GROUNDED
EXPERTISE

Report on Preliminary Groundwater Assessment

Proposed Redevelopment of Compass Centre

83-99 North Terrace, Bankstown NSW

Prepared for Essence Project Management Pty Ltd

Project 213805.04

27 May 2024

Douglas Partners Pty Ltd
ABN 75 053 980 117
douglaspartners.com.au
96 Hermitage Road, West Ryde,
NSW 2114
(02) 9809 0666

Document History

Details

Project No.	213805.04
Document Title	Report on Preliminary Groundwater Assessment
Site Address	83-99 North Terrace, Bankstown NSW
Report Prepared For	Essence Project Management Pty Ltd
Filename	213805.04.R.001.Rev2

Status and Review

Status	Prepared by	Reviewed by	Date issued
Revision 0	Jun Yan	Scott Easton	1 March 2024
Revision 1	Jun Yan	Scott Easton	19 April 2024
Revision 2	Jun Yan	Scott Easton	27 May 2024

Distribution of Copies

Status	Issued to
Revision 0	Craig Sanders, Essence Project Management Pty Ltd
Revision 1	Craig Sanders, Essence Project Management Pty Ltd
Revision 2	Craig Sanders, Essence Project Management Pty Ltd

The undersigned, on behalf of Douglas Partners Pty Ltd, confirm that this document and all attached drawings, logs and test results have been checked and reviewed for errors, omissions and inaccuracies.

Signature		Date
Author	闫俊	27 May 2024
Reviewer	闫俊 pp Scott Easton	27 May 2024

Table of Contents

Page No

Report on Preliminary Groundwater Assessment
Proposed Redevelopment of Compass Centre
83-99 North Terrace, Bankstown NSW

1. Introduction

This report prepared by Douglas Partners (Douglas) outlines the preliminary groundwater assessment for the proposed redevelopment of compass centre at 83-99 North Terrace, Bankstown NSW. The work was commissioned in an email dated 15 February 2024 by Craig Sanders of Essence Project Management Pty Ltd and was undertaken in accordance with Douglas's proposal 213805.02.P.002.Rev0 dated 8 February 2024.

It is understood that the proposed development includes the demolition of existing buildings and construction of four high-rise towers over a shared two -level basement. The basement is proposed to be drained and will intersect the water table.

This assessment is based on the results of recent geotechnical investigation carried out by Douglas on the site (ref: 213805.02.R.001.Rev0 dated November 2023) as well as the long term groundwater level monitoring between 29 September 2023 and 12 January 2024 (ref: 213805.02.R.003.Rev0, dated January 2024).

The groundwater assessment included two-dimensional numerical modelling of the proposed excavation to assess groundwater inflow rates. Details of the numerical modelling are given in this report.

This report must be read in conjunction with all appendices including the notes provided in Appendix A.

2. Site Description

The site is located at 83-99 North Terrace, Bankstown and occupies an area of approximately 10,100 m^2. The site slopes down gently from the north-west to south-east, varying between about RL 22 m and RL 24 m AHD. The site is bounded by The Mall, The Appian Way, North Terrace and Fetherstone Street (to the north, east, south and west respectively) but the site excludes the existing multi-storey Fetherstone Apartments on the western boundary of the site (at 3-7 Fetherstone Street). It is understood from the supplied drawings that the multi-storey building at 3-7 Fetherstone Street has a single-level basement, and is founded above the proposed excavation level for this development.

The site is currently occupied by the Altis Compass Centre on the southern and eastern portions of the site, which comprises single and double storey commercial premises and a multi-storey tower, a vacant three-storey building in the north-western portion of the site which was formerly in use as a library and an on-grade asphaltic concrete carpark across the northern and western portion of the site. A single-level basement is present beneath the Compass Centre on the southern portion of the site and is accessed via Fetherstone Street. The site setting is shown in Figure 1.

