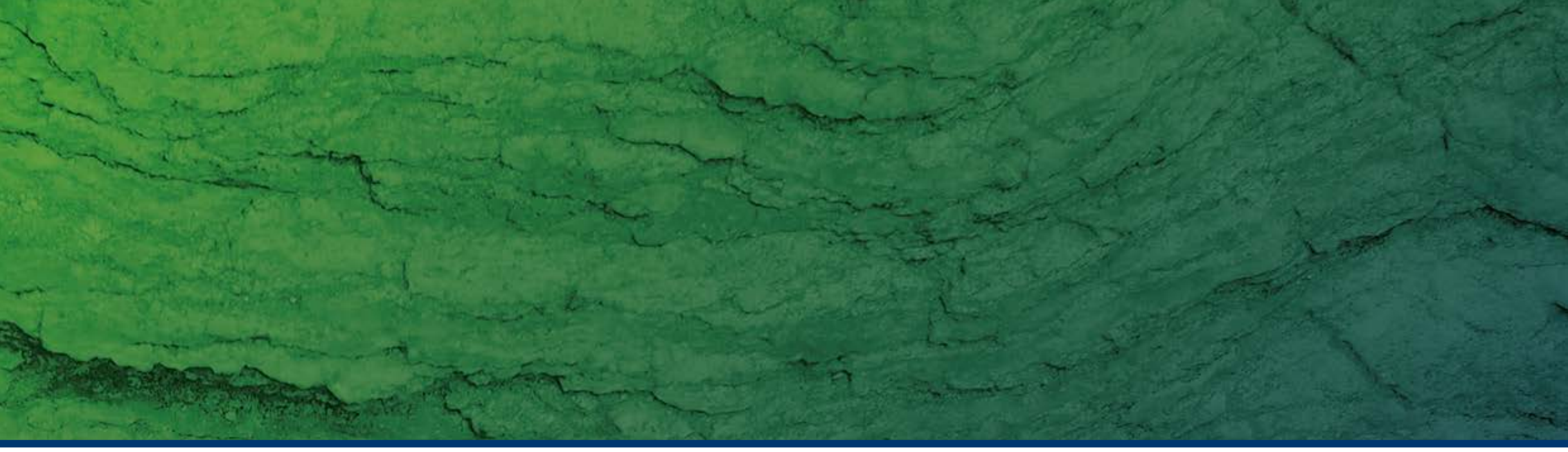
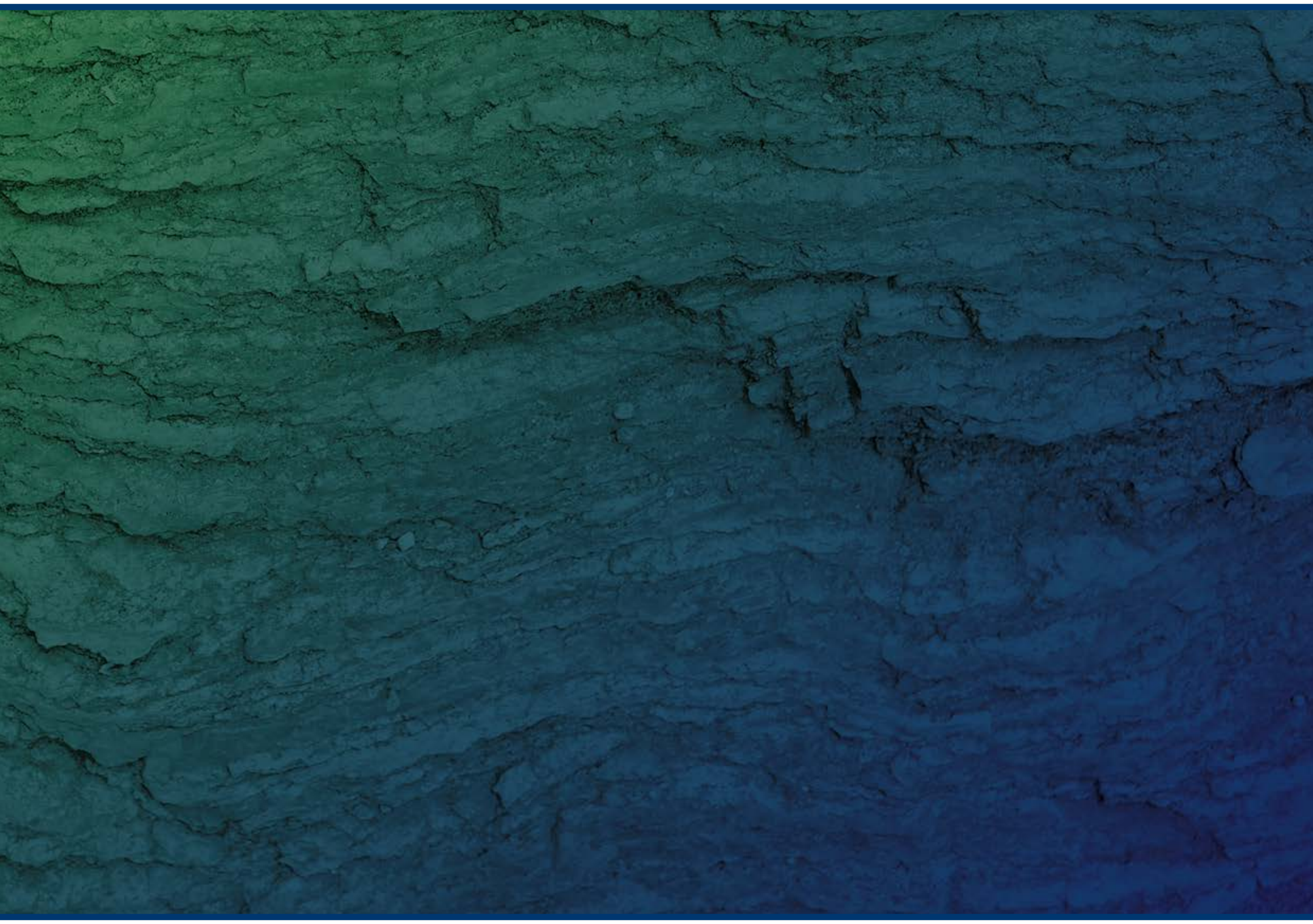


