G3 56 Delhi Road
North Ryde NSW 2113
P +61-2 9812 5000
F +61-2 9812 5001
E mailbox@psm.com.au
www.psm.com.au

Our Ref: PSM4580-006L Rev 1

9 May 2022

Development Manager
Gurner
3 Goldman Lane
Double Bay NSW 2028
michael@gurner.com.au

Attention: Michael Skala

Dear Michael

**RE: 12 HASSALL STREET, PARRAMATTA
GROUNDWATER ASSESSMENT**

1. Introduction

This letter presents the results of PSM's groundwater assessment for the proposed development at 12 Hassall Street, Parramatta (**the Site**). This groundwater assessment has been prepared in accordance with our fee proposal (ref. PSM4580-005L REV 1, dated 31 January 2022) and provides an assessment of the potential impact of the proposed development on the groundwater resource at the Site.

In late September 2021 PSM undertook a due diligence geotechnical investigation at the Site (ref. PSM4580-003L, dated 12 October 2021). During our investigation, only the western half of the site was accessible due to the presence of existing buildings on site covering the eastern half. Accordingly, our boreholes were located in the western half of the site.

PSM have reviewed the following documentation for geotechnical and groundwater investigations undertaken by other consultants at the Site and at nearby sites:

- Architectural drawing set by PTW Architects, not dated (drawings DC-00-0000 to DC-110-1500)
- Geotechnical information we have in the vicinity of the Site and at nearby sites.

PSM have prior experience with basement excavations and groundwater assessments at nearby sites.

Based on the above, and our investigation in late September 2021, we note the following about the proposed re-development:

- The proposed site comprises Lot 156 (DP1240854)
- The total site plan area is approximately 2,050 m^2
- The development proposes the construction of a 61 storey mixed use building, comprising:
  - 5 levels of basement parking (not including the B1 mezzanine and a pit maintenance level) containing 175 car parking spaces
  - A 4 storey podium providing commercial floor space and residents' amenities
  - Level 4 will provide residential amenities including a podium terrace with a swimming pool
  - A residential tower (levels 5 to 59) providing 391 build-to-rent apartments
  - Level 60 will provide residential amenities including a rooftop terrace, pool and lounge.

- The finished ground floor level will be RL 10.55 m AHD
- The ground conditions in the boreholes near the site show an alluvial soil profile of up to approximately 8 m thick overlying shale bedrock
- The PSM boreholes during the September 2021 investigation encountered soils of thickness 6 m.

Other details of the proposed re-development (i.e. loads, basement levels, etc.) are not known by PSM.

2. Factual Data

2.1 Geological Setting and Ground Model

PSM have previously completed a due diligence geotechnical investigation at the Site in September 2021. The geological setting and ground model at the Site is summarised in our report detailing the investigation (ref. PSM4580-003L, dated 12 October 2021).

2.2 Groundwater levels at the Site

2.2.1 2021 – Present Investigation by PSM

Four (4) boreholes (denoted BH01 to BH04) were advanced during the site investigation completed by PSM in September 2021. Groundwater was not encountered during the augering of these boreholes during the investigation. Following the drilling of BH03, a piezometer (PZ01) was installed. A groundwater monitoring device (HOBO) was placed in the piezometer to record water levels.

Table 1 presents a summary of the PSM monitoring well at the Site.

Appendix A presents the locality plan of the PSM monitoring well at the Site.

Table 1 – PSM Monitoring Wells at 12 Hassall Street

Location	Monitoring Well ID	Water Level Measurement Date	Standing Water RL (m AHD)	Bottom of Well RL (m AHD)	Screened Water Bearing Zone
12 Hassall Street	BH03	17/02/2022	-2.8	-11.3	Residual, Weathered Shale, & Fractured Shale

Appendix C presents the results of continuous groundwater level monitoring in BH03 between 24 September 2021 and 17 February 2022. Rainfall data was taken from Australian Government Bureau of Meteorology (BOM), station 066124 Parramatta North (Mansons Drive).

2.2.2 Other known monitoring wells

We are aware of three (3) historic groundwater monitoring wells at the Site. Table 2 presents a summary of these other known historic monitoring wells at the Site.

Appendix A presents the locality plan of the other known historic monitoring wells at the Site.

Table 2 – Other Known Historic Monitoring Wells at 12 Hassall Street

Location	Monitoring Well ID	Water Level Measurement Date	Standing Water RL (m AHD)	Bottom of Well RL (m AHD)	Screened Water Bearing Zone
12 Hassall Street	HW02s	11/07/2017	10.3	8.1	Alluvium / Residual
	HW02i	11/07/2017	5.4	4.9	Weathered Shale
	HW02d	11/07/2017	-4.4	-15.1	Fractured Shale

Given the relatively little standing groundwater (pressure head) within HW02i compared to both HW02s and HW02d, a capillary break (zone of desaturation) is inferred to exist both above and below the HW02i instrument. Therefore, it is assessed that the vertical hydraulic conductivity of the materials overlying HW02i are lower than the hydraulic conductivity at HW02i.

2.3 Groundwater levels around the Site

Numerous historic monitoring wells located close to the Site are known to PSM. Table 3 presents a summary of the other known historic monitoring wells around the Site.

Appendix B presents the locality plan of known historic monitoring wells around the Site.