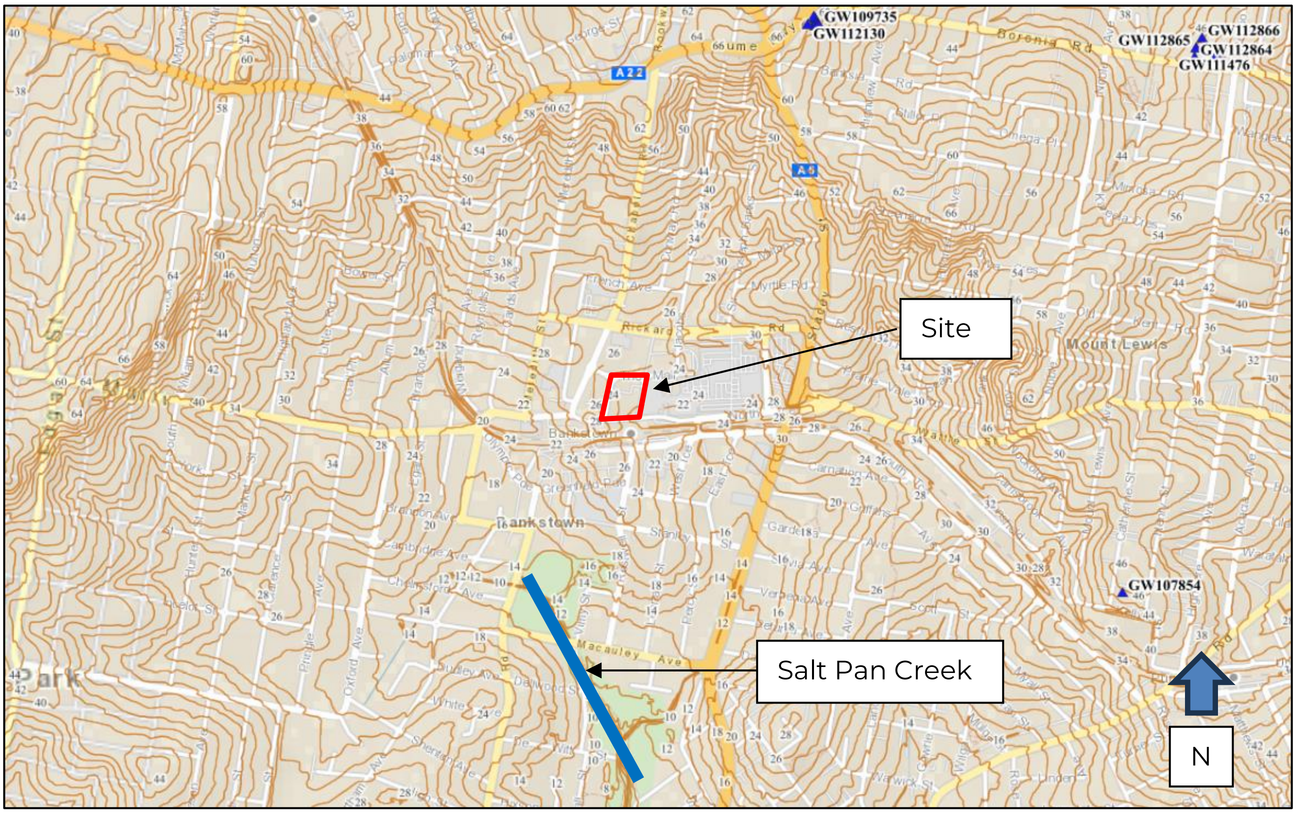


Figure 1: Site Location with 2 m Surface Contours and Registered Groundwater Bores

3. Background information

3.1 Geology

Reference to the Sydney 1:100,000 Geological Series Sheet indicates that the site is underlain by Ashfield Shale of the Wianamatta Group. The Ashfield Shale formation typically comprise dark grey shale and laminite (finely interbedded sandstones and siltstones).

3.2 Hydrogeology

The primary permeability of the Ashfield Shale is known to be low. Therefore, groundwater within the project area is understood to mostly occur along defects where secondary permeability of the rock has been enhanced.

The aquifer within the Ashfield Shale in the project area appears to be unconfined and is mostly recharged by rainfall infiltration.

3.3 Hydrology

The site is located about 600 m north of Salt Pan Creek. No other significant hydrological features are present in the vicinity of the project.

3.4 Groundwater receptors

A search of the NSW Office of Water groundwater database revealed that no registered water wells are located within 500 m of the site.

A review of the groundwater dependent ecosystems (GDE) Atlas from Australian Bureau of Meteorology showed that there are no mapped GDEs within 1 km of the site.

4. Previous investigations

4.1 Geotechnical investigations

Detailed information and results of the geotechnical investigations on the site are provided in the geotechnical investigation report prepared by Douglas (ref: 213805.02.R.001.Rev0 dated November 2023). The most recent investigation included cone penetration tests (CPTs) at six locations to depths of between 3.92 m and 4.70 m, drilling of six cored boreholes (BH1 to BH6) to a maximum depth of about 20.34 m. Groundwater monitoring wells were installed in BH1 to BH4.

Table 1 summarises the levels at which different materials were encountered in the boreholes and CPTs.

Table 1: Summary of Material Strata Levels

Material	Depth to Top of Unit (m BGL) [RL at Top of Unit (m AHD)]							
	1	2	3 / 3A	4	5	6	7	8
FILL / PAVEMENT	0.0 [22.3]	0.0 [22.3]	0.0 [22.4]	0.0 [22.5]	0.0 [22.5]	0.0 [22.7]	0.0 [22.0]	0.0 [22.0]
RESIDUAL CLAY	0.6 [21.7]	0.8 [21.5]	0.6 [21.8]	1.0 [21.5]	0.7 [21.8]	1.3 [21.9]	1.0 [21.6]	0.5 [21.5]
WEATHERED SILTSTONE	1.0 [21.3]	1.0 [21.3]	3.8 [18.6]	4.3 [18.2]	3.5 [19.0]	3.0 [19.7]	3.0 [19.0]	3.3 [18.7]
SILTSTONE: VL-L	1.2 [21.1]	1.3 [21.0]	4.8 [17.6]	4.6 [17.9]	3.9 [18.6]	3.5 [19.2]	NE	NE
SILTSTONE: L-M	3.3 [19.0]	5.2 [17.1]	6.0 [16.4]	5.2 [17.3]	5.4 [17.1]	4.4 [18.3]	NE	NE
INTERBEDDED SILTSTONE AND SANDSTONE: H-VH	NE	NE	12.2 [10.2]	12.2 [10.3]	12.6 [9.9]	12.7 [10.0]	NE	NE
Termination Depth	11.61 [10.7]	12.10 [10.2]	15.00 [7.4]	15.18 [11.1]	15.00 [11.1]	20.35 [2.4]	4.12 [17.9]	3.96 [18.0]

Note: NE = not encountered VL = very low strength, L = low strength, M = medium strength, H = high strength, VH = very high strength

4.2 Groundwater investigation

4.2.1 Groundwater levels

After installation, the four groundwater monitoring wells (BH1 to BH4) were purged of drilling fluid using a submersible pump and digital data loggers were installed.

A summary of the measured groundwater levels within the site is provided in Table 2. The locations of the groundwater monitoring wells are shown on Drawing 1 (Appendix B). It should be noted that groundwater levels vary over time due to seasonal, climatic and other factors.

Table 2: Summary of Groundwater Level Monitoring

Well	Datalogger Installed	Manual Groundwater Level Measurement (m BGL) [m AHD]				Highest Groundwater Level Recorded During Monitoring Period (m AHD)
		9/10/2023	11/10/2023	7/12/2023	12/1/2024	
BH1	29/9/2023	4.65 [17.65]	-	4.53 [17.77]	4.49 [17.81]	17.91 (29/11/2023)
BH2	29/9/2023	4.65 [17.65]	-	4.50 [17.80]	4.46 [17.84]	17.92 (29/11/2023)
BH3	11/10/2023	-	4.74 [17.66]	4.55 [17.85]	4.60 [17.80]	17.97 (29/11/2023)
BH4	11/10/2023	-	6.27 [16.23]	6.23 [16.27]	6.28 [16.22]	16.40 (8/1/2024)

4.2.2 Permeability testing

Rising head permeability tests were carried out in the monitoring wells. This was carried out by purging the well then measuring the rise in water level at regular time intervals using a data logger.

