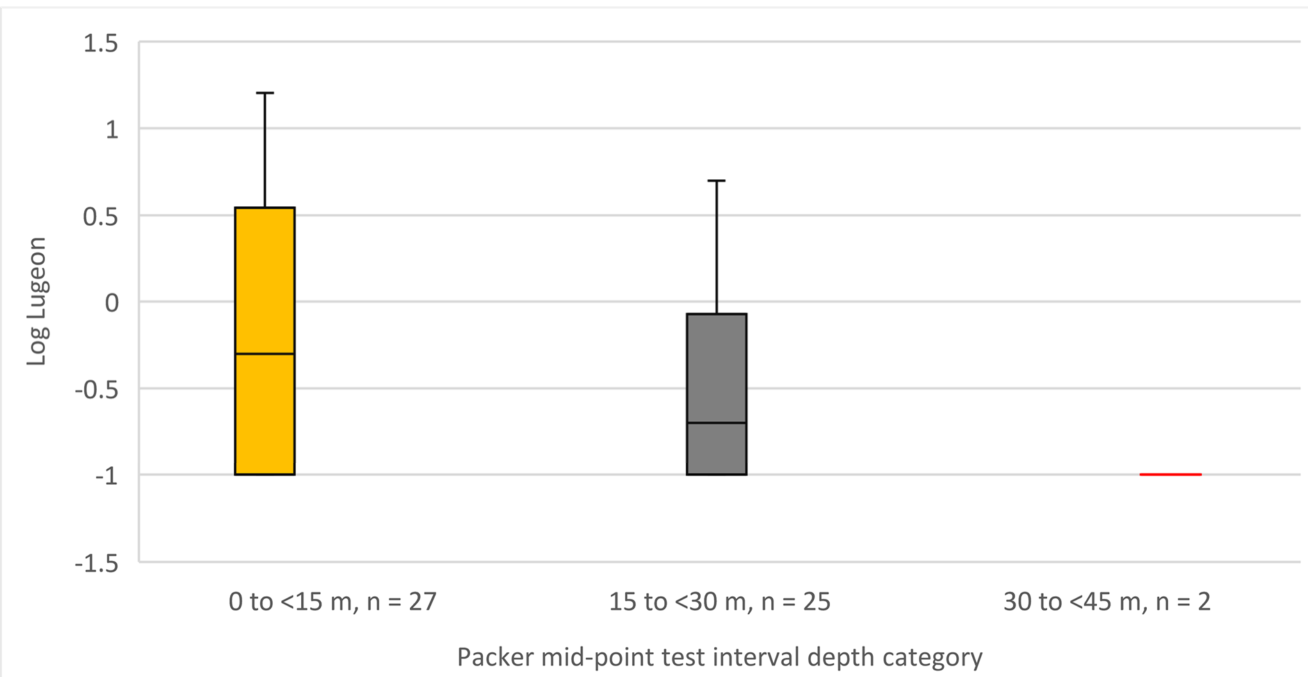


FIGURE 3-2: LOG LUGEON VALUES FOR SILTSTONE TEST INTERVALS BY DEPTH CATEGORY

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

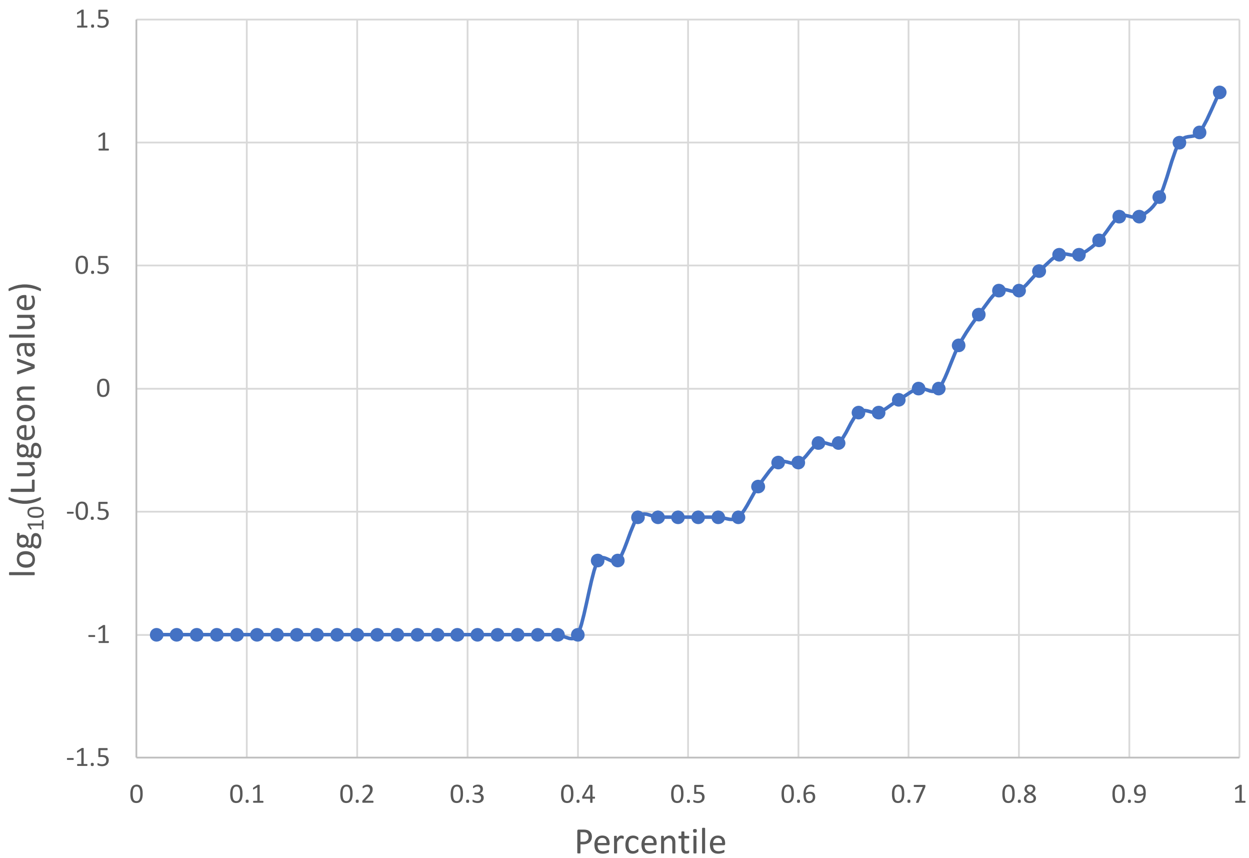


FIGURE 3-3: CUMULATIVE DISTRIBUTION OF LUGEON VALUES FOR SILTSTONE TEST INTERVALS

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

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

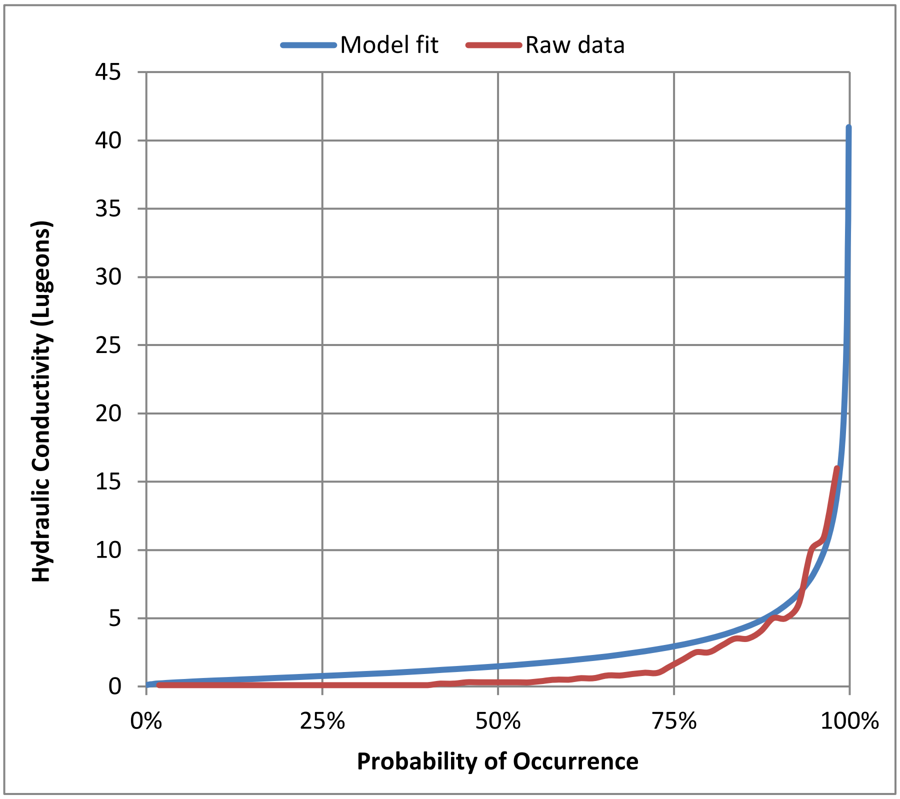


FIGURE 3-4: CUMULATIVE DISTRIBUTION OF LUGEON VALUES FOR SILTSTONE TEST INTERVALS AND NORMAL DISTRIBUTION MODEL FIT TO DATA

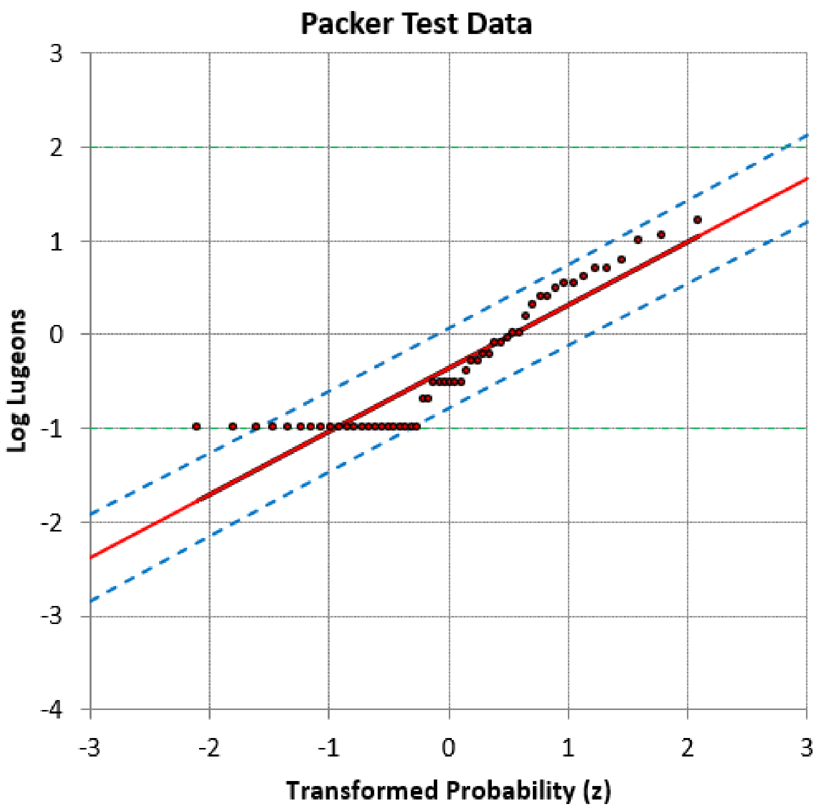


FIGURE 3-5: QUANTILE PLOT OF LUGEON VALUES FOR SILTSTONE TEST INTERVALS

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

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

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

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

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

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

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

3.2.5. Mittagong Formation

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

3.2.6. Hawkesbury Sandstone

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

TABLE 3-5: HAWKESBURY SANDSTONE GROUNDWATER SYSTEM PROPERTIES FROM CTP PROJECT DATA AND LITERATURE REVIEW

Statistic	Packer testing					Lit				
	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				
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}		
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}					
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}					
Single value										
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}		
N (number of tests)	150	196	205	363	300					
K_v/K_h										
Minimum							0.01			
Single value										
Maximum							0.10			
Specific storage range (m^{-1})										
Minimum						5.00×10^{-6}	5.00×10^{-6}		1.00×10^{-5}	3.70×10^{-3}
Single value										
Maximum						1.00×10^{-5}	5.00×10^{-5}		1.00×10^{-4}	1.00×10^{-1} [c]
Specific yield (-)										
Minimum						0.02				
Single value							0.025			
Maximum						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)
Summary										
Parameter	**Minimum value**	**Maximum value**	**Adopted representative value**							

