Groundwater Impact Assessment

68-80 Beauchamp Road, Hillsdale, NSW

PSM5745-018R 20 February 2026

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1. Introduction

This report presents the initial Groundwater Impact Assessment for the proposed residential development at 68 – 80 Beauchamp Road, Hillsdale, NSW (the **Site**). The work was undertaken in accordance with PSM proposal PSM5745-015L dated 21 January 2026.

1.1 Response to Authority requests

This report has been prepared in response to the requirements raised by the following authorities, as noted in the document prepared by FPD Planning:

- Department of Climate Change, Energy, the Environment and Water – Water Group
  - Prior to determination, the DPHI should request the proponent to quantify the maximum annual volume of water take due to aquifer interference activities and demonstrate the ability to acquire sufficient water entitlement unless an exemption applies.
  - Prior to determination, DPHI should request the proponent to provide an impact assessment of water supply work construction for dewatering, if proposed.
  - If the take of groundwater is found to be greater than 3ML per year, prior to determination DPHI should request the proponent to assess impacts due to aquifer interference activities in accordance with the NSW Aquifer Interference Policy and Framework (2012).
- Department of Planning, Housing and Infrastructure (DPHI)

Table 1 - Clauses in the DPHI Key Issues Letter addressed by this report

Clause	Key Issue	Report Section
4b	Provide a groundwater assessment that analyses groundwater inflow during construction and operation of the site. Quantification of maximum potential inflow volumes is required in accordance with the Department's *Minimum requirements for building site groundwater investigations and reporting*.	9
4c	If the findings result in a groundwater take greater than 3 ML/year, the impacts due to aquifer interference are to be assessed in accordance with the NSW Aquifer Interference Policy and Framework 2012.	9

2. Background

2.1 Documentation

We have been provided with the following information for the proposed development:

- Specification document summarising the background and scope of works (ref. LAHC 2025/186 dated September 2024)
- Architectural drawings (floorplans and elevations) by Studio.SC dated October 2025 (*Issued for SSDA*).

2.2 Previous Site Investigations

PSM have previously undertaken intrusive geotechnical investigations for this Site with the results detailed in PSM5745-010R REV3 dated 7 November 2025. These investigations included the following:

- Drilling of six (6) augered boreholes up to 8 m below the ground surface
- Installation of standpipe piezometers with groundwater monitoring devices (HOBOs) at BH-PZ1, BH-PZ2, BH-PZ3 and BH-PZ4
- Six (6) cone penetrometer tests (CPTs) to depths ranging between approximately 3 m and 6 m.

3. The Site

3.1 Location

The Site is located at 68-80 Beauchamp Road, Hillsdale in the Bayside Local Government Area (**LGA**).

The Site is bound by the following structures/properties:

- Beauchamp Road to the south which is a Regional Road. To the south across Beauchamp Road are low rise single dwellings,
- Matraville Public School to the north and east,
- A two-storey single dwelling to the west which fronts Beauchamp Road and low-rise apartment buildings which front Flack Avenue as well as Flack Avenue reserve which connects Beauchamp Road to Flack Avenue.

It is currently occupied by two storey residential buildings containing 179 social housing dwellings

Inset 1 presents a locality plan of the Site.

Inset 1: Locality plan with approximate site boundaries shown in red (aerial imagery from Nearmap dated 3 December 2025)

3.1.1 Proposed Development

The proposed development will comprise:

- Two (2) residential flat buildings seven to eight storeys comprising 179 social housing dwellings
- A single access from Beauchamp Road providing servicing and basement car park access
- Associated landscaping and communal open space
- Infrastructure servicing
- Two basement levels as per Studio.SC drawings.

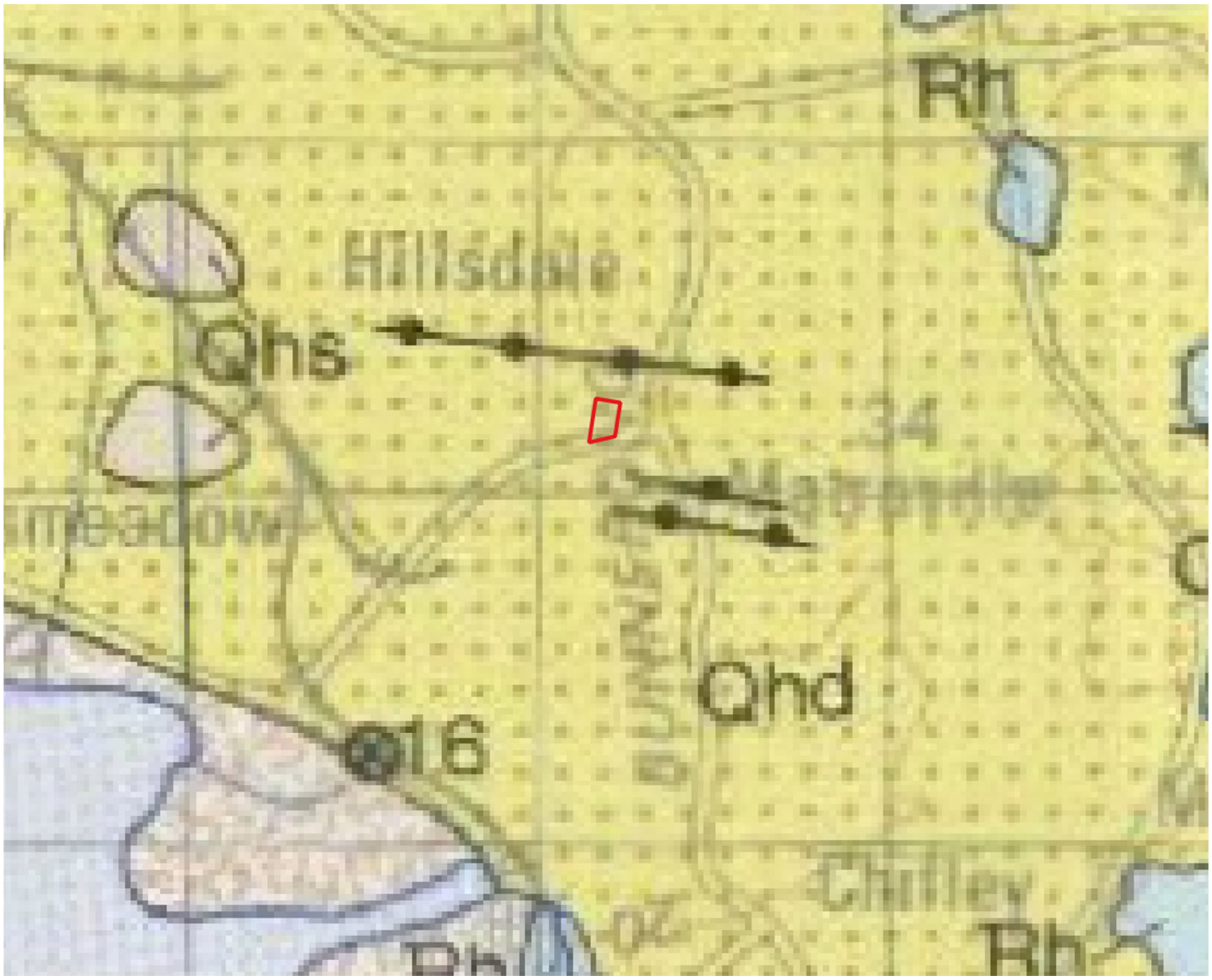
3.2 Topography

The Site generally grades up towards the south-east. There is evidence of minor cut and fill having occurred on site as part of the existing developments, based on the site investigation results.

3.3 Geological Setting

The Sydney 1:100,000 geological map (1966), Inset 2, indicates that the Site is underlain by (Qhd) medium to fine-grained "marine" sand with podsols.

This is consistent with observations made during the PSM investigation undertaken in 2025.



Inset 2: Approximate extents of Site on the Sydney 1:100,000 geological map (1966)

3.4 Neighbouring Developments

Near to and surrounding the Site, there are various existing residential structures and Matraville Public School adjoins the Site to the north and east. It is understood that these immediate developments do not contain basement levels.

To the north-west of the Site there are low rise apartment buildings that can be accessed from Flack Avenue and Rhodes Street. A review of Google street view imagery indicates that these buildings have basements, which could be up to 2 levels based on information found on the NSW planning portal website.

The Site has frontage to Beauchamp Road to the south which is a Regional Road. South from Beauchamp Road, there are low rise single dwellings which are understood to not have basement levels and a retail outlet area which may have basement levels.

This is discussed further in Section 8.1.1 Groundwater recharge and discharge.

4. Regulatory Considerations

4.1 General

Constructing buildings with basements that require excavation which intercepts an aquifer is an aquifer interference activity. Therefore, it is subject to the Water Management Act 2000, relevant water sharing plans, and the NSW Aquifer Interference Policy.

4.2 Water Licensing

Applicants must seek appropriate licences and approvals for a building project before construction starts.

Where predicted maximum groundwater inflows into an excavation during construction are expected to be less than 3 ML/year, such developments could be considered as a 'minor aquifer interference activity' which are generally exempt from the full extent of the Water Management Act and therefore may not require a licence.

This is discussed in the WaterNSW Fact Sheet titled '*Water access licence exemption for aquifer interference activities taking 3ML or less of groundwater per year*'.

This fact sheet includes a list of common aquifer interference activities to which the exemption may apply, one of which is the '*excavation to construct or maintain a building, road or infrastructure'*.

Ultimately the decision as to whether a licence will be required for the construction of the basement will be at the discretion of WaterNSW.

4.3 Minimum Requirements for Building Site Groundwater Investigations and Reporting (NSW DPIE, 2022)

This document sets out groundwater investigation, modelling and reporting requirements to be undertaken when assessing the effect of a proposed building basement on an aquifer.

The document indicates that a tanked basement construction would result in no ongoing dewatering and as such reduces/eliminates the following:

- Energy demand from the continual operation of the pump-out system.
- The required maintenance demand to keep the pumps operating and drainage lines free of clogging from the aeration of mineral-rich groundwater.
- The water demand on the surrounding groundwater system and optimise the availability of groundwater for all users.
- The additional administrative demand to retain records and maintain valid approvals, licences, or both, that would otherwise be imposed on the future managers of the property.

4.4 Disposal/reuse of groundwater

Groundwater flowing into the excavation during construction will need to be either re-used on site for irrigation purposes, other grey water usage or be disposed of either in the stormwater or sewer systems.

Where disposal is proposed this will need to be agreed with the appropriate authorities, i.e., Local Council for stormwater and Sydney Water for sewer. Each authority will have their requirements with regards to peak and average flows and water quality.