Permeability tests were analysed with Hvorslev (1951) solution for slug testing interpretation. Results are presented in Table 3. The detailed results of the permeability tests are attached in Appendix D.

Table 3: Interpreted permeability results

Well ID	Screen Interval (m)	Screened lithology	Interpreted Permeability (m/s)
BH1	1.00 – 11.61	Siltstone	2.0×10^{-6}
BH2	1.20 – 12.10	Siltstone	2.4×10^{-8}
BH3	0.90 – 10.51	Clay / Siltstone	4.5×10^{-7}
BH4	2.60 – 15.18	Clay / Siltstone / Sandstone	6.1×10^{-7}
Geometric Mean			**3.4×10^{-7}**

5. Conceptual hydrogeological model

The site is underlain by Ashfield Shale below fill and residual clay. The permanent groundwater table is anticipated to occur within the underlying Ashfield Shale along defects where secondary permeability has been enhanced (e.g. in fractures and cracks).

Free groundwater was not encountered during augering of the boreholes. Depths to groundwater were measured in the monitoring wells following installation and well development. Water levels measured in the monitoring wells ranged between RL 16 m and RL 18 m and were within the bedrock. The groundwater is expected to flow towards the south to Salt Pan Creek.

It should be noted that groundwater levels fluctuate with climatic variations and other factors (e.g. adjacent basements). Therefore, the water levels will temporarily rise during periods of heavy or prolonged rainfall and fall during dry periods.

6. Groundwater modelling

6.1 Methodology

Groundwater modelling was undertaken to assess the potential inflow rates into the proposed basement during and after construction, as well as induced groundwater level changes in adjacent and surrounding areas.

A 2-dimensional (2D) numerical groundwater model was developed. The modelling was carried out using the 2D finite element hydrogeological software SEEP/W (a component of GeoStudio) developed by GEOSLOPE International Ltd. The transient conditions during construction dewatering and the long-term 'steady-state' flow to the 'drained' excavation were modelled in the analysis.

6.2 Model Geometry

Douglas selected a cross-section in the north to south direction, intercepting the basement footprint to run the 2D analysis. The cross-section is presented in Drawing 1, Appendix B.

The aquifer surrounding the proposed development was simulated as a multi-layered numerical model to represent the subsurface conditions beneath the site and to allow simulation of the vertical flow components. The layers comprised from top to bottom: soil (fill and residual), weathered siltstone and fresh siltstone bedrock.

The model boundaries were extended 300 m from the southern and northern edges of the proposed excavation.

The SEEP/W results are based on a 2D model, which assumes the excavation is much longer than its width and provide seepage rates per unit metre. For the analysis, the excavation was assumed to have an approximately rectangular configuration with an effective width of about 92 m and a length of about 110 m.

It is understood that the proposed Basement 2 level is at about RL14.9 m. The model assumed dewatering to about RL14 m below the Basement two level. It is anticipated that the excavation will be supported by soldier pile walls.

6.3 Model Set-up

6.3.1 Model Layers

The hydrogeological units were simulated by assigning/combining multiple materials with different hydraulic properties (permeability and storage) to the model layers. The soil and rock layers were assigned as saturated/unsaturated materials in the hydrogeological model analysed using SEEP/W.

Model layers and adopted hydraulic properties are presented in Table 4. Parameters were selected based on the results of the permeability testing (Section 0), ranges of values documented in the available literature for similar lithologies (e.g. Fetter 1994 and Hoek & Bray 1981) and previous experience with similar materials. The anisotropy ratio of the permeability of the fresh siltstone was estimated based on the approximate amount of the horizontal and vertical defects observed in the boreholes.

Table 4: Model layers and adopted hydraulic properties

Layer	Material	Saturated Kh (m/s)	Anisotropy Ratio (Kv/Kh)	Porosity (%)	VWC model	Material Model
1	Soil and weathered siltstone	1×10^{-7}	1	40	Silty clay	Unsaturated / saturated
2	Fresh siltstone	3.5×10^{-7}	0.33	20	Sand	Unsaturated / saturated

6.3.2 Initial Groundwater Elevation and Boundary Conditions

As the proposed development is located within an urban setting with parks around the site, an annual rainfall of 1.2 m and net infiltration percentage of 3% was assigned as water influx over the ground surface. The model adopted a constant head of RL 17 m at the base of the model (RL - 50m). These boundary conditions were assigned and adjusted to reproduce the highest groundwater levels (RL 18 m) within the bedrock as recorded from the wells installed on the site.

6.3.3 Basement design and dewatering

The proposed soldier pile walls (with strip drains) and exposed rock faces were simulated as 'Seepage Faces' in SEEP/W with zero hydraulic pressure head along the full height. The subsoil drainage and the sump were simulated by a horizontal boundary condition of 'Zero hydraulic pressure head'.

The model set-up is presented in Figure 2 (for Basement 3 option).