Statistic	Groundwater models					
	SS [a]	SS [a]	SS [a]	T [b]	SS [a] / T [b]	T [b]
Horizontal hydraulic conductivity (m/d)						
Minimum			1.00×10^{-3}	1.00×10^{-3}	1.50×10^{-3}	8.64×10^{-4} (deeper zones)
5th percentile						
10th percentile						
25th percentile						
Median					6.00×10^{-3}	
Harmonic mean						
Geomean						
Average					3.02×10^{-2}	
Single value	1.00×10^{-2}	1.00×10^{-2}				8.64×10^{-3} (excludes 'deeper zones'
75th percentile						
90th percentile						
95th percentile						
Maximum			5.16×10^{-3}	5.00×10^{-2}	1.30×10^{-1}	6.91×10^{-3} (deeper zones)
N (number of tests)						
K_v/K_h						
Minimum					0.02	
Single value	1	0.05				0.1
Maximum					0.50	
Specific storage range (m^{-1})						
Minimum					1.00×10^{-6}	
Single value						5.00×10^{-6}
Maximum					1.00×10^{-5}	
Specific yield (-)						
Minimum					0.02	
Single value						0.01
Maximum					0.05	
Source	Golder (2016)	CDM Smith (2016)	GHD (2015)	GHD (2015)	Hydro Simulations (2017)	LSBJV (2020)
Summary						
Parameter						

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

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

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

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

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

- The 75th percentile value is classified as a low Lugeon value (1 to 5 Lugeon), with the rock mass characterised as tight.
- The median, geometric mean and mean value is 0.4 Lugeons, 0.6 Lugeons and 5.9 Lugeons, respectively. The median and geometric mean values are classified as very low Lugeon values (<1 Lugeon), with the rock mass characterised as very tight. The mean value is classified as a moderate Lugeon value, with the rock mass characterised as having 'a few partly open' discontinuities.
- Out of a total of 211 tests, the maximum test result of >100 Lugeons occurred for three tests at SMW_BH502, a single test at SMW_BH717 and a single test at SMW_BH719

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

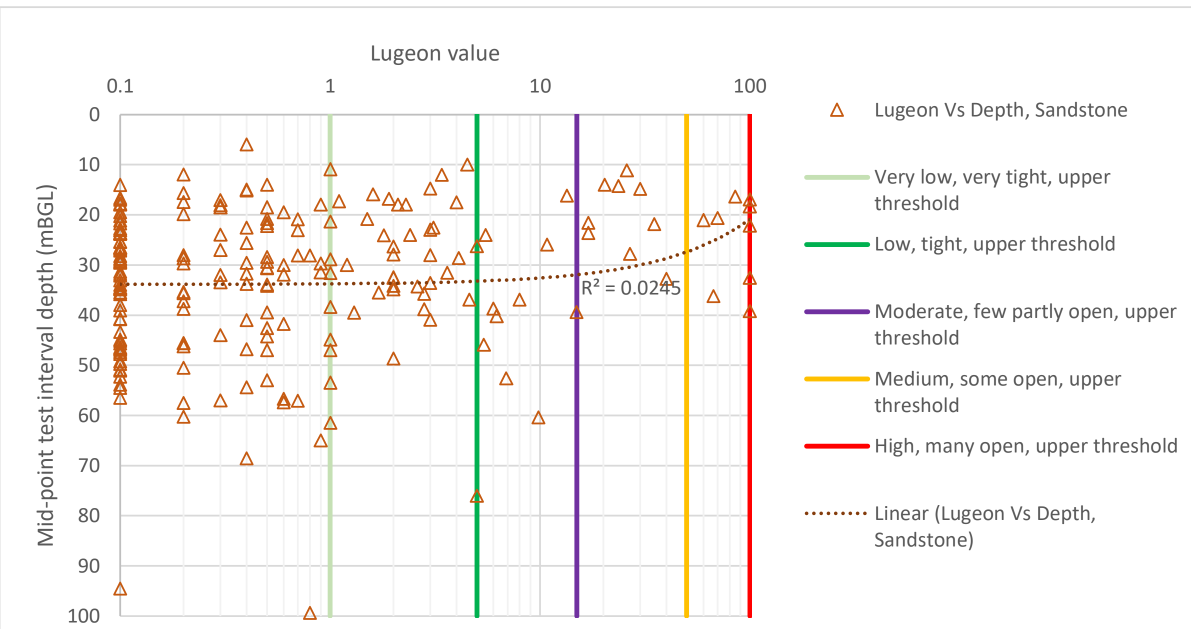


FIGURE 3-6: LUGEON VALUES FOR SANDSTONE TEST INTERVALS, CLASSED ACCORDING TO QUINONES-ROZO (2010)

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

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

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

Packer mid-point test interval depth category	N	Lugeon value			Horizontal hydraulic conductivity (m/d)		
		Median	Geometric mean	Arithmetic mean	Median	Geometric mean	Arithmetic mean
0 to <15 m	13	3.0	2.4	7.5	2.60×10^{-2}	2.10×10^{-2}	6.54×10^{-2}
15 to <30 m	90	0.5	0.7	8.0	4.33×10^{-3}	6.27×10^{-3}	6.92×10^{-2}
30 to <45 m	65	0.4	0.5	3.8	3.47×10^{-3}	4.28×10^{-3}	3.28×10^{-2}
45 to <60 m	34	0.4	0.5	5.3	3.47×10^{-3}	4.66×10^{-3}	4.59×10^{-2}
60 to 105.9 m (max)	9	0.1	0.1	0.2	8.67×10^{-4}	1.14×10^{-3}	1.35×10^{-3}

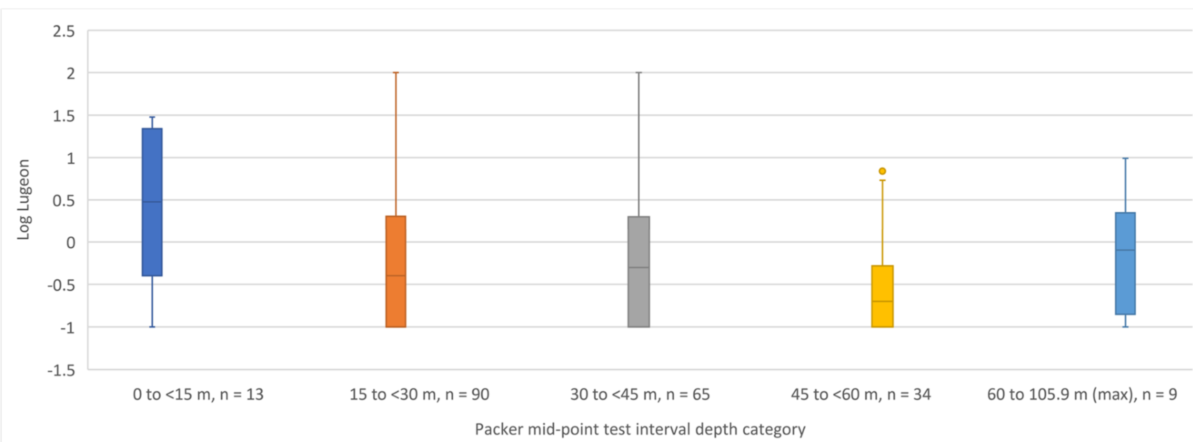


FIGURE 3-7: LOG LUGEON VALUES FOR SANDSTONE TEST INTERVALS BY DEPTH CATEGORY

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

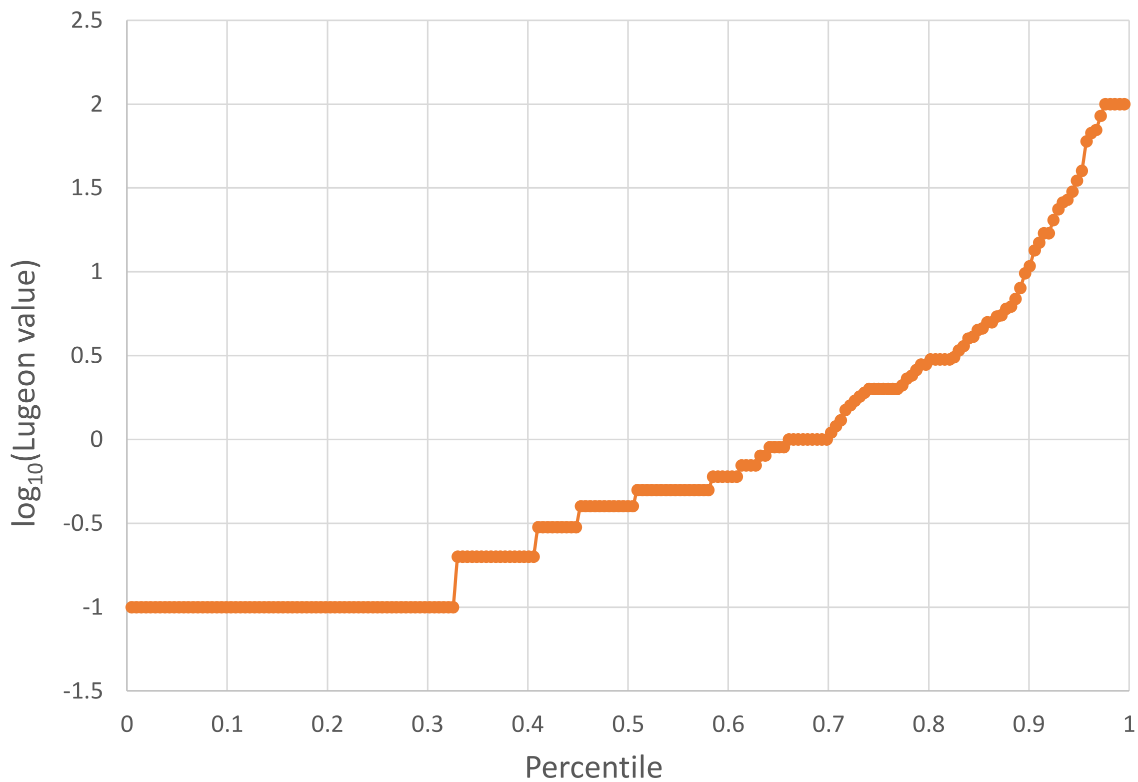


FIGURE 3-8: CUMULATIVE DISTRIBUTION OF LUGEON VALUES FOR SANDSTONE TEST INTERVALS

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

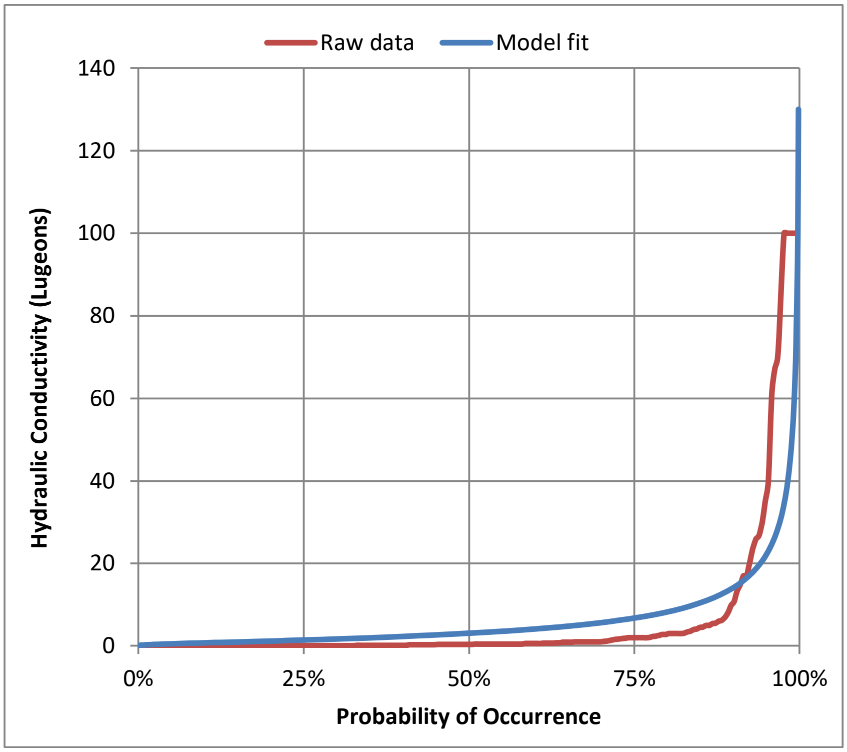


FIGURE 3-9: CUMULATIVE DISTRIBUTION OF LUGEON VALUES FOR SANDSTONE TEST INTERVALS AND NORMAL DISTRIBUTION MODEL FIT TO DATA