Table 3 – Known Historic Monitoring Wells Around the Site

Location	Monitoring Well ID	Water Level Measurement Date	Standing Water RL (m AHD)	Bottom of Well RL (m AHD)	Screened Water Bearing Zone
189 Macquarie Street (130 m from the Site)	HB103	15/11/2011	2.6	-1.0	Alluvium / Residual, Fractured Shale
	HB104	15/11/2011	-0.9	-1.0	Residual, Weathered Shale, & Fractured Shale
191-193 Macquarie Street (130 m from the Site)	HB01s	11/07/2017	1.8	-1.3	Alluvium / Residual
	HW01d	11/07/2017	-3.4	-15.1	Fractured Shale
290 m from the Site	HW101	26/07/2017	1.9	-0.6	Alluvium / Residual
290 m from the Site	HW102	26/07/2017	1.6	-5.3	Weathered Shale & Fractured Shale
420 m from the Site (Near Parramatta River)	HW103	26/07/2017	0.7	-2.9	Fractured Shale
240 m from the Site	HW104	26/07/2017	2.6	-7.4	Fractured Shale
240 m from the Site	HW105	26/07/2017	2.9	-2.0	Alluvium / Residual

2.4 Known basements around the Site

2.4.1 189 Macquarie Street, Parramatta

PSM understand that during 2017, groundwater investigations and assessments during the redevelopment at 189 Macquarie Street, Parramatta were completed.

PSM understand the following regarding the development at 189 Macquarie Street:

- The development consisted of five (5) levels of basement, with two sub-terrain plant rooms below basement level 5 at a finished floor level of RL – 13.8 m.
- The basement plan area was approximately 5,200 m^2.
- A drained basement design was implemented.
- The basement was excavated and constructed prior to the construction and installation of the monitoring wells.
- Manual and automated volume measurements measured an average discharge flow between 25 m^3/day and 30 m^3/day.

2.4.2 Other nearby basements

On 14 February 2022 a PSM geotechnical engineer undertook a survey of the basements from selected surrounding buildings. The following observations were made:

- 60 Station St contains 2 basement levels
- 9 Hassall St contains 6 basement levels
- 14 Hassall St contains 3 basement levels
- 13-15 Hassall St likely contains 4-5 basement levels (unable to be confirmed).

Other buildings in the area are also likely to have multiple levels of basements.

The retaining walls inside the basements of 60 Station St and 9 Hassall St appeared to be soldier pile walls, i.e., regularly spaced piles with shotcrete and possibly steel mesh between the piles. Based on this we consider it likely that these basements are drained rather than tanked, noting that tanked basements require substantially thicker retention than shotcrete infill can provide. The approximate location of the 60 Station St and 9 Hassall St basements are shown on Appendix B.

It was not obvious what retention or groundwater systems were present in the 14 Hassall St basement.

2.5 Hydraulic Conductivity tests at the Site

PSM are aware of rising head (slug) tests in each of the other known historic monitoring wells at the Site. The slug tests were used to provide an order of magnitude estimate of the horizontal hydraulic conductivity (K) of the aquifer material immediately surrounding the monitoring well screen.

Based on the observed geology and water bearing zones, the shallow alluvium / residual and weathered shale water bearing zones were assumed to be unconfined, and the deeper fractured shale aquifer was assumed to be confined. Bower-Rice (1976) and Hvorslev (1951) solutions were implemented in software package AQTESOLV for both unconfined and confined aquifers. Table 4 presents the estimated hydraulic conductivity averaged over the two solution methodologies and all tests conducted in each monitoring well.

Table 4 – Average Estimated Horizontal Hydraulic Conductivity (m/day) at the Site

Monitoring Well ID	Average Estimated Hydraulic Conductivity (m/day)	Water Bearing Zone	Assumed Aquifer Confinement Condition
HW02s	4.10E-01	Alluvium / Residual	Unconfined
HW02i	2.22E-02	Weathered Shale	Unconfined
HW02d	2.85E-01	Fractured Shale	Confined

2.6 Hydraulic Conductivity of Shale in Parramatta

2.6.1 Around the Site

PSM are aware of rising head (slug) tests in each of the known historic monitoring wells installed at 191-193 Macquarie Street. The same assumptions and procedures for testing and interpretation were applied as those conducted at 12 Hassall Street.

PSM are aware of slug tests in five (5) other historic monitoring wells around the Site which estimate horizontal hydraulic conductivity. The assumptions and interpretation for these tests are unknown to PSM.

Table 5 presents the average estimated hydraulic conductivity for testing undertaken around the Site in each historic monitoring well.

Table 5 – Average Estimated Horizontal Hydraulic Conductivity (m/day) Around the Site

Monitoring Well ID	Location	Average Estimated Hydraulic Conductivity (m/day)	Water Bearing Zone	Assumed Aquifer Confinement Condition
HW01s	191-193 Macquarie Street	5.48E-02	Alluvium / Residual	Unconfined
HW01d		6.93E-03	Fractured Shale	Confined
HW101	290 m from the Site	2.90E-01	Alluvium / Residual	Unknown
HW105	240 m from the Site	2.00E-02	Alluvium / Residual	Unknown
HW102	290 m from the Site	5.90E-01	Weathered Shale & Fractured Shale	Unknown
HW103	420 m from the Site (Near Parramatta River)	6.00E-03	Fractured Shale	Unknown
HW104	240 m from the Site	3.00E-02	Fractured Shale	Unknown

2.6.2 General

Ashfield shale hydraulic conductivity is typically 10^{-7} to10^{-9} m/s (10^{-2} to 10^{-4} m/day) in fresh shale, and 10^{-6} to10^{-9} m/s (10^{-1} to 10^{-4} m/day) in weathered shale (Hewitt, P. 2012, Groundwater Control for Sydney Rock Tunnels). These estimates of conductivity are routinely used to estimate flows into excavations in Sydney rock tunnels.

3. Conceptual Groundwater Model

3.1 Conceptual Groundwater Model of the Site

The conceptual groundwater model comprises water bearing zones within the different geological units at the Site. From the site investigation undertaken by PSM in September 2021 and other known historic monitoring wells, the conceptual groundwater model of the Site comprises:

- A perched water table in the alluvium / residual material (observed approximately at RL 10 m in 2017 in historic monitoring well HW02s). The extent and thickness of this perched water table is unknown, especially within the eastern portion of the Site, where no investigations have been undertaken due to existing structures.
- Transient groundwater flows through fractures in the weathered and fresh shale. The intensity and aperture of the fractured zones in the shale can greatly vary spatially, both laterally and vertically, and fractured zones can be discontinuous.
- The permanent groundwater in the shale unit (observed approximately at RL-3 m in 2022 in PSM monitoring well BH03 and at RL -4 m in 2017 in historic monitoring well HW02d). The permanent groundwater level observed at the Site is affected by the drained basement systems from neighbouring buildings adjacent to and in the vicinity of the Site. These neighbouring drained basements may be causing the groundwater table at the Site to be below its natural level. Nevertheless, we consider that it is unlikely that in the future the groundwater will return to the natural level as this would require the drained basements to stop removing water from the aquifer.