PROCEEDINGS

2019 AUSTRALIAN GEOMECHANICS SOCIETY VICTORIAN SYMPOSIUM

Geotechnical characterisation – managing design and construction risk

Wednesday, 30 October 2019, 8:00am – 7:00pm
Rydges Hotel, 186 Exhibition Street, Melbourne



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PREFACE

The Victorian chapter of the Australian Geomechanics Society invited academics and practitioners in the field of geotechnical and ground engineering to attend the 2019 Australian Geomechanics Society Victorian Symposium held on 30 October 2019.

In recent years Victoria has seen significant growth in the construction industry. Investment in both public infrastructure and commercial real estate is growing, and as our cities and infrastructure grow, so too does the need to develop parcels of land with challenging ground conditions. Economical and safe geotechnical design requires efficient and well thought through ground investigation and characterisation to identify and manage ground risks and opportunities.

The 2019 Australian Geomechanics Society Victorian Symposium presents an overview of current state of-the-art practices, innovation, new research results and case studies relating to geotechnical characterisation with an emphasis on its implications for addressing and managing design and construction risk. The 2019 Symposium brought together professional engineers, researchers, specialist contractors, regulators, educators and students to share and discuss their experiences on the topic of ground characterisation.

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Site characterisation and permeation curtain grouting for deep basement excavation

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ABSTRACT

This paper describes the geological site characterisation for the design and construction of a 26m deep basement excavation in Fitzroy, Melbourne. Key geotechnical features include an extensive, 10-40m thick felsic dyke within Melbourne Formation rock which generated geotechnical implications for excavation stability and high groundwater inflows. As a mitigation effort a hybrid retention system consisting of a solider pile wall and an infill grout curtain was subsequently developed.

Four rounds of geotechnical and hydrogeological investigations were completed to delineate the extent of the dyke and identify groundwater flow characteristics. Site trials were completed to provide permeation grouting parameters within the dyke. Pumping tests are proposed to validate the effectiveness of the grout curtain in mitigating groundwater inflow to the excavation.

A Discrete Fracture Network (DFN) of rock defects was parametrically scripted in three dimensions to assist with the design of grout volumes and wedge failure modelling. The DFN was developed from seed boreholes using investigation results from acoustic and optical televiewer borehole imagery.

The grout curtain design consisted of a primary and secondary grouting programme based on the DFN model and results of the grout trial.

Keywords: Deep Basement, Permeation grouting, Discrete Fracture Network.

1 INTRODUCTION

This paper describes the geological site characterisation for the design and construction of 7-storey basement excavation in Fitzroy, Melbourne extending down to 26m below the surrounding street levels.

Key geotechnical features identified during site investigations include an extensive, 10-40m thick felsic dyke within Melbourne Formation rock which generated geotechnical implications to excavation stability and high groundwater inflows.

