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02 September 2025
E25834.G12

Caddens Pty Ltd
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Groundwater Seepage Analysis
Proposed Mixed-Used Development
68&80 O'connell Street, Caddens NSW

1. INTRODUCTION

1.1. BACKGROUND

At the request of Caddens Pty Ltd (the Client), EI Australia (EI) has prepared this Groundwater Seepage Analysis for 68&80 O'connell Street, Caddens NSW (the site).

EI has prepared the following documents

- Geotechnical Investigation (GI) report for the site, referenced E25834.G03, dated 18 January 2023
- Additional Geotechnical Investigation (AGI) report for the site, referenced E25834.G04_Rev5, dated 18 August 2025

This is a preliminary groundwater seepage analysis report prepared to support SSDA lodgement.

1.2. PROPOSED DEVELOPMENT

The following documents, supplied by the Client, were used to assist with the preparation of this GI report:

- Architectural Drawings prepared by Group GSA, project number A252106, Drawing Nos: SSDA0000-SSDA3026, Issue P2,dated 22 July 2025;
- Site survey plan prepared by Ramsay Surveyors Pty Ltd – Drawing Ref No. 8966, Sheet 1 to 7, dated on 5 February 2021; and
- Request for Geotechnical Proposal prepared by Northrop, dated 19 October 2022.

Based on the provided documents and clarifications from Holdmark, EI understands that the proposed development involves the following:

- Demolition of an existing at grade car park
- Construction of 4 x 4 - 6 storey shop top housing buildings at 68 O'Connell Street, Caddens, comprising:
    - 177 residential dwellings
    - Ground floor commercial retail, and medial uses
- Construction of a single storey commercial premises (shop) located at the southwestern corner of 68 O'Connell Street
- Construction of basement car parking to service the shop top housing, ground floor commercial and commercial premises at the southwestern corner

- Construction of 11 x 4 – 7 storey residential flat buildings at 80 O'Connell Street, including:
  - 305 residential dwellings
  - Basement car parking
- Internal roads
- Associated infrastructure and services
- Landscaping and communal open spaces including a 'public square'

EI understand that No.68&80O'Connell Street has a proposed layout of the basement and the design Finish Floor Level (FFL) of the lowest basement level is RL 38.90m and RL 55.3m to 49.6 AHD, respectively . A bulk excavation Level (BEL) is assumed to be 38.6m and 55.0m to 49.3m AHD, respectively, including the basement slab thickness.

To achieve the BEL, excavation depths from 4.4m to 10.9m Below Existing Ground Level (BEGL) have been estimated. Locally deeper excavations may be required for footings, lift overrun pits, crane pads, and service trenches.

1.3. ASSESSMENT OBJECTIVES

The objective of this GSA is to provide an estimation of the groundwater take volumes that require pumping out during the construction and operational stage of the development, estimation of the groundwater drawdown as a result of the dewatering, and its associated ground settlements (if any).

2. SITE MODEL

2.1. MODELLED SECTIONS

One section as shown below were modelled and analysed as a part of GSA because of the sloping topography, the location of the sections is shown in the attached **Figure 2**:

- Section A-A: A half-section of the site through the eastern site boundary where excavation and dewatering will be deepest. For Buildings U, T, S, P, R2 and R.
- Section B-B: A half-section of the site through the eastern site boundary where excavation and dewatering will be deepest. For Buildings B, C, E, J, H, G and F.
- Section C-C: A half-section of the site through the eastern site boundary where excavation and dewatering will be deepest. For Buildings N, M, L and K.

EI understands the building B, C, E, J, H, G and F share one basement outline, buildings U, T, S, P, R2 and R share one basement outline and building N, M, L and K share one basement outline. Therefore, each half section was adopted for each basement outline.

2.2. SUBSURFACE CONDITIONS PERMEABILITY

For the purpose of the groundwater seepage analysis, the subsurface conditions outlined EI's GI report referenced above have been used. For modelling Section A-A, the subsurface condition were adopted from BH101M along the eastern boundary and both Sections B-B and C-C the subsurface conditions were adopted from BH103M and BHH7M, respectively, both located along the western boundary. A summary of the permeability values which were adopted for the assessment of groundwater take volumes are presented in **Table 1** below.

Table 1 Summary of Subsurface Conditions and Adopted Design Parameters

Material [1]	Adopted RL of Top of Unit (m AHD) [2]			Adopted Permeability (m/s)	Permeability Anisotropy $k_{vertical}/k_{horizontal}$
	Section A-	Section B-B	Section C-C		

Material [1]	Adopted RL of Top of Unit (m AHD) [2]			Adopted Permeability	Permeability Anisotropy
	A				
	East	West	West		
Fill	49.20	58.00	63.00	1×10^{-5}	1.0
Residual Soil	48.60	57.00	62.90	1×10^{-8}	1.0
Very Low Strength Siltstone/Sandstone	43.90	52.80	62.10	2.8×10^{-7}	0.5
Low Strength Siltstone/Sandstone/Shale	35.80	48.23	-	0.5×10^{-7}	0.5
Medium Strength Bedrock (Sandstone / Siltstone	-	-	60.70	1.0×10^{-7}	0.3

The permeability the Siltstone bedrock was calculated based on the pump-out test rests completed within monitoring wells and average number was adopted. The monitoring wells and pump out test results are summarised in Error! Reference source not found. below.