3.2 General Conceptual Groundwater Model of Parramatta

The general conceptual groundwater model of Parramatta comprises:

- Parramatta River, located approximately 420 m from the Site, provides a control for groundwater levels around Parramatta. In general, the groundwater at Parramatta River varies between RL 1 m AHD and RL 3 m AHD.
- Other monitoring data in and around the Paramatta CBD indicate water levels between 4 m and 7 m below ground level, which at the Site would indicate natural groundwater levels between RL 3 m AHD and RL 6 m AHD.
- The natural flow direction from the Site is north-east towards Parramatta River, which flows out into Sydney Harbour.
- Groundwater levels observed in the known historic monitoring wells around the Site vary spatially and are also dependent on the construction stage (e.g. pre-basement construction or post-basement construction) at the time of measurement.

4. Groundwater Assessment and Advice

4.1 Groundwater Assessment During Construction

Numerous groundwater considerations must be made during construction of the proposed basements.

In this assessment, we have assumed that the perched water table observed in the alluvium / residual layer will be cut-off during excavation works by means of an impermeable cut-off retaining wall structure down to the shale to prevent seepage into the excavation.

Given that the current permanent water table in the shale is observed approximately at RL -3 m AHD, and the assumed basement finished floor level is at RL -10 m AHD, transient inflows from the weathered and fractured shale during construction works are expected to occur. This will result in temporary drawdown of the groundwater within the shale during construction.

Disposal of the groundwater to the stormwater network during construction of the basement would be subject to quality assessment outside the scope of this letter.

Steady-state analytical modelling was undertaken to estimate groundwater influx rate and to predict the groundwater drawdown as a result of the drained basement excavation during construction. Steady-state modelling was done in accordance with Marinelli and Niccoli (2000), which makes the following simplifications and assumptions:

- The proposed basement excavation is modelled as an equivalent area circular cylinder with steady-state, axisymmetric, horizontal radial flow through the unconfined aquifer.
- Groundwater flow occurs through two zones separated by a “no-flow boundary”:
  - Zone 1 captures horizontal flow to the basement walls
  - Zone 2 captures flow from beneath the base of the basement excavation.
- Uniformly distributed recharge at the water table.

Additionally, based on the sedimentary nature of the aquifer formation, the vertical hydraulic conductivity (K_v) was assumed to be an order of magnitude lower than the horizontal hydraulic conductivity (K_h) in Zone 2.

The calculation was completed assuming full drawdown to the assumed finished floor level. A groundwater level of RL -2 m AHD was conservatively assumed based on known current groundwater levels. The basement plan area was measured from the provided Architectural drawing set by PTW Architects.

Monthly rainfall data was obtained from the Australian Government Bureau of Meteorology (BOM), station 67019 (Prospect Reservoir), from a weathered station approximately 8km from the site. The historic mean annual rainfall (based on monthly observations) from a period between 1887 and 2021 was 878.6 mm. This converted to an assumed average daily rainfall of 0.00241 m/day. Based on the high density localised setting, a recharge rate of 1% of the rainfall was assumed.

Table 6 provides a summary of input parameters used for the model.

Table 6 – Analytical model input parameters

Parameter	Value	Unit
Basement plan area	1750	m^2
Equivalent circular radius, r_p	23.6	m
Recharge rate, W	2.41E-05	m/day
Target dewatering level	-10	m RL
Groundwater level, H_0	-2 (1)	m RL
Initial saturated thickness above base of excavation, h_0	8 (2)	m
Saturated thickness above base of excavation at radius r_p, h_p	0	m
Depth of water above target dewatering level within final excavation, d	0	m

1 Groundwater level was at RL 5 m AHD for sensitivity Case 4, Case 5a, and Case 5b.

2 Initial saturated thickness above the base of excavation is 15 m for sensitivity Case 4, Case 5a, and Case 5b.

From the geotechnical investigation completed in September 2021 by PSM, it is assumed that the dewatering zone (between RL -2 m and RL -10 m) is within weathered shale and fractured shale. Different estimates for the hydraulic conductivity within the dewatering zone were used to estimate the groundwater influx and predict the groundwater drawdown.

Table 7 provides a summary of the estimated groundwater influx due for different values of hydraulic conductivity. Figure 1 presents a plot of the groundwater drawdown profile for the different cases determined in accordance with Marinelli and Niccoli (2000).

Table 7 – Estimated groundwater influx

Case	Hydraulic Conductivity, K_h (m/day)		Drawdown radius, r_0 (m)	Groundwater influx, Q (m^3/day)
	Zone 1	Zone 2		
Design Cases				
Case 1 – Average K_h around the Site (without outliers)	1.43E-02	1.43E-02	162	5.4
Case 2 – Scaled solution from observed groundwater inflow at 189 Macquarie Street	1.75E-02	1.75E-02	175	6.4
Case 3a – Average K_h around the Site: • In weathered and fractured shale for Zone 1 • In fractured shale for Zone 2	1.57E-01	8.20E-02	419	32.8
Case 3b – Geomean K_h around the Site: • In weathered and fractured shale for Zone 1 • In fractured shale for Zone 2	4.09E-02	2.44E-02	243	10.3
Sensitivity Cases				
Case 4 – Case 2 with initial groundwater at RL 5 m	1.75E-02	1.75E-02	286	14.0
Case 5a – Case 3a with initial groundwater at RL 5 m	1.57E-01	8.20E-02	711	74.9
Case 5b – Case 3b with initial groundwater at RL 5 m	4.09E-02	2.44E-02	404	23.2

It is clear that the groundwater volume estimates are sensitive to the assumed hydraulic conductivity, which is used to iteratively calculate the drawdown extent (r_0) at the defined groundwater level. The larger the radius of drawdown (primarily defined by the hydraulic conductivity through iteration calculations), the amount of groundwater inflow estimated increases rapidly.