Due to the extensive site constraints including access, groundwater discharge both during construction and permanent conditions, a hybrid temporary retention system of a solider pile wall with an infill grout curtain was developed following a number of investigations and site grouting trials.

2 SITE AND GEOLOGICAL SETTING

The proposed excavation has plan dimensions approximately 49m long by 45m wide and is bound by an existing heritage listed six-storey brick masonry building along its northern and western elevations. The southern elevation is bound by a major road and tram easement, while the eastern elevation is bound by a two-lane council road (refer Figure 1). Several buried services are located beneath the pavements adjacent, or immediately on, the eastern and southern boundaries. The heritage building was constructed in the early 1920's and supported with large (approximately 3m square) pad footings, founded on extremely weathered siltstone.

The site is located within an area underlain by Silurian age siltstone and sandstone referred to as the Melbourne Formation (MF), and typically overlain by a residual clay with a surficial layer of fill. The MF is indicated to be tightly bedded and folded by east-west tectonic compression, with fold axis typically oriented in a NNE-SSW direction.

Igneous intrusions are commonly encountered within the MF, and generally are more weathered than the host rock having a consistency of a firm to stiff clay. However, the igneous rock may also have higher strength than the host rock in thicker intrusions. The presence of significant intrusions can result in hydrothermally altered siltstone and sandstone with altered characteristics and properties than that typically observed with the MF.

2.1 Summary of ground conditions

The typical soil units for the site consist of medium dense to very dense gravelly and sandy fill underlain by stiff to hard silty clay residual soil. Thickness of the encountered fill and residual soil ranges from 0.3m to 1.8m and 0.1m to 1.9m respectively.

The encountered rock units include variably weathered siltstone, sandstone and felsic dyke. The medium grained sandstone represents a minor portion of the lithology encountered. The siltstone unit dips down towards the west.

The dyke was predominantly encountered over the western portion of the site and is interpreted to be dipping gently to moderately towards the west. The vertical thickness of the dyke ranges from 3.5m at the eastern portion of the site to 27m at the western portion of the site (refer Figure 2a). The felsic dyke rock at depth was encountered generally as a higher strength material than the surrounding host rock. Weathered siltstone and sandstone adjacent to the dyke have been hydrothermally altered ie. exhibiting higher strengths.