2.3. GROUNDWATER OBSERVATIONS AND PUMP OUT TESTS

EI have previously installed monitoring wells BH101M,BH103M and BH104M for this site. Data loggers were installed in the groundwater monitoring wells (BH101M, BH103M and BH104M) for the long-term groundwater monitoring on the13 August 2025 and at the time of writing this GSA no long-term water data was adopted. A Summary of groundwater levels recorded by EI at different occasions for BH101M, BH103M andBH104M are presented in **.**

EI carried out the Pump Out test on 12 June 2025 and 13 August 2025 within the monitoring wells BH101M, BH103M and BH104M installed by EI. The details of the rising head tests are presented in **Table 2** below. A Summary of groundwater levels recorded by EI at different occasions including BH1M prior to damage is presented in **.**

Table 2 Monitoring Well Details and Pump Out Test Results

Monitoring Well/ Test ID	Total Well Depth (m BEGL)	Screen Length (m)	Screened Section	Date of Test	Approximate RL of Groundwater Level (m AHD)	Adopted Permeability (m/s) [3]
BH101M	11.6	6	Very Low Strength Siltstone	12-June-25 13 August -25	45.30 45.87	3.11×10^{-7} 2.40×10^{-7}
BH103M	8.0	3	Very Low Strength Siltstone	12-June-25 13 August -25	50.05 50.95	1.07×10^{-8} 1.62×10^{-8}
BH104M	5.2	3	Residual Soil	12-June-25 13 August -25	54.37 54.48	-[1]

Note 1- No recharge observed after bailing well dry.

Table 2 Summary of Groundwater Levels

Monitoring Well/ Borehole ID	Date of Observation	Approximate Depth to Groundwater Level (m BEGL)	Approximate RL of Groundwater Level (m AHD)
BH101M	12-June-25 13-Aug-25	3.90 3.33	45.30 45.87

BH103M	12-June-25 13-Aug-25	5.77 4.87	50.05 50.95
BH104M	12-June-25 13- Aug-25	4.33 4.22	54.37 54.48
BHA7M	21-Nov-22	4.60	46.70
BHD4M	21-Nov-22	3.80	49.90
BHG1M	21-Nov-22	2.80	53.30
BHH7M	21-Nov-22	3.30	58.90

For the purpose of the assessment, to account for seasonal fluctuation a design groundwater level in of RL between 46.90-59.90 m AHD (which is 1 m above the highest recorded water level encountered in BH3M as shown in **Table 3** above).

2.4. THE MAXIMUM EXTENT OF THE CONE OF DEPRSSION

The estimated maximum extent of the cone of depression has been determined using Sichardt's Formula, as follows:

$$R = 3000\ (H - h)\sqrt{k}$$

Where:

- R = Maximum extent of the cone of depression (m).
- H = Highest design groundwater level (m).
- h = Estimated groundwater level at the BEL due to full duration of dewatering (m).
- k = Hydraulic conductivity of the aquifer (m/s).

The coefficient of 3000 is based on empirical observations, predominantly for sandy aquifers. The lowest coefficient outlined in Sichardt's formula is approximately 1000, which would result in a smaller radius of influence. Therefore, using a coefficient of 3000 in this case is considered conservative.

Based on the formula above, the maximum extent of the cone of depression is estimated to be approximately <10m, which is unrealistic. Hence, EI have adopted a conservative 40m distance of influence to assess the possible effects of drawdown and settlements.

2.5. SHORING SYSTEM

At the time of this assessment, no detailed structural designs were available. However, based on the recommendations of the geotechnical report and the site subsurface conditions of shallow bedrock, EI have assumed a drained contiguous pile shoring wall system for this site.

This assessment does not assess the overall stability and embedment depth of the shoring system. Once final designs are made available, this assessment should be revised accordingly.

3. GROUNDWATER TAKE ASSESSMENT

3.1. GROUNDWATER SEEPAGE VOLUMES DURING CONSTRUCTION PHASE

Groundwater seepage analysis for flow through and beneath the shoring wall during construction has been undertaken using PLAXIS 2D, a finite element groundwater seepage analysis software. PLAXIS 2D estimates the seepage rate of water entering the excavation from beneath the shoring wall. This model estimates the volume of water which will be required to be dewatered during the construction of the basement and until the dewatering is turned off.