4.5 SEARs

The Planning Secretary's Environmental Assessment Requirements (SEARs) issued for the project (SSD-83256478) on 12 May 2025, relating to the geotechnical and groundwater aspects for the State Significant Development application (SSDA) requires the following clause , Inset 3, to be assessed with respect to groundwater.

This report provides a response on the effect of the proposed basement on groundwater resources.

12. Ground and Groundwater Conditions • Assess potential impacts on soil resources and related infrastructure and riparian lands on and near the site and including soil erosion. • Where required provide a Groundwater Impact Assessment in accordance with relevant Groundwater Guidelines. If the proposed development is on land identified as having high salinity or acid sulfate soil potential in an EPI provide a Salinity Management Plan or Acid Sulfate Soil Management Plan that includes appropriate management measures and strategies.	Geotechnical Assessment <u>If required:</u> Groundwater Impact Assessment Salinity Management Plan Acid Sulfate Soils Management Plan

Inset 3: Industry SEARs Checklist for Item 12 – Ground and Groundwater Conditions

5. Proposed basement configuration

5.1 Geometry

From the provided documents, we understand that the Site covers an area of approximately 7,857 m^2 and the proposed basement footprint is approximately 2,700 m^2.

Inset 2 presents a plan view of the proposed basement excavation footprint.

The provided Architectural drawings do not indicate the datum for the shown basement elevations. Additionally, the Bulk Excavation Plan in the Architectural drawing set showing the proposed ground levels also do not indicate the datum. On this basis, it has been assumed that the existing topography remains unchanged for the proposed development, and the basement levels have therefore been expressed in terms of metres below ground level (**mBGL**), as summarised below:

- Basement elevation = RL 15.97 – RL 9.37 = 6.6 mBGL

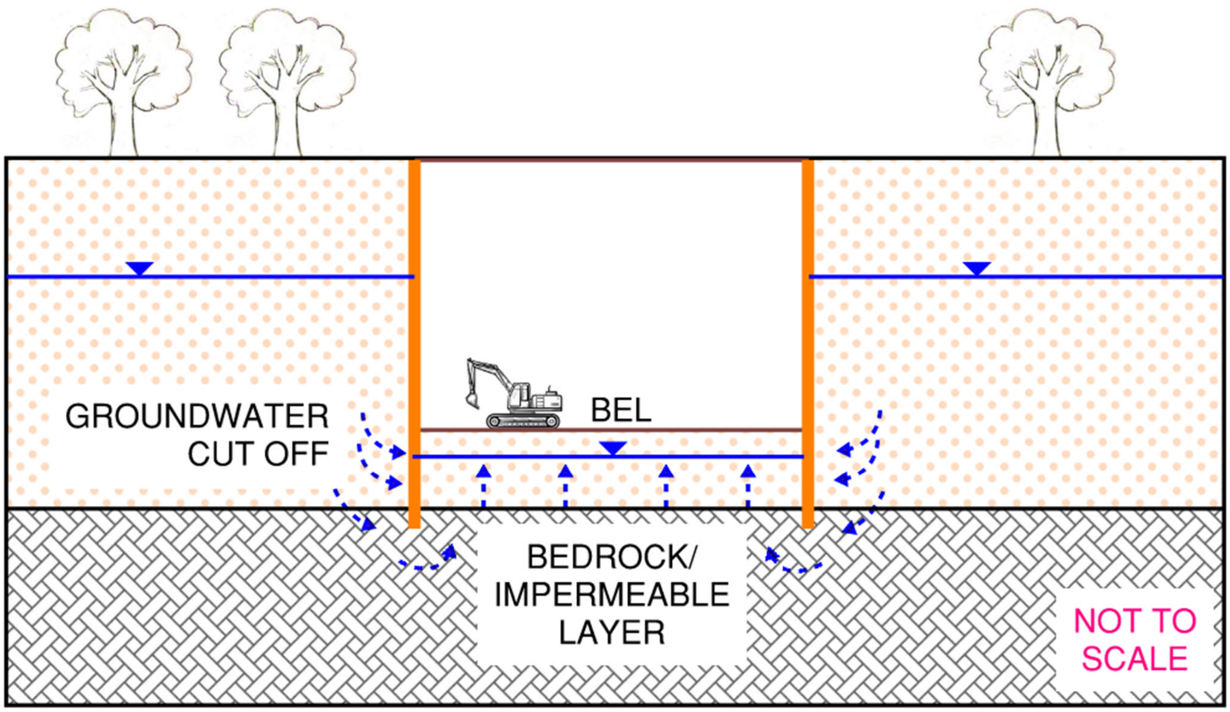
Inset 4: **Plan view of proposed excavation footprint overlaid on aerial imagery from Nearmap dated 3 December 2025**

5.2 Drainage condition

5.2.1 During construction

The proposed basement geometry will result in excavation below the measured water levels in the piezometers installed on Site. As discussed in Section 7 and Section 8, the sand is expected to have a high permeability while the underlying bedrock unit is expected to have a relatively lower permeability. On this basis, we have recommended that a tanked basement should be adopted where water-tight shoring walls will need to be designed and constructed around the perimeter of excavation to below bulk excavation level. This is also noted in the Bayside Council response. Nonetheless, we have not received details of the proposed retention system for the excavation.

During construction, the excavation will need to be dewatered progressively to allow plant and machinery to operate on the bulk excavation level. The groundwater inflows will need to be limited and groundwater within the excavation should be lowered to at least 1 m below the bulk excavation levels and as such, the water-tight shoring system may need to extend to the underlying bedrock / impermeable layer to cutoff the inflows. There would still be some groundwater inflow at the base, however the cutoff would minimise the inflows and also control the groundwater drawdown and mitigates the risk of drawdown induced settlement behind the excavation. A schematic is shown in Inset 5.



Inset 5 Simple Schematic showing the cut off and groundwater flow (support elements not shown)

5.2.2 In perpetuity

As per the conditions by the Bayside Council, a fully tanked basement is to be adopted. As such, negligible groundwater inflow is expected in perpetuity. However, allowance should be made for minor inflows into the basement through joints and cracks in wall and slab structure. Additionally, the shoring and base slab would need to be designed to withstand the hydrostatic pressures and buoyancy forces. Advice on the groundwater level and the associated design consideration is provided in Section 10.

6. Geotechnical investigations at the Site

PSM previously completed geotechnical site investigations including drilling of six (6) augered boreholes up to 8.0 m below the ground surface supplemented by six (6) cone penetration tests (CPT), which were advanced up to 6.0 m below ground. The results of the geotechnical investigation and inferred subsurface conditions at the Site are presented in PSM5745-010R REV3 dated 7 November 2025.

PSM installed a total of four (4) standpipe piezometers (groundwater monitoring wells) during the geotechnical investigation works at BH-PZ1, BH-PZ2, BH-PZ3 and BH-PZ4.

Inset 1 shows the approximate locations of the completed boreholes relative to the approximate Site boundaries.

Sandy material was encountered during the site investigations and practical refusal was encountered in all the CPTs. The practical refusal may be inferred as the top of bedrock. However, this is to be confirmed with further borehole investigations.

7. Hydrogeological investigations

7.1 Piezometers

PSM has previously installed (4) piezometers in BH-PZ1, BH-PZ2, BH-PZ3 and BH-PZ4 during the site investigations in 2025.

As a component of the geotechnical investigation work, a total of four (4) HOBO water level loggers were installed to monitor the groundwater levels:

- BH-PZ1: standpipe piezometer with HOBO logger collecting data in the period between 20 June 2025 and 28 January 2026.
- BH-PZ2: standpipe piezometer with HOBO logger collecting data in the period between 24 June 2025 to 9 July 2025. The HOBO data logger was uninstalled on 9 July 2025 and transferred to BH-PZ4.
- BH-PZ3: standpipe piezometer with HOBO logger collecting data in the period between 20 June 2025 and 28 January 2026.
- BH-PZ4: standpipe piezometer with HOBO logger collecting data in the period between 9 July 2025 to 28 January 2025.

Construction records for the aforementioned piezometers are included in Appendix A.

7.2 Hydraulic Conductivity Tests

"Falling head" tests were performed on 28 January 2026 within the piezometers installed in BH-PZ1, BH-PZ3 and BH-PZ4. No test was performed at BH-PZ2 as the HOBO logger in this piezometer had been transferred to BH-PZ4 on 9 July 2025.

Falling head tests were undertaken by adding water to the piezometer, and the water discharge (falling of the water level) was monitored using the HOBO water level logger. The falling head (slug) tests assessed the hydraulic conductivity of the NATURAL SOIL unit at BH-PZ1 and BH-PZ4. In BH-PZ3, the screened lithology comprises of the NATURAL SOIL unit and also the SANDSTONE unit.

The results of the falling head tests are presented in Appendix C and summarised in Table 2.

Table 2 – Results of falling head tests undertaken

BH ID	k_h BEST ESTIMATE (m/s)	Adopted k_h (m/s)
BH-PZ1	3.3×10^{-5}	3.5×10^{-5}
BH-PZ3	2.1×10^{-6}	2.5×10^{-6}
BH-PZ4	1.3×10^{-5}	1.5×10^{-5}

8. Conceptual groundwater model

8.1 Regional model – Botany Sandbed Aquifer (BSA)

Within the Botany Basin, there are two primary regional groundwater systems (Hatley, 2004):

- A groundwater system within the unconsolidated sediments of the Botany Sandbed Aquifer (**BSA**) deposited within the basin
- A groundwater system within the underlying bedrock sequences that constitute the Botany Basin erosional depression.

The groundwater system encountered during the previous geotechnical investigations undertaken at the Site is assessed to be associated with the BSA, rather than the underlying bedrock units.

The Site is underlain by the Upper Unit of the Botany Basin sedimentary sequence, which forms part of the BSA (Hatley, 2004). This unit comprises a heterogeneous sequence of fine to medium grained loose sands with interbedded silt and clay layers. The sediments were deposited in a range of environments including transgressive dune, estuarine bay, beach ridge, tidal flat and terrestrial swamp settings. The thickness of the unit varies from 0 m to greater than 20 m. These unconsolidated and largely uncemented sands of the Botany Basin sequence generally form effective aquifer materials (Hatley, 2004).

8.1.1 Groundwater recharge and discharge

Recharge to the BSA occurs predominantly through direct infiltration of rainfall, with minor contributions from irrigation, leakage from buried services, and upward flow from the underlying bedrock units although the interaction of the bedrock with the BSA is not well understood (Hatley, 2004). The recharge is highest in areas of the basin least affected by urban development, such as the golf courses and parks which surround the wetland and ponds of the East Lakes area. Less significant contributions come from the numerous smaller open areas, residential backyards and runoff from the covered areas (Hatley, 2004).