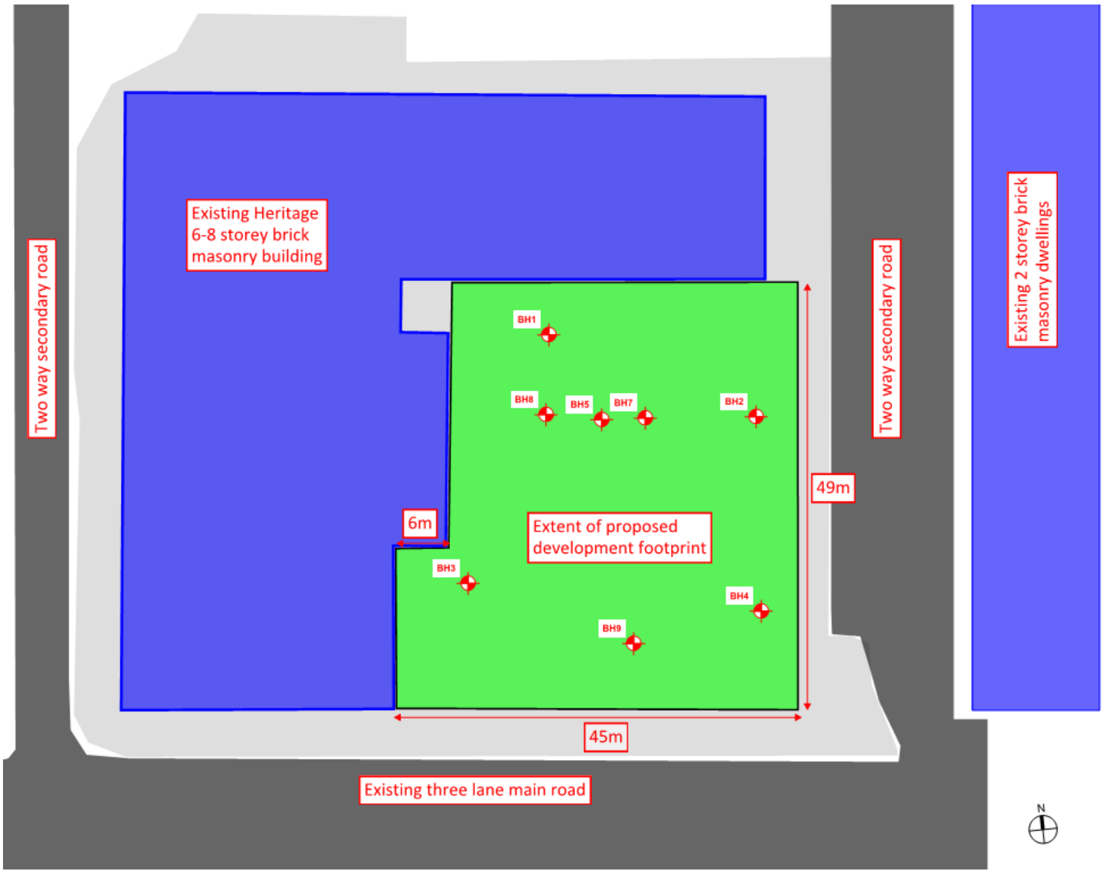

Figure 1 - Site layout and test location plan

Groundwater levels measured ranged between 5 m below ground level (bgl) along the western boundary and 8.5 m bgl along the eastern boundary providing considerable variation of groundwater level across the site. This discrepancy in groundwater levels, has been attributed to potential dewatering beyond the site boundaries.

3 GROUND INVESTIGATION SCOPE

Prior to the initial investigation, consideration was given to reducing design uncertainty on defect dip and dip directions of the MF to mitigate instability within the rock mass and optimise the design, as per previous local examples in Melbourne. Therefore, the investigation was tailored to improve the understanding of how these risks may impact the proposed development. Nine vertical boreholes were completed varying in depth between 1.0 m up to 35 m. The investigation included in-situ pressuremeter testing and borehole imaging.

The in-situ pressuremeter testing confirmed the variation of the material stiffness and inferred the strength of weathered rock at shallow depths well above bulk excavation level, with typical modulus values ranging from 150 MPa to 600 MPa. Pressuremeter test results of material at and below bulk excavation indicating modulus values varying between 1600 MPa to 3900 MPa.

Downhole imaging included an acoustic televiewer (ATV) was conducted in four of the boreholes to identify discontinuities within the rock mass. The imaging data indicated two likely joint sets plus one bedding orientation. The adopted defect orientations are summarised in Table 1.

Table 1 - Summary of defects in the Melbourne Formation

Structures	Dip Angle (°)	Dip Direction (°)
Siltstone Bedding	30 to 45	244 to 278
Dyke Joint Set 1	26 to 32	042 to 065
Dyke Joint Set 2	65	256
Dyke Joint Random	31 & random	088 and random

Permeability testing initially consisted of falling head, rising head and air-lift testing across six of the nine investigation locations on site (refer Table 2). Typical permeability values within the less fractured felsic dyke are in the order of 10^{-8} to 10^{-9} m/s. However, for the fractured dyke, permeability values up to 10^{-4} m/s have been recorded. The permeability of the siltstone unit is in the order of 10^{-8} m/s.

Preliminary seepage analyses, assuming the rock mass permeabilities and an axisymmetric simplified ground model indicated that due to the large variation of permeabilities encountered, inflow rates during excavation could vary from 0.2 l/s to 125 l/s.

The results of the air-lift testing indicate a significant zone of high permeability occurring within the dyke material in the vicinity of BH1. Peak initial pumping rates of 12 litres per minute were recorded, decreasing to approximately 10.5 litres per minute for the remainder of the test. The relative consistent pumping rate over the eight-hour testing period suggested that the of groundwater storage and/or recharge present within this zone is significant. Water levels measured periodically at

Table 2 - Results of groundwater testing prior to grout trials

Bore Hole ID	Recorded GWL (m AHD)	Screen Elevation From (m AHD)	Screen Elevation To (m AHD)	Permeability (m/s)	Screened Geology[1]	Testing Type[2]
BH1	35.5	23.75	25.95	1.0 x $10^{-□}$	Felsic Dyke (Fractured)	FHST
				8 x 10^{-5}		FHST
				7.5 x 10^{-5}		RHST
BH2	32.8	27.77	30.97	9.5 x 10^{-9}	Siltstone (MW*)	FHST
BH3	35.8	15.77	17.97	4.8 x $10^{-□}$	Felsic Dyke (Less Fractured)	RHST
				2.1 x 10^{-9}		FHST
BH4	32.5	19.26	22.46	1.3 x 10^{-8}	Siltstone (SW**)	FHST
BH7	34.9	19.3	26	-	Siltstone	-
BH8	35.8	22.3	27.02	1.2 x $10^{-□}$	Felsic Dyke	FHST + RHST
BH9	35.7	19.64	27.94	2.7 x $10^{-□}$	Felsic Dyke	FHST + RHST

1. *MW = Moderately weathered; ** SW = Slightly weathered.
2. FHST – Falling Head Slug Test, RHST – Rising Test Slug Test

BH2 to BH4 did not show significant changes over the testing period suggesting the high permeability zone may be localised within the dyke material and not continuous across the encountered siltstone.