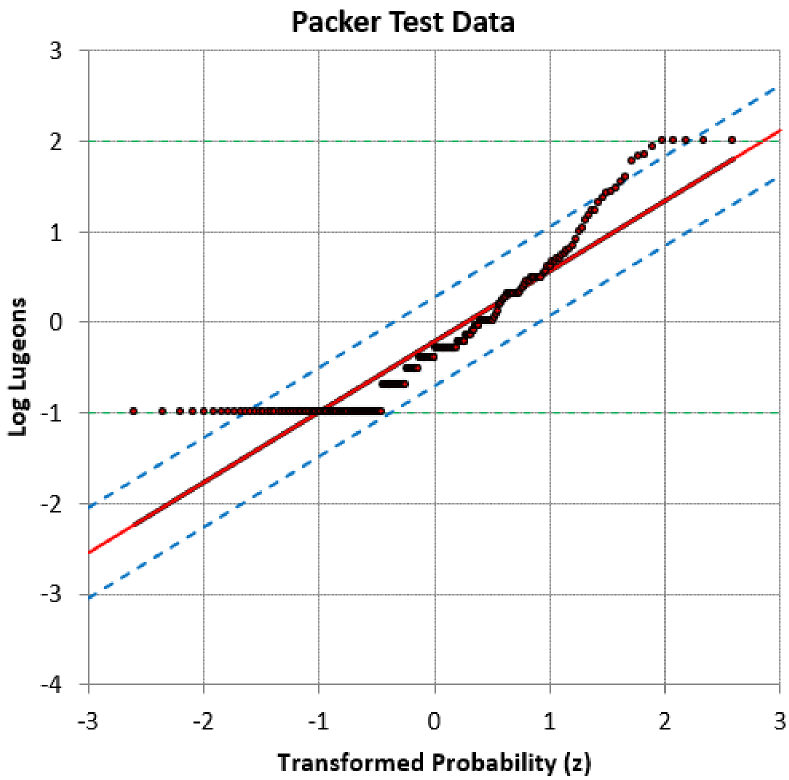


FIGURE 3-10: QUANTILE PLOT OF LUGEON VALUES FOR SANDSTONE TEST INTERVALS

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

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

Table 3-7: Lugeon and hydraulic conductivity results for sandstone test intervals

Statistic	Sandstone test intervals	
	Lugeon	Horizontal hydraulic conductivity (m/d)
Minimum	0.10	8.64×10^{-4}
5th percentile	0.10	8.64×10^{-4}
10th percentile	0.10	8.64×10^{-4}
25th percentile	0.10	8.64×10^{-4}
Median	0.40	3.46×10^{-3}
Geometric mean	0.61	5.27×10^{-3}
Arithmetic mean	5.90	5.10×10^{-2}
75th percentile	2.00	1.73×10^{-2}
90th percentile	9.80	8.47×10^{-2}
95th percentile	32.50	2.81×10^{-1}
Maximum	100.00	8.64×10^{-1}
Log-normally distributed fit		
Arithmetic mean	3.10	2.68×10^{-2}
K_{3D}	1.00	8.64×10^{-3}
N (number of tests)	150	

3.2.1. Dykes and Faults

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

4. Adopted representative hydrogeological parameter values

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

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

Hydrogeological unit	Typical Horizontal hydraulic conductivity range (m/day)	K_v/K_h range	Specific storage range (m^{-1})	Specific yield range (-)
Typical range				
Quaternary alluvium	5.00×10^{-3} to 1.00×10^{0}	0.1 to 0.5	1.00×10^{-5}	0.06 to 0.20
Ashfield Shale	3.80×10^{-3} to 1.20×10^{-2} (0.4 to 1.4 Lugeons) (geometric mean to 75th percentile) (Log-normally distributed arithmetic mean is 1.64×10^{-2} = 1.9 Lugeons; K_{3D} value is 6.05×10^{-3} m/d = 0.7 Lugeons)	0.1 to 1.0	5.00×10^{-6} to 1.00×10^{-5}	0.01 to 0.025
Mittagong Formation and Hawkesbury Sandstone	5.27×10^{-3} to 1.73×10^{-2} (0.6 to 2.0 Lugeons) (geometric mean to 75th percentile) (Log-normally distributed arithmetic mean is 2.65×10^{-2} m/d = 3.1 Lugeons; K_{3D} value is 9.06×10^{-3} m/d = 1.0 Lugeons)	0.01 to 1	1.00×10^{-6} to 1.00×10^{-5}	0.02 to 0.05
Adopted representative value				
Quaternary alluvium	1.00×10^{0} (predominantly sandy) 5.00×10^{-3} (predominantly clayey)	0.1	1.00×10^{-5}	0.20 (predominantly sandy) 0.06 (predominantly clayey)
Ashfield Shale	Conservative: 1.21×10^{-2} (1.4 Lugeons; 75th percentile) Likely: 6.05×10^{-3} m/d (0.7 Lugeons; K_{3D} value)	0.1	5.00×10^{-6}	0.02
Mittagong Formation and Hawkesbury Sandstone	Conservative: 1.73×10^{-2} (2.0 Lugeons; 75th percentile) Likely: 8.64×10^{-3} m/d (1.0 Lugeons; K_{3D} value)	0.1	5.00×10^{-6}	0.05

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

5. References

Aecom (2015), New M5 Environmental Impact Statement, Technical working paper: Groundwater, Appendix Q

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

Technical Memo

To	Fernando Martinez Ceballos Eduardo Torralba Marsutha Somasundaram	Date	23 February 2022
Copies	Fernando Lopez Asensio Dan Worrall Jiun Dar Ong	Document ID	SMWSTCTP-AFJ-SWD-SN250,SN300,SN350,SN400-ST-RPT-Generic Appendix G Annexure XX
From	Ben Rotter	Revision	C
Subject	Design groundwater loads for station soil retaining walls – accidental load cases – burst water main and blocked drainage		

1. Introduction

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

2. Particular Specifications

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

4.1 Civil and Structural

4.1.3 Design Loading

4.1.3.1 General

(d) The Tunnelling Contractor must design all civil and structural works to accommodate the potential impact of groundwater levels and hydrostatic pressures of floodwater plains or a burst water main where existing or new water utilities are within proximity to the Project Works and Temporary Works. [SM-W-CTP-PS-709]

(i) The Tunnelling Contractor must not allow for any reduction in hydrostatic loadings due to localised lowering of groundwater levels in the design of the Works. The reduction of hydrostatic loading due to localised lowering of groundwater levels is permitted in the design of the support of Station Excavations and Station Shaft Excavations that are drained in accordance with the requirements in Section 4.1.7(a). [SM-W-CTP-PS-715]

4.1.8 Groundwater Seepage

(b) The Tunnelling Contractor must design for the risk of water pressure build-up as a result of blocked drainage. [SM-W-CTP-PS-1030]

3. Design groundwater load conditions

3.1. CTP project works conditions

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

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

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

3.2. CTP project works exceptional conditions

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

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

4. Exceptional load condition: burst water mains

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

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

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

The burst water main would then saturate the soils.

Two scenarios have been considered:

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

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

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

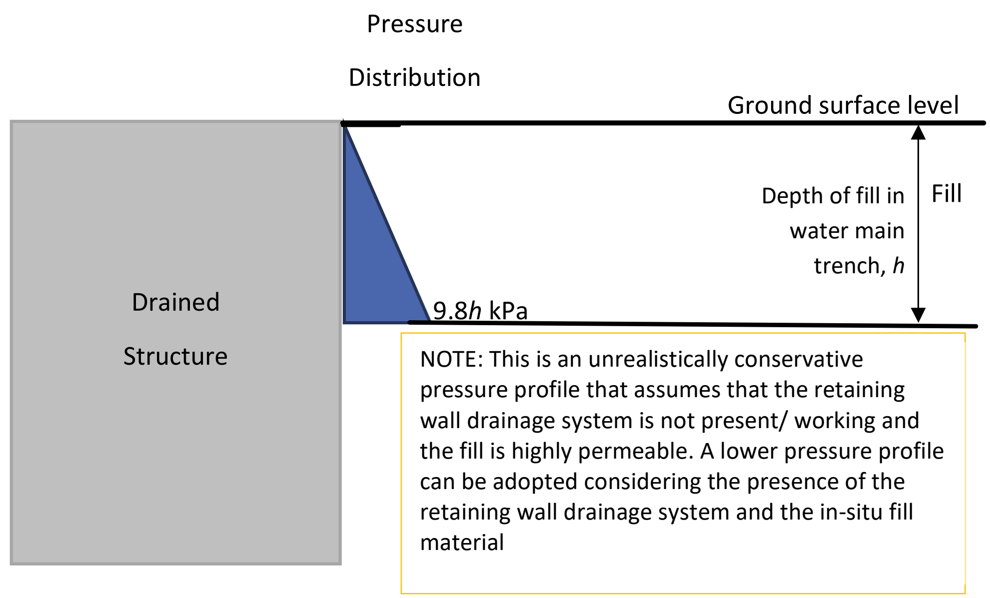

FIGURE 1: EXCEPTIONAL GROUNDWATER PRESSURE CONDITION FOR BURST WATER MAIN

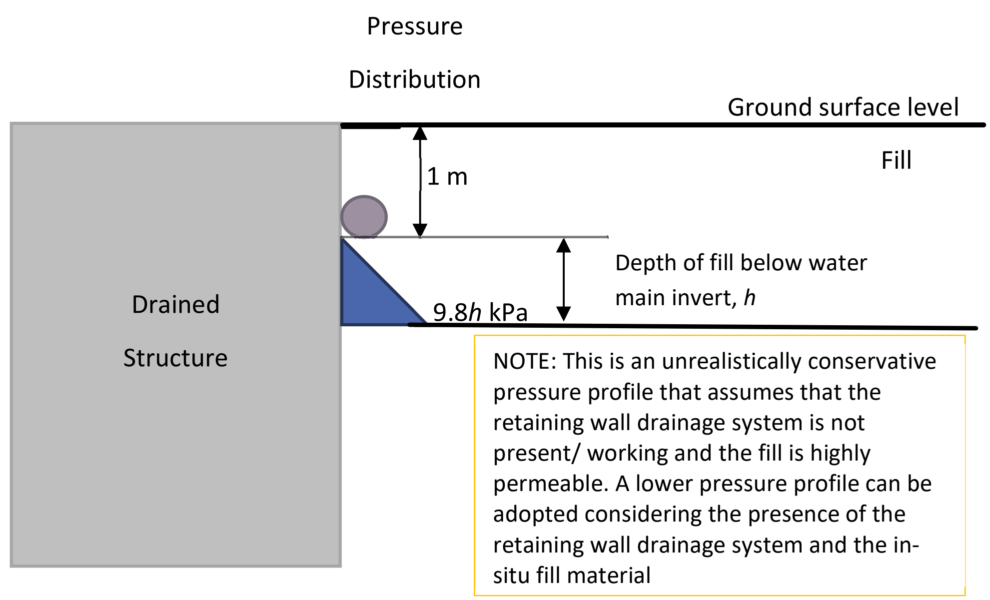

FIGURE 2: EXCEPTIONAL GROUNDWATER PRESSURE CONDITION FOR BURST WATER MAIN CONSIDERING PIPE INVERT LEVEL

5. Exceptional load condition: flood

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

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

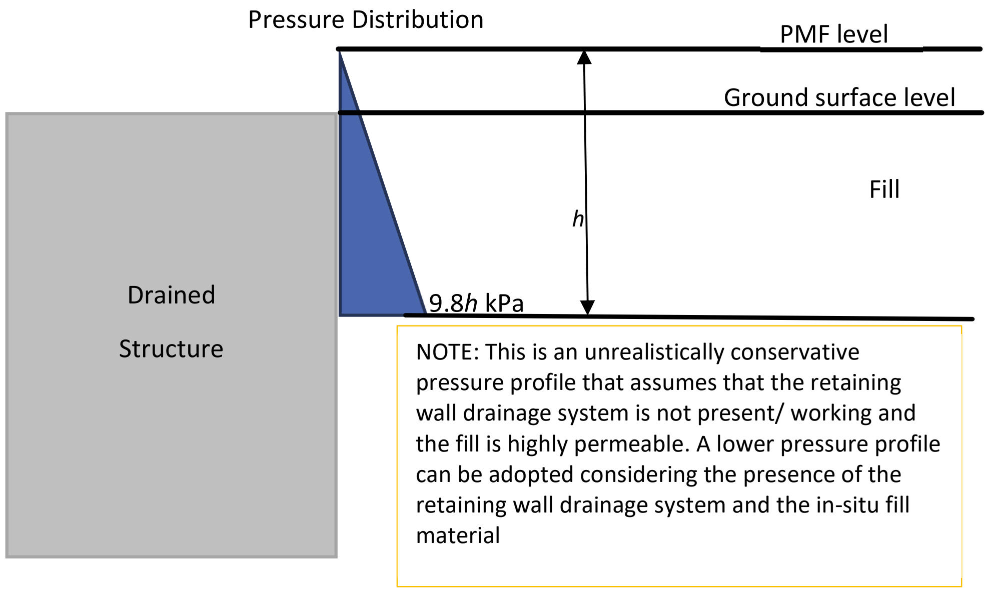

FIGURE 3: EXCEPTIONAL GROUNDWATER PRESSURE CONDITION FOR FLOOD SCENARIO

6. Exceptional load condition: blocked drainage

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

This section describes the development of the general load condition.

6.1. Retaining wall design

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

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

6.1. Approach to developing load condition

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

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

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

6.2. Modelling approach

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

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

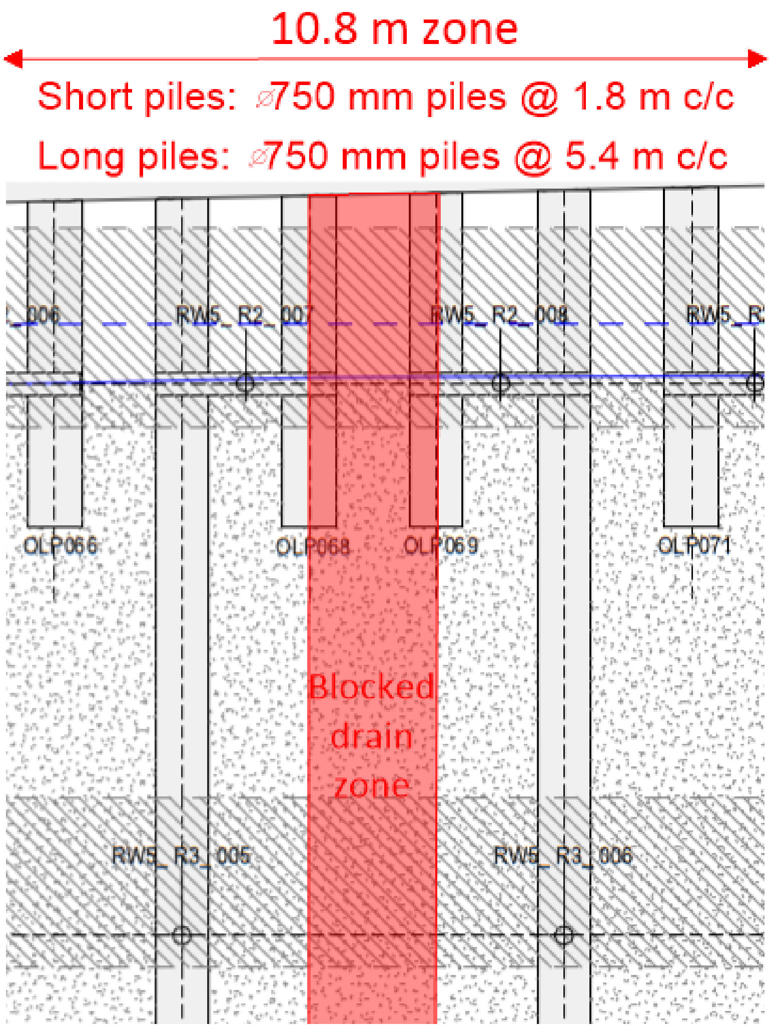

FIGURE 4: TYPICAL PILE LAYOUT AND BLOCKED DRAINAGE ZONE

6.3. Model parameter values

Adopted hydrogeological parameter values are provided in Table 1.