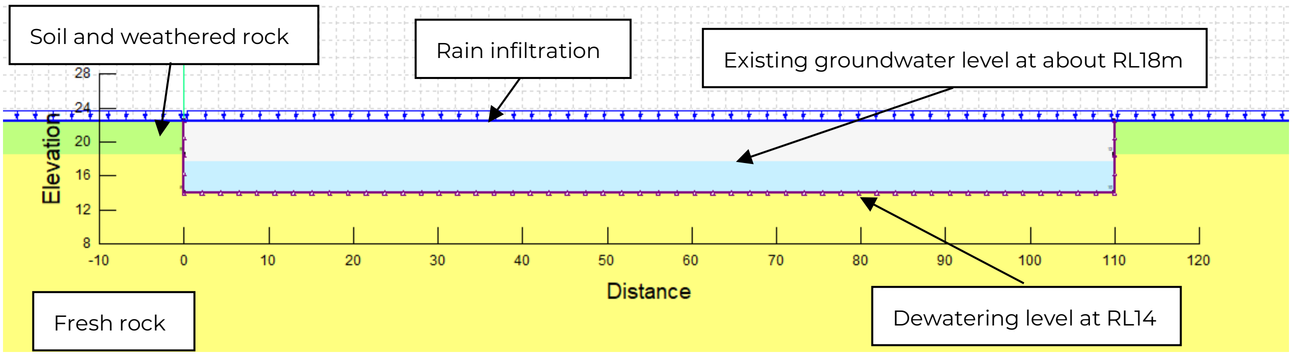


Figure 2: Model set-up

6.3.4 Simulation

To simulate the temporary dewatering required during construction, transient flow conditions were modelled in the analysis. For the purpose of the analysis, excavation was undertaken in a single stage (i.e. instantaneously) and the subsequent temporary dewatering period was assumed to be up to one year, subdivided into multiple time intervals.

The analysis was also run under 'steady state' conditions for the long term (i.e. assuming permanent drained basement, after construction).

6.4 Modelling Results

6.4.1 Groundwater Inflow

The inflow rates obtained from the modelling represent the estimated total rate of groundwater flowing into the excavation and the volume (per unit time) requiring extraction via the dewatering system, in order to dewater the basement during and after construction.

The SEEP/W results are based on a two-dimensional model, which assumes the excavation is much longer than its width. In order to convert these predictions to more realistic values, the following expression proposed by Kavvadas et al (1992) was used to adjust the analytical results, to allow for the actual length to width ratio of the proposed basement.

$$\frac{qo}{q} = 0.7 + 0.3\left(1 - \frac{B}{L}\right)$$

where: q_0 is the predicted 'realistic' inflow rate q is the inflow rate from 2D modelling factored for the perimeter length of the excavation

B is the width of the excavation (in-plane of the 2D model)

L is the length of the excavation (out-of-plane of the 2D model)

The basement has an approximately rectangular configuration with an effective width of about 110 m (B) and a length of about 92 m (L). A reduction factor of 0.64 was therefore calculated using the expression above. The average inflow rates predicted from the model have been reduced using this reduction factor to account for the basement shape.

The estimated inflows from the analysis for the first year (i.e. during construction) and long term are summarised in Table 5.

Table 5: Simulated inflow results for dewatering level at RL 14 m

Elapsed Time	Baseline case – predicted inflow	
	m^3/day	Cumulative (ML)
1 day	48.6	0.0
3 days	29.3	0.1
7 days	27.1	0.2
14 days	26.3	0.4
30 days	25.1	0.8
60 days	23.0	1.6
90 days	21.8	2.2
180 days	19.7	4.1
365 days	17.0	7.5
Long term	**m^3/day**	**ML/year**
	13.6	5.0

The results of the groundwater inflow analysis indicate:

- For dewatering level at RL 14 m, inflow of about 7.5 ML in the first year of construction; inflow will decrease in the long term, after construction, to about 5.0 ML/year;

The inflow rate presented above is a prediction only and may vary significantly to the assumptions presented. The actual flow rate will only be known once the excavation is completed, and the inflow can be observed and measured. Appropriate planning should be in place to monitor and compensate for greater variations in actual inflows.

6.4.2 Drawdown and Settlement

For the drained basement, the groundwater levels are anticipated to be lowered by a maximum of 4 m outside the excavation. However, groundwater drawdown occurs within the rock units and therefore is not expected to result in any significant settlement on surrounding sites due to drawdown.

7. Discussion

The geotechnical investigation on the site has identified fill and residual soils, overlying the Ashfield Shale bedrock.

Groundwater levels have been observed in the bedrock at depths of between 4.5 m and 6.3 m in the monitoring wells at the site.

The groundwater level will be required to be lowered by about 4 m for Basement 2 Level.

Based on the results of the numerical analysis, the long-term inflow rates are about 5.0 ML/year for the proposed 2-level basement.

From a geotechnical point of view, it is considered that drained basements are feasible without any significant impact to surrounding properties. In addition, the use of a drained basement is not considered likely to influence any GDEs or groundwater supply works.

Based on the inflow estimate of over 3 ML/year, the proposed drained basement requires a Water Access License and a Water Supply Works Approval from WaterNSW, assuming that the groundwater take is approved.

The groundwater inflow is sensitive to the modelled aquifer permeability. The adopted permeability of the fresh rock in the analysis (3.5×10^{-7} m/s) was based on the geometric mean of the results of permeability tests. The flow estimates would be about 10% and 10 times of the results in this analysis if the lower bound permeability of 2.4×10^{-8} m/s and the upper bound permeability of 2.0×10^{-6} m/s were adopted, respectively.

Given that groundwater is expected to flow through fractures and defects within the bedrock, it is likely that both the permeability and storage will vary significantly across the site. There is a reasonable degree of uncertainty associated with the extent and connectivity of the fractures at the site. Larger scale pumping tests with observation wells could be carried out over an extended period to establish if the water bearing zones are well connected to the aquifer and will continue to yield over time, or quite localised and likely to dry up relatively quickly.

From a construction perspective however, groundwater ingress occurring through discrete fractured zones is much simpler to deal with, compared with diffuse flow through a more porous material (i.e. upward leakage through a uniform sand sequence). Groundwater flow occurring through defects or fractured zones may be able to be controlled through the pressure grouting and shotcreting of perimeter excavations, dependent upon the extent, or aperture of the features.

8. Limitations

Douglas has prepared this report for this project at 83-99 North Terrace, Bankstown NSW in accordance with Douglas' proposal 213805.02.P.002.Rev0 dated 8 February 2024 and acceptance received from Craig Sanders. This report is provided for the exclusive use of Essence Project

Management Pty Ltd for this project only and for the purposes as described in the report. It should not be used by or relied upon for other projects or purposes on the same or other site or by a third party. Any party so relying upon this report beyond its exclusive use and purpose as stated above, and without the express written consent of Douglas, does so entirely at its own risk and without recourse to Douglas for any loss or damage. In preparing this report Douglas has necessarily relied upon information provided by the client and/or their agents.