The permeability testing indicated that the northwest corner of the site would be a problematic for controlling the inflow rates due to high permeability values (1.0 x 10^{-}□m/s) at BH1 and the extensive thickness of the felsic dyke (>20 m) at the western site boundary.

4 RETENTION OPTIONS

A number of retentions schemes were initially discussed including:

- diaphragm walls with either a rock cutter or grab which would require pre-drilling with a bored piled rig;
- contiguous or secant piles with grout infill between piles;
- a hybrid system comprising of both of the aforementioned; and
- traditional soldier pile and shotcrete infill panels.

Both diaphragm wall schemes were seen as being cost and program prohibitive and therefore not considered as the best outcome for the project. A retention system constructed using bored piles appeared to be the most feasible and cost-effective. The main concern would be achieving verticality, and therefore maintain water-tightness, over the full height of the excavation and over the depth to achieve cut-off below the felsic dyke. To overcome this a bored pile solution supplemented with permeation grouting was also included for consideration.

5 GROUT TRIAL

Due to the permeability uncertainties identified during the investigation period, a grout trial was proposed to address the effectiveness of grouting the rock mass and initial estimation of grout take, serving as a basis to determining the viability of this retention approach.

The grout trial was divided into two separate test areas over the northern portion of the site, with the intent of creating a grout curtain surrounding each of the existing boreholes BH1 and BH2. The northwest grouting test site (BH1) will be the critical site to determine if grouting is effective for reducing the permeability of the highly permeable zone to the target permeability of $10^{-}8$ m/s.

The grouting test at the northeast site (BH2) aimed to verify whether grouting to 2 m below the base of the felsic dyke is sufficient to achieve the target permeability of $10^{-}8$ m/s or if grouting full depth to 3 metres below the final excavation level will be required. The grouting test at the northeast site will also provide an estimation of the grout takes for the less permeable areas of the site.

5.1 Grout trial test phases

The following outlines the grouting process and validation testing completed during the grout trial.

- Step 1**:** Commence drilling of primary holes at 6m spacing to form a square. On completion, carry out down hole imaging to enable assessment of encountered rock quality. All grout holes must extend 2m below the base of the dyke. Anticipated grouting depths for the northwest and northeast grouting test area were 37mbgl and 15mbgl. Sequential complete packer testing from the toe of the hole to the surface in 6m maximum length intervals.
- Step 2: Commence pressure grouting of primary grout holes.
- Step 3: Commence drilling at secondary locations at 3m infill spacing between primary holes. Carry out down hole imaging to enable assessment of encountered rock quality. Carry out in-situ packer validation testing over maximum 3m length intervals to verify the effectiveness of the primary grouting works.
- Step 4: Commence secondary pressure grouting, if required.
- Step 5: Proceed with further grouting and repeat as required until a permeability of < 10^{-8}m/s is achieved in packer testing. Additional infill grouting stages may be required with each stage at half the previous stage spacing.

5.2 Grout Mix

A 1:1 water to cement ratio by weight using standard general purpose (GP) cement was adopted for the trial. Superplasticiser additive was also adopted to decrease the apparent viscosity of grout without increasing the bleed. The intent of maintaining the grout mix consistent during the trial was to determine whether a stable mix that wouldn't sediment in injected defects while still being of sufficiently low viscosity to be able to penetrate narrow defects that require grouting. The use of finer grained cement, such as micro fines, was not considered during the trial. Should the trial be unsuccessful using GP cement, then the feasibility of the scheme as a whole would quickly become cost prohibitive due to the increased cost of finer cements compared to GP cement.

5.3 Grout trial results

Over one month, eight grout locations (four primaries and four secondaries, had been drilled and grout injected around both BH1 and BH2. Grouting extending to 37 m bgl and 15 m bgl respectively, with grout being injected over relative equal depth increments from the base of hole.

For the trail, a maximum grout volume was estimated using an empirical porosity value by the grouting contractor and agreed to be used as the primary 'stop criteria'. However, the majority of the 'stop criteria' achieved at both trial locations was "Minimum flow" ie the minimum flow encountered under constant pressure, indicating that the closure of rock mass defects had been achieved prior to the target volume being reached.

The grout placement in the primary holes at trial location of BH1 resulted in an average of 3000 L per hole, while the grout placement in the secondary holes took an average of 1600 L per hole. The grout placement in the primary holes at trial location of BH2 took an average of 1300 L per hole, while the secondaries took an average of 940 L per hole. At both trial locations, it is observed that grout intake reduced significantly between primaries and secondaries, which indicates that the grout had permeated to the rock mass sufficiently such that it could not accept any further grout intake.

A number of down hole packer tests were carried out in the grout holes and the original borehole locations following the completion of grouting. At the trial location BH1, almost all water pressure tests for grout holes resulted in a permeability in the order of 1 x 10^{-7} m/s or less (with a single test carried out in a secondary grout hole resulted in a permeability of 1 x 10^{-6} m/s). Permeability testing carried out at BH1 post grouting returned permeabilities in the order of 1 x 10^{-7} m/s. The falling head test at BH1 resulted in a permeability of 1 x 10^{-8} m/s. This is significantly less than the investigation stage test results in Table 1.

At trial location BH2, the water pressure testing in the grout holes resulted in permeabilities ranging between 1.0 x 10^{-6} m/s to 1 x 10^{-9} m/s. The permeability tests carried out at BH2 post grouting returned permeabilities in the order of 1 x 10^{-7} m/s. The grout trial had been successful in establishing the effectiveness of grouting to reduce the rock mass permeabilities of the felsic dyke encountered on site to an acceptable level (with a potential to achieve the initial target permeability of 1 x 10^{-8} m/s). It also provided an indication of the grout take and penetration within the felsic dyke and siltstone.

6 DEVELOPING THE DISCRETE FRACTURE NETWORK

The Discrete Fracture Network (DFN) model for the site was generated from a combination of borehole logs and televiewer logs, using a workflow developed inhouse (Wang and Vecchiarelli, 2019). The DFN was created to assist with the estimation of grout volume for the grout curtain and the calculation of retention pressures from rock wedges. Defect characteristics such as orientation, aperture, persistence and termination are incorporated into the model. Figure 2 provides snapshots of the workflow. The DFN model is separated into two geological domains: (1) Dyke and (2) Siltstone.

Defects such as faults and seams are modelled deterministically while bedding partings and joints sets are modelled non-deterministically (i.e. based on simulations). Within the Siltstone domain, defect types include two joint sets plus bedding partings (refer Table 3). An additional two joints sets within the siltstone have been observed from the borehole logging and included in the site geological structural model. The defects within the dyke domain consist of two joint sets. Joints and bedding partings are randomly generated using stochastic processes and follow the principles of the conventional Poisson model (Dershowitz and Einstein, 1988).

Lineal fracture intensity (P10, number of fractures per unit length from boreholes) inform the number of fracture seeds within each voxel (block) with the two geology domains. P10 for bedding partings and joint sets are randomly assigned into each voxel adhering to their probabilistic distribution of P10 derived from the borehole and televiewer logs (Refer to Figure 2b).

Joint types are allocated based on their probability of occurrence identified from the borehole and televiewer logs. Defect orientation and persistence are simulated at each fracture seed location using a normal distribution derived from their respective statistical attributes (Refer to Table 3 and Figure 2c). The generated defects are calibrated and validated against the averaged lineal fracture intensity (P10) obtained from the input borehole and televiewer logs. The calibration process consists of cleaning fracture seeds (remove/ add) and manipulation of the fracture persistence's statistical attributes (mean, standard deviation and skewness).

7 RETENTION DESIGN

Following the success of the grout trial, a soldier piled wall with shotcrete panel infills was adopted as the preferred retention scheme. The soldier piles would be supplemented with a full depth grout curtain to assist with groundwater management. The retention design was carried out using a soil-structure interaction analyses and the modelling of construction sequences. The retaining wall design has been carried out using the *Oasys* software package FREW and select analyses have been modelled with Plaxis 2D.

A combination of site specific data, published data (Johnston 1992), previous local experience and RocLab rock mass calculation was used to estimate the material parameters. The retention piles were 750 mm diameter bored piles installed at 1.2 m centres on all elevations. Piles were generally embedded 3 m below bulk excavation level and each pile restrained by multiple rows of temporary prestressed cable anchors. Anchor free lengths were extended beyond a line taken up at 45° from the bulk excavation levels.

In addition to the earth pressure distribution a 'minimum rock pressure' was adopted, for rock units which are highly weathered and better, based on a statistical kinematic assessment of potential rock wedges. The kinematic assessment was carried out using Unwedge

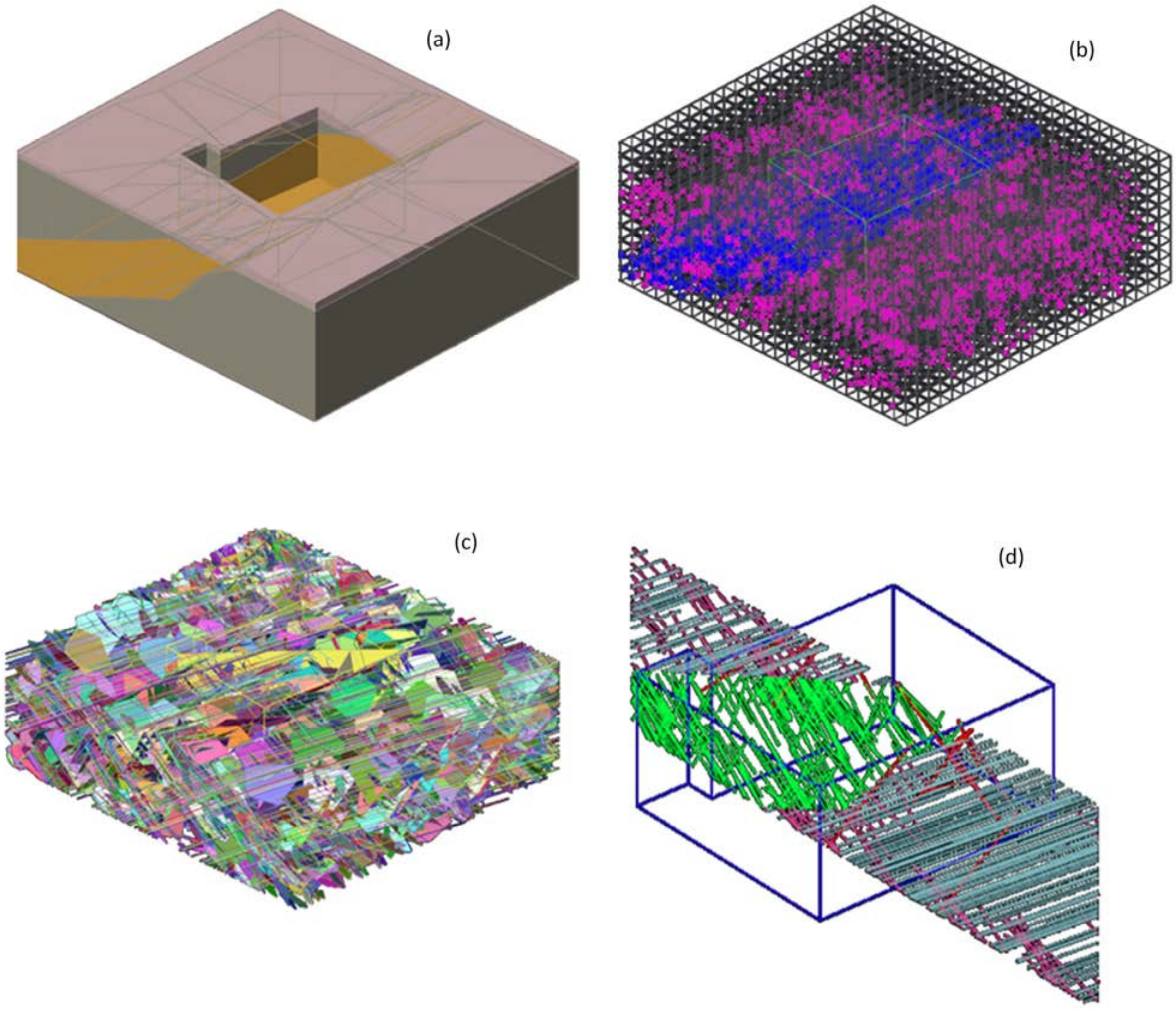

Figure 2 - Schematic of the DFN Modelling process; (a) Site model showing Dyke within Siltstone, (b) Voxelization showing assigned fracture seeds in Siltstone and Dyke, (c) Three dimensional DFN of statically generated defects based on parameters in Table 3, (d) Two dimensional slice through the DFN showing the dominant bedding in the siltstone and distributed defects in the dyke. The DFN was calibrated by comparing the number of defects intersecting boreholes from the investigation and within the DFN model.

Table 3 - Summary of defect type, probabilities and orientations for non-deterministic defects

Defect Type	Probability	Dip Angle (°)		Dip Direction (°)	
		Mean	Stdev	Mean	Stdev
Siltstone Joint 1	50%	16	8	81	27
Siltstone Joint 2	50%	46	3	146	4
Siltstone Bedding Parting	100%	40	0	255	0
Dyke Joint 1	70%	34	18	86	35
Dyke Joint 2	30%	58	5	242	8

and assuming joint persistence varying between 10 m to 40 m in length. A minimum 40 kPa equivalent rock pressure was adopted.

Due to the identified siltstone bedding planes dipping relatively perpendicular to the eastern boundary, the retention along the eastern boundary was designed for a full wedge failure along a bedding plane, just above the bulk excavation level and a third of the excavation height above the bulk excavation level. The free length for the bottom four rows of anchors along the eastern elevation have been taken beyond a line taken up at 30° from the bulk excavation level based on borehole imaging test.

The grout curtain surrounding the site consisted of a single line of holes located behind the retention piles. Grout holes were nominated to be drilled 5° from vertical orientated outside the site. A minimum of 33 primary holes and 32 secondary holes were proposed, resulting in a final centre to centre spacing of 3 m between primaries and secondaries. Grout hole lengths varied to either extend 3 m below bulk excavation or 3 m below the felsic dyke, therefore the grout holes ranged between

30 m to 48 m bgl. Subsequent rounds of grouting were assessed by nominating water pressure tests in areas where the.
grout take has reach the maximum volume criteria to ensure that the rock mass had reached the required rock mas permeability

Up to five rounds of grouting have been completed in the high permeability area in the NW corner. Reviews of the grout takes from the primary, secondary and any further rounds of grouting were compared with the site model as part of the validation process.

As a contingency measure tube-a-manchette (TAM) pipes were nominated at a 2.5 m spacing around the site to allow for targeted grouting during excavation should excessive groundwater inflow be encountered, or the integrity of the grout curtain compromised.

The generated DFN model is intersected with the planned grouting works to estimate the volume of grout take for a nominal treatment volume (Refer to Figure 3). The DFN model was calibrated with the initial grout trial programme to provide more accurate defect volumes (i.e. maximum grout takes) for the subsequent grouting works.

Following grouting works, a validation period was specified which included a pumping from a minimum of five wells and monitoring the groundwater drawdown internal and external to the grout curtain. The results of the validation testing were compared against the results encountered during investigation determining the effectiveness of the primary and secondary grouting.

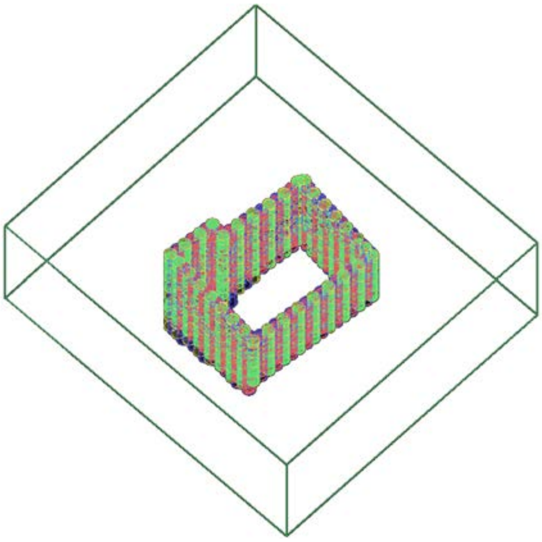

Figure 3 - DFN Model intersected with Grout treatment volumes

8 CONCLUSION

The proposed deep basement has presented particular challenges, including deep retaining walls, significant groundwater inflows, considerable variability in geological and geotechnical conditions and excavation carried out adjacent to a major road occupied heritage structures and critical utilities.

A comprehensive and diverse investigation including full scale on site grout trials was completed for the proposed development. This scope of work has facilitated a hybrid retention solution with traditional bored soldier pile retaining wall with infill permeation grout curtain.
A construction stage review process of grouting records and packer testing during the installation of the grout curtain has provided confidence in the integrity of the grout curtain prior to excavation.

The design has also allowed for the provision of post-excavation grouting if such is shown to be needed as the excavation progresses.

9 ACKNOWLEDGMENTS

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