This hydraulic conductivity testing at HW02d and HW102 suggests that these monitoring wells have intercepted a more permeable fracture zone in the shale compared to the other nearby boreholes intercepting fractured shale or the general Parramatta hydraulic conductivity in the literature. The hydraulic conductivity of the fractured shale water bearing zone is predominantly controlled by the secondary porosity (flow through fractures), which is influenced by the intensity and aperture of the fractured zone through the shale profile. The intensity and aperture of the fractured zones can greatly vary spatially, both laterally and vertically, and fractured zones can be discontinuous. As such, the hydraulic conductivity values determined from testing within these two monitoring wells are considered outliers unlikely to be representative of the shale aquifer conditions throughout the Site. Case 1 excludes the outliers in the average hydraulic conductivity used for the analytical solution.

Case 1 presents a reasonable steady state analytical solution based on appropriate hydraulic conductivity testing and typical conductivity values for shale.

Case 2 presents a solution obtained by scaling the solution for the nearby basement at 189 Macquarie Street, which was based on observation of groundwater inflow rates. It is assumed that the aquifer average hydraulic conductivity is consistent between 189 Macquarie Street and 12 Hassall Street.

Case 3a is a design case which presents an analytical solution which takes different average (arithmetic average) hydraulic conductivity values in Zone 1 and Zone 2 of the model. This case provides a reasonable upper-bound estimate of the long-term average seepage risks.

Case 3b is a design case which presents an analytical solution which takes different geomean (geometric average) hydraulic conductivity values in Zone 1 and Zone 2 of the model. This case provides a reasonable lower-bound estimate of the long-term average seepage risks.

Case 4, Case 5a, and Case 5b are sensitivity cases which consider an unlikely but upper bounded water table (not effected by neighbouring basements) with expected hydraulic conductivities from Case 2, Case 3a, and Case 3b, respectively. These can also be used to assess the effect of short term fluctuations in groundwater elevation associated with transient flows due to wet weather or broken services.

On the above basis, we consider that:

- Case 1, Case 2, and Case 3b are best estimates of the steady state inflows into the excavation during construction. Higher flows may be expected during excavation as the water is drawn down and steady state established.
- Cases 4, 5a and Case 5b represent a range of values for transient conditions during wet weather or initial stages of drawdown.
- The preliminary estimate of inflows into the excavation during construction is between 6 and 30 m^3/day. With the upper limit likely to represent the seepage rates that may occur prior to steady state conditions being reached. We note that no calculations have been made to estimate the rates of groundwater seepage that may occur prior to steady-state conditions being reached. Based on PSM experience modelling at other sites, we assume early time seepage risks may typically be five times larger than steady-state.

Disposal of the groundwater to the stormwater network during construction of the basement would be subject to quality assessment outside the scope of this letter.

4.2 Long-term Groundwater Assessment

Following construction of the proposed basement, we understand the permanent condition will encompass tanking (waterproofing) of the basement.

We note that for a tanked basement the groundwater inflow into the tanked basement will be limited primarily by the performance of the basement walls and / or drainage system and nearby drainage assets, and not governed by the hydraulic conductivity of the surrounding water bearing zones in the ground.

We recommend that at this stage the design level for basement tanking be at RL -1 m AHD. The design level for basement tanking is higher than the current observed groundwater level of RL -3 m AHD but lower than the groundwater elevation should the area not be affected by drawdown from neighbouring basements (assumed to be RL 5 m AHD).

Above this design level, an appropriate drainage system should be implemented to capture transient flows through the weathered and fractured shale layers, as well as temporarily higher water tables associated with periods of prolonged wet weather and changes in neighbouring basement conditions or broken services.

Both surface and sub-surface drainage needs to be designed and constructed properly to prevent pore water pressures from building up beyond design levels behind the retaining walls and in the retained material. Otherwise, appropriate water pressures must be included in the design. Inflows from surface run-off and from drainage of transient flows above the tanked basement must be considered in the long-term when sizing the sump and pump during design. While the numerical modelling assessment has not explicitly quantified such transient inflow risks, we do note our assessment (sensitivity cases Case 4, Case 5a, and Case 5b) consider the risk of a permanent 7 m increase in the height of the regional water table from RL -2 m AHD to RL 5 m AHD at some time during the design life of the asset. For this assumption, this assessment calculates an approximate factor of 2.2 increase in seepage compared to the design cases. These cases can be used for preliminary assessment of the likely transient flows and appropriately size the sump and pump structures.

We recommend that flows during construction be monitored to inform the final designs of the tank structures and sump and pump systems.

Below the tanking design level, the basement slab, basement walls and building itself will need to be designed for the uplift and lateral pressures from the groundwater. Above the tanking level, the designer should make an allowance for a nominal lateral loading due to groundwater of at least 10 kPa on the basement walls. This allowance captures potential transient flows on the basement structure from rainwater, potential broken services, and other sources of transient groundwater. The cut-off wall in the upper materials should be designed for hydrostatic loads as monitored in the shallow aquifer piezometers.

Disposal of the groundwater to the stormwater network following construction of the basement would be subject to quality assessment outside the scope of this letter. Given the tanking of the basement, the disposal volumes would be expected to be small and be associated with rainfall periods.

4.3 Effects on Neighbouring Structures

4.3.1 During Construction

Dewatering will cause drawdown of the permanent groundwater table within and locally around the basement excavation. The extent of dewatering is shown in Table 7 and Figure 1 and Figure 2.

This drawdown is expected to occur primarily in the shale layer. It is therefore expected that the localised drawdown and strength and weathering of the shale will result in negligible ground settlement unlikely to affect the neighbouring structures and infrastructure.