Discharge of aquifer may include seepage from the sediments into deep basements that are constructed within the unit. To the north-west of the Site there are low rise apartment buildings, and we understand these buildings have basements which could act as potential discharge points. Additionally, south from Beauchamp Road, is a retail outlet area which may have basement levels.

8.1.2 Regional groundwater flow direction

The regional groundwater flow direction generally reflects topographic trends, surface drainages and ponds which occur across the basin.

The regional groundwater flows are generally towards Botany Bay which is west to south-west of the Site. Inset 6 outlines the general flow direction within the BSA.

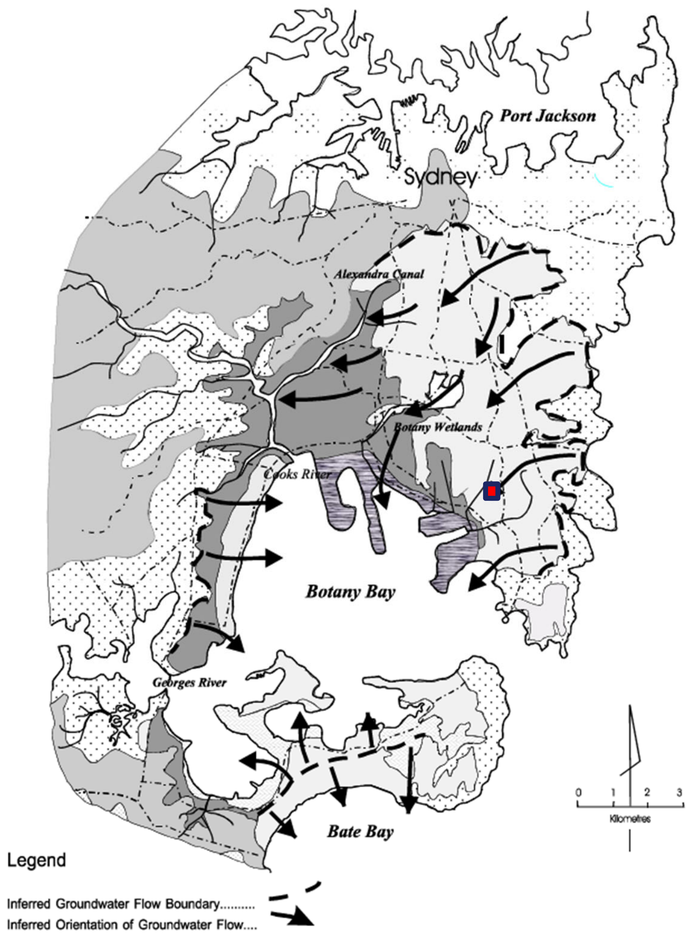


Figure 5: Map of the Botany Basin showing inferred groundwater flow directions in the Botany Sandbed Aquifer unit (Bish et al., 2000, McKibbin and Russell, 2002).

Inset 6: Inferred groundwater flow direction in the BSA (Figure 5, Hatley, 2004) with approximate Site location marked in red

8.2 Groundwater levels

The groundwater levels based on the data recorded by the HOBO data loggers are presented in Appendix B.

The groundwater data collected in the latest data collection periods between 20 June 2025 to 28 January 2026 (for BH-PZ1, BH-PZ3), 24 June 2025 to 9 July 2025 (for BH-PZ2), and 9 July 2025 to 28 January 2026 (for BH-PZ4) show the fluctuations in groundwater level with time:

- BH-PZ1: Up to approximately 1.7 m
- BH-PZ2: Up to approximately 0.5 m
- BH-PZ3: Up to approximately 2.0 m
- BH-PZ4: Up to approximately 1.5 m.

The data from the installed standpipe piezometers indicates that the groundwater has a slight fall in the west direction. The recorded groundwater levels are as below and appear to be located within the NATURAL SOIL unit which is part of the BSA. We note that the groundwater encountered at BH-PZ4 is around the interface of the NATURAL SOIL unit and to some degree, the underlying inferred SANDSTONE.

- BH-PZ1: 10.0 mAHD to 11.7 mAHD
- BH-PZ2: 10.2 mAHD to 10.7 mAHD
- BH-PZ3: 11.0 mAHD to 13.0 mAHD
- BH-PZ4: 9.8 mAHD to 11.4 mAHD

It is anticipated that the groundwater encountered at the Site comprise a regional scale aquifer (see Section 8 below).

Table 3 – Summary of Groundwater Level Monitoring

Borehole ID	Installed	Top of Ground	Screened Interval		Last steady groundwater level up to 28 Jan 2026	
		m AHD	m bgl	m AHD	m bgl	m AHD
BH-PZ1	20 June 2025	14.0	0.4 – 5.4	13.6 – 8.6	3.4	10.6
BH-PZ2	24 June 2025	13.8	0.5 – 5.3	13.3 – 8.5	3.3	10.5
BH-PZ3	20 June 2025	15.3	0.7 – 7.2	14.6 – 8.1	4.0	11.3
BH-PZ4	9 July 2025	13.7	0.5 – 5.2	13.2 – 8.5	3.7	10.0

Note:
(1) Screened interval measured from top of gravel pack to the base of the screen.

8.3 Inferred groundwater conditions at the Site

At this Site, the the Upper Unit of the Botany Basin sedimentary sequence, which forms part of the BSA has been identified. This is referred to herein to as the Upper Aquifer and is expected to intersect the proposed basement:

- **Upper Aquifer** – This aquifer is inferred to follow the topography with groundwater expected to be flowing west to south-west in the direction of Botany Bay. The elevation of the groundwater in this Upper Aquifer has been measured in BH-PZ1 to BH-PZ4.

We expect this aquifer to be recharged by surface infiltration and could be affected by variations in ground conditions and presence of local discharge points such as other drained basements.

As discussed in Section 6, practical refusal was encountered in all of the CPTs conducted as part of the SI. For the purpose of this assessment, we assume that the ground conditions at the Site can be divided into two main strata:

- Regolith, i.e., Soil Unit

  Based on our previous investigation (PSM5745-010R REV 3), the soil unit comprised predominantly SAND and was noted to be medium to coarse grained with a consistency of loose to medium dense.
- Bedrock Unit, which we infer to be Hawkesbury Sandstone based on the geological setting and our experience in the area.

8.4 Aquifer hydraulic conductivity

With regards to the hydraulic conductivity, we consider that site conditions can be characterised as two distinct units:

- UPPER UNIT (Regolith) – Comprising NATURAL SOIL (SAND: medium to coarse grained)
- LOWER UNIT (Bedrock) – Comprising SANDSTONE bedrock.

When assessing the hydraulic conductivity of these two units we have considered:

- The geotechnical characteristics of the units
- Typical conductivities adopted for these units in projects across Sydney both published in the literature and that we are aware of from our involvement in many hydrogeological models for projects in Sydney. For the above units, typical ranges of horizontal hydraulic conductivity are:
  - UPPER UNIT (Regolith): $k_h = 10^{-4}$ to 10^{-6} m/s, $k_h/k_v = 1$
  - LOWER UNIT (Bedrock): $k_h = 10^{-7}$ to 10^{-8} m/s, $k_h/k_v = 5$.
- Results of localised hydraulic conductivity tests conducted in the UPPER UNIT (Regolith) is presented in Table 2.
- Discrepancies between the results of the hydraulic conductivity tests and published literature is likely due to the difficulty in undertaking hydraulic conductivity tests in high permeability materials. These tests assume instant displacement of the water level whereas during PSM's field testing, water was added to the well with a bucket or hose. In high permeability material, water can flow away from the well faster than it is added causing the measured response to be slower than the actual soil behaviour. As a result, the field tests can result in underestimation of the in-situ permeability. On the above basis, we consider that the hydraulic conductivity tests should be treated as lower bound estimates of the permeability.

For the purpose of this assessment, we have adopted the following horizontal hydraulic conductivity:

- UPPER UNIT (Regolith):
  - Best estimate: $k_h = 3x10^{-5}$
  - Upper bound: $k_h = 1x10^{-4}$
- LOWER UNIT (Bedrock):
  - Best estimate: $k_h = 1x10^{-7}$
  - Upper bound: $k_h = 5x10^{-7}$

The adopted hydraulic conductivity parameters for the Sandstone bedrock are within the range provided by Pells et al (2019).

9. Inflow and groundwater drawdown assessment

9.1 General

We consider that the proposed excavation geometry would intercept the Upper Aquifer.

In Section 9.2 we have investigated possible inflows from this Upper Aquifer into the basement excavation at 6.6 mBGL and provided an estimate of the inflow into the excavation using the following approach:

- An analytical solution for unconfined flow into an excavation with recharge which we have used to provide an estimate of inflows during construction.

It is important to note that inflow values are sensitive to a number of inputs for which an exact value is not available. On this basis, it is usual to complete sensitivity analyses.

We note that in perpetuity, the basement will be tanked, hence seepage into the basement will be minimal and limited to seepage through cracks and defects in the basement walls and slabs.

9.2 Effect of excavation on the Groundwater

9.2.1 Inflow estimate – Analytical Approach

It is assumed that the basement shoring and cut off structure will be installed and socketed into the Sandstone bedrock prior to excavations. The shoring and cut off would be an effective impermeable system (e.g. secant piles, diaphragm walls, sheet piles, jet grouting etc). The designer and contractor should ensure that a robust design and construction QA is in place to achieve this requirement.

In consideration of the above assumption, we have used the Forchheimer 1914, Hvorslev 1951 Case 3/B and Hvorslev 1951, Case 4/C analytical solutions to estimate the inflow through the basement floor.

The adopted ratio of horizontal conductivity to vertical conductivity is 5:1 in the Sandstone bedrock i.e. $k_h/k_v = 5$.

We have considered three scenarios:

- Scenario 1 assumes:
  - The water table being 2 m higher than the basement elevation of 6.6 m BGL across the entire basement footprint
  - Best estimate hydraulic conductivity
- Scenario 2 assumes:
  - The water table being 4 m higher than the basement elevation of 6.6 m BGL allowing for groundwater fluctuation across the entire basement footprint
  - Best estimate hydraulic conductivity
- Scenario 3 assumes:
  - The water table being 2 m higher than the basement elevation of 6.6 m BGL across the entire basement footprint
  - Upper bound hydraulic conductivity.

The analyses have been undertaken assuming the following:

- An impermeable wall is constructed prior to excavation (e.g., secant piles, diaphragm walls, sheet piles etc)
- The wall is constructed into the bedrock to provide a cut-off. Based on the CPT results and boreholes, we infer that the bedrock could be between 4 m and 6 m depth below the ground level within the proposed basement footprint.

The inputs to the analytical solution and the resulting inflow estimates are presented in Table 4.