For the purpose of this modelling, it has been assumed that:

- The subsurface conditions were horizontal along the site. The permeability values presented in **Table 1** above were adopted for each unit.
- The contiguous pile shoring walls are assumed to be impermeable.
- For the simplicity of this model, temporary dewatering will be undertaken within the basement retaining wall perimeter to BEL, or about RL 38.6m and 55.0m to 49.3m.
- An external design groundwater level of RL 46.90m,section A-A,RL 54.30m and 55.5m, Section B-B and section C-C 59.90m and 61.6m were assumed to be constant ranging between 0.0m-11.4m away from the shoring wall.
- A “No-Flow” boundary is defined along the symmetric line (the centre of the excavation), at Section A-A,40m, Section B-B, 80m and Section C-C, 64.50m from the perimeter shoring wall.
- The shoring walls surrounding the basement excavation has a total length of about 520m total basement parameter for buildings U,T,S,P,R1 and R2, about 400m basement parameter for buildings B,C,E J,,H,G and F and about 345m total basement parameter for buildings K,L,M and N.
- The basement will be constructed in 365 days.

The PLAXIS 2D model is presented in **Appendix A. Table 3** below provides the estimated groundwater inflow rate into the basement.

Table 3 Summary of Analysis Results

Section	Inflow per m length of perimeter wall (m^3/day)	Inflow into excavation (m^3/day)	Total Inflow during construction (ML/365 days)
A-A	0.021	5.46	1.99
B-B	0.027	5.40	1.97
C-C	0.040	6.88	2.51

3.2. ASSESSMENT OF GROUNDWATER TAKE DURING OPERATIONAL PHASE

A drained basement using sub-soil drainage and a sump-and-pump system was assumed. Based on the PLAXIS 2D results, the estimated volume of groundwater removed beneath the basement during the operational phase of the development is expected to be approximately section A-A 1.99ML per year, section B-B 1.97ML per year and section C-C 2.51ML per year.

3.3. GROUNDWATER DRAWDOWN AND DRAWDOWN INDUCED SETTLEMENT

EI have completed a settlement analysis using PLAXIS 2D to estimate the potential drawdown-induced settlements as a result of the above predicted drawdown. The soil parameters used in the settlement analysis are presented. The predicted drawdown over the zone of influence and settlement is shown in **Table 4**.

Table 4 Predicted Ground Settlement due to Groundwater Drawdown

Section	Maximum Drawdown (m)	Ground settlement (mm)
A-A	10.8	<1
B-B	6.5	<1
C-C	11.0	<1

The maximum predicted ground settlements are predicted to be less than 1 mm. Ground experiencing settlements less than 10mm are considered to be a 'Negligible' risk in regards to category of damage risk due to dewatering, as defined in Cashman and Preene (2021), as shown in the excerpt in **Plate 1**.

Although the PLAXIS 2D modelling provides predicted drawdown-induced ground settlement values, it would be prudent for a thorough assessment of potential risks posed on neighbouring structures to be completed by a qualified and experienced structural engineer.

Risk category[a]	*Maximum settlement (mm)*[b]	*Building tilt*[c]	*Anticipated effects*
Negligible	<10	<1/500	Superficial damage unlikely
Slight	10–50	1/500–1/200	Possible superficial damage; unlikely to have structural significance
Moderate	50–75	1/200–1/50	Expected superficial damage and possible structural damage to buildings; possible damage to rigid pipelines
Severe	75	>1/50	Expected structural damage to buildings and expected damage to rigid pipelines or possible damage to other pipelines

Source: Preene, M., *Proceedings of the Institution of Civil Engineers—Geotechnical Engineering*, 143(4), 177–190, 2000. With permission.

[a] The risk category is to be based on the more severe of the settlement or tilt criteria.
[b] Maximum settlement is based on the nearest edge of the structure to the groundwater control system.
[c] Tilt is based on rigid body rotation, assuming that all of the maximum settlement occurs as differential settlement across the width of the structure or across an element of the structure.

Plate 1 Excerpt from Cashman and Preene (2021)

4. CONCLUSIONS AND COMMENTS

Based on the findings of this report and within the limitations of available data, EI concludes that:

- Construction and operational phase groundwater take will be approximately:
  - Section A-A 1.99ML per year, section B-B 1.97ML per year and section C-C 2.51ML per year.
- The above estimate is based on the following assumptions:
  - The shoring wall system is an fully drained contiguous pile wall retention system;
  - Continuous dewatering in order to maintain the groundwater at a depth of BEL during construction, and construction of the basement will take 365 days;
  - The basement walls and slab will be designed as drained for the developments lifetime.
  - This assessment does not take into consideration any excavation that may be required for footings, service trenches, lift pits, or crane pads. This additional excavation, if required, is not expected to affect the retention or the dewatering system.
- Based on the predicted settlements of <1mm, the drawdown as a result of the dewatering will have negligible, if any, adverse impact on the neighbouring properties.
- Based on our assessment, the groundwater volumes expected per year appear to be manageable using a drained basement system for its lifetime. Hence in our opinion “tanking” of the basement is not warranted and a drained basement is possible for the development.