The results provided in the report are indicative of the sub-surface conditions on the site only at the specific sampling and/or testing locations, and then only to the depths investigated and at the time the work was carried out. Sub-surface conditions can change abruptly due to variable geological processes and also as a result of human influences. Such changes may occur after Douglas's field testing has been completed.

Douglas's advice is based upon the conditions encountered during this investigation. The accuracy of the advice provided by Douglas in this report may be affected by undetected variations in ground conditions across the site between and beyond the sampling and/or testing locations. The advice may also be limited by budget constraints imposed by others or by site accessibility.

The assessment of atypical safety hazards arising from this advice is restricted to the geotechnical and groundwater components set out in this report and based on known project conditions and stated design advice and assumptions. While some recommendations for safe controls may be provided, detailed 'safety in design' assessment is outside the current scope of this report and requires additional project data and assessment.

This report must be read in conjunction with all of the attached and should be kept in its entirety without separation of individual pages or sections. Douglas cannot be held responsible for interpretations or conclusions made by others unless they are supported by an expressed statement, interpretation, outcome or conclusion stated in this report.

This report, or sections from this report, should not be used as part of a specification for a project, without review and agreement by Douglas. This is because this report has been written as advice and opinion rather than instructions for construction.

The scope of work for this investigation/report did not include the assessment of surface or sub-surface materials or groundwater for contaminants, within or adjacent to the site. Should evidence of fill of unknown origin be noted in the report, and in particular the presence of building demolition materials, it should be recognised that there may be some risk that such fill may contain contaminants and hazardous building materials.

Appendix A

About this Report

About this Report

Introduction

These notes have been provided to amplify DP's report in regard to classification methods, field procedures and the comments section. Not all are necessarily relevant to all reports.

DP's reports are based on information gained from limited subsurface excavations and sampling, supplemented by knowledge of local geology and experience. For this reason, they must be regarded as interpretive rather than factual documents, limited to some extent by the scope of information on which they rely.

Copyright

This report is the property of Douglas Partners Pty Ltd. The report may only be used for the purpose for which it was commissioned and in accordance with the Conditions of Engagement for the commission supplied at the time of proposal. Unauthorised use of this report in any form whatsoever is prohibited.

Borehole and Test Pit Logs

The borehole and test pit logs presented in this report are an engineering and/or geological interpretation of the subsurface conditions, and their reliability will depend to some extent on frequency of sampling and the method of drilling or excavation. Ideally, continuous undisturbed sampling or core drilling will provide the most reliable assessment, but this is not always practicable or possible to justify on economic grounds. In any case the boreholes and test pits represent only a very small sample of the total subsurface profile.

Interpretation of the information and its application to design and construction should therefore take into account the spacing of boreholes or pits, the frequency of sampling, and the possibility of other than 'straight line' variations between the test locations.

Groundwater

Where groundwater levels are measured in boreholes there are several potential problems, namely:

- In low permeability soils groundwater may enter the hole very slowly or perhaps not at all during the time the hole is left open;
- A localised, perched water table may lead to an erroneous indication of the true water table;
- Water table levels will vary from time to time with seasons or recent weather changes. They may not be the same at the time of construction as are indicated in the report; and
- The use of water or mud as a drilling fluid will mask any groundwater inflow. Water has to be blown out of the hole and drilling mud must first be washed out of the hole if water measurements are to be made.

More reliable measurements can be made by installing standpipes which are read at intervals over several days, or perhaps weeks for low permeability soils. Piezometers, sealed in a particular stratum, may be advisable in low permeability soils or where there may be interference from a perched water table.

Reports

The report has been prepared by qualified personnel, is based on the information obtained from field and laboratory testing, and has been undertaken to current engineering standards of interpretation and analysis. Where the report has been prepared for a specific design proposal, the information and interpretation may not be relevant if the design proposal is changed. If this happens, DP will be pleased to review the report and the sufficiency of the investigation work.

Every care is taken with the report as it relates to interpretation of subsurface conditions, discussion of geotechnical and environmental aspects, and recommendations or suggestions for design and construction. However, DP cannot always anticipate or assume responsibility for:

- Unexpected variations in ground conditions. The potential for this will depend partly on borehole or pit spacing and sampling frequency;
- Changes in policy or interpretations of policy by statutory authorities; or
- The actions of contractors responding to commercial pressures.

If these occur, DP will be pleased to assist with investigations or advice to resolve the matter.

continued next page

About this Report

Site Anomalies

In the event that conditions encountered on site during construction appear to vary from those which were expected from the information contained in the report, DP requests that it be immediately notified. Most problems are much more readily resolved when conditions are exposed rather than at some later stage, well after the event.

Information for Contractual Purposes

Where information obtained from this report is provided for tendering purposes, it is recommended that all information, including the written report and discussion, be made available. In circumstances where the discussion or comments section is not relevant to the contractual situation, it may be appropriate to prepare a specially edited document. DP would be pleased to assist in this regard and/or to make additional report copies available for contract purposes at a nominal charge.