Table 4 – Inputs to Analytical Solution and Estimated Inflows

Properties	DURING CONSTRUCTION – BEST ESTIMATE (SCENARIO 1)	DURING CONSTRUCTION – SCENARIO 2	DURING CONSTRUCTION – SCENARIO 3 (UPPER BOUND)
Hydraulic conductivity – horizontal, k_h (m/s)	1.00E-07	1.00E-07	5.00E-07
Hydraulic conductivity – vertical, k_v (m/s)	2.00E-08	2.00E-08	1.00E-07
Head difference (m)	2	4	2
Percentage of excavation footprint below water table	100%	100%	100%
Estimated Inflow (ML/year)	0.21	0.41	1.03

9.2.2 Drawdown effects

Dewatering of the Site during excavation works will likely cause localised drawdown of the groundwater table within the vicinity of the Site. However, we do not expect the groundwater level post drawdown to be lower than what have been experienced in the past.

Further, with a water-tight shoring system that extends to the underlying bedrock/ impermeable layer to cutoff the inflows, the extent of the drawdown and any associated drawdown induced settlement behind the excavation will be minimised as shown by the small inflow expected.

Note that the estimated inflows are minor and below the 3 ML/yr limit that is typically adopted as the upper limit to minimal impact on the existing groundwater conditions.

10. Groundwater Impact Assessment

10.1 Design groundwater levels

Based on the groundwater monitoring results, we recommend the following design groundwater levels to be adopted for the design of the structures:

- During construction: Approximately 2 to 3 m above the proposed basement level (i.e., RL 12 m AHD)
- In perpetuity: Design groundwater elevation at the existing ground surface (approx. 13.7 mAHD to 14.4 m AHD).

  Review of the long-term groundwater monitoring in combination with consideration of the effects of storm surge, climate change and high tides may allow for a lower groundwater level (i.e., below the surface level).

  Alternatively, hydrostatic relief valves can be installed at the adopted design water level. Further discussion is provided below.

10.1.1 Basement in perpetuity

The shoring and base slab would need to be designed to withstand hydrostatic pressures and buoyancy forces. Uplift is typically resisted by the building weight and deadload. Any net uplift will need to be designed for, and we envisaged that piles are required to resist the uplift load. In the temporary stage during construction, the net uplift force due to partial building dead load must be considered, or otherwise, pressure relief “valves” shall be installed.

The tanked basement shall also be designed such that there is no long-term pumping requirements or drawdown of the water table surrounding the site. Internal surface drainage and a system to remove minor inflows into the basement through joints and cracks in wall and slab structure from the final basement will be required as a tanked basement typically experiences some residual inflows.

It is not possible to accurately predict all future groundwater levels, so where an elevation lower than the ground surface is adopted, to eliminate the risk of buoyancy issues for the tanked basement and overloading of the slab and walls, the design could incorporate hydrostatic relief ‘valves or portals’ at the adopted design water level. These would prevent overloading and structural damage to the building should the water levels rise to above the design level. Alternatively, the walls and slabs can be designed for water rising to the existing ground surface.

Dewatering should be in place whenever inflows or relief “valves” are present.

10.2 Licensing Requirements

Under the assumption that the basement walls will be installed and socketed into the Sandstone bedrock, the predicted maximum groundwater inflows into the proposed basement during construction are expected to be below 3 ML/year, as shown in Section 9.

The proposed excavation can thus be considered as a ‘minor aquifer interference activity’ which are generally exempt from the full extent of the Water Management Act and therefore may not require a licence. This is discussed in the WaterNSW Fact Sheet titled ‘*Water access licence exemption for aquifer interference activities taking 3ML or less of groundwater per year*’.

Ultimately however, the decision as to whether a licence will be required for the construction of the basement will be at the discretion of WaterNSW.

10.3 Effect on neighbouring structures

As discussed in Section 9, a water-tight shoring system that extends to the underlying bedrock/ impermeable layer, will control the groundwater drawdown and mitigate the risk of drawdown induced settlement behind the excavation. Further, as shown in the assessment, the inflow is expected to be small during construction. Then in perpetuity, the inflow is expected to be small given that the basement will be tanked.

On this basis, we consider that any minor drawdown resulting from the excavation (i.e., construction) at the Site / in perpetuity would be very unlikely to result in settlements that would damage neighbouring properties. Further, the groundwater level will not be lower than what it has experienced in the past.

10.4 Disposal/Reuse of Groundwater

Groundwater flowing into the excavation during construction will need to be either re-used on site as grey water or for irrigation purposes or disposed of either in stormwater or sewer.

Where disposal is proposed this will need to be agreed with the appropriate authorities i.e. Local Council for stormwater and Sydney Water for sewer. Each authority will have their requirements with regards to peak and average flows and water quality.

The predicted yearly inflows are small and would not be expected to result in overloading of the stormwater system. Should capacity of the stormwater be considered an issue this could be addressed by sizing of the sump and pump to allow collection and disposal at low flow rates during periods of dry weather.

Water quality issues would need to be addressed by environmental consultant separately, and local water treatment plants can be used to treat the water if required.

11. Exclusions

This report does not consider the groundwater quality, contamination and environmental issues associated with the proposed basement at the Site. We understand that these are being addressed separately by others.

12. Recommendations for further assessments at the detailed design stage

As part of the design development, the following is recommended:

- Design of shoring system and cut off should consider the recommended groundwater level and shall be designed to be impermeable.
- The cut off walls shall be designed to be installed adequately into the underlying bedrock strata to provide an adequate groundwater cut-off.
- Permeability test of the bedrock strata via hydraulic packer testing should be undertaken to confirm the permeability of the bedrock.

If the above design recommendations are not adopted or the verification is not undertaken, a revised assessment will be required.

Yours Sincerely

JANE SHEN
SENIOR GEOTECHNICAL ENGINEER

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Appendix A
Piezometer Construction Records

Pells Sullivan Meynink
Engineering Consultants
Rock - Soil - Water

JOB no: **PSM5745**

PROJECT: **Homes NSW**
68-80 Beauchamp Road, Hillsdale

PIEZOMETER CONSTRUCTION RECORD

HOLE NUMBER: BH-PZ1
PIEZOMETER: BH-PZ1
COLLAR EASTING: 336307
COLLAR NORTHING: 6241432
COLLAR RL(m): 14.00
DATUM: AHD GDA2020/MGA56

DRILLING CONTRACTOR: Legion Drilling
RIG: Geoprobe 7720R
DEPTH OF HOLE (m): 8
BOREHOLE INCLINATION: -90
PIEZO INSTALLATION DATE: 20/06/2025
SUPERVISED BY: BTA

Tick boxes

Steel protective well cover [X]

PVC cap [X]

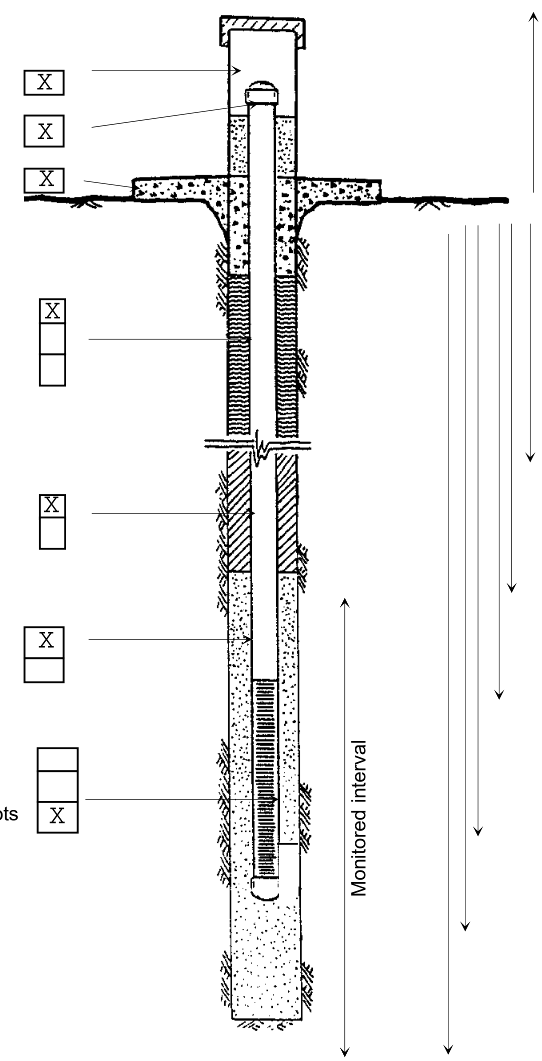
Concrete colar [X]

Back fill type:
Cement bentonite [X]
Soil [ ]
None [ ]

Seal:
Bentonite pellets [X]
Other [ ]

Gravel type:
2-5mm gravel [X]
Other [ ]

Perforation type:
Drill holes [ ]
Hack saw cuts [ ]
40um machine slots [X]



Complete dimensions if appropriate

Height of stickup (m)	0
Diameter of PVC (mm)	50
Depth to top of seal (m)	0
Depth to top of gravel pack (m)	0.4
Depth to top of screen (m)	0.9
Depth to base of screen (m)	5.4
Depth to base of piezo (m)	5.4
Depth to base of gravel (m)	8.0

Sandstone 5.2 m bgl

COMMENTS:
HOBO placed in standpipe for long-term groundwater monitoring at 4.0 m below surface level.