Should any design or construction conditions differ from that adopted in this report; this GSA should be reviewed and updated as required.

5. LIMITATIONS

This report has been prepared for the exclusive use of Caddens Pty Ltd who is the only intended beneficiary of EI’s work. The scope of the inspections carried out for the purpose of this report is limited to those agreed with Caddens Pty Ltd.

No other party should rely on the document without the prior written consent of EI, and EI undertakes no duty, or accepts any responsibility or liability, to any third party who purports to rely upon this document without EI's approval.

EI has used a degree of care and skill ordinarily exercised in similar tasks by reputable members of the geotechnical industry in Australia as at the date of this document. No other warranty, expressed or implied, is made or intended. Each section of this report must be read in conjunction with the whole of this report, including its appendices and attachments.

The conclusions presented in this report are based on a limited assessment of conditions, with specific locations chosen to be as representative as possible under the given circumstances.

EI's professional opinions are reasonable and based on its professional judgment, experience, training and results from analytical data. EI may also have relied upon information provided by the Client and other third parties to prepare this document, some of which may not have been verified by EI.

EI's professional opinions contained in this document are subject to modification if additional information is obtained through further investigation, observations, or validation testing and analysis during remedial activities. In some cases, further testing and analysis may be required, which may result in a further report with different conclusions.

6. CLOSURE

Please do not hesitate to contact the undersigned should you have any questions.

For and on behalf of
EI Australia

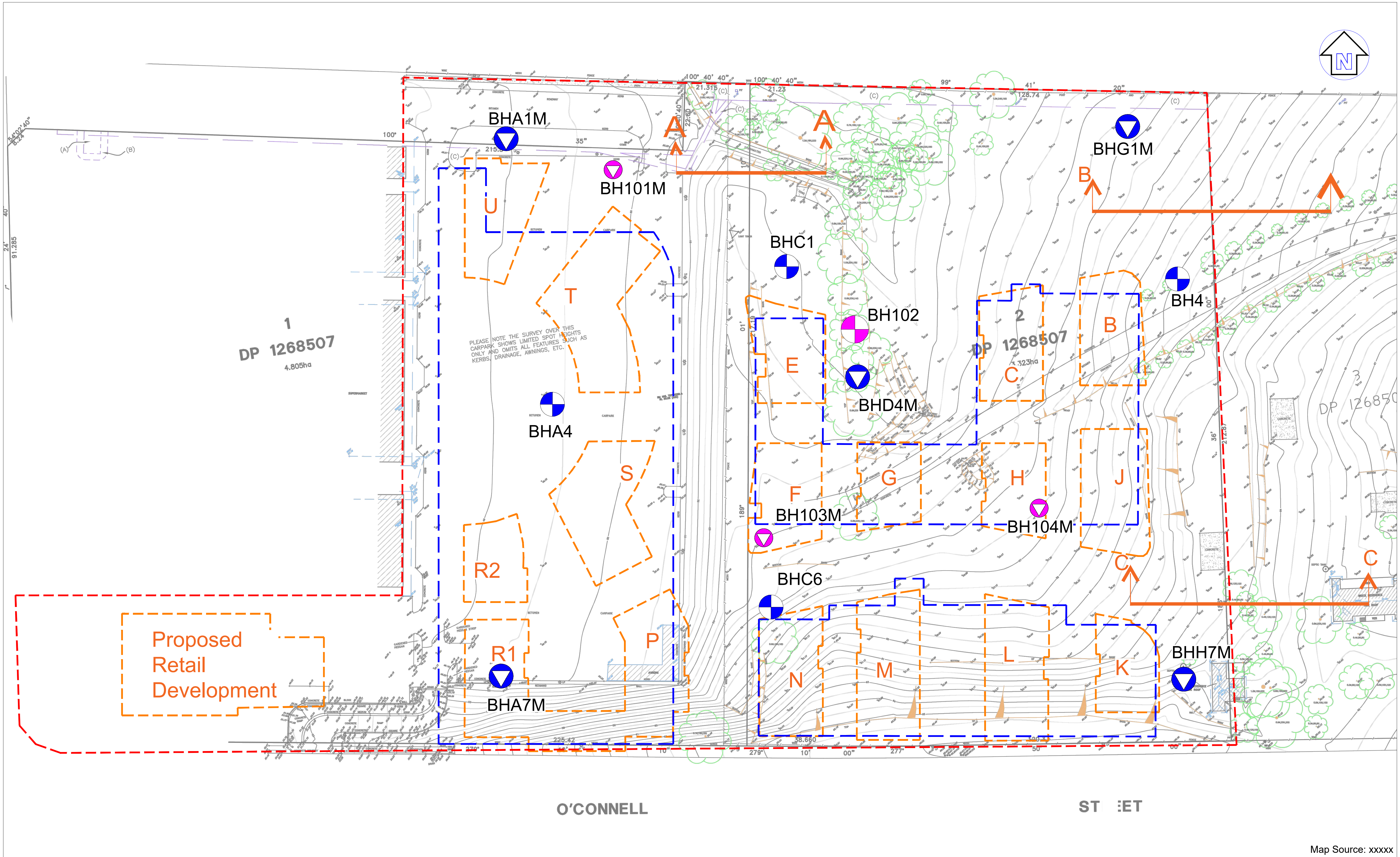
Author

Salah Khalifa
Geotechnical Engineer

Figure 1 – Analysed Section Plan
Appendix A – Pump-out Test Results
Appendix B – Important Information