Should future investigation indicate the presence of deeper alluvium / residual materials on the eastern portion of the Site, dewatering within this layer could result in drawdowns that can cause minor settlement. However, given observations during previous investigations of the stiff consistency in the alluvium / residual material, there would still be small settlements which are unlikely to affect the serviceability of services and roads. Furthermore, most of the neighbouring structures are likely to be founded below the alluvium / residual layer and within the shale layer and thus would be unaffected by such settlements.

4.3.2 Long-term

Following construction works and in perpetuity, it is understood that the basement will be tanked. The groundwater would thus be expected to recover to pre-construction levels. Drawdown effects post tanking are thus expected to be negligible.

4.4 Recommendations for Additional Work

The groundwater assessment made in this letter is considered preliminary. Further details regarding the proposed basement depth and design would be required to confirm the groundwater inflows and drawdown estimates.

In particular, details about the temporary cut-off retaining wall structure in the alluvium and upper materials would be required to confirm the estimates. This would need to be informed by further investigations to confirm ground and groundwater conditions on the eastern portion of the Site, which is currently occupied by existing structures. The presence of deeper alluvium or more permeable materials in this area can affect the advice in this letter.

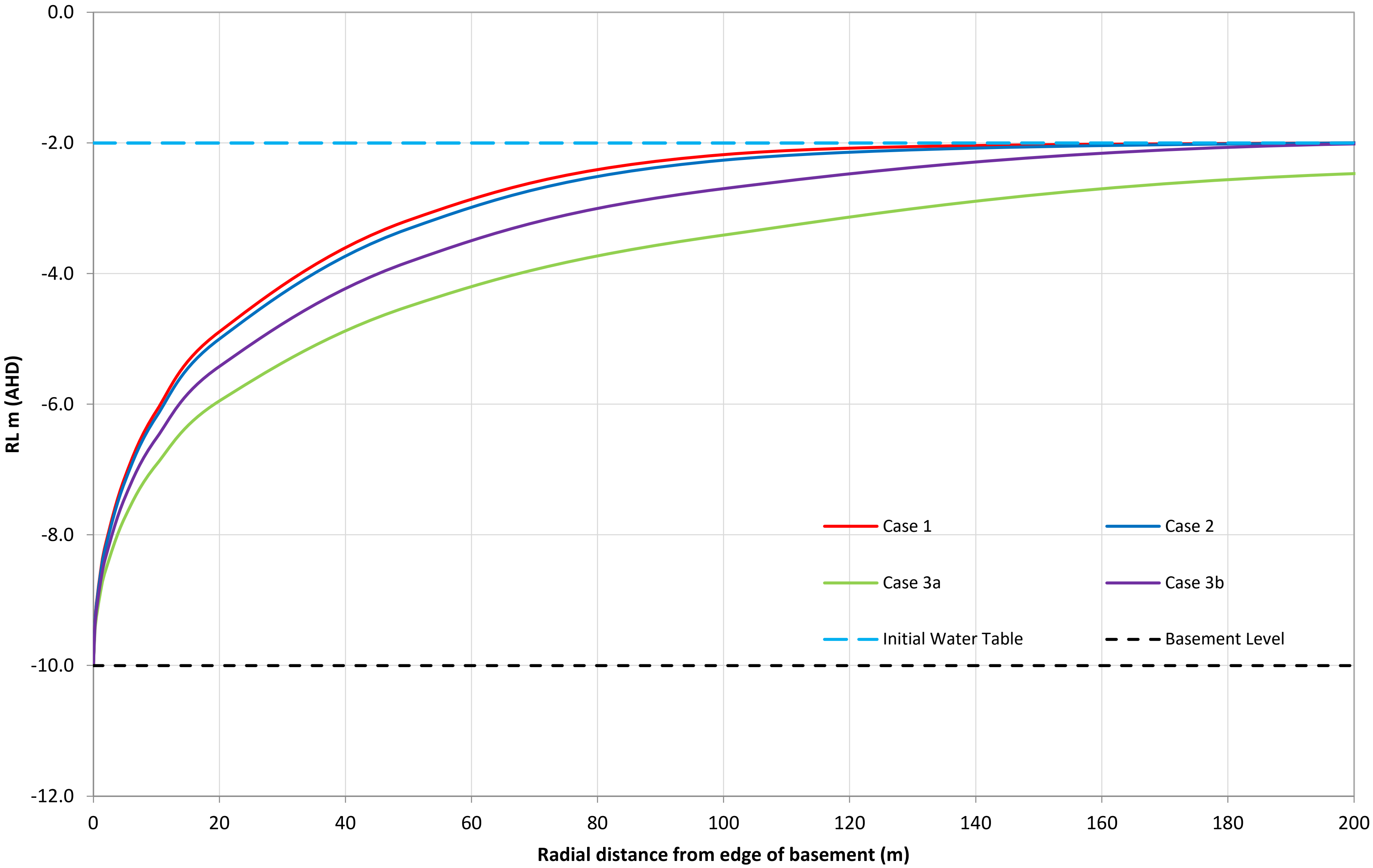
Groundwater monitoring should be continued during further site investigations and during the construction stage. Additional groundwater monitoring wells should be installed on the eastern portion of the Site during investigations in the area. Further rising head (slug) tests should be conducted at the Site within existing and additional monitoring wells. A detailed groundwater assessment should be undertaken using results of further investigations, slug testing, and borehole data. This should include details of the proposed tanked basement excavation, drainage system, and temporary cut-off retaining wall structure.

Yours Sincerely

GREG FAZZONE
GEOTECHNICAL ENGINEER

DAVID PICCOLO
PRINCIPAL

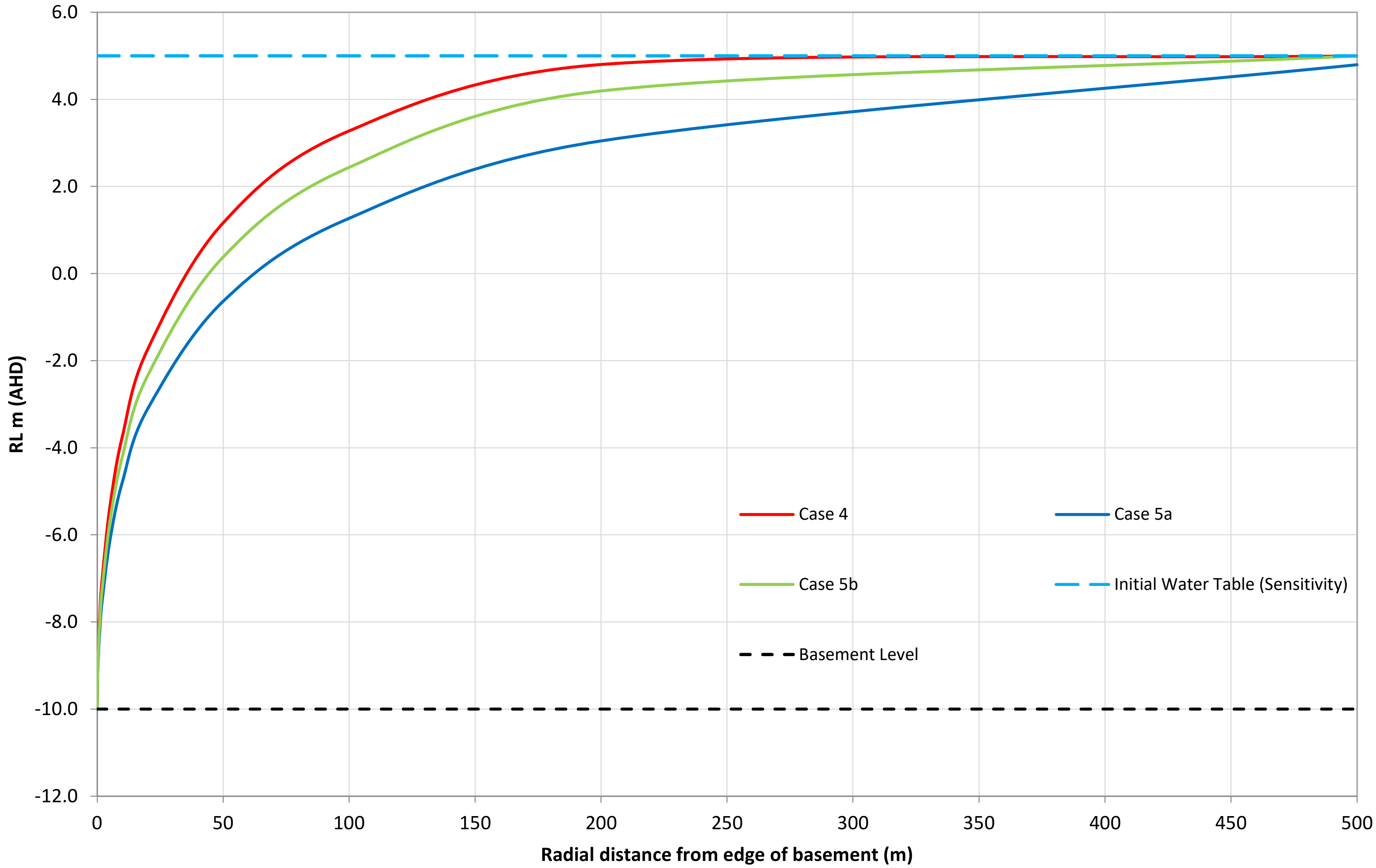
Encl.	Figure 1	Groundwater Drawdown Profiles – Design Cases
	Figure 2	Groundwater Drawdown Profiles – Sensitivity Cases
	Appendix A	Site Locality plan of Monitoring Wells
	Appendix B	Locality plan of Monitoring Wells around the Site
	Appendix C	PSM BH03 standing groundwater levels



Notes:

1. Analytical solution and drawdown profile calculated in accordance with Marinelli and Niccoli (2000).

Gurner **12 Hassall Street** **Parramatta** **GROUNDWATER DRAWDOWN PROFILES** **Design Cases**	
PSM4580-006L	**Figure 1**



Notes:

1. Analytical solution and drawdown profile calculated in accordance with Marinelli and Niccoli (2000).

Gurner

12 Hassall Street

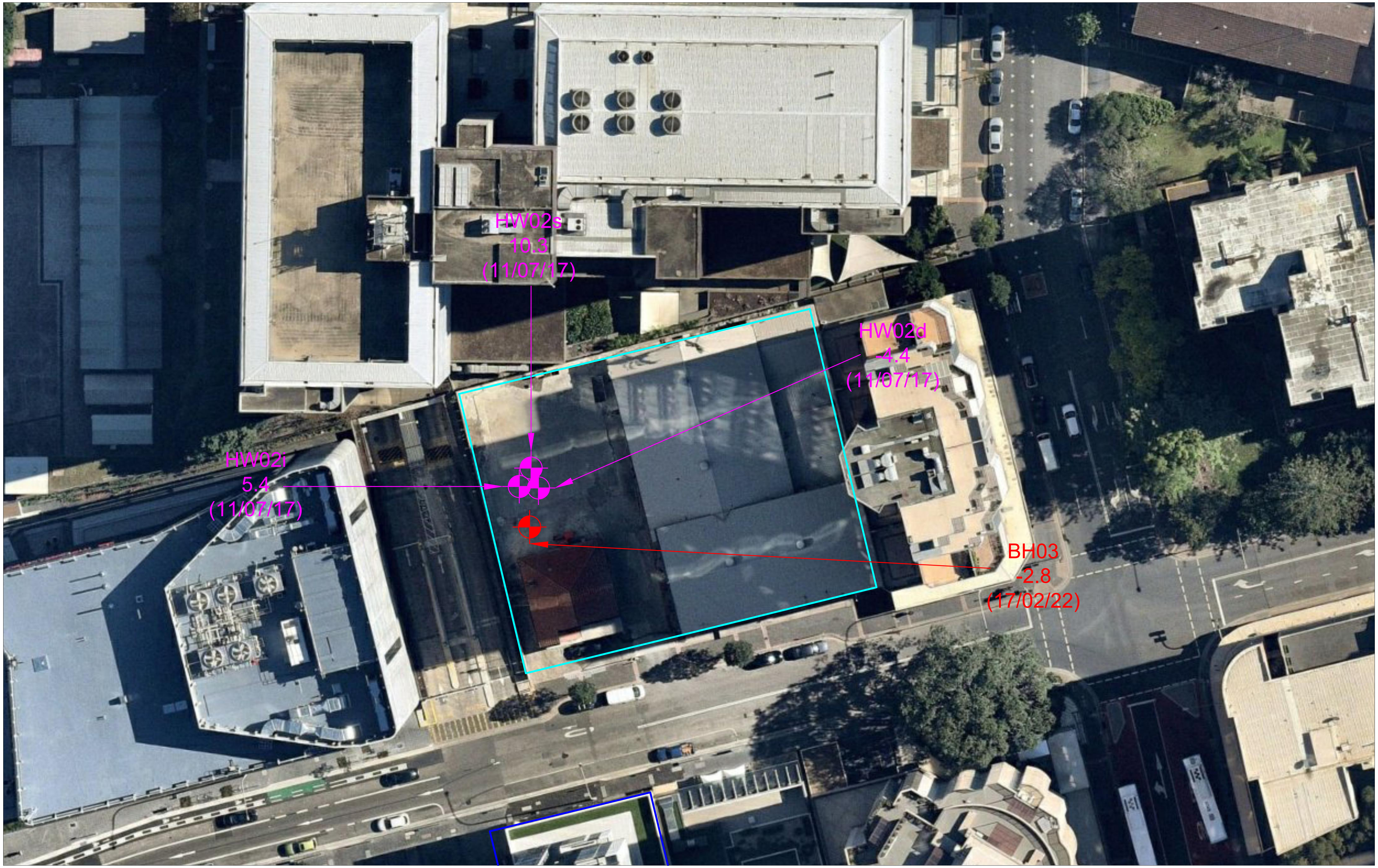
Parramatta

GROUNDWATER DRAWDOWN PROFILES

Sensitivity Cases

PSM4580-006L	**Figure 2**

Appendix A
Site Locality plan of Monitoring Wells



Legend:

- PSM MW
- Other known historic wells at the Site
- 12 Hassall Street Site Boundary

Notes:

1. Latest groundwater level (m RL [AHD]) shown for each Borehole / Monitoring Well ID.

N

0 2.5 5.0 7.5 10.0 12.5
SCALE (m)