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

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

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

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

TABLE 1 ADOPTED HYDROGEOLOGICAL PARAMETER VALUES

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

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

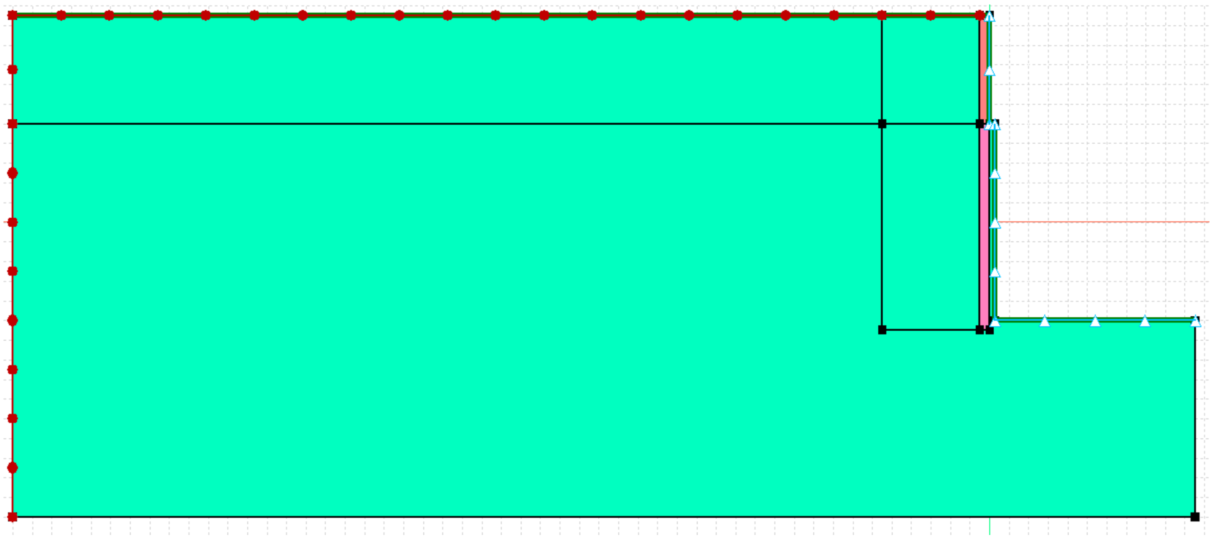

FIGURE 5: MODEL DOMAIN

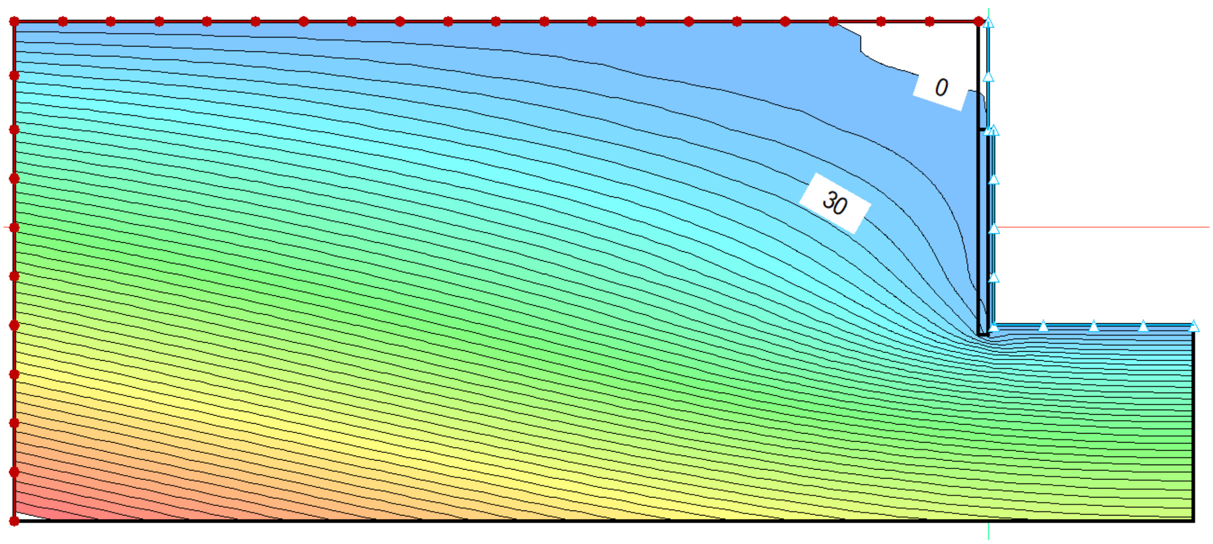

FIGURE 6: EXAMPLE MODEL OUTPUT

6.1. Modelling results

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

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

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

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

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

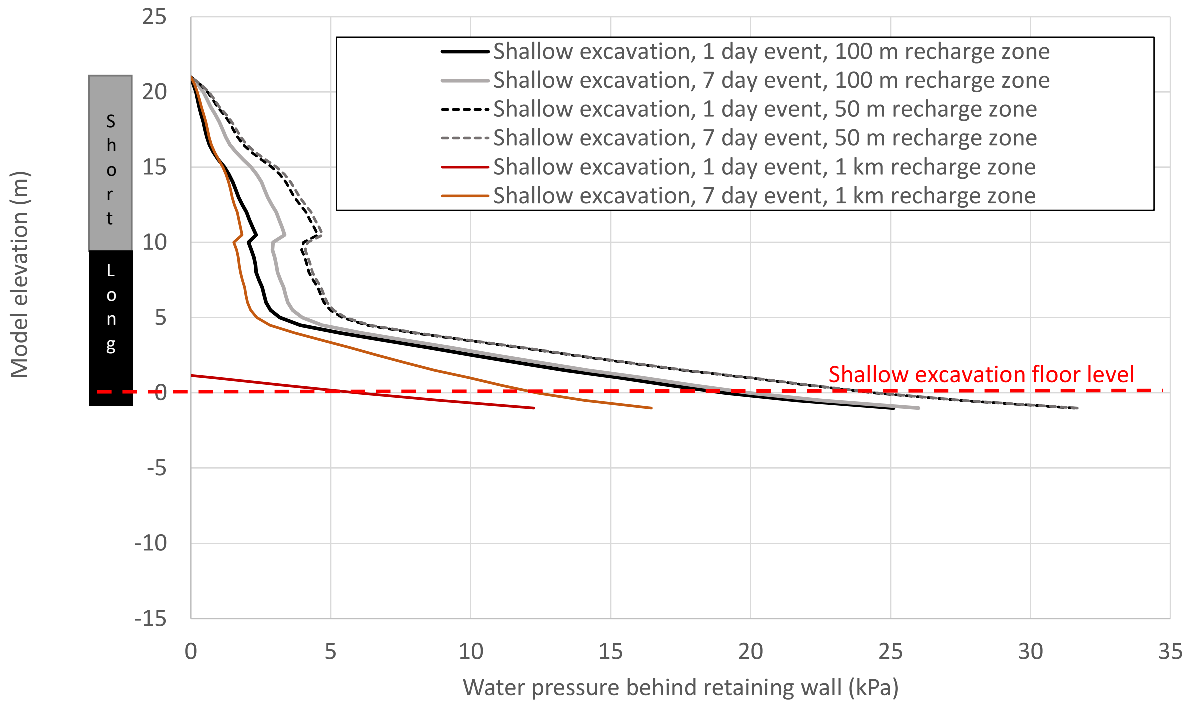

FIGURE 7: MODEL RESULTS – GROUNDWATER PRESSURE PROFILE ALONG PILED WALL – SHALLOW EXCAVATION

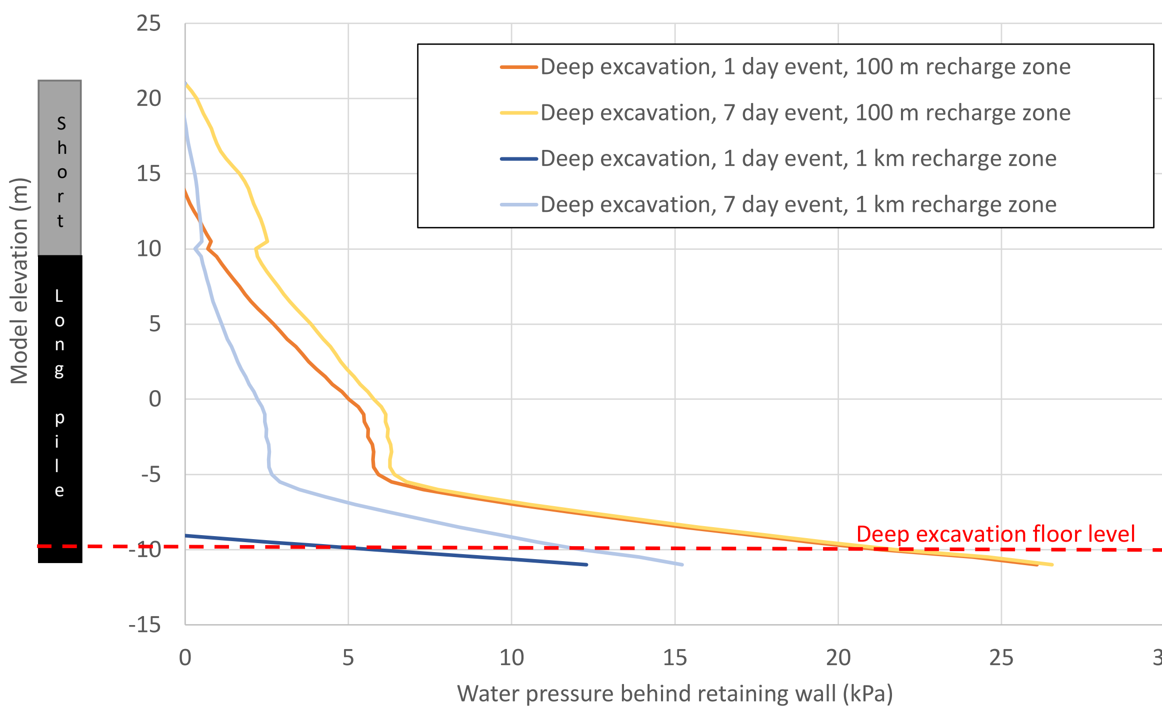

FIGURE 8: MODEL RESULTS – GROUNDWATER PRESSURE PROFILE ALONG PILED WALL – DEEP EXCAVATION

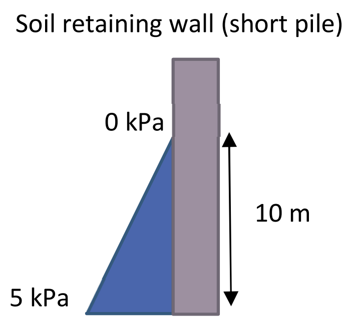

FIGURE 9: PRESSURE PROFILE DIAGRAM BASED ON MODEL RESULTS

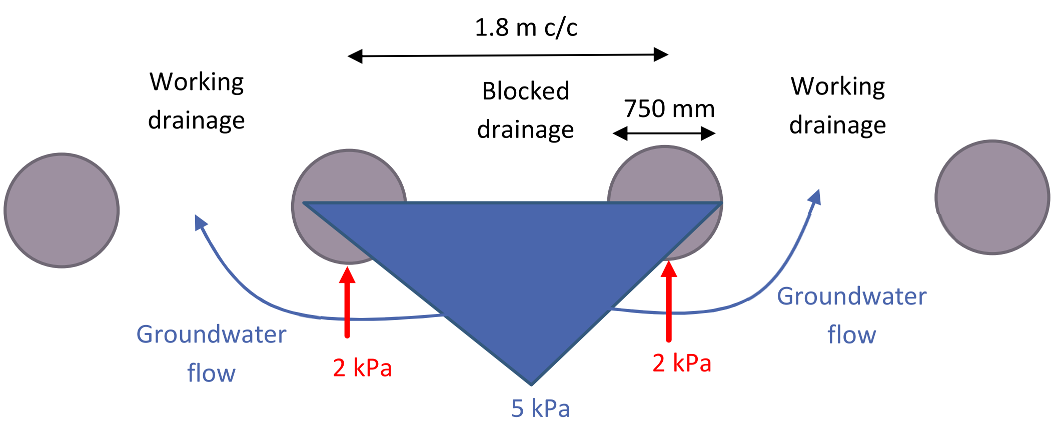

FIGURE 10: PRESSURE PROFILE DIAGRAM (IN PLAN VIEW)

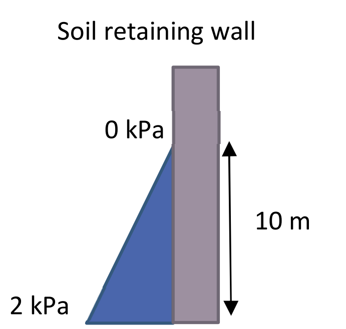

FIGURE 11: PRESSURE PROFILE TO ADOPT IN DESIGN OF SOIL RETAINING WALLS FOR EXCEPTIONAL LOAD CONDITION (GROUNDWATER) REPRESENTING BLOCKED DRAINAGE

ANNEXURE D: GROUNDWATER MODELLING REPORT

Jacobs TYPSA
Jacobs Typsa Joint Venture

Technical Memo

To	Ahmed Elsayed	Date 18 March 2022
Copies		Document ID SMWSTCTP-AFJ-BWT-SN300-ST-RPT-003030 Appendix G Annexure D
From	Ben Rotter	Revision A
Subject	Burwood North Station Groundwater Modelling – Stage 3 – Annexure D	

1. Introduction

The objective of this memorandum is to summarise groundwater modelling undertaken in support of the Stage 3 Burwood North 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 Design Stage 3 Burwood North Station Hydrogeological Assessment Report.

2. Groundwater modelling

Numerical groundwater flow models have been developed in support of the Stage 3 Burwood North Station design. The modelling objectives were to:

- Predict groundwater inflow rates to the Burwood North Station excavation
- Predict associated propagation of groundwater level drawdown

The models have been developed in the Geostudio software package, SEEP/W. SEEP/W is a finite difference modelling package for modelling groundwater flow.

Two-dimensional cross-sectional models were developed:

- Station models, which consider:
    - A cross section through the station box excavation, pedestrian adit and shaft
    - A cross section through the station cavern excavation
- Palaeochannel model, which considers:
    - A cross section running from the eastern end of the station box, through the alluvium to the east of the station

2.1. Model cross sections

The location of the model cross sections represented in the SEEP/W model is shown in Figure 1 b.

The station models follow a cross section that extends from a ridgeline in the southwest to the Parramatta River in the northeast.

The palaeochannel model follows a cross section that extends from the eastern end of the station box to a ridgeline located approximately 500 m east of the station box.

Each cross section intersects the station excavations, and was selected to provide reasonable representation of distant boundary conditions.

For the station models, the station box excavation model lies exactly on the cross section shown in Figure 1 b, while the station cavern excavation model section line runs through the cavern at the station location.

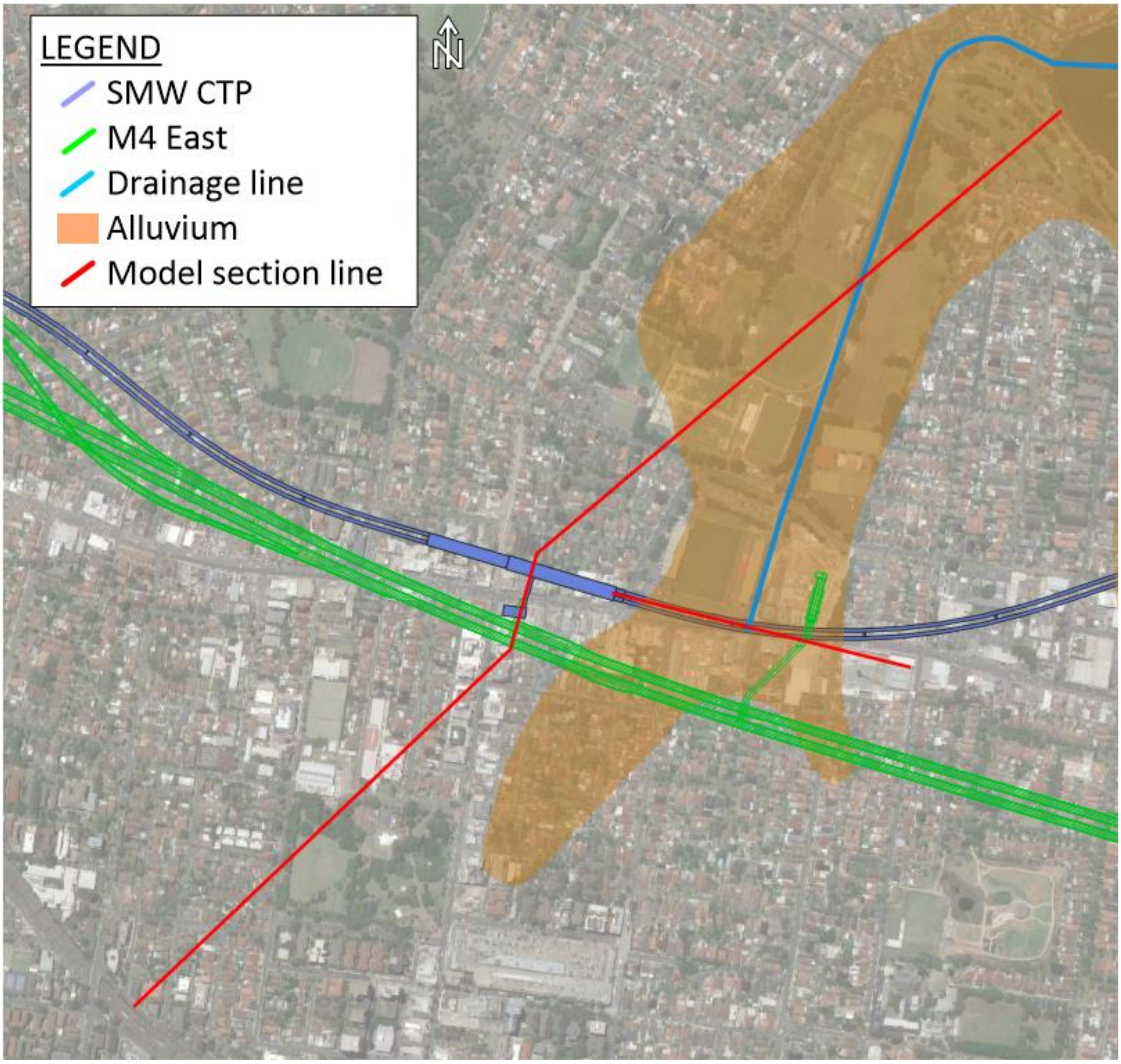

FIGURE 1 BURWOOD NORTH STATION – STATION MODEL AND PALAEOCHANNEL MODEL CROSS SECTION LOCATIONS

3. Station models

Two-dimensional cross-sectional models were developed approximately southwest to northeast through Burwood North Station and extended to appropriate boundaries. The model was calibrated to existing representative groundwater levels at Burwood North Station in steady state.

Upon achieving suitable calibration, a transient model was developed, which incorporated boundary conditions to simulate groundwater drainage associated with instantaneous ("wished-in-place") station excavations.