Figures

Figure 1 Analysed Section Plan



Map Source: xxxxx

LEGEND (All Locations are Approximate)

- Site boundary
- Basement boundary
- Section
- Borehole location (EI Australia 2023)
- Monitoring well location (EI Australia, 2023)
- Monitoring well location (EI Australia 2025)
- Borehole location (EI Australia 2025)

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Caddens Estate Development Pty Ltd
Ground Seepage Analysis
68&80 O'cnnell Street, Caddens, NSW
Borehole Location Plan

Figure: 2

Project: E25834.G12

Appendix A – Pump out test results

Rising Head Permeability Test

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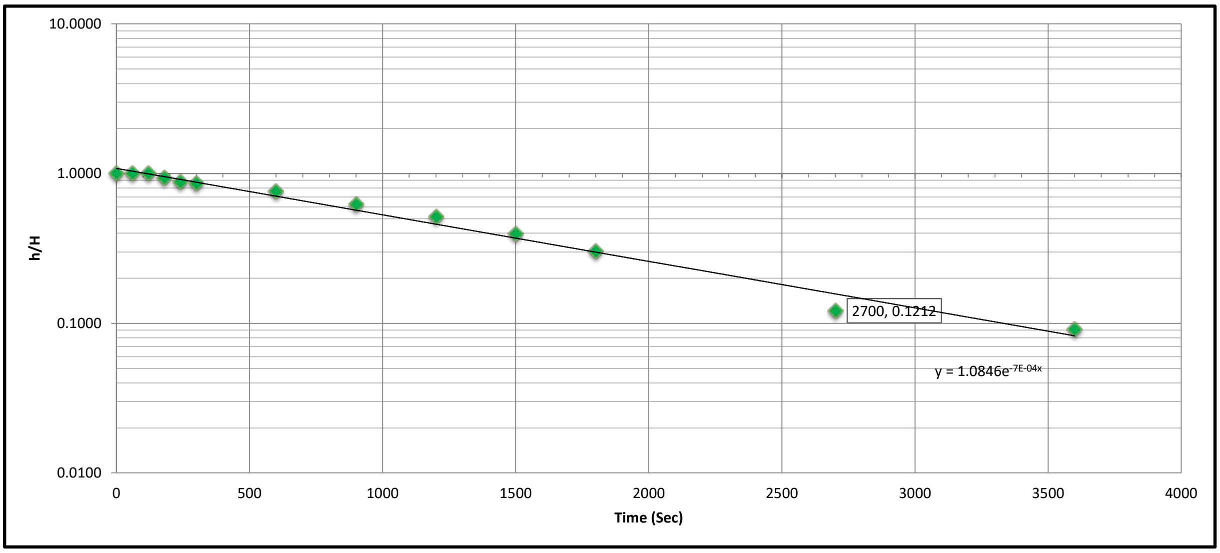
EI Job No.	E25834.G12		Test Date	12/06/2025
By	S2		Location	BH101M

Borehole Detail

BH No.	BH101M			Based on Hvorslev Method
Casing Stick-up (m)	-0.12			
Effective Piezo Screen Length (m)	3			$K = \frac{r^2 \ln(L/R)}{2LT_o}$
Piezo Radius r (m)	0.025			
Bore radium (Auger Radius) (m)	0.038	auger radius = 0.05; core radius is 0.038		
Depth of the piezometer (m BGL)	11.6			
Static Water Level (m BToC)	3.9		t1	180
Lag time T0 (sec)	4569.91		t2	3600
			h1	0.9394
Calculated Permeability k	**3.11E-07**	m/sec	h2	0.0909

Time (mins)	Time (sec)	Depth to water (ft BToC)	Depth to water (m BToC)	Change in Level (m)	h/H
Static			3.90		
0	0		7.2	3.30	1.0000
1	60		7.2	3.30	1.0000
2	120		7.2	3.30	1.0000
3	180		7	3.10	0.9394
4	240		6.8	2.90	0.8788
5	300		6.75	2.85	0.8636
10	600		6.4	2.50	0.7576
15	900		5.95	2.05	0.6212
20	1200		5.6	1.70	0.5152
25	1500		5.2	1.30	0.3939
30	1800		4.9	1.00	0.3030
45	2700		4.30	0.40	0.1212
60	3600		4.20	0.30	0.0909