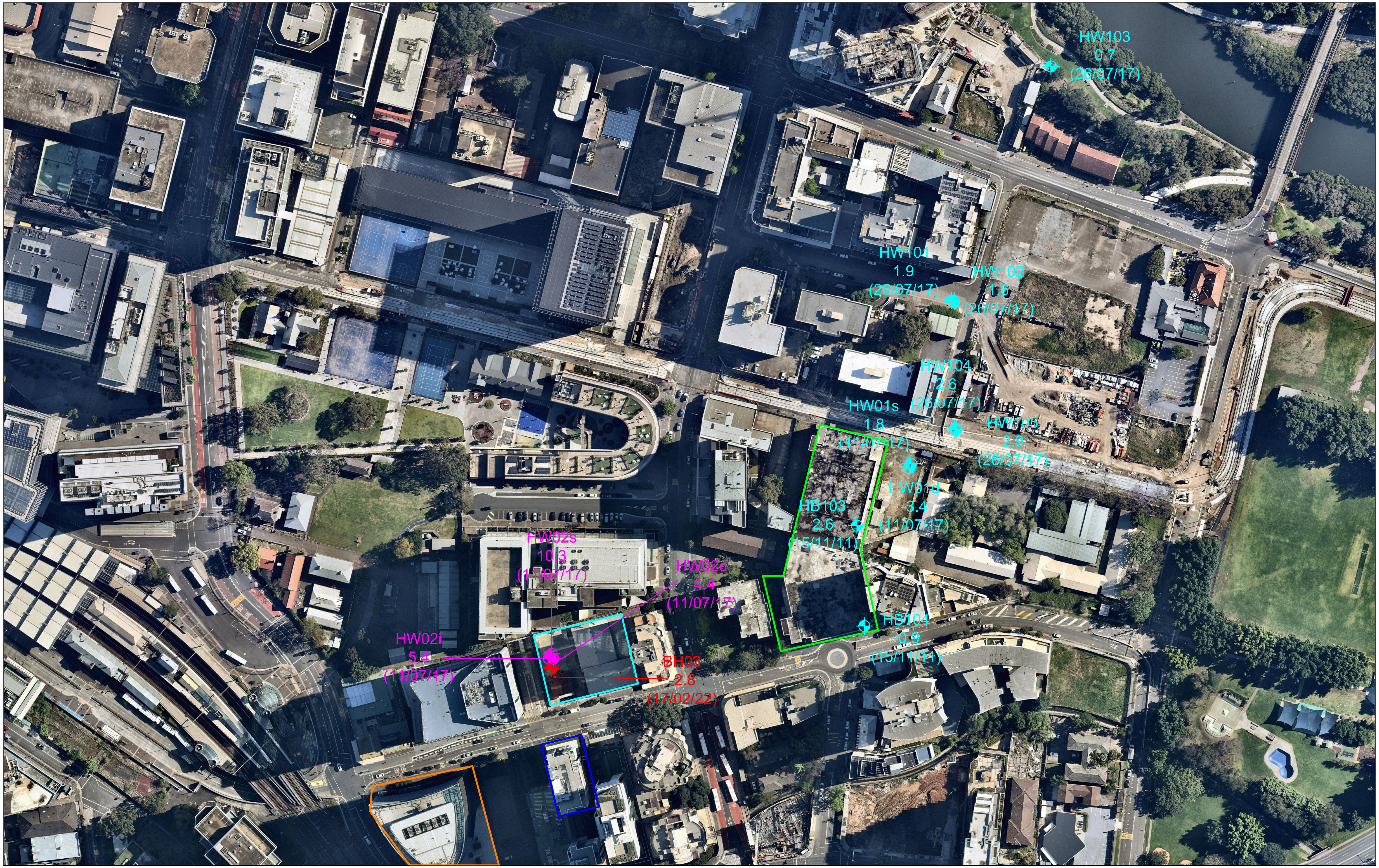
PSM

Gurner
12 Hassall Street
Parramatta

MONITORING WELLS
SITE LOCALITY PLAN

PSM4580-006L	Appendix A

Appendix B
Locality plan of Monitoring Wells around the Site



Legend:

- PSM MW
- Other known historic wells at the Site
- Other known historic wells around the Site
- 12 Hassall Street Site Boundary
- 189 Macquarie Street Site Boundary
- 9 Hassall Street Boundary
- 60 Station Street Boundary

Notes:

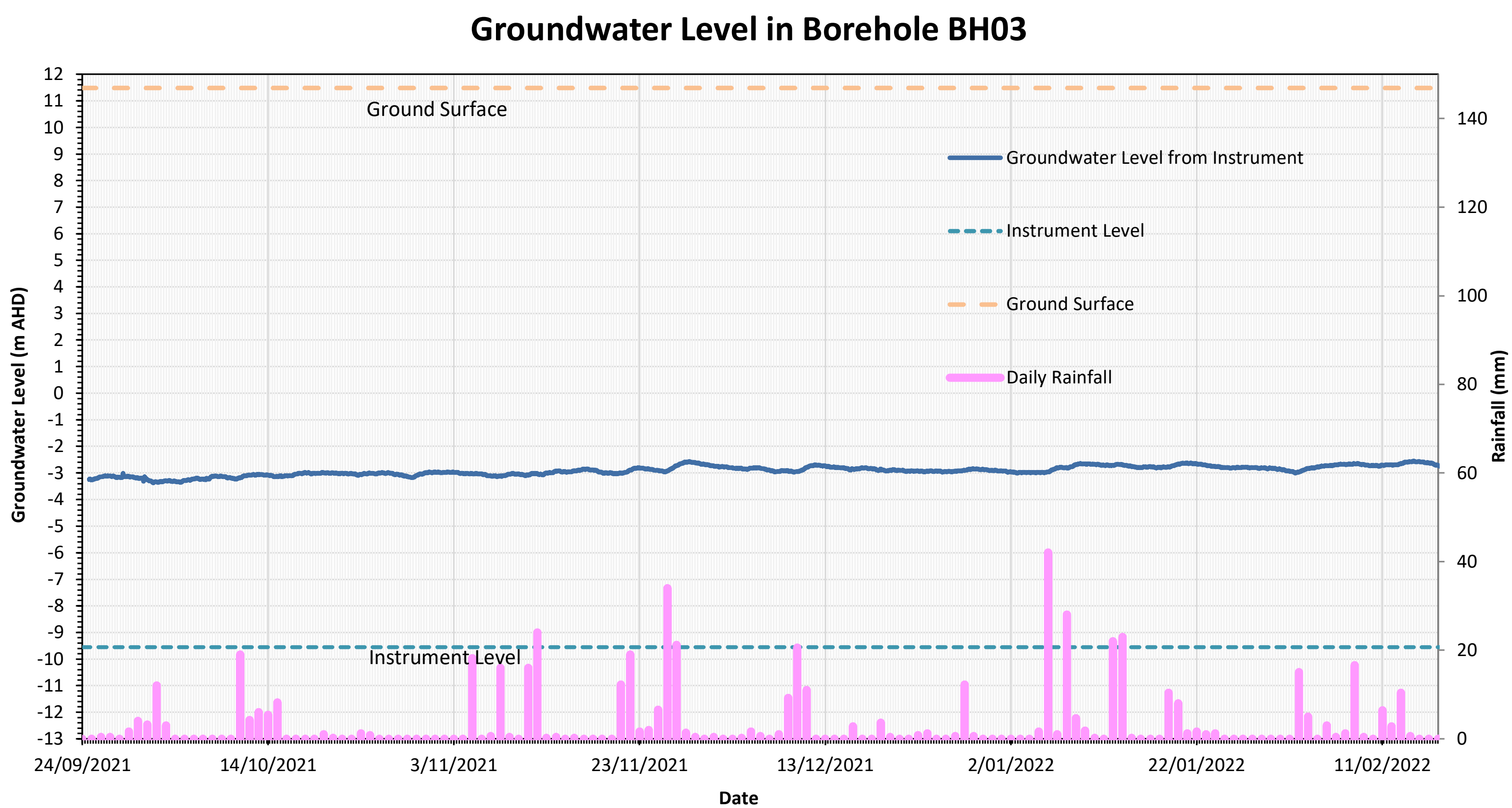
1. Latest groundwater level (m RL [AHD]) shown for each Borehole / Monitoring Well ID.

N

0 10 20 30 40 50
SCALE (m)

Gurner 12 Hassall Street Parramatta HISTORIC MONITORING WELLS LOCALITY PLAN	
PSM4580-006L	Appendix B

Appendix C
PSM BH03 standing groundwater levels



Notes:

1. Instrument elevation (m RL): -9.55
2. Ground surface elevation (mRL): 11.5
3. Water level was measured manually on 07/10/2021
4. Data logger instrument installed on 24/09/2021
5. Rainfall data from BOM, station 066124, downloaded 18/02/2022

Gurner **12 Hassall Street** **Parramatta, NSW** **GROUNDWATER MONITORING** **PZ01 (BH03)**	
PSM4580-006L	**Appendix C**