Site Inspection

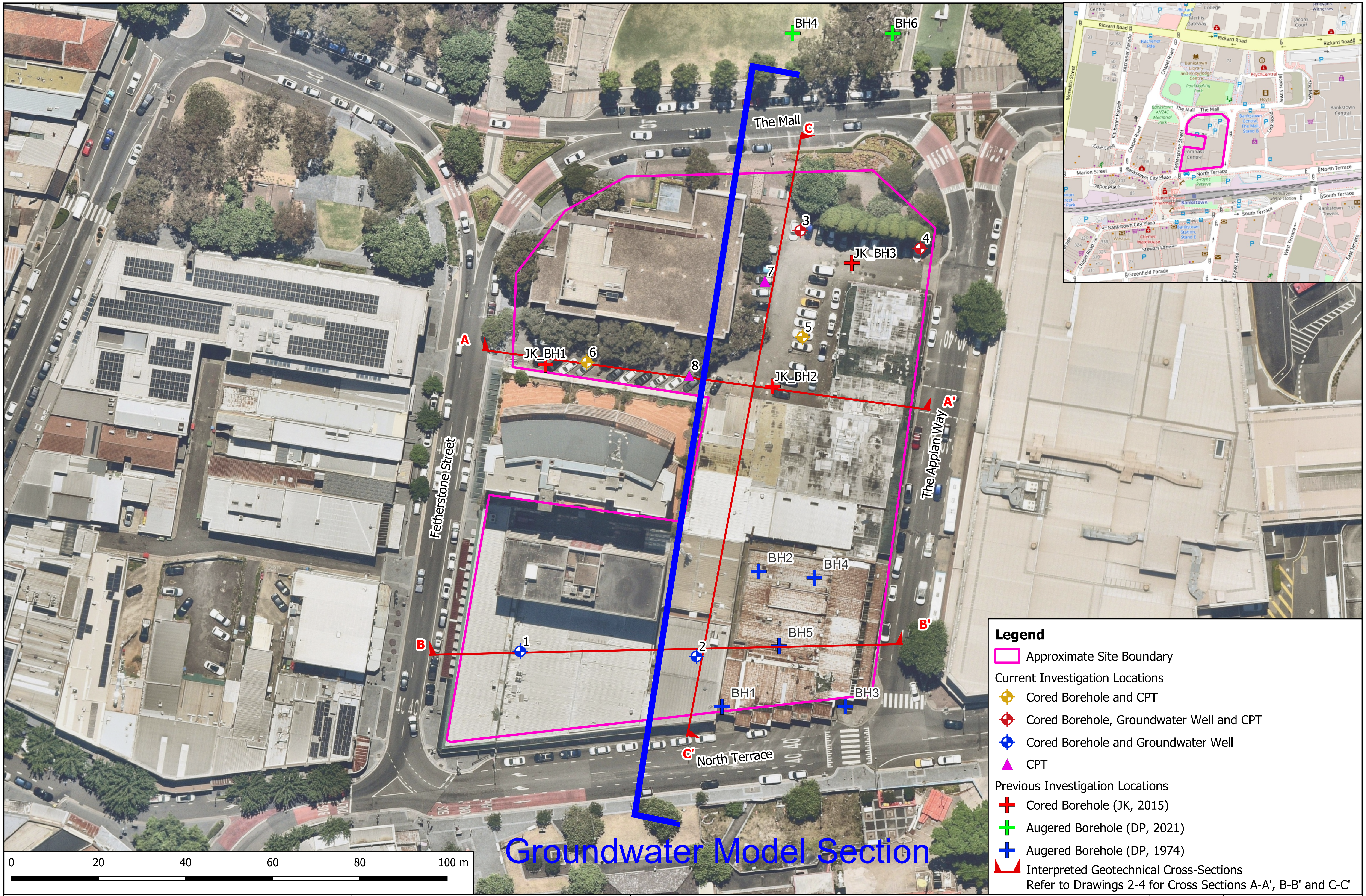
The company will always be pleased to provide engineering inspection services for geotechnical and environmental aspects of work to which this report is related. This could range from a site visit to confirm that conditions exposed are as expected, to full time engineering presence on site.

intentionally blank

intentionally blank

Appendix B

Drawing



Douglas Partners Geotechnics / Environment / Groundwater	CLIENT: Essence Project Management		TITLE: Test Location Plan Proposed Redevelopment of Altis Compass Centre 83-99 North Terrace, Bankstown	PROJECT No: 213805.02
	OFFICE: Sydney	DRAWN BY: JDB		DRAWING No: 1
	SCALE: 1:800 @A3	DATE: 19/10/2023		REVISION: 0

Appendix C

Permeability Testing Results

Douglas Partners Pty Ltd
ABN 75 053 980 117
www.douglaspartners.com.au

Permeability Testing - Rising Head Test Report

Client:	Essence Project Management Pty Ltd	Project No:	213805.02
Project:	Proposed Redevelopment of Compass Centre	Test date:	29-Sep-23
Location:	83-99 North Terrace, Bankstown	Tested by:	LL

Test Location

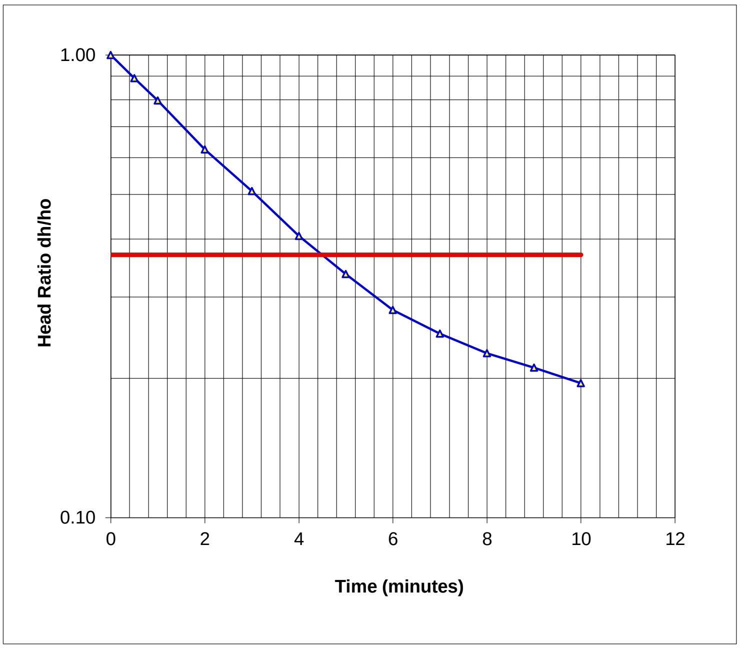
		Test No.	BH1	
Description:	Standpipe in borehole	Easting:	318338.9	m
Material type:	Siltstone	Northing	6245290.8	m
		Surface Level:	22.3	m AHD