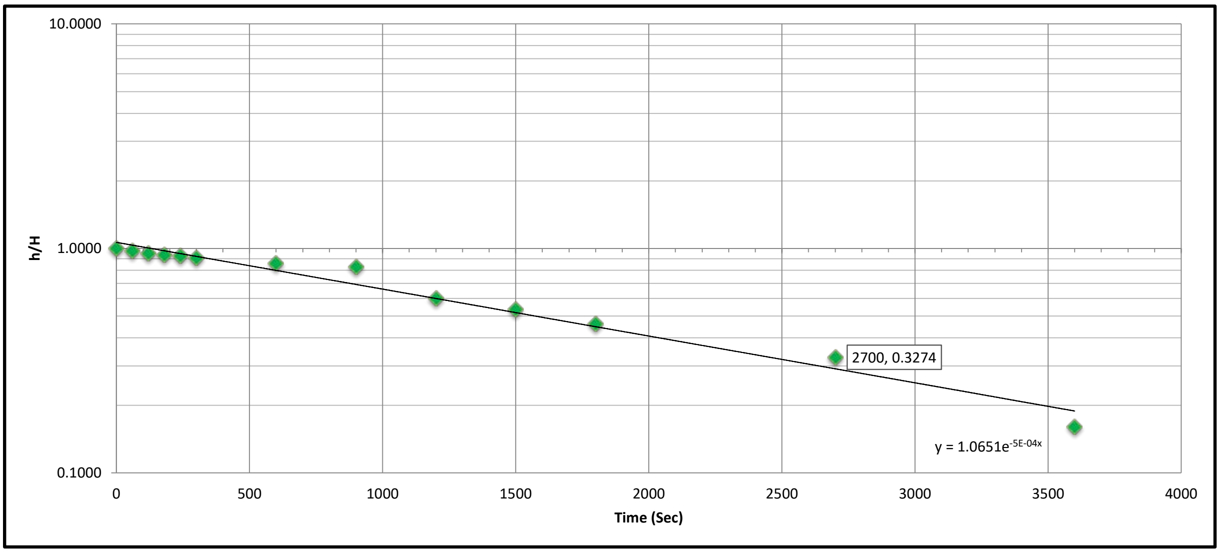
Rising Head Permeability Test

EI Job No.	E25834.G12		Test Date	13/08/2025
By	S2		Location	BH101M

Borehole Detail

BH No.	BH101M			Based on Hvorslev Method
Casing Stick-up (m)	-0.12			
Effective Piezo Screen Length (m)	3			$K = \frac{r^2 \ln(L/R)}{2LT_o}$
Piezo Radius r (m)	0.025			
Bore radium (Auger Radius) (m)	0.038	auger radius = 0.05; core radius is 0.038		
Depth of the piezometer (m BGL)	11.6			
Static Water Level (m BToC)	3.33		t1	600
Lag time T0 (sec)	4569.91		t2	1500
			h1	0.8574
Calculated Permeability k	**2.40E-07**	m/sec	h2	0.5332

Time (mins)	Time (sec)	Depth to water (ft BToC)	Depth to water (m BToC)	Change in Level (m)	h/H
Static			3.33		
0	0		9.5	6.17	1.0000
1	60		9.35	6.02	0.9757
2	120		9.2	5.87	0.9514
3	180		9.1	5.77	0.9352
4	240		9.05	5.72	0.9271
5	300		8.92	5.59	0.9060
10	600		8.62	5.29	0.8574
15	900		8.43	5.10	0.8266
20	1200		7.02	3.69	0.5981
25	1500		6.62	3.29	0.5332
30	1800		6.18	2.85	0.4619
45	2700		5.35	2.02	0.3274
60	3600		4.32	0.99	0.1605