Pells Sullivan Meynink
Engineering Consultants
Rock - Soil - Water

JOB no: **PSM5745**

PROJECT: **Homes NSW**
68-80 Beauchamp Road, Hillsdale

PIEZOMETER CONSTRUCTION RECORD

HOLE NUMBER: BH-PZ2	DRILLING CONTRACTOR: Legion Drilling
PIEZOMETER: BH-PZ2	RIG: Geoprobe 7720R
COLLAR EASTING: 336280	DEPTH OF HOLE (m): 8
COLLAR NORTHING: 6241358	BOREHOLE INCLINATION: -90
COLLAR RL(m): 13.80	PIEZO INSTALLATION DATE: 20/06/2025
DATUM: AHD GDA2020/MGA56	SUPERVISED BY: BTA

Tick boxes

Steel protective well cover [X]

PVC cap [X]

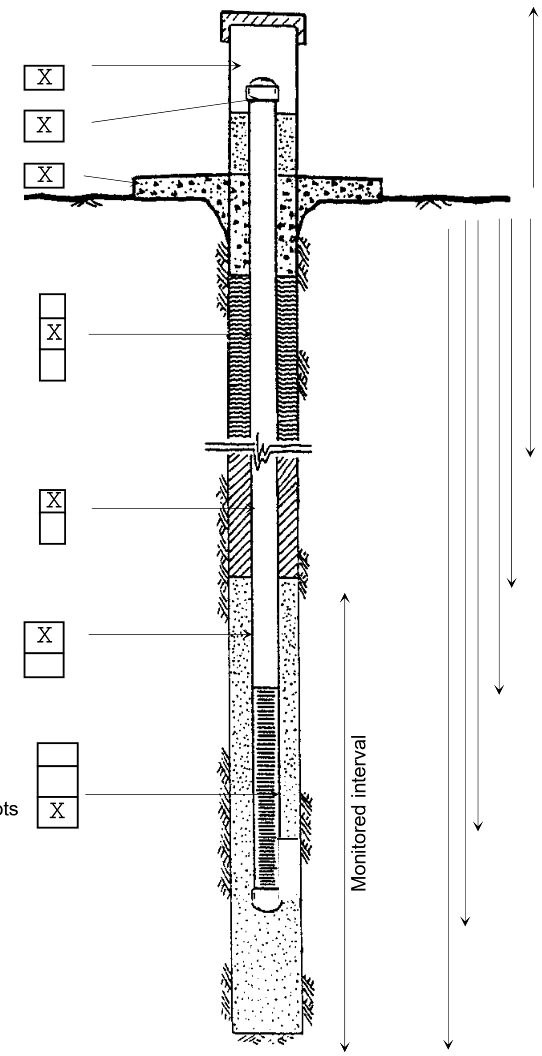
Concrete colar [X]

Back fill type:
Cement bentonite [ ]
Soil [X]
None [ ]

Seal:
Bentonite pellets [X]
Other [ ]

Gravel type:
2-5mm gravel [X]
Other [ ]

Perforation type:
Drill holes [ ]
Hack saw cuts [ ]
40um machine slots [X]



Complete dimensions if appropriate

Height of stickup (m)	0
Diameter of PVC (mm)	50
Depth to top of seal (m)	0
Depth to top of gravel pack (m)	0.5
Depth to top of screen (m)	0.8
Depth to base of screen (m)	5.3
Depth to base of piezo (m)	5.3
Depth to base of gravel (m)	8.0

Sandstone 5.0 m bgl

COMMENTS:
HOBO placed in standpipe for long-term groundwater monitoring at 4.0 m below ground level.
During a ground water monitoring trip on 9 July 2025, the HOBO was removed from this standpipe and relaunched into BH-PZ4.

Pells Sullivan Meynink
Engineering Consultants
Rock - Soil - Water

JOB no: **PSM5745**

PROJECT: **Homes NSW**
68-80 Beauchamp Road, Hillsdale

PIEZOMETER CONSTRUCTION RECORD

HOLE NUMBER: BH-PZ3
PIEZOMETER: BH-PZ3
COLLAR EASTING: 336308
COLLAR NORTHING: 6241332
COLLAR RL(m): 15.3
DATUM: AHD GDA2020/MGA56

DRILLING CONTRACTOR: Legion Drilling
RIG: Geoprobe 7720R
DEPTH OF HOLE (m): 8
BOREHOLE INCLINATION: -90
PIEZO INSTALLATION DATE: 20/06/2025
SUPERVISED BY: BTA

Tick boxes

Steel protective well cover [X]

PVC cap [X]

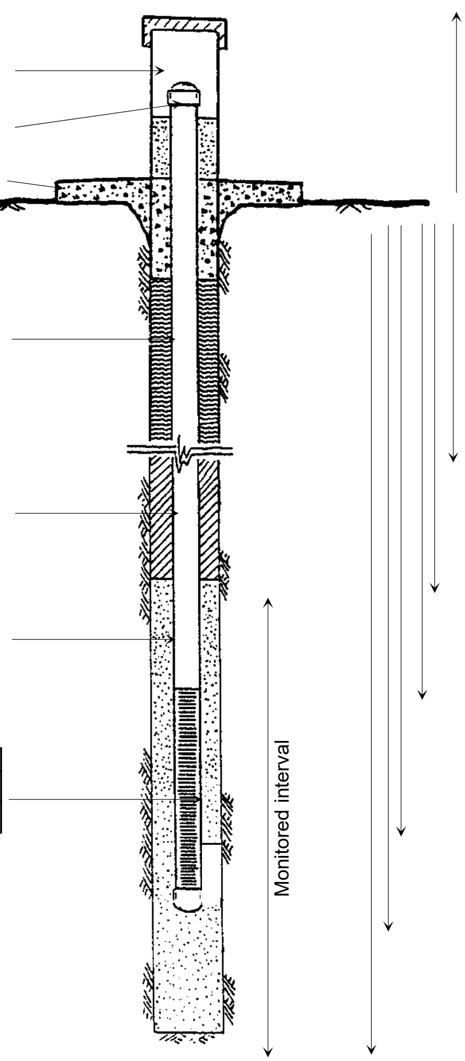
Concrete colar [X]

Back fill type:
Cement bentonite [ ]
Soil [X]
None [ ]

Seal:
Bentonite pellets [X]
Other [ ]

Gravel type:
2-5mm gravel [X]
Other [ ]

Perforation type:
Drill holes [ ]
Hack saw cuts [ ]
40um machine slots [X]



Complete dimensions if appropriate

Height of stickup (m)	0
Diameter of PVC (mm)	50
Depth to top of seal (m)	0
Depth to top of gravel pack (m)	0.7
Depth to top of screen (m)	1.2
Depth to base of screen (m)	7.2
Depth to base of piezo (m)	7.2
Depth to base of gravel (m)	8.0

Sandstone 3.2 m bgl

COMMENTS:
HOBO placed in standpipe for long-term groundwater monitoring at 6.0 m below surface level.

PIEZOMETER CONSTRUCTION RECORD

HOLE NUMBER: BH-PZ4
PIEZOMETER: BH-PZ4
COLLAR EASTING: 336292
COLLAR NORTHING: 6241386
COLLAR RL(m): 13.70
DATUM: AHD GDA2020/MGA56

DRILLING CONTRACTOR: Legion Drilling
RIG: Geoprobe 7720R
DEPTH OF HOLE (m): 8
BOREHOLE INCLINATION: -90
PIEZO INSTALLATION DATE: 20/06/2025
SUPERVISED BY: BTA

Tick boxes

Steel protective well cover [X]

PVC cap [X]

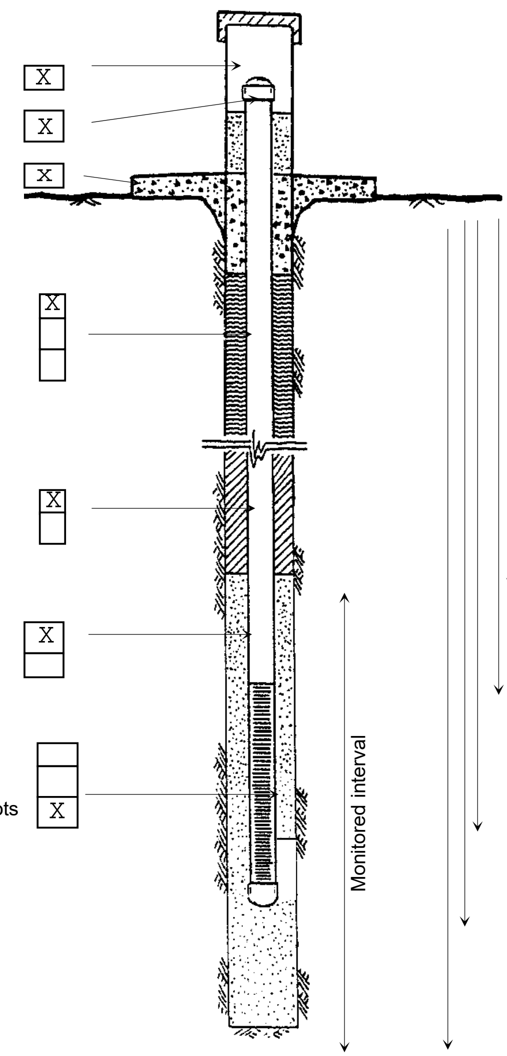
Concrete colar [X]

Back fill type:
Cement bentonite [X]
Soil [ ]
None [ ]

Seal:
Bentonite pellets [X]
Other [ ]

Gravel type:
2-5mm gravel [X]
Other [ ]

Perforation type:
Drill holes [ ]
Hack saw cuts [ ]
40um machine slots [X]