Details of Well Installation

Effective diameter (2re)	76	mm	Depth to water before test	4.65	m
borehole diameter (2R)	76	mm	Depth to water at start of test	5.93	m
Effective Length of well screen (Le)	6.96	m	Depth of top of gravel pack	1.00	m
			Depth of base of gravel pack	11.61	m

Test Results

Time (min)	Depth (m)	Change in Head dH (m)	dH/Ho
0.0	5.93	1.28	1.000
0.5	5.79	1.14	0.891
1.0	5.67	1.02	0.797
2.0	5.45	0.80	0.625
3.0	5.30	0.65	0.508
4.0	5.17	0.52	0.406
5.0	5.08	0.43	0.336
6.0	5.01	0.36	0.281
7.0	4.97	0.32	0.250
8.0	4.94	0.29	0.227
9.0	4.92	0.27	0.211
10	4.90	0.25	0.195



To = 4.5 mins
270 secs

Theory: Falling Head Permeability calculated using equation by Hvorslev

$k = [r^2 \ln(Le/R)]/2Le\ To$

where r = radius of casing
R = radius of well screen
Le = length of well screen
To = time taken to rise or fall to 37% of initial change

Hydraulic Conductivity	**k =**	**2.0E-06**	m/sec
	=	**1.7E-01**	m/day

Douglas Partners
Geotechnics / Environment / Groundwater

Douglas Partners Pty Ltd
ABN 75 053 980 117
www.douglaspartners.com.au

Permeability Testing - Rising Head Test Report

Client:	Essence Project Management Pty Ltd	Project No:	213805.02
Project:	Proposed Redevelopment of Compass Centre	Test date:	29-Sep-23
Location:	83-99 North Terrace, Bankstown	Tested by:	LL

Test Location | **Test No.** BH2

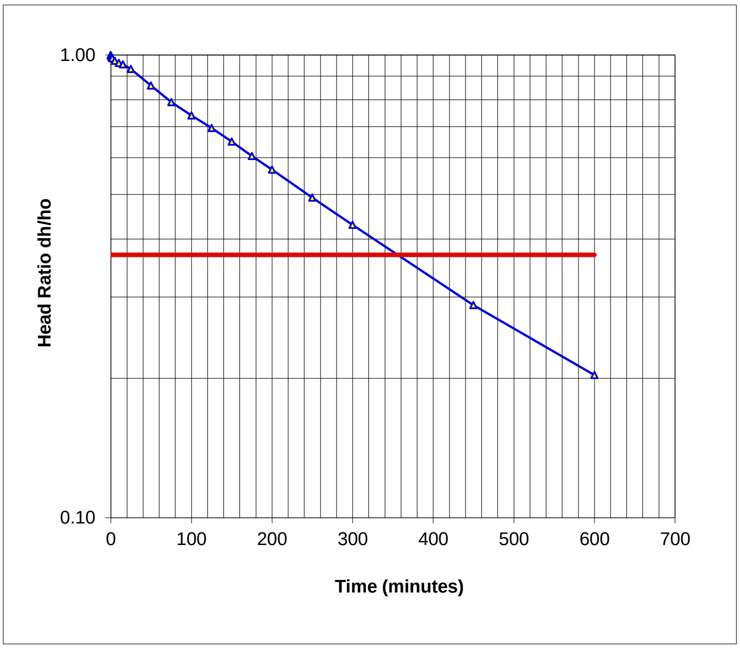
Description:	Standpipe in borehole	Easting:	318338.9	m
Material type:	Siltstone/Sandstone Interbedded	Northing	6245290.8	m
		Surface Level:	22.3	m AHD

Details of Well Installation

Effective diameter (2re)	76	mm	Depth to water before test	4.65	m
borehole diameter (2R)	76	mm	Depth to water at start of test	6.42	m
Effective Length of well screen (Le)	7.45	m	Depth of top of gravel pack	1.20	m
			Depth of base of gravel pack	12.10	m

Test Results

Time (min)	Depth (m)	Change in Head dH (m)	dH/Ho
0.00	6.42	1.77	1.000
0.50	6.41	1.76	0.994
1	6.40	1.75	0.989
2	6.39	1.74	0.983
5	6.37	1.72	0.972
10	6.35	1.70	0.960
15	6.34	1.69	0.955
25	6.30	1.65	0.932
50	6.17	1.52	0.859
75	6.05	1.40	0.791
100	5.96	1.31	0.740
125	5.88	1.23	0.695
150	5.80	1.15	0.650
175	5.72	1.07	0.605
200	5.65	1.00	0.565
250	5.52	0.87	0.492
300	5.41	0.76	0.429
450	5.16	0.51	0.288
600	5.01	0.36	0.203



To = 360 mins
21600 secs

Theory: Falling Head Permeability calculated using equation by Hvorslev

$k = [r^2 \ln(Le/R)]/2Le\ To$

where r = radius of casing
R = radius of well screen
Le = length of well screen
To = time taken to rise or fall to 37% of initial change

Hydraulic Conductivity	**k =**	**2.4E-08**	m/sec
	=	**2.0E-03**	m/day

Permeability Testing - Rising Head Test Report

Client:	Essence Project Management Pty Ltd	Project No:	213805.02
Project:	Proposed Redevelopment of Compass Centre	Test date:	29-Sep-23
Location:	83-99 North Terrace, Bankstown	Tested by:	LL