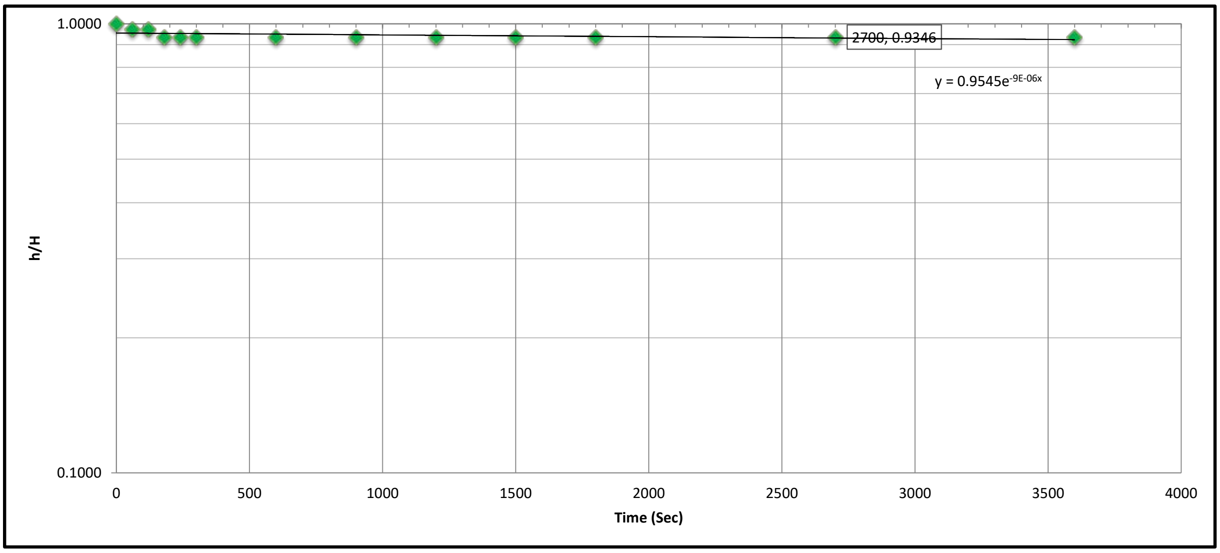
Rising Head Permeability Test

EI Job No.	E25834.G12		Test Date	12/06/2025
By	S2		Location	BH103M

Borehole Detail

BH No.	BH103M			Based on Hvorslev Method
Casing Stick-up (m)	0.92			
Effective Piezo Screen Length (m)	2.23			$K = \frac{r^2 \ln(L/R)}{2LT_o}$
Piezo Radius r (m)	0.025			
Bore radium (Auger Radius) (m)	0.038	auger radius = 0.05; core radius is 0.038		
Depth of the piezometer (m BGL)	8.37			
Static Water Level (m BToC)	5.77		t1	0
Lag time T0 (sec)	4569.91		t2	3600
			h1	1.0000
Calculated Permeability k	**1.07E-08**	m/sec	h2	0.9346

Time (mins)	Time (sec)	Depth to water (ft BToC)	Depth to water (m BToC)	Change in Level (m)	h/H
Static			5.77		
0	0		8.37	2.60	1.0000
1	60		8.3	2.53	0.9731
2	120		8.3	2.53	0.9731
3	180		8.2	2.43	0.9346
4	240		8.2	2.43	0.9346
5	300		8.2	2.43	0.9346
10	600		8.2	2.43	0.9346
15	900		8.2	2.43	0.9346
20	1200		8.2	2.43	0.9346
25	1500		8.2	2.43	0.9346
30	1800		8.2	2.43	0.9346
45	2700		8.2	2.43	0.9346
60	3600		8.2	2.43	0.9346



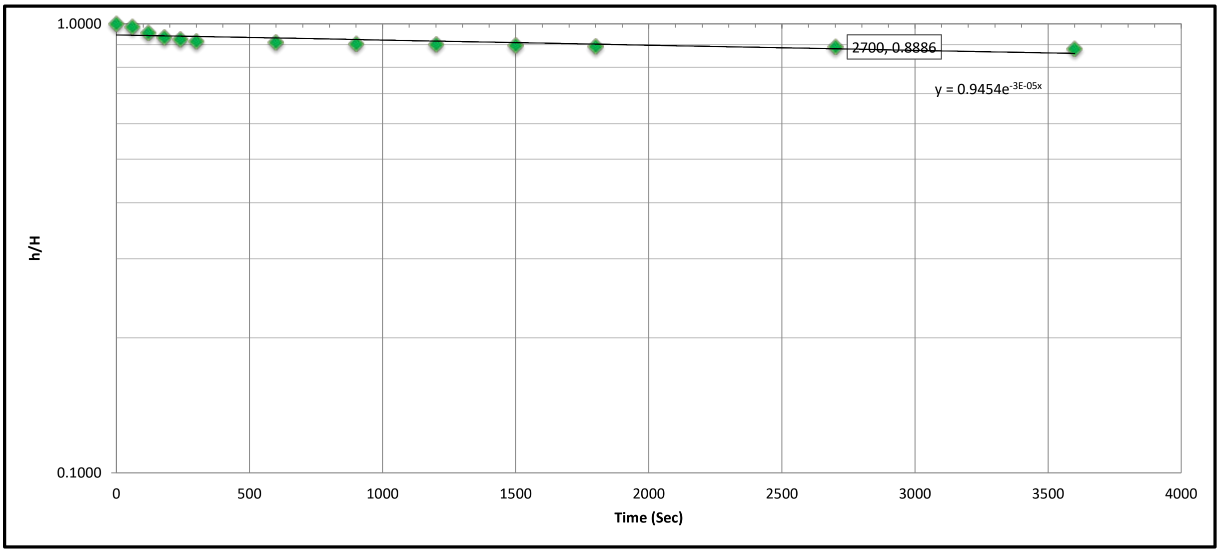
Rising Head Permeability Test

EI Job No.	E25834.G12		Test Date	13/08/2025
By	S2		Location	BH103M

Borehole Detail

BH No.	BH103M			Based on Hvorslev Method
Casing Stick-up (m)	0.92			
Effective Piezo Screen Length (m)	3			$K = \frac{r^2 \ln(L/R)}{2LT_o}$
Piezo Radius r (m)	0.025			
Bore radium (Auger Radius) (m)	0.038	auger radius = 0.05; core radius is 0.038		
Depth of the piezometer (m BGL)	8.37			
Static Water Level (m BToC)	4.87		t1	0
Lag time T0 (sec)	4569.91		t2	3600
			h1	1.0000
Calculated Permeability k	**1.62E-08**	m/sec	h2	0.8800