3.1. Model layers

At the station site, the ground profiles reported in the Geotechnical Interpretive Report were considered. These were extended at distance along the cross section.

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

The Ashfield Shale layer is represented from ground surface level to a uniform level of -3 m AHD along the entire section and is based on the level of the Ashfield Shale/Mittagong Formation interface at the approximate centre of the station. The Hawkesbury Sandstone/Mittagong Formation model layer occurs beneath the Ashfield Shale layer and extended to a depth of -50 m AHD. This base level is about 30 m below the base of the station excavation and therefore provides sufficient model thickness to enable interaction of the station excavations with the underlying groundwater system.

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

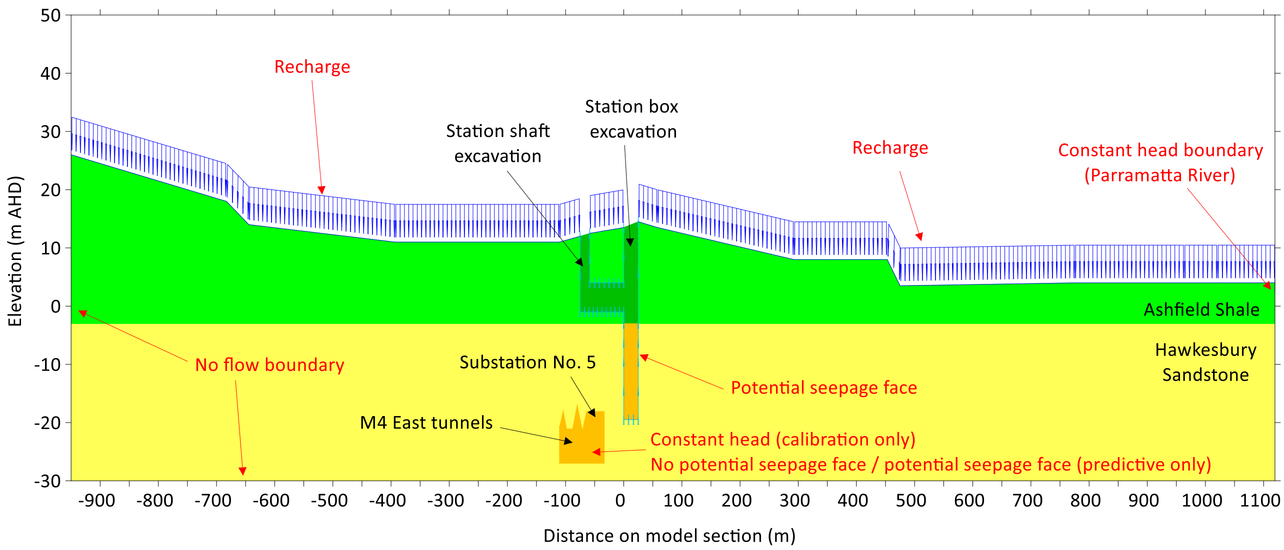

FIGURE 2 BURWOOD NORTH STATION MODEL CONDITIONS

3.2. Model conditions

3.2.1. Model layer hydrogeological properties

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

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 Design Stage 3 Hydrogeological Assessment Report).

TABLE 1 HYDROGEOLOGICAL PARAMETER VALUES APPLIED IN STATION MODELS

Parameter	Ashfield Shale	Hawkesbury Sandstone	Justification
Saturated horizontal hydraulic conductivity (m/d)	0.012	0.0173	Equivalent to 75th percentile of CTP packer testing for siltstone and sandstone intervals, respectively, as documented in hydrogeological properties annexure, Annexure B
Ratio of saturated vertical to horizontal hydraulic conductivity (-)	0.1	0.1	Based on regional literature review, as documented in hydrogeological properties annexure, Annexure B
Saturated horizontal and vertical hydraulic conductivity (m/d) applied over excavation	100	100	Applied over Sydney Olympic Park Station excavation area to represent free drainage within the excavation that would occur during excavation
Specific yield (-)	0.02	0.05	Based on regional literature review, as documented in hydrogeological properties annexure, Annexure B
Coefficient of volume compressibility (kPa^{-1})	$5.1{\times}10^{-7}$	$5.1{\times}10^{-7}$	Calculated based on specific storage values derived from regional literature review, as documented in hydrogeological properties annexure, Annexure B

3.2.2. Boundary conditions

Boundary conditions are shown in Figure 2 and included:

- For the M4 East tunnels and Substation No. 5:
  - For calibration, the groundwater level drawdown observed at the M4 East tunnels and Substation No. 5 are represented by a constant head of 0 m AHD. This boundary condition reflects groundwater levels observed in the immediate vicinity of the M4 infrastructure (refer to Section 4.3.3 of the main Hydrogeology Design Report,). Given that it is not known whether groundwater level drawdown associated with the M4 infrastructure is at (near) steady state or ongoing, this is considered a suitable approach.
  - For predictive modelling, two different predictive models were used because it is not known whether groundwater level drawdown associated with the M4 infrastructure is at (near) steady state or ongoing. One first model did not apply any boundary condition to the M4 infrastructure. This assumes that the M4 infrastructure have already induced the maximum drawdown they are capable of, and the predictive model adopts the initial groundwater levels to account for the existing drawdown. The second model assumes that the M4 infrastructure have not already induced the maximum drawdown they are capable of, and the predictive model adopts a potential seepage face boundary condition around these structures
- The piezometric pressure head analysis discussed in the main body of the Hydrogeology Design Report does not support the presence of a consistent perched water table near the station box and indicates variable hydraulic connection between the Hawkesbury Sandstone and Ashfield. For this reason, a watertable (unconfined) aquifer was modelled
- Constant head of 1 mAHD applied over a depth of 1 m from ground level, at the northeastern extent of model, representing the Parramatta River
- Recharge applied at a rate equivalent to 4% of mean annual rainfall over the whole section, except where open excavation is present. This recharge rate was arrived at during model calibration by matching modelled groundwater levels to existing conditions. The rate is slightly high, which is likely due to the particular combination of boundary conditions at this location. Lower recharge values did not provide a favourable match to observed groundwater levels for the hydraulic conductivity values adopted. The higher recharge rate (higher hydraulic conductivity for the rock) provides a more conservative estimate of groundwater inflows to the excavations

- No flow boundaries applied at base of model; and at southwestern and northeastern extents of model, except where the seepage face and constant head boundaries were applied

3.2.3. Modelling calibration and inflow prediction

The model calibration to existing groundwater levels was solved in steady state mode.

A transient model was developed that adopted initial groundwater heads based on the steady state (calibrated) model to begin the transient simulation. Transient simulation ran for a duration of 3,650 days (10 years).

The predictive transient model applied internal seepage face boundaries 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.

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

3.3. Model calibration

The model was calibrated by adjusting the recharge rate to achieve the targeted existing representative watertable as shown in Figure 3. The interpreted observed groundwater levels in the immediate vicinity of the M4 infrastructure, as discussed in Section 4.3.3 of the main Hydrogeology Design Report are also shown on the figure.

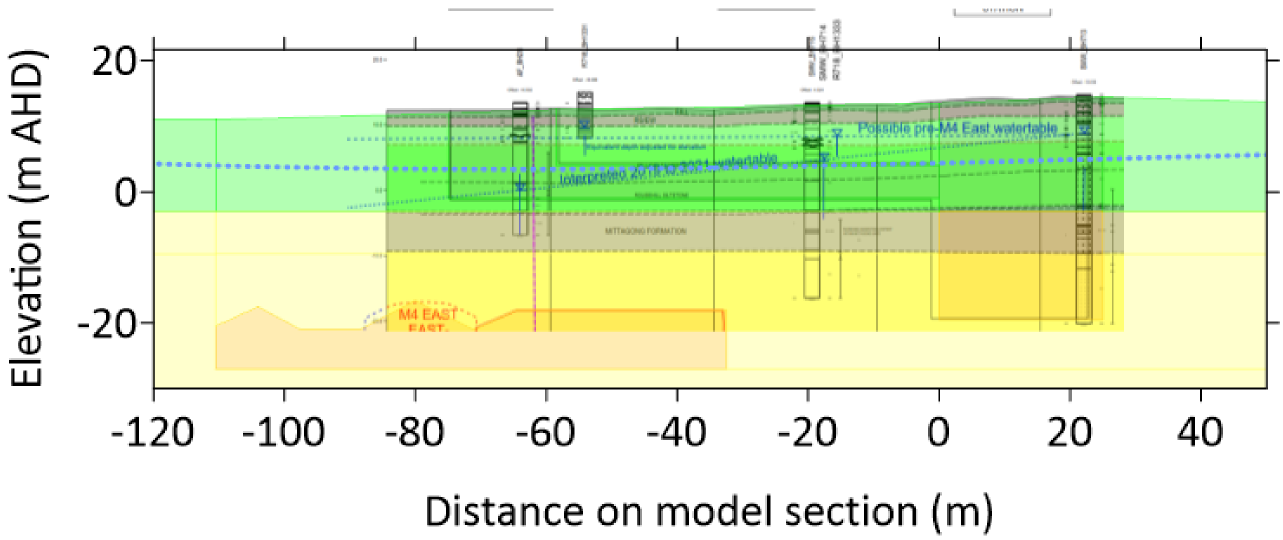

FIGURE 3 CALIBRATED WATERTABLE LEVEL (BLUE DASHED LINE)

3.4. Predictive modelling

3.4.1. Groundwater inflows

Model-predicted groundwater inflow rates to the station box and cavern excavations are shown in Figure 4. Predicted inflows to the station box excavation are up to approximately 0.85 L/s (70 m³/d) and to the cavern are up to 0.55 L/s (50 m³/d). The long-term inflows are predicted to be approximately 0.4 L/s (35 m³/d) to the station box excavation and 0.3 L/s (25 m³/d) to the cavern excavation for the adopted hydrogeological conditions.

Predicted inflows to the shaft and pedestrian adit are negligible. These excavations are underdrained by the station box.

These predictions assume that the drawdown induced by the M4 East infrastructure, as observed in January 2022, reflects the maximum drawdown that the M4 East infrastructure will induce. Should the M4 East infrastructure induce additional drawdown, groundwater inflows to the CTP excavations may differ from those noted above.

As shown in Figure 4, the modelled groundwater inflow rates vary with time. It is noted that the early time groundwater inflow rates are considered to be higher than would occur in reality under the assumed hydrogeological conditions and are considered to be elevated, in part, because the full excavation occurs instantaneously (the excavation is "wished in place") in the model. In reality, the excavation would deepen progressively, and peak groundwater inflows would be lower than those reported here.

Geotechnical interpretations indicate the presence of a potential fault zone at approximately the interface of the cavern and the station box excavations; and the presence of two potential dykes, one intersecting the western end of the cavern, and one intersecting the eastern end of the station box. Available packer test data does not indicate that the rock is of high permeability. There is a possibility that the hydraulic conductivity of rock may be relatively high in the vicinity of fault zones, dykes, joint swarms, or in other unidentified geological features. Should this be the case, groundwater inflows to the excavations may be higher than those predicted here. The potential implications of this are discussed in the main body of the Hydrogeological Assessment Report.

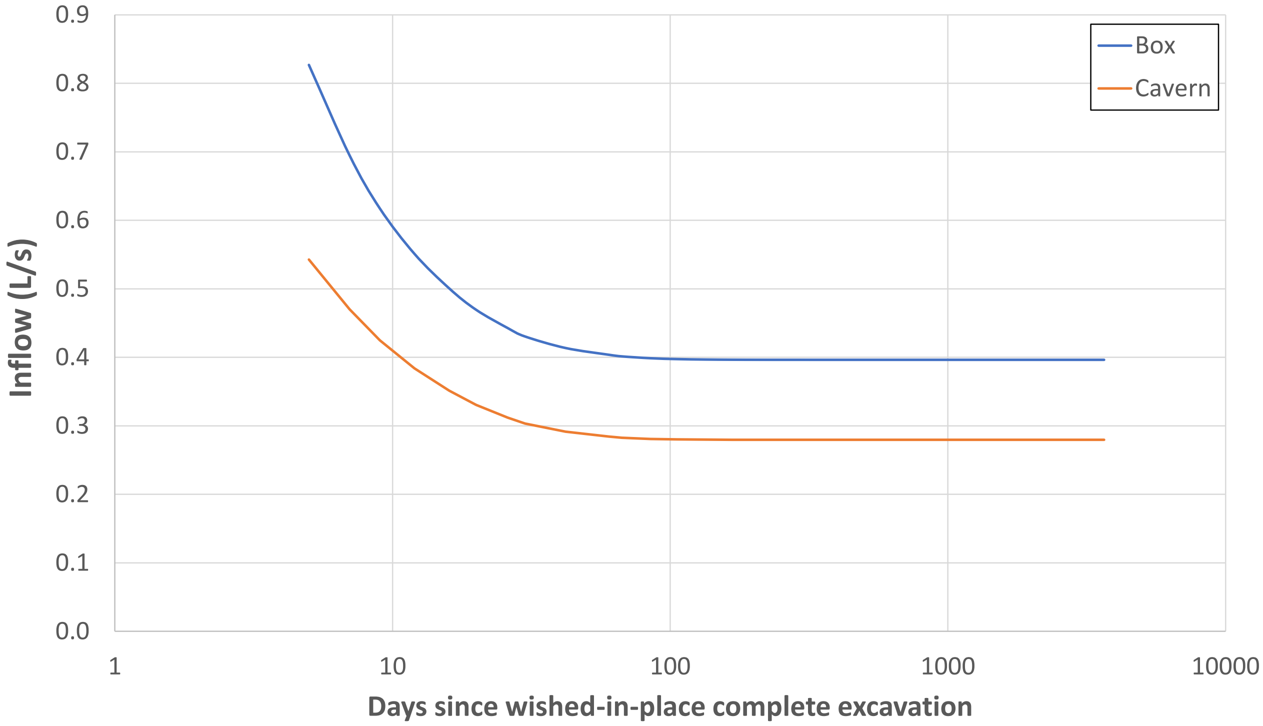

FIGURE 4 PREDICTED GROUNDWATER INFLOW RATES

3.4.2. Watertable drawdown

As noted above, it is not known whether groundwater level drawdown associated with the M4 infrastructure is at (near) steady state or ongoing.

For predictive modelling, two different predictive models were used because it is not known whether groundwater level drawdown associated with the M4 infrastructure is at (near) steady state or ongoing:

- The first model did not apply any boundary condition to the M4 infrastructure. This assumes that the M4 infrastructure have already induced the maximum drawdown they are capable of, and the predictive model adopts the initial groundwater levels to account for the existing drawdown. This scenario/model is considered more likely to be representative of actual conditions than the second scenario/model
- The second model assumes that the M4 infrastructure have not already induced the maximum drawdown they are capable of, and the predictive model adopts a potential seepage face boundary condition around these structures

The modelled initial watertable and the predicted long-term watertable (two years after wished in place excavation) surfaces are shown in Figure 5 for the first model and Figure 6 for the second model. The predicted long-term watertable at 10 years after wished in place excavation is the same as the predicted long-term watertable at two years after wished in place excavation (i.e., near-steady state conditions have been reached after two years) for both models. Drawdown is greater in the second model, because the M4 East infrastructure are deeper and cause greater drawdown than the CTP excavations alone.

Figure 7 shows the predicted drawdown of the watertable for both models.

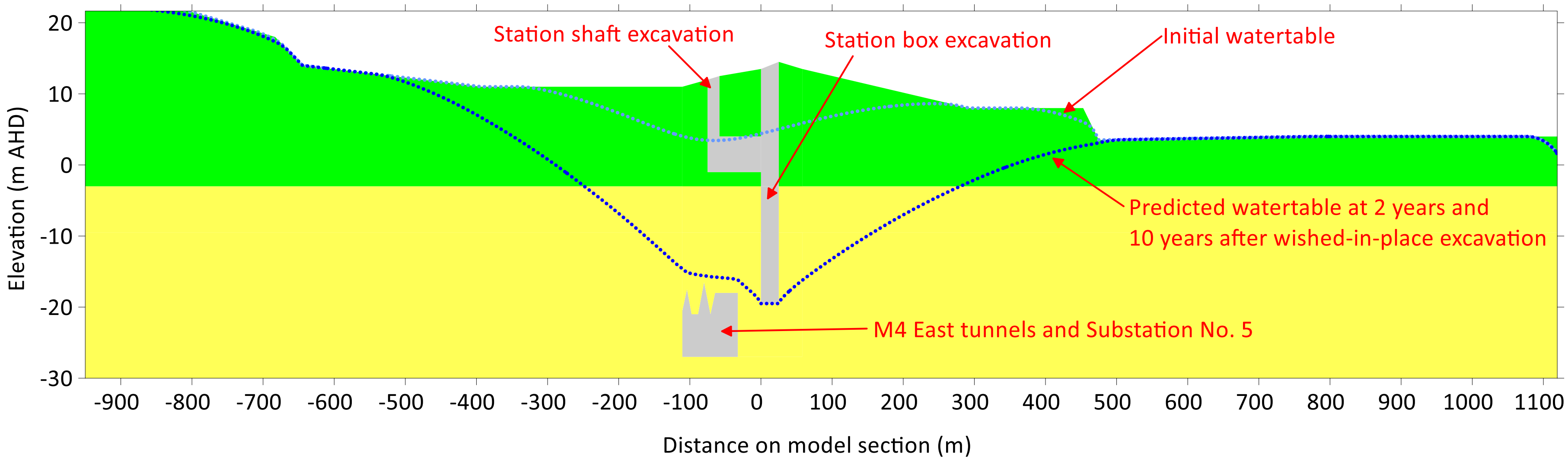

FIGURE 5 INTERPRETED PRE-CTP-WORKS EXCAVATION (INITIAL) WATERTABLE AND PREDICTED WATERTABLE AFTER (2 AND 10 YEARS SINCE) EXCAVATION, WITH SCENARIO IN WHICH DRAWDOWN AT M4 EAST INFRASTRUCTURE REACHED STEADY STATE PRIOR TO 2022

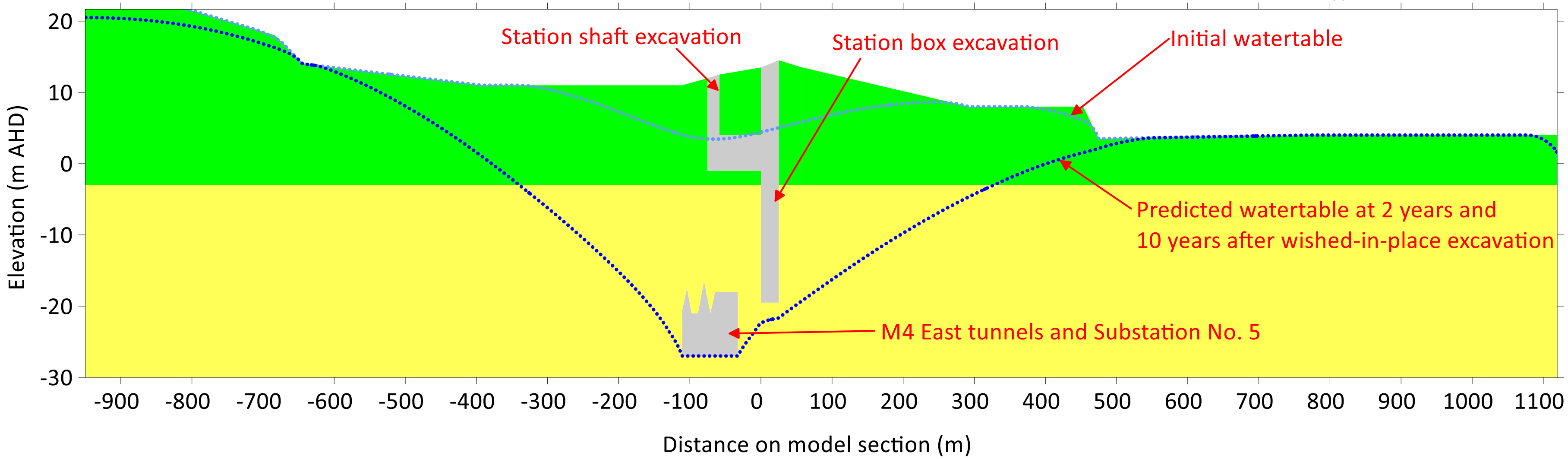

FIGURE 6 INTERPRETED PRE-CTP-WORKS EXCAVATION (INITIAL) WATERTABLE AND PREDICTED WATERTABLE AFTER (2 AND 10 YEARS SINCE) EXCAVATION, WITH SCENARIO IN WHICH DRAWDOWN AT M4 EAST INFRASTRUCTURE IS ONGOING IN 2022

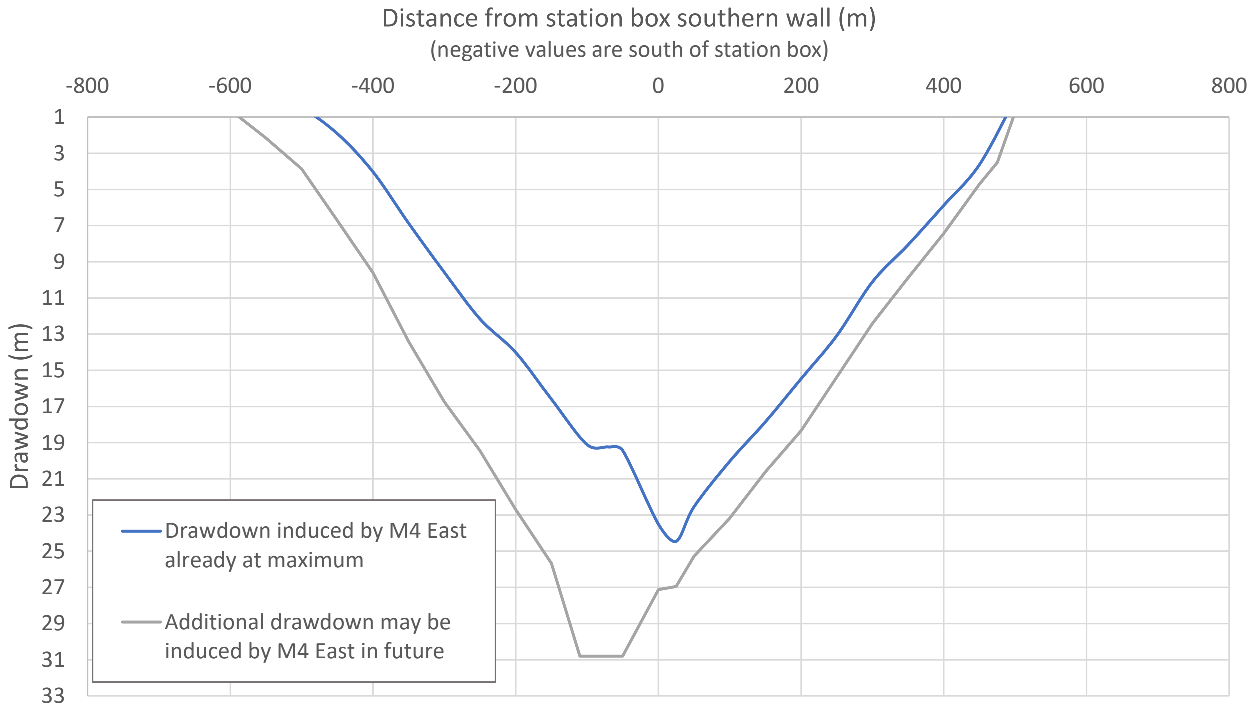

FIGURE 7 PREDICTED DRAWDOWN OF THE WATERTABLE

4. Palaeochannel model

A two-dimensional cross-sectional model was developed approximately eastwards along the alignment, from the eastern end of Burwood North Station, through the alluvium to the east, extending to an appropriate eastern boundary.

The model was calibrated to groundwater levels in steady state under assumed existing conditions, with predictive modelling adopting a transient approach that considered the presence of the station excavation.

4.1. Model layers

At the station site and across the palaeochannel, the ground profiles reported in the Geotechnical Interpretive Report were adopted.

Three hydrogeological units are represented in the model: Alluvium, Ashfield Shale and Hawkesbury Sandstone.

Fill and residual soil units are not included in the model because, at the station site, the water table is situated below these units, and is inferred to similarly lie below these units at the peripheries of the palaeochannel.

The Mittagong Formation is not explicitly represented in the model and is instead represented by the Hawkesbury Sandstone unit. This approach was adopted because the Mittagong Formation is thin at the station (e.g. about 5 m thick) and is characteristically similar to the Hawkesbury Sandstone in its hydrogeological properties.

The Hawkesbury Sandstone/Mittagong Formation model layer occurs beneath the Ashfield Shale layer and extended to a depth of -40 m AHD. This base level is about 20 m below the base of the station excavation and therefore provides sufficient model thickness to enable interaction of the station excavations with the underlying groundwater system.

The model layers and boundary conditions are shown in Figure 8.

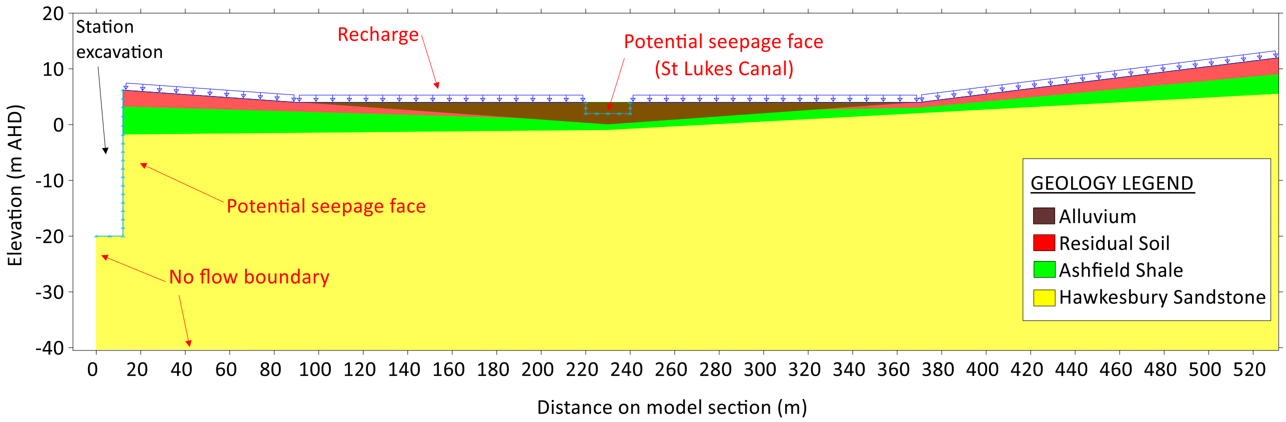

FIGURE 8 BURWOOD NORTH STATION – PALAEOCHANNEL MODEL CONDITIONS

4.2. Model conditions

4.2.1. Modelled flow conditions

Saturated flow conditions were simulated.

Representation of unsaturated flow within the fill and residual soil was not required because these units are relatively thin, unsaturated at available monitoring locations, and are not significant with respect to the groundwater flow regime.

Representation of only saturated flow within the alluvium provides a conservative modelling basis with regard to predicted desaturation/depressurisation of groundwater in the alluvium.

A radial model was adopted for this section, since a two-dimensional cross sectional model through the short (eastern) end of the station box would significantly overestimate the influence of the excavation on groundwater flow conditions to the east of the station. The adopted radius of the station box in the model is 12 m, which is equal to half the width of the station box. Seepage to the station box occurs in the predictive model over this zone.

4.2.2. Model layer hydrogeological properties

Hydrogeological parameter values applied in the models are shown in Table 2. 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 Design Stage 3 Hydrogeological Assessment Report).

There are no hydrogeological test data for the alluvium or residual soils. Available borehole logs for the alluvium (R255 BH104A, drilled for the M4 RTA investigations) record the alluvium as a sandy clay. Parameter values consistent with this material description were adopted for the alluvium. Residual soils are recorded as clay, which is typical of residual soils derived from Ashfield Shale. They are assumed to be of relatively low permeability (about half the horizontal hydraulic conductivity value of the Ashfield Shale has been adopted) – this provides conservative estimates of drawdown, since rainfall recharge is less able to re-saturate the formation.

TABLE 2 HYDROGEOLOGICAL PARAMETER VALUES APPLIED IN PALAEOCHANNEL MODEL

Parameter	Alluvium	Residual Soil	Ashfield Shale	Hawkesbury Sandstone	Justification
Saturated horizontal hydraulic conductivity (m/d)	0.1	0.005	0.012	0.0173	Alluvium considered to comprise sandy clay based on available borehole logs. Residual soils comprise clay and are assumed to be relatively low permeability (about half the value of the Ashfield Shale) – this is conservative from a drawdown perspective. Siltstone and sandstone values equivalent to 75th percentile of CTP packer testing for siltstone and sandstone intervals, as documented in hydrogeological properties annexure, Annexure B
Ratio of saturated vertical to horizontal hydraulic conductivity (-)	0.5	0.1	0.1	0.1	Based on regional literature review, as documented in hydrogeological properties annexure, Annexure B
Specific yield (-)	0.06	0.02	0.02	0.05	Based on regional literature review, as documented in hydrogeological properties annexure, Annexure B
Coefficient of volume compressibility (kPa^{-1})	1.0×10^{-5}	5.1×10^{-7}	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

4.2.3. Boundary conditions

Boundary conditions are shown in Figure 8 and included:

- Potential seepage face boundary condition at St Lukes Canal, representing potential baseflow contribution (seepage of groundwater) to the canal surface waters
- Potential seepage face boundary condition at the station excavation (predictive model only)
- Since the rainfall recharge will have a significant influence on predicted groundwater level drawdown, and actual rainfall recharge in the area is unknown, models with two different rates were adopted:
    - The first model considered a recharge a rate equivalent to 4% of mean annual rainfall over the whole section, except where open excavation is present. This recharge rate was arrived at during calibration of the station excavation model and provides a less conservative scenario
    - The second model considered a recharge a rate equivalent to 1% of mean annual rainfall over the whole section, except where open excavation is present. This is considered a low rate for urban settings in the Sydney Basin on Ashfield Shale/Hawkesbury Sandstone
- No flow boundaries applied at base of model, and at eastern extent of model, except where the seepage face and constant head boundaries were applied

4.3. Predictive modelling

The initial groundwater level, prior to CTP excavation works, was modelled based on steady state conditions in the absence of the excavation (i.e. no potential seepage face boundary condition at the proposed excavation location).

A transient model was developed that adopted initial groundwater heads based on the steady state (calibrated) model to begin the transient simulation. Transient simulation ran for a duration of 3,650 days (10 years).

The predictive transient model applied internal seepage face boundaries around the eastern end of the station excavation as represented in the model cross section (see Figure 8).

The modelled initial watertable and the predicted long-term watertable (two years after wished in place excavation) surfaces are shown in Figure 9 and Figure 10 for, respectively, the models with recharge of 4% and 1% of mean annual rainfall.

The predicted long-term watertable at 10 years after wished in place excavation is the same as the predicted long-term watertable at two years after wished in place excavation (i.e., near-steady state conditions have been reached after two years) for both models. Drawdown is greater in the second model, because the M4 East infrastructure are deeper and cause greater drawdown than the CTP excavations alone.

The models indicate that the alluvium could be partially desaturated across its western extent (west of St Lukes Canal) due to the CTP station excavation; and that, under low rainfall recharge conditions, the alluvium could be partially desaturated across much of its extent.

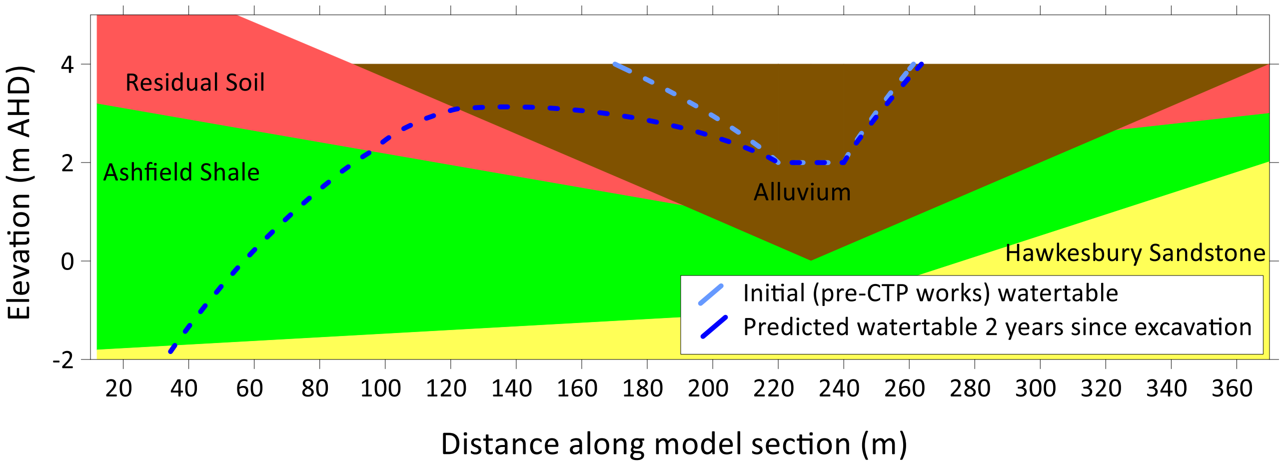

FIGURE 9 INTERPRETED PRE-CTP-WORKS EXCAVATION (INITIAL) WATERTABLE AND PREDICTED WATERTABLE AFTER (2 AND 10 YEARS SINCE) EXCAVATION FOR RECHARGE AT 4% OF MEAN ANNUAL RAINFALL

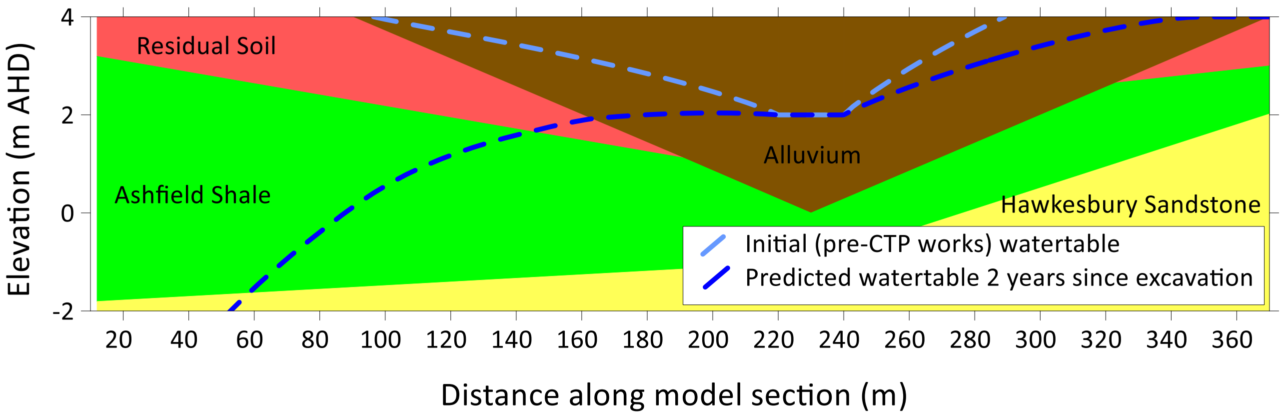

FIGURE 10 INTERPRETED PRE-CTP-WORKS EXCAVATION (INITIAL) WATERTABLE AND PREDICTED WATERTABLE AFTER (2 AND 10 YEARS SINCE) EXCAVATION FOR RECHARGE AT 1% OF MEAN ANNUAL RAINFALL

4.4. Uncertainty

It should be noted that there is significant uncertainty in the predicted groundwater level drawdown in the alluvium. The nature, extent and depth of the alluvium is unknown. Numerous assumptions have been made with regard to hydrogeological conditions. In the case that actual condition vary from those adopted in this modelling, groundwater levels may be significantly different to those predicted here.

Various conditions have been modelled to consevratively assess potential groundwater level drawdown in the alluvium.