Complete dimensions if appropriate

Height of stickup (m): 0

Diameter of PVC (mm): 50

Depth to top of seal (m): 0

Depth to top of gravel pack (m): 0.5

Depth to top of screen (m): 1

Sandstone 5.2 m bgl

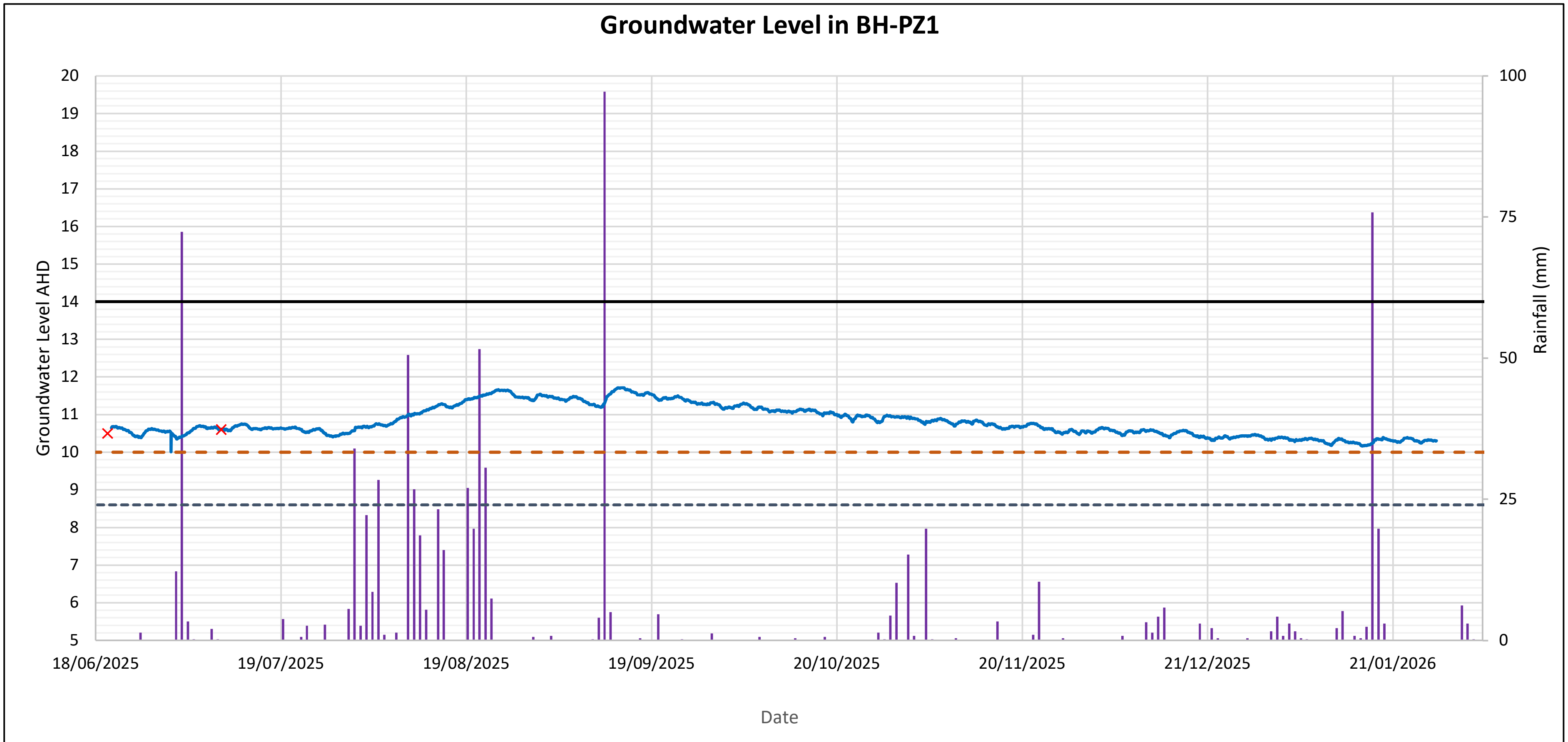
Depth to base of screen (m): 5.2

Depth to base of piezo (m): 5.2

Depth to base of gravel (m): 5.2

COMMENTS:
During a ground water monitoring trip on 9 July 2025, a HOBO was removed from BH-PZ2 on 9 July 2025 and relaunched into BH-PZ4.
The HOBO sits at 4.0 m below surface level.

Appendix B
Groundwater Monitoring



Notes:

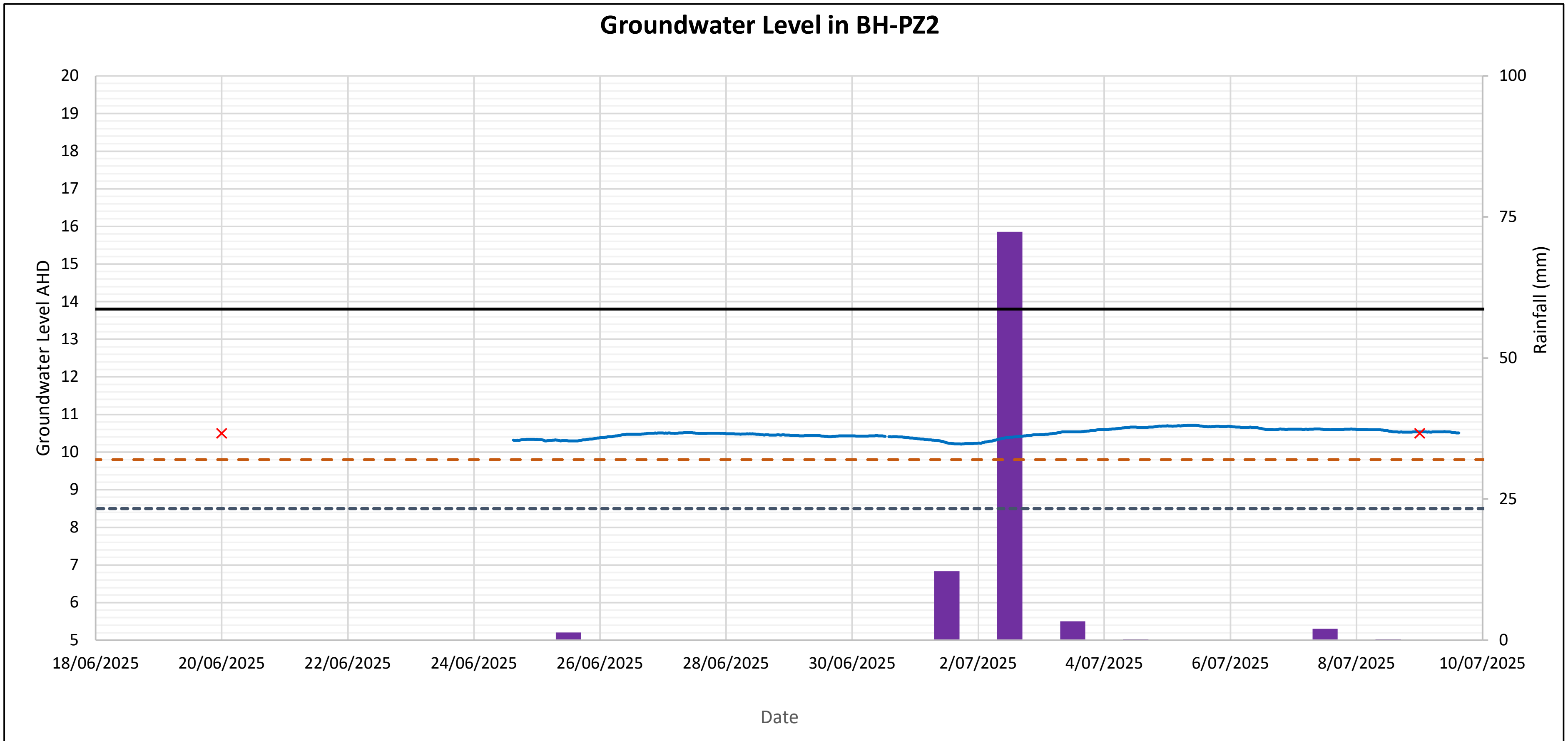
1. Ground Surface Elevation (m AHD)	14.0
2. Instrument Elevation (m AHD)	10.0
3. Data Logger Instrument Installed on 20/06/2025	
4. Bottom of Seal (m AHD)	13.6
5. Bottom of Screen (m AHD)	8.6
6. Rainfall data from BOM station 066037, downloaded 05/02/2026	

Rainfall
GW Level
Ground Surface
Instrument Level
Bottom of Screen
Manual Water Reading

JBS&G
Homes NSW
68-80 Beauchamp Road, Hillsdale
BH-PZ1

Groundwater Levels

PSM5745-018R	Appendix B



Notes:

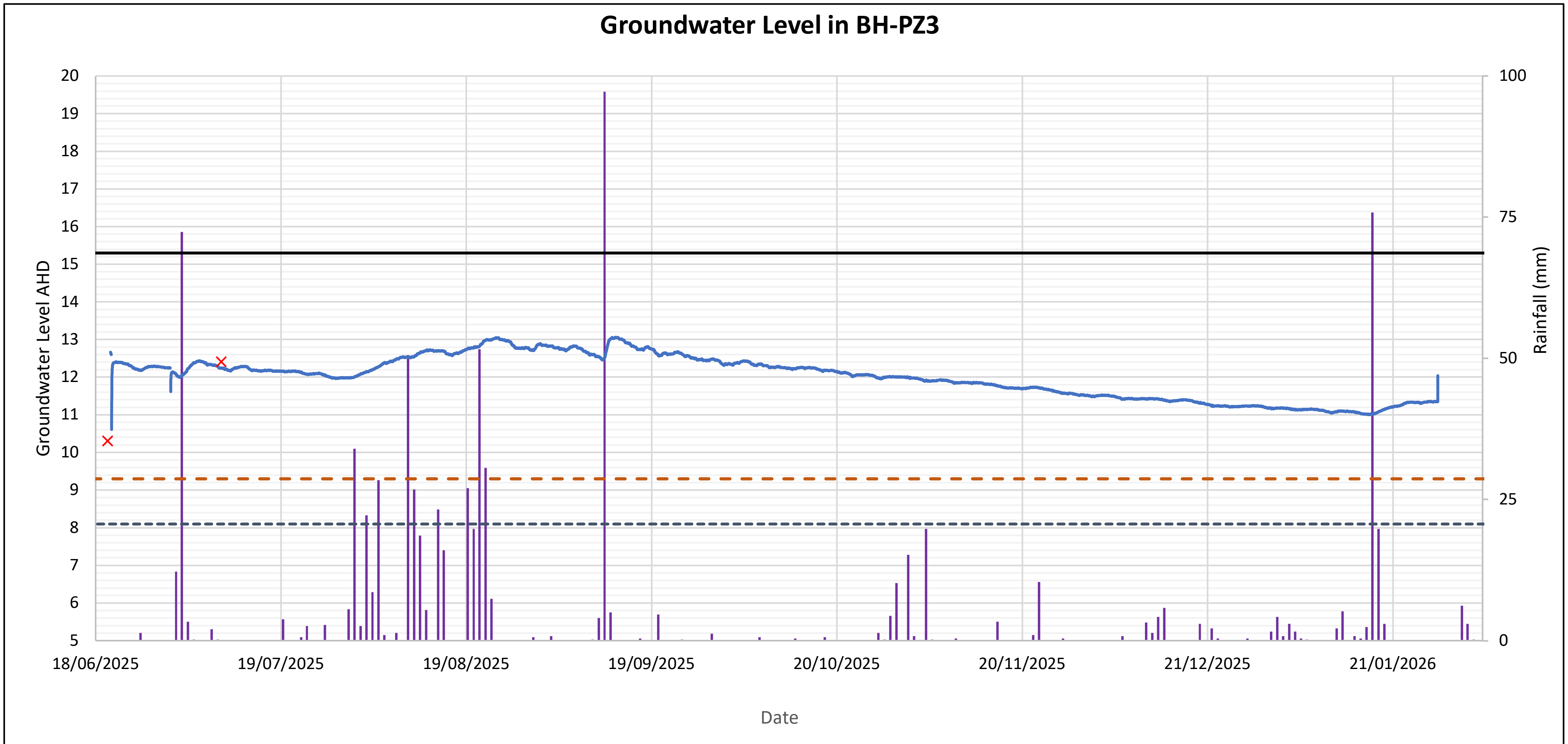
1. Ground Surface Elevation (m AHD)	13.8
2. Instrument Elevation (m AHD)	9.8
3. Data Logger Instrument Installed on 24/06/2025	
4. Bottom of Seal (m AHD)	13.3
5. Bottom of Screen (m AHD)	8.5
6. Rainfall data from BOM station 066037, downloaded 15/07/2025	

Rainfall
GW Level
Ground Surface
Instrument Level
Bottom of Screen
Manual Water Reading

JBS&G
Homes NSW
68-80 Beauchamp Road, Hillsdale
BH-PZ2

Groundwater Levels

PSM5745-018R	Appendix B



Notes:

1. Ground Surface Elevation (m AHD)	15.3
2. Instrument Elevation (m AHD)	9.3
3. Data Logger Instrument Installed on 20/06/2025	
4. Bottom of Seal (m AHD)	14.6
5. Bottom of Screen (m AHD)	8.1
6. Rainfall data from BOM station 066037 , downloaded 05/02/2026	