Time (mins)	Time (sec)	Depth to water (ft BToC)	Depth to water (m BToC)	Change in Level (m)	h/H
Static			4.87		
0	0		8.37	3.50	1.0000
1	60		8.32	3.45	0.9857
2	120		8.21	3.34	0.9543
3	180		8.13	3.26	0.9314
4	240		8.1	3.23	0.9229
5	300		8.07	3.20	0.9143
10	600		8.05	3.18	0.9086
15	900		8.03	3.16	0.9029
20	1200		8.02	3.15	0.9000
25	1500		8	3.13	0.8943
30	1800		7.99	3.12	0.8914
45	2700		7.98	3.11	0.8886
60	3600		7.95	3.08	0.8800



Appendix B – Important Information

Important Information

SCOPE OF SERVICES

The geotechnical report ("the report") has been prepared in accordance with the scope of services as set out in the contract, or as otherwise agreed, between the Client And EI Australia ("EI"). The scope of work may have been limited by a range of factors such as time, budget, access and/or site disturbance constraints.

RELIANCE ON DATA

EI has relied on data provided by the Client and other individuals and organizations, to prepare the report. Such data may include surveys, analyses, designs, maps and plans. EI has not verified the accuracy or completeness of the data except as stated in the report. To the extent that the statements, opinions, facts, information, conclusions and/or recommendations ("conclusions") are based in whole or part on the data, EI will not be liable in relation to incorrect conclusions should any data, information or condition be incorrect or have been concealed, withheld, misrepresented or otherwise not fully disclosed to EI.

GEOTECHNICAL ENGINEERING

Geotechnical engineering is based extensively on judgment and opinion. It is far less exact than other engineering disciplines. Geotechnical engineering reports are prepared for a specific client, for a specific project and to meet specific needs, and may not be adequate for other clients or other purposes (e.g. a report prepared for a consulting civil engineer may not be adequate for a construction contractor). The report should not be used for other than its intended purpose without seeking additional geotechnical advice. Also, unless further geotechnical advice is obtained, the report cannot be used where the nature and/or details of the proposed development are changed.

LIMITATIONS OF SITE INVESTIGATION

The investigation programme undertaken is a professional estimate of the scope of investigation required to provide a general profile of subsurface conditions. The data derived from the site investigation programme and subsequent laboratory testing are extrapolated across the site to form an inferred geological model, and an engineering opinion is rendered about overall subsurface conditions and their likely behaviour with regard to the proposed development. Despite investigation, the actual conditions at the site might differ from those inferred to exist, since no subsurface exploration program, no matter how comprehensive, can reveal all subsurface details and anomalies. The engineering logs are the subjective interpretation of subsurface conditions at a particular location and time, made by trained personnel. The actual interface between materials may be more gradual or abrupt than a report indicates.

SUBSURFACE CONDITIONS ARE TIME DEPENDENT

Subsurface conditions can be modified by changing natural forces or man-made influences. The report is based on conditions that existed at the time of subsurface exploration. Construction operations adjacent to the site, and natural events such as floods, or ground water fluctuations, may also affect subsurface conditions, and thus the continuing adequacy of a geotechnical report. EI should be kept appraised of any such events, and should be consulted to determine if any additional tests are necessary.

VERIFICATION OF SITE CONDITIONS

Where ground conditions encountered at the site differ significantly from those anticipated in the report, either due to natural variability of subsurface conditions or construction activities, it is a condition of the report that EI be notified of any variations and be provided with an opportunity to review the recommendations of this report. Recognition of change of soil and rock conditions requires experience and it is recommended that a suitably experienced geotechnical engineer be engaged to visit the site with sufficient frequency to detect if conditions have changed significantly.

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REPORT FOR BENEFIT OF CLIENT

The report has been prepared for the benefit of the Client and no other party. EI assumes no responsibility and will not be liable to any other person or organisation for or in relation to any matter dealt with or conclusions expressed in the report, or for any loss or damage suffered by any other person or organisation arising from matters dealt with or conclusions expressed in the report (including without limitation matters arising from any negligent act or omission of EI or for any loss or damage suffered by any other party relying upon the matters dealt with or conclusions expressed in the report). Other parties should not rely upon the report or the accuracy or completeness of any conclusions and should make their own inquiries and obtain independent advice in relation to such matters.

OTHER LIMITATIONS

EI will not be liable to update or revise the report to take into account any events or emergent circumstances or fact occurring or becoming apparent after the date of the report.