Test Location

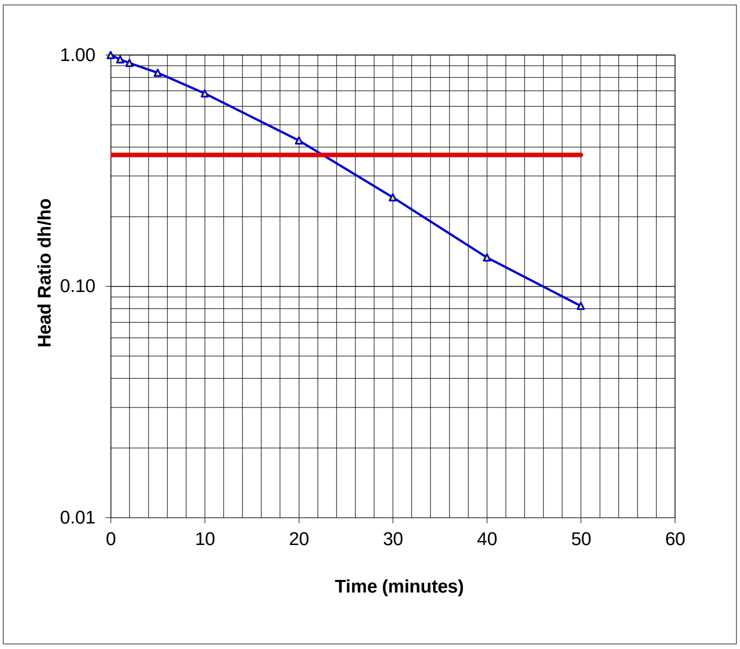
Description:	Standpipe in borehole	**Test No.**	BH3	
Material type:	Clay and Siltstone	Easting:	318362.8	m
		Northing	6245388.6	m
		Surface Level:	22.4	m AHD

Details of Well Installation

Effective diameter (2re)	76	mm	Depth to water before test	4.73	m
borehole diameter (2R)	76	mm	Depth to water at start of test	9.23	m
Effective Length of well screen (Le)	5.78	m	Depth of top of gravel pack	0.90	m
			Depth of base of gravel pack	10.51	m

Test Results

Time (min)	Depth (m)	Change in Head dH (m)	dH/Ho
0.0	9.23	4.50	1.000
1	9.03	4.30	0.956
2	8.89	4.16	0.924
5	8.50	3.77	0.838
10	7.80	3.07	0.682
20	6.65	1.92	0.427
30	5.82	1.09	0.242
40	5.33	0.60	0.133
50	5.10	0.37	0.082



To = 23.0 mins
1380 secs

Theory: Falling Head Permeability calculated using equation by Hvorslev

$k = [r^2 \ln(Le/R)]/2Le\ To$

where r = radius of casing
R = radius of well screen
Le = length of well screen
To = time taken to rise or fall to 37% of initial change

Hydraulic Conductivity	**k =**	**4.5E-07**	m/sec
	=	**3.9E-02**	m/day

Douglas Partners
Geotechnics / Environment / Groundwater

Douglas Partners Pty Ltd
ABN 75 053 980 117
www.douglaspartners.com.au

Permeability Testing - Rising Head Test Report

Client:	Essence Project Management Pty Ltd	Project No:	213805.02
Project:	Proposed Redevelopment of Compass Centre	Test date:	29-Sep-23
Location:	83-99 North Terrace, Bankstown	Tested by:	LL

Test Location

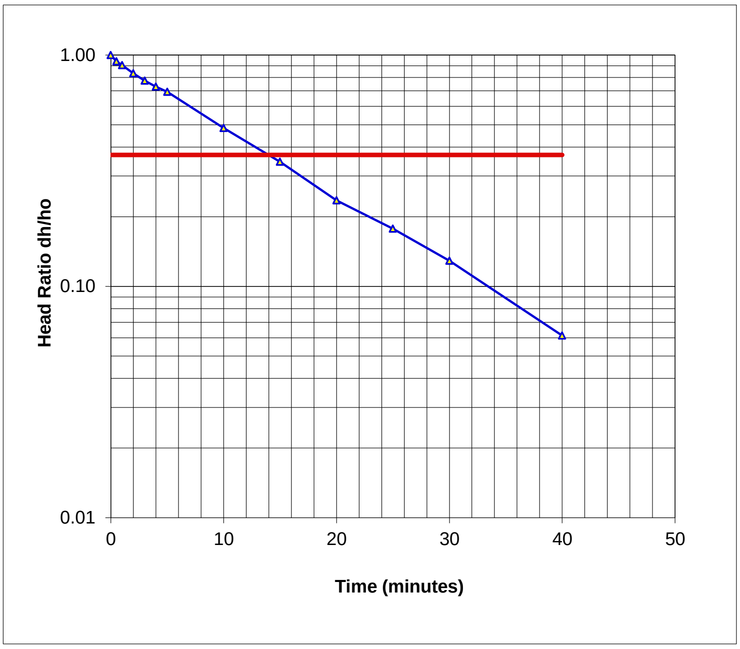
Description:	Standpipe in borehole	**Test No.**	BH4	
Material type:	Clay, Siltstone and Sandstone	Easting:	318390.3	m
		Northing	6245384.6	m
		Surface Level:	22.5	m AHD

Details of Well Installation

Effective diameter (2re)	76	mm	Depth to water before test	5.80	m
borehole diameter (2R)	76	mm	Depth to water at start of test	8.90	m
Effective Length of well screen (Le)	9.38	m	Depth of top of gravel pack	2.60	m
			Depth of base of gravel pack	15.18	m

Test Results

Time (min)	Depth (m)	Change in Head dH (m)	dH/Ho
0.0	8.9	3.10	1.000
0.5	8.71	2.91	0.939
1	8.60	2.80	0.903
2	8.38	2.58	0.832
3	8.20	2.40	0.774
4	8.06	2.26	0.729
5	7.95	2.15	0.694
10	7.30	1.50	0.484
15	6.87	1.07	0.345
20	6.53	0.73	0.235
25	6.35	0.55	0.177
30	6.20	0.40	0.129
40	5.99	0.19	0.061



To = 11.5 mins
690 secs

Theory: Falling Head Permeability calculated using equation by Hvorslev

$k = [r^2 \ln(Le/R)]/2Le\ To$

where r = radius of casing
R = radius of well screen
Le = length of well screen
To = time taken to rise or fall to 37% of initial change

Hydraulic Conductivity **k = 6.1E-07** m/sec
= 5.3E-02 m/day