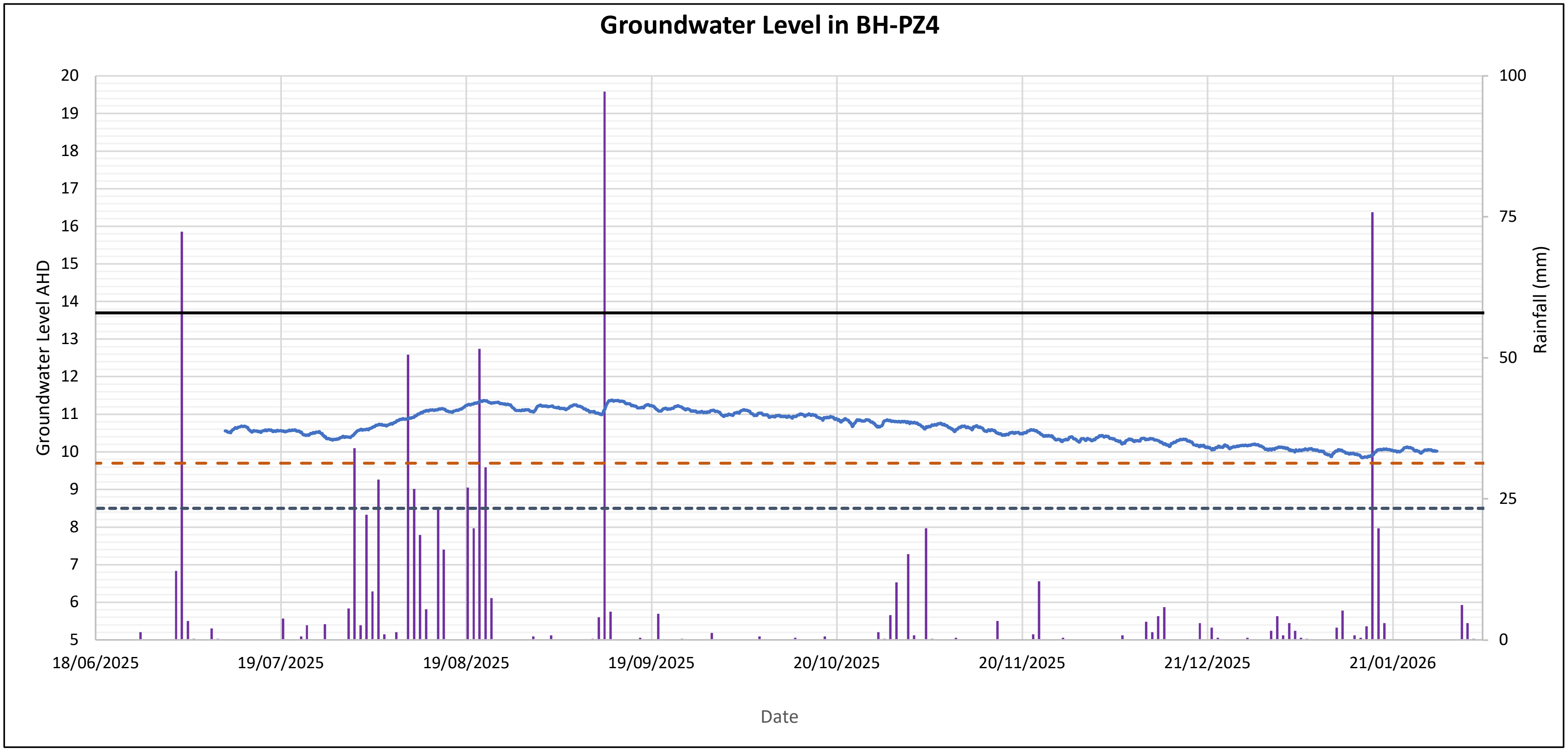
Rainfall
GW Level
Ground Surface
Instrument Level
Bottom of Screen
× Manual Water Reading

PSM

JBS&G
Homes NSW
68-80 Beauchamp Road, Hillsdale
BH-PZ3

Groundwater Levels

PSM5745-018R	Appendix B



Notes:

1. Ground Surface Elevation (m AHD)	13.7
2. Instrument Elevation (m AHD)	9.7
3. Data Logger Instrument Installed on 20/06/2025	
4. Bottom of Seal (m AHD)	13.2
5. Bottom of Screen (m AHD)	8.5
6. Rainfall data from BOM station 066037 , downloaded 05/02/2026	

Rainfall
GW Level
Ground Surface
Instrument Level
Bottom of Screen
Manual Water Reading

JBS&G
Homes NSW
68-80 Beauchamp Road, Hillsdale
BH-PZ4
Groundwater Levels

PSM5745-018R	Appendix B

Appendix C
Slug Tests

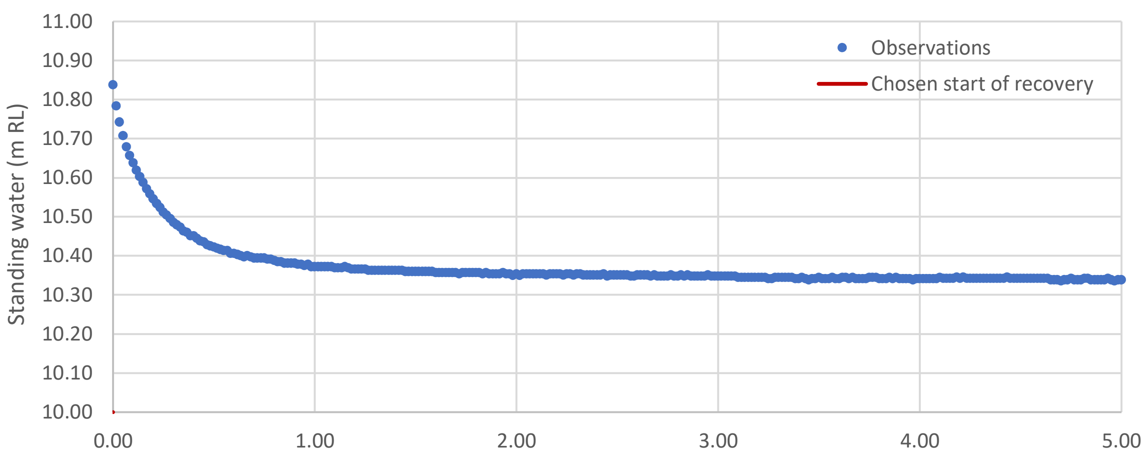
Pells Sullivan Meynink
Engineering Consultants
Rock - Soil - Water

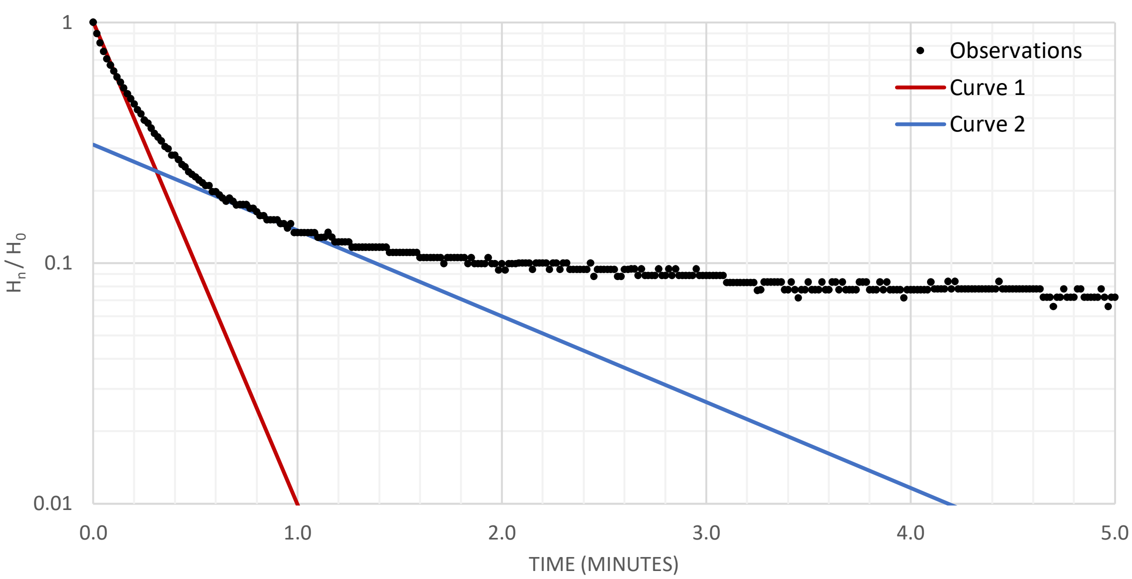
JOB no.: **PSM5745**

PROJECT: **Hillsdale**

Slug Test

HOLE NUMBER:	**BH-PZ1**	DEPTH OF HOLE (m):	**5.40**
BORE EASTING:	**336307**	SCREEN LENGTH 'L' (m):	**4.5**
BORE NORTHING:	**6241432**	INTERNAL DIAMETER 'D_I' (m):	**0.050**
COLLAR RL(m):	**14.0**	EXTERNAL DIAMETER 'Do' (m):	**0.06**
DATUM:	**m AHD**	SCREEN SHAPE FACTOR 'F'	**5.64**
CONTRACTOR	**Legion Drilling**	WATER DIP (m below collar)	**3.70**
WELL TESTED BY:	**JY**	STANDING WATER (m AHD)	**10.30**
SCREENED LITHOLOGY:	**NATURAL SOIL**	DATE	**28-Jan-26**





	Time lag 'T'	Hydraulic conductivity 'k'	
	mins	m/s	m/day
Curve 1	0.2	3.3E-05	2.9E+00
Curve 2	1.2	4.8E-06	4.1E-01

From Hvorslev (1951) $k = \frac{A}{FT}$

where $A = \pi D_I^2/4$

$$F = \frac{2\pi L}{\ln\left(\frac{L}{D_o} + \sqrt{1 + \left(\frac{L}{D_o}\right)^2}\right)}$$

$$T = \frac{t_2 - t_1}{\ln(H_1/H_0) - \ln(H_2/H_0)}$$

H_n = drawdown at time t_n

COMMENTS:

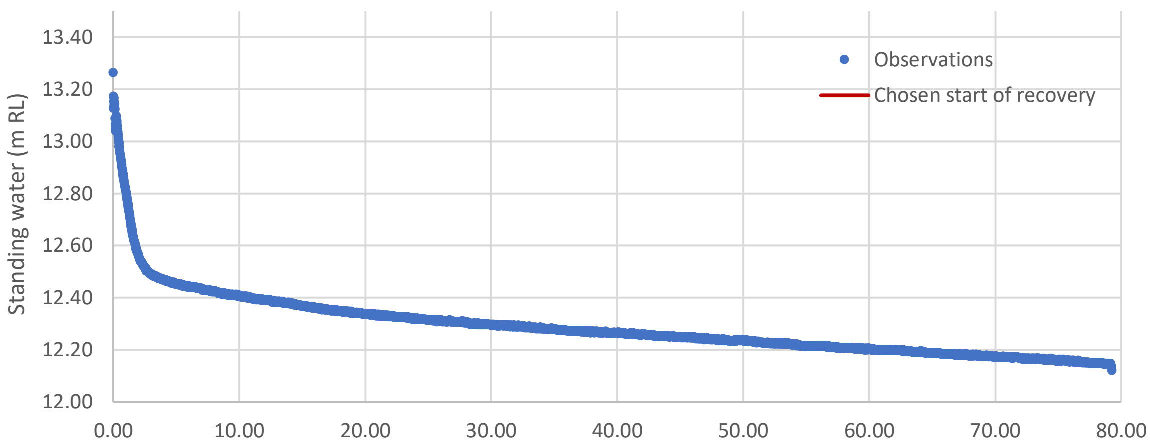
Pells Sullivan Meynink
Engineering Consultants
Rock - Soil - Water

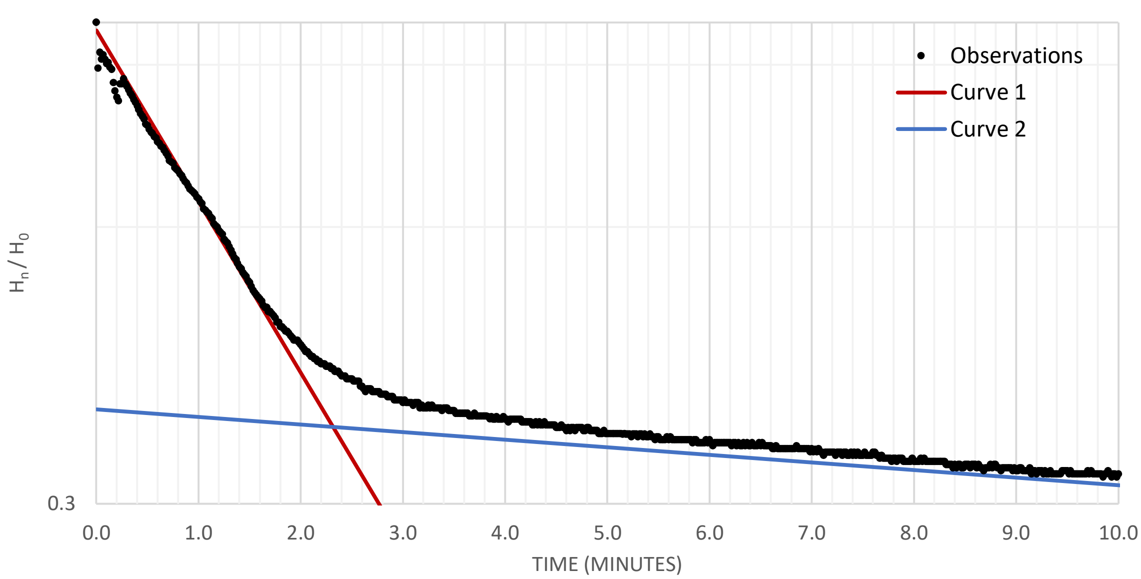
JOB no.: **PSM5745**

PROJECT: **Hillsdale**

Slug Test

HOLE NUMBER:	**BH-PZ3**	DEPTH OF HOLE (m):	**7.20**
BORE EASTING:	**336308**	SCREEN LENGTH 'L' (m):	**6.0**
BORE NORTHING:	**6241332**	INTERNAL DIAMETER 'D_I' (m):	**0.050**
COLLAR RL(m):	**15.3**	EXTERNAL DIAMETER 'Do' (m):	**0.06**
DATUM:	**m AHD**	SCREEN SHAPE FACTOR 'F'	**7.12**
CONTRACTOR	**Legion Drilling**	WATER DIP (m below collar)	**3.30**
WELL TESTED BY:	**JY**	STANDING WATER (m AHD)	**12.00**
SCREENED LITHOLOGY:	**NATURAL SOIL, SANDSTONE**	DATE	**28-Jan-26**





	Time lag 'T'	Hydraulic conductivity 'k'	
	mins	m/s	m/day
Curve 1	2.2	2.1E-06	1.8E-01
Curve 2	52.7	8.7E-08	7.5E-03

From Hvorslev (1951) $k = \frac{A}{FT}$

where $A = \pi D_I^2/4$

$$F = \frac{2\pi L}{\ln\left(\frac{L}{D_o} + \sqrt{1 + \left(\frac{L}{D_o}\right)^2}\right)}$$

$$T = \frac{t_2 - t_1}{\ln(H_1/H_0) - \ln(H_2/H_0)}$$

H_n = drawdown at time t_n

COMMENTS:
Screened lithology (1.2 mBGL to 7.2 mBGL):
NATURAL SOIL between 1.2 mBGL to 3.2 mBGL.
SANDSTONE between 3.2 mBGL to 7.2 mBGL.

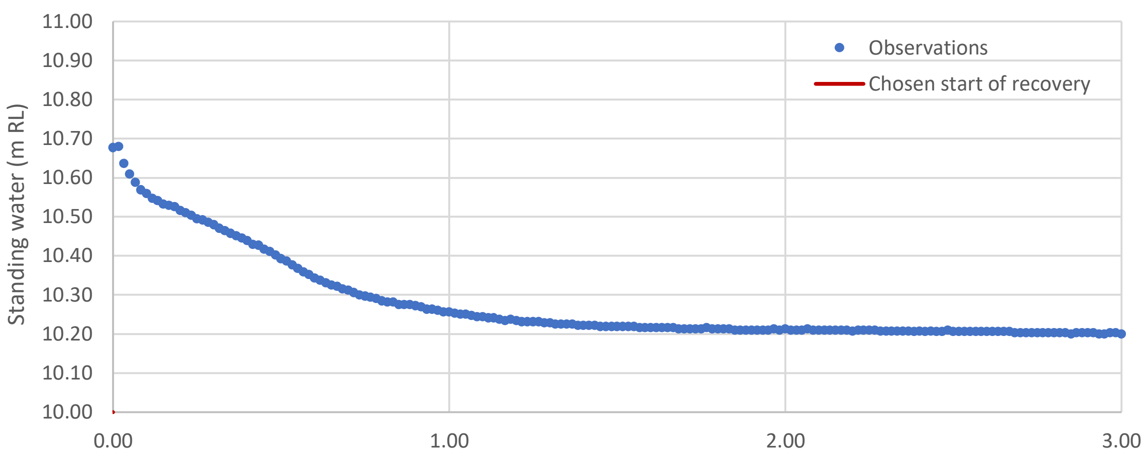
Pells Sullivan Meynink
Engineering Consultants
Rock - Soil - Water

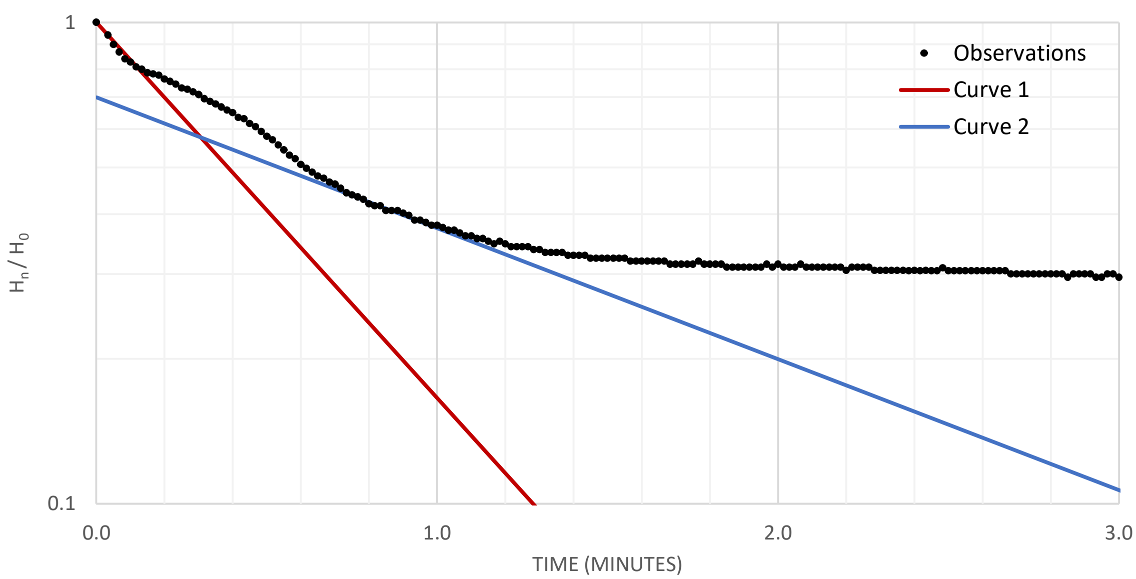
JOB no.: **PSM5745**

PROJECT: **Hillsdale**

Slug Test

HOLE NUMBER:	**BH-PZ4**	DEPTH OF HOLE (m):	**5.20**
BORE EASTING:	**336292**	SCREEN LENGTH 'L' (m):	**4.2**
BORE NORTHING:	**6241386**	INTERNAL DIAMETER 'D_I' (m):	**0.050**
COLLAR RL(m):	**13.7**	EXTERNAL DIAMETER 'Do' (m):	**0.06**
DATUM:	**m AHD**	SCREEN SHAPE FACTOR 'F'	**5.34**
CONTRACTOR	**Legion Drilling**	WATER DIP (m below collar)	**3.70**
WELL TESTED BY:	**JY**	STANDING WATER (m AHD)	**10.00**
SCREENED LITHOLOGY:	**NATURAL SOIL**	DATE	**28-Jan-26**





	Time lag 'T'	Hydraulic conductivity 'k'	
	mins	m/s	m/day
Curve 1	0.5	1.3E-05	1.1E+00
Curve 2	1.6	3.8E-06	3.3E-01

From Hvorslev (1951) $k = \frac{A}{FT}$

where $A = \pi D_I^2/4$

$$F = \frac{2\pi L}{\ln\left(\frac{L}{D_o} + \sqrt{1 + \left(\frac{L}{D_o}\right)^2}\right)}$$

$$T = \frac{t_2 - t_1}{\ln(H_1/H_0) - \ln(H_2/H_0)}$$

H_n = drawdown at time t_n

COMMENTS: