REPORT TO

FRASERS PROPERTY AUSTRALIA

ON

PREDICTION OF GROUND MOVEMENTS UNDER RAIL CORRIDOR

FOR

PROPOSED HOUSING RENEWAL

AT

Stage 1A, LOT 5-7 POLDING PLACE, TELOPEA, NSW

Date: 14 July 2021
Ref: 33079YCrptFEM

JKGeotechnics
www.jkgeotechnics.com.au

T: +61 2 9888 5000
JK Geotechnics Pty Ltd
ABN 17 003 550 801

Report prepared by:

Thomas Clent
Senior Engineering Geologist

Report reviewed by:

Woodie Theunissen
Principal Associate | Geotechnical Engineer

For and on behalf of
JK GEOTECHNICS
PO BOX 976
NORTH RYDE BC NSW 1670

DOCUMENT REVISION RECORD

Report Reference	Report Status	Report Date
33079YCrptFEM	Final Report	14 July 2021

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<u>ATTACHMENTS</u>

1 INTRODUCTION

This report presents the results of our Finite Element Analysis (FEA) for the proposed residential development at 14 Polding Place, Telopea, NSW. The modelling was commissioned by Mr Chris Koukoutaris of Frasers Property Australia on 19 April 2021 as a variation to PO 4400019206, and was carried out in accordance with our proposal dated 16 April 2021, Ref: P51260YC.

Stage 1A of the development consists of two residential towers with up to 10 above ground levels over two to three basement levels. The lowest basement level (B2) has a proposed finished floor level (FFL) of RL48.4m, which will require excavation to depths ranging from about 5m to 9m below existing ground surface levels but is set well back from the western boundary and the adjoining rail corridor. However, the Lower Ground Floor Level, which has a finished floor level of RL52.5m and requires excavation to a depth of about 2m, extends closer to the western boundary and the adjoining rail corridor. At its closest point excavation to a maximum depth of about 2m will be required and will be set back about 8m from the site boundary. A temporary batter is proposed along the boundary during bulk excavation with permanent support provided by a 0.25m thick concrete wall that will be and subsequently backfilled following completion of the structure.

The purpose of the FEA was to model the proposed Stage 1a development and its potential impact on the adjoining rail corridor. This analysis has been used to predicted the magnitude of movements induced below the adjacent railway line as a result of the proposed development.

2 FINITE ELEMENT ANALYSIS

2.1 Development of Geotechnical Model

To allow the impact of the development on the adjoining railway line to be assessed, a geotechnical model of the site and its surrounds was interpreted at one cross section. This geotechnical model was then used in our numerical analysis to predict the magnitude of displacements induced below the railway line.

The location of the section is shown on the attached Figure 1. The detailed cross section is shown in Figure 2. The geotechnical model is based on the subsurface conditions presented in our geotechnical investigation (Ref: 33079SCrpt, dated 7 May 2020). The borehole locations are shown on the attached Figure 1 while the borehole logs and laboratory test results are presented in Appendix A.

The model divides the subsurface profile into a number of soil and bedrock units. Geotechnical parameters were selected for each geological unit based on the borehole information, the results of field and laboratory strength testing and well-established empirical correlations. While we have not been able to specifically investigate the subsurface conditions present below the rail corridor, Borehole 1 was located approximately 10m east of the rail corridor. We consider that the conditions encountered in this borehole are representative of the materials present below the rail corridor.

The groundwater level adopted for the model was based on the latest monitoring of standpipes which was completed on 29 April 2020. Groundwater levels were modelled at RL52m.

This geotechnical model was then used as the basis of our analysis. PLAXIS 2D, a two-dimensional finite element (FE) computer program was used to complete numerical analysis using the small strain hardening soil model for the soil and the Mohr-Coulomb model for the rock. Staged modelling was completed to simulate the construction steps that are anticipated to be taken during the proposed development.

2.2 Model Geometry and Applied Loads

Cross Section A-A presents the analysed model geometry and is shown in Figure 2. The surface levels were based on the reduced levels shown in the supplied detailed survey plan prepared by Craig and Rhodes (Project: 191-19, Amend No. 01, dated 20 February 2020). The survey provides spot heights and contours within the site but not over the rail corridor. In this regard, the rail corridor levels have been assumed based on the supplied information. The survey plan is attached in Appendix C.

The proposed building location, temporary batters, basement walls, slabs and bulk excavation levels are based on email correspondence with and structural drawings prepared by Robert Bird Group (Job No. 20137, Drawing No. SK012, and SK013, Rev A, dated 7 July 2021). During construction temporary batters will be formed at about 1 Vertical (V): 1 Horizontal (H) to allow the excavation of the Lower Ground Floor Level. Permanent basement walls will then be constructed and subsequently backfilled.

At the time of completing the finite element modelling, the adjacent railway line was under construction. Notwithstanding this, track loads for each of the two rail lines were modelled as a 60kPa uniformly distributed load applied over a 3m width located centrally below the tracks and applied at the base of the ballast. A 10kPa uniformly distributed load extending from the railway line to the site boundary has also been applied. We have been provided with the working loads for the building by Robert Bird Group, however no footing dimensions have been provided. A design allowable bearing pressure of 600kPa has been applied at the three footing locations across the bulk excavation level and footings appropriately sized based on this pressure. Due to the 2D nature of analysis the column loads have, in effect been modelled as strip footings rather than discrete pad footing loads. The footing locations and loads are shown on the attached geotechnical model.

For the purpose of the model, the footing loads have been located within the Class V siltstone bedrock.

2.3 Geotechnical Parameters

A small strain soil hardening model was used to model the behaviour of the soils while the ballast was modelled using a hardening soil model. The siltstone bedrock was modelled using the Mohr-Coulomb model and the tables below detail the parameters adopted for the soils and the bedrock respectively.

Geotechnical Parameters Adopted for Soils			
Parameter	**Ballast**	**Fill-Silty Clay**	**Silty Clay (Hard)**
Unsaturated Unit Weight (kN/m^3)	16	18	18
Saturated Unit Weight (kN/m^3)	19	20	21
Cohesion (c') (kPa)	0.5	0	5
Internal Angle of Friction (ϕ')	42	28°	28°
Dilation Angle(Ψ)	12°	0°	0°
Modulus (E_{50}) (MPa)	21	15	75
Modulus (E_{oed}) (MPa)	10.5	7.5	37.5
Unload/Reload Modulus (E_{ur}) (MPa)	63	45	225
Shear Strain at 0.7G$_0$	-	1.5 x 10^{-4}	1.5 x 10^{-4}
Reference Shear Modulus G_o^{ref}(MPa)	-	46.87	234.3

Geotechnical Parameters Adopted for Siltstone and Sandstone Bedrock					
Unit	**Unit Weight (kN/m^3)**	**Cohesion (c) (kPa)**	**Internal Angle of Friction (ϕ)**	**Young's Modulus (E) (MPa)**	**Poisson's Ratio**
Siltstone Class V	22	20	28°	150	0.25
Siltstone Class IV	23	50	30°	250	0.2
Siltstone/Sandstone Class III	24	250	30°	900	0.2

Where soil or bedrock is in contact with structural elements, a reduction factor (R_{inter}) of 0.67 has been adopted. This is applied to the soil or bedrock strength parameters to model the reduction in shear strength between the two dissimilar materials.

2.4 Structural Parameters

The following structural parameters have been adopted for the structural elements

Structural Element	*Youngs Modulus (E) (MPa)*	*Second Moment of Inertia (I) per meter run (m^4/m)*	*Cross Sectional Area (A)* (m^2/m run)	*Poissons Ratio*
Core Filled Concrete Walls	20,000	1.3×10^{-3}	0.25	0.15
Concrete Slabs (200mm thick)	20,000	-	0.2	0.15

2.5 Initial Stress State

The initial stress field has been modelled by the adoption of K_o values relating horizontal and vertical stresses for specific units. The K_o value adopted has been calculated on the basis of the relationship Ko = 1- sinϕ.

2.6 Model Stages

A nil step was then run after the initial calculation stage. The purpose of this nil stage was to allow the stresses to re-orientate themselves to more accurately reflect the stress state that will occur where a non-horizontal surface exists.

The model was run through a number of stages in an attempt to simulate the existing conditions and the construction procedure. The stages are set out below.

Section A-A

1. Initial phase to develop stress state,

1b. Nil Step,

2. Existing Conditions: Apply surcharge loads to the rail track.

   Total movements are reset at this point such that only the movements induced by the development are reported.

3. Excavate to BEL RL52.5m forming a temporary slope batter at 1V:1H
4. Construction of basement walls and floor slabs
5. Apply Footing Loads
6. Backfill behind basement wall.

3 ANALYSIS RESULT

The analysis results for the modelled section are tabulated below. We note that on completion of analysis *Stage 3 Existing Conditions,* all displacements were reset to zero. This zeroing of movements allows only the movement induced by the excavation, retaining walls, floor slab, building loads and backfill to be analysed. In the table below we have provided results of the modelling for Stages 3, 5 and 6. The results summarise the movements induced below the rail line.

Section A-A - Maximum Cumulative Displacement Cumulative Maximum Displacement below Railway Track (mm)			
Stage	Predicted Cumulative Track Movements		
	Vertical	Horizontal	*Twist
3	0.01	0.01	0.003
5	0.03	0.04	0.010
6	0.01	0.04	0.011

**Twist is the differential settlement between the eastern and western tracks on the section line drawn through the corridor.*

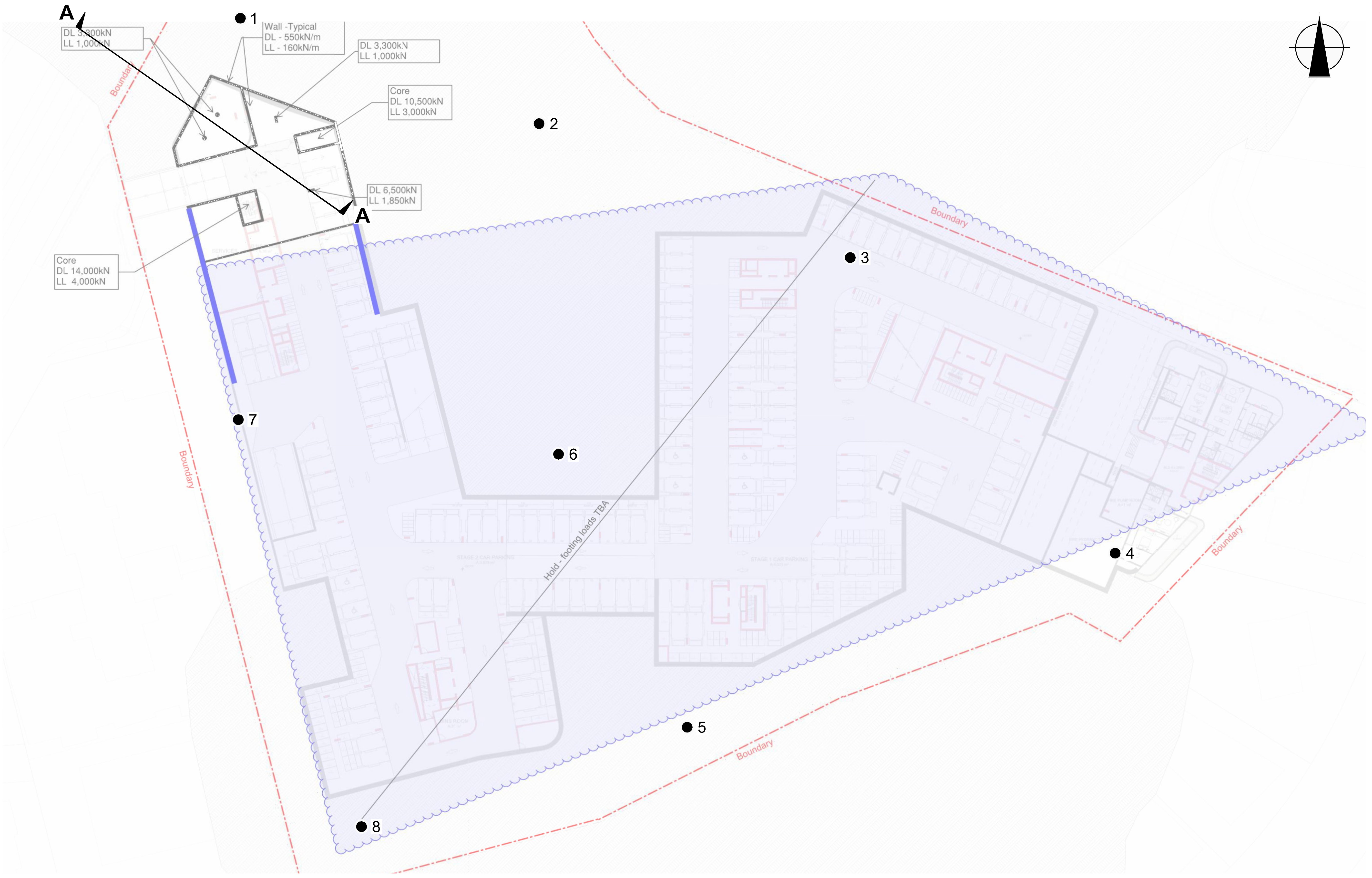
The modelling has shown that the effect of the proposed excavation and construction of the building for the Stage 1A development at 5-7 Polding Place will induce negligible total movements (no greater than 0.04mm) below the railway tracks. As discussed above, we believe that due to the limitations of two-dimensional modelling that actual movements are likely to be less than the predicted movements although the extent to which this is the case is difficult to quantify.

4 GENERAL COMMENTS

Plaxis 2D has been used to model the effect of the proposed development on Sydney Trains assets. Whilst all efforts have been made to check the reasonableness of the reported results the simulation of geotechnical problems by means of the finite element method implicitly involves some inevitable numerical approximations. Consequently, while results have been calculated to three decimal places, it is unlikely that their accuracy is to this order. Observation of displacements during the proposed stages of construction should be used to verify the accuracy of the analysis.

The modelling has been based on information available to us, which has been checked for accuracy to the extent reasonably possible. If additional information becomes available at any stage during the project which appears in conflict with current assumptions then we should immediately be notified and asked to review our analysis.

This report has been prepared for the particular project described and no responsibility is accepted for the use of any part of this report in any other context or for any other purpose. If there is any change in the proposed development described in this report then all recommendations should be reviewed. Copyright in this report is the property of JK Geotechnics. We have used a degree of care, skill and diligence normally exercised by consulting engineers in similar circumstances and locality. No other warranty expressed or implied is made or intended. Subject to payment of all fees due for the investigation, the client alone shall have a licence to use this report. The report shall not be reproduced except in full.



0 6 12 18 24 30
SCALE 1:600 @A3 METRES

This plan should be read in conjunction with the JK Geotechnics report.

Title: **BOREHOLE LOCATION PLAN**

Location: POLDING PLACE, TELOPEA, NSW

Report No: 33079YC

Figure: 1

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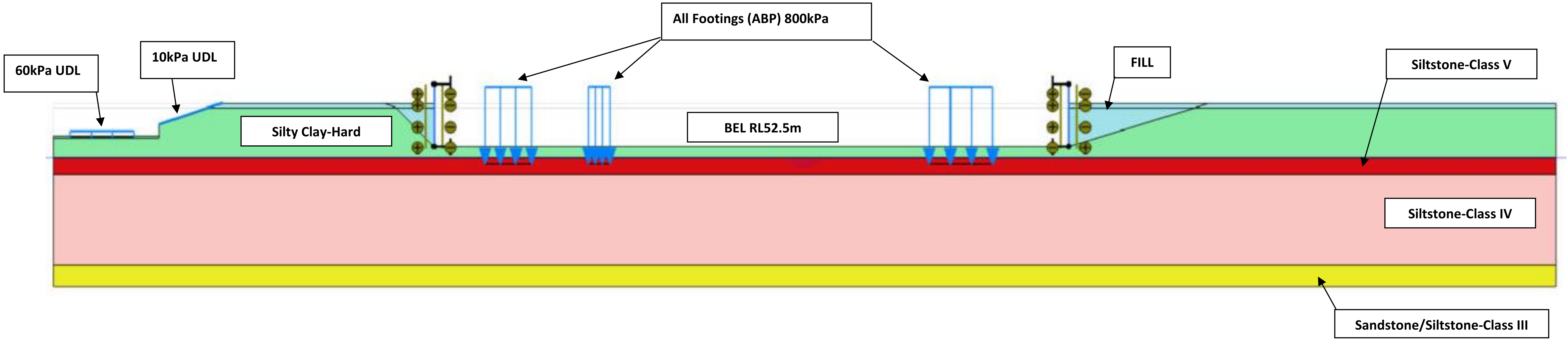
PLOT DATE: 14/07/2021 4:53:14 PM DWG FILE: Y:\33000'S\33079SC TELOPEA\CAD\33079YC.DWG



Geotechnical Model-Cross Section A-A

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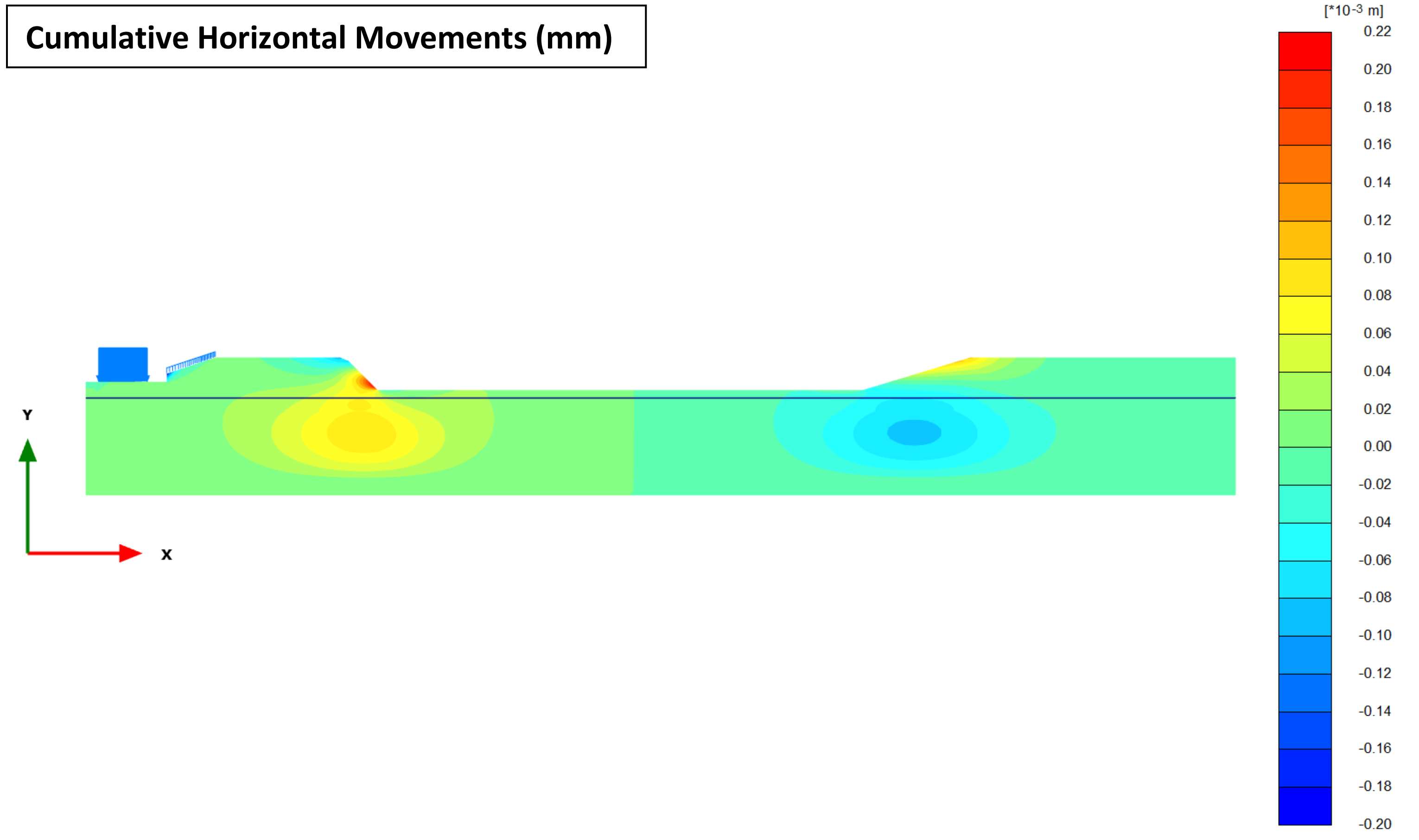
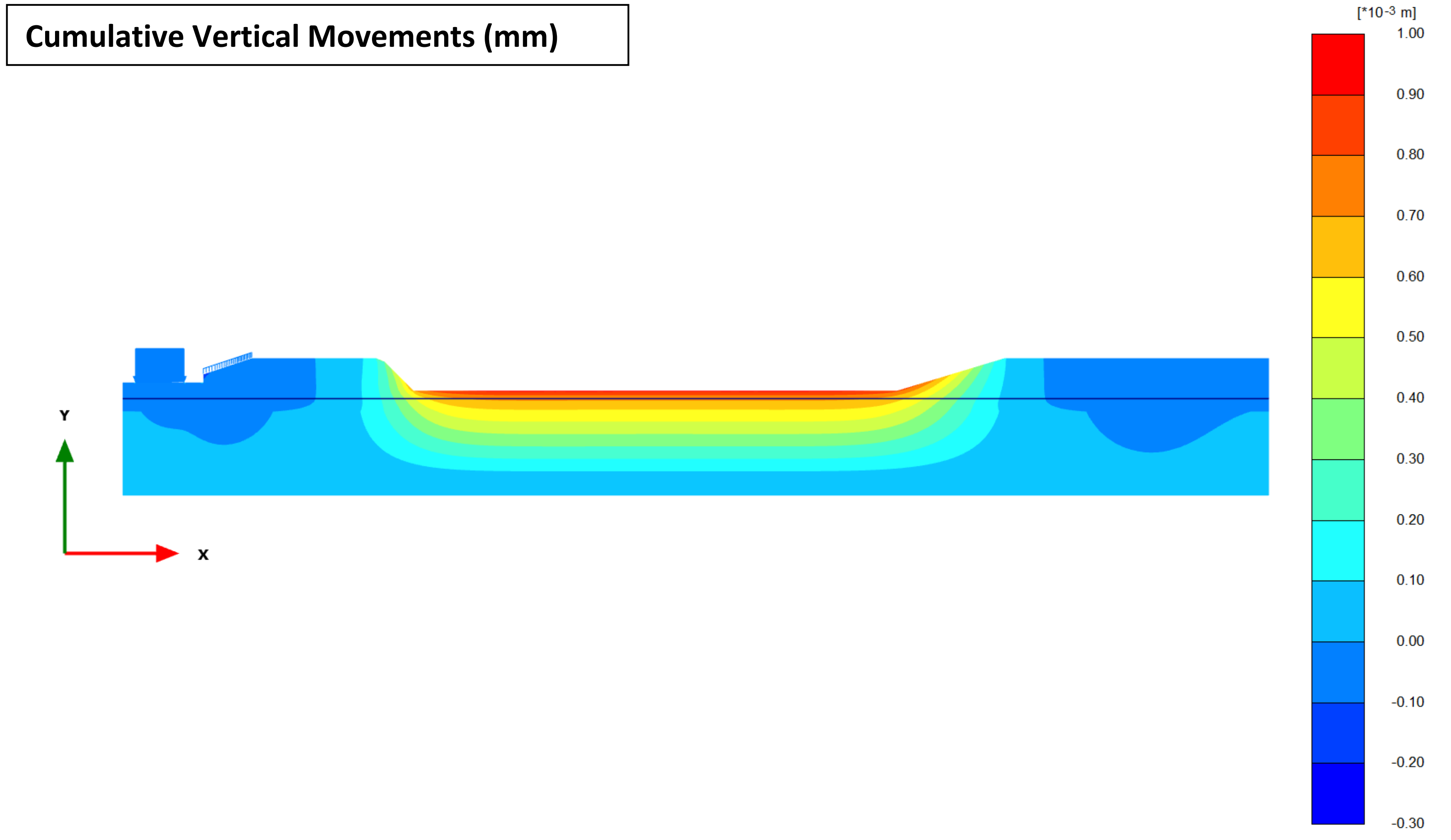
Report No 33079YC Figure No. 2



Cumulative Horizontal Movements (mm)



Cumulative Vertical Movements (mm)

Cumulative Horizontal and Vertical Movements (mm)
Stage 3: Excavate to RL52.5m

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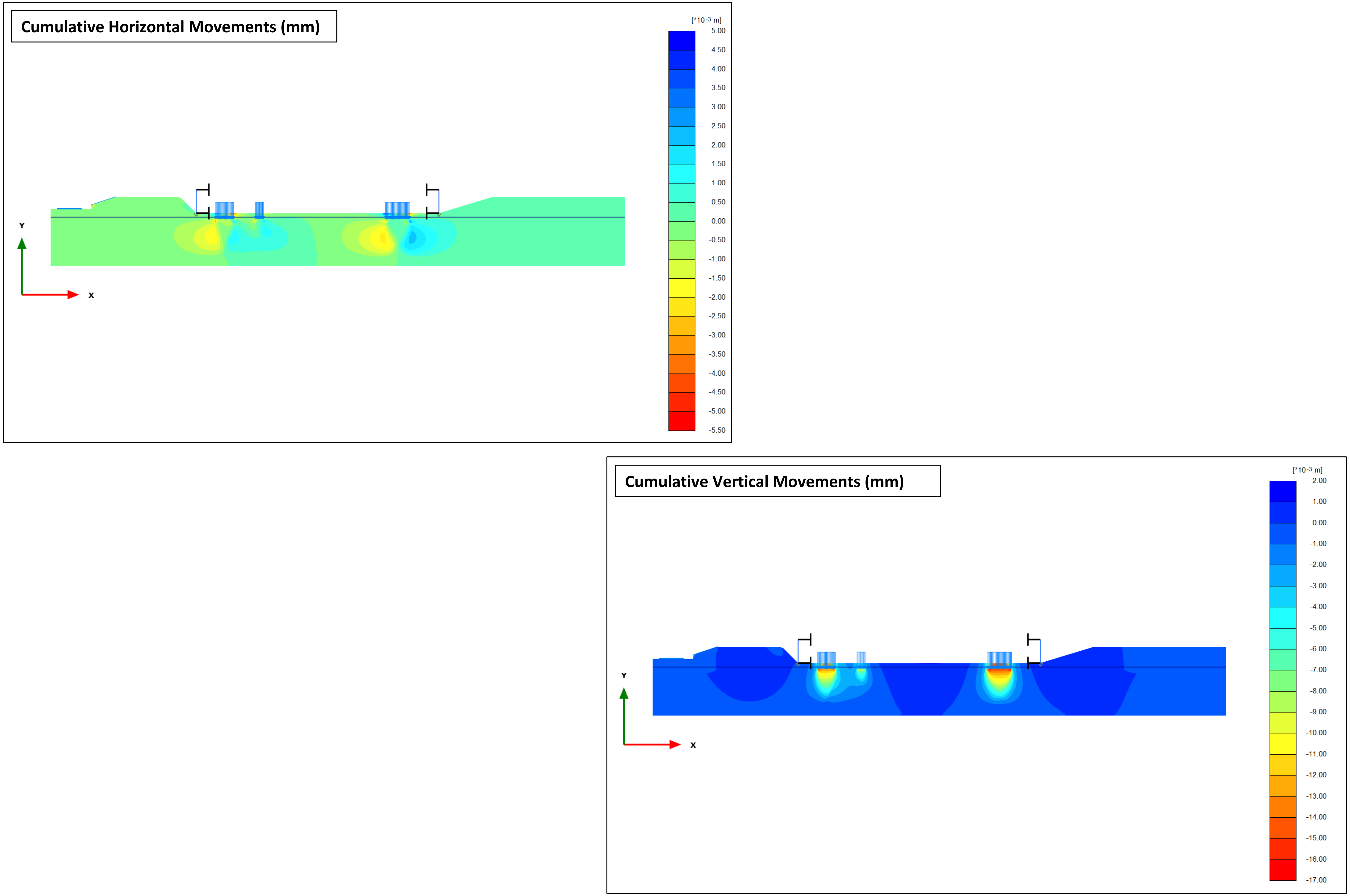
Figure No. 3


Cumulative Horizontal Movements (mm)

Cumulative Vertical Movements (mm)

Cumulative Horizontal and Vertical Movements (mm)
Stage 5: Apply Footing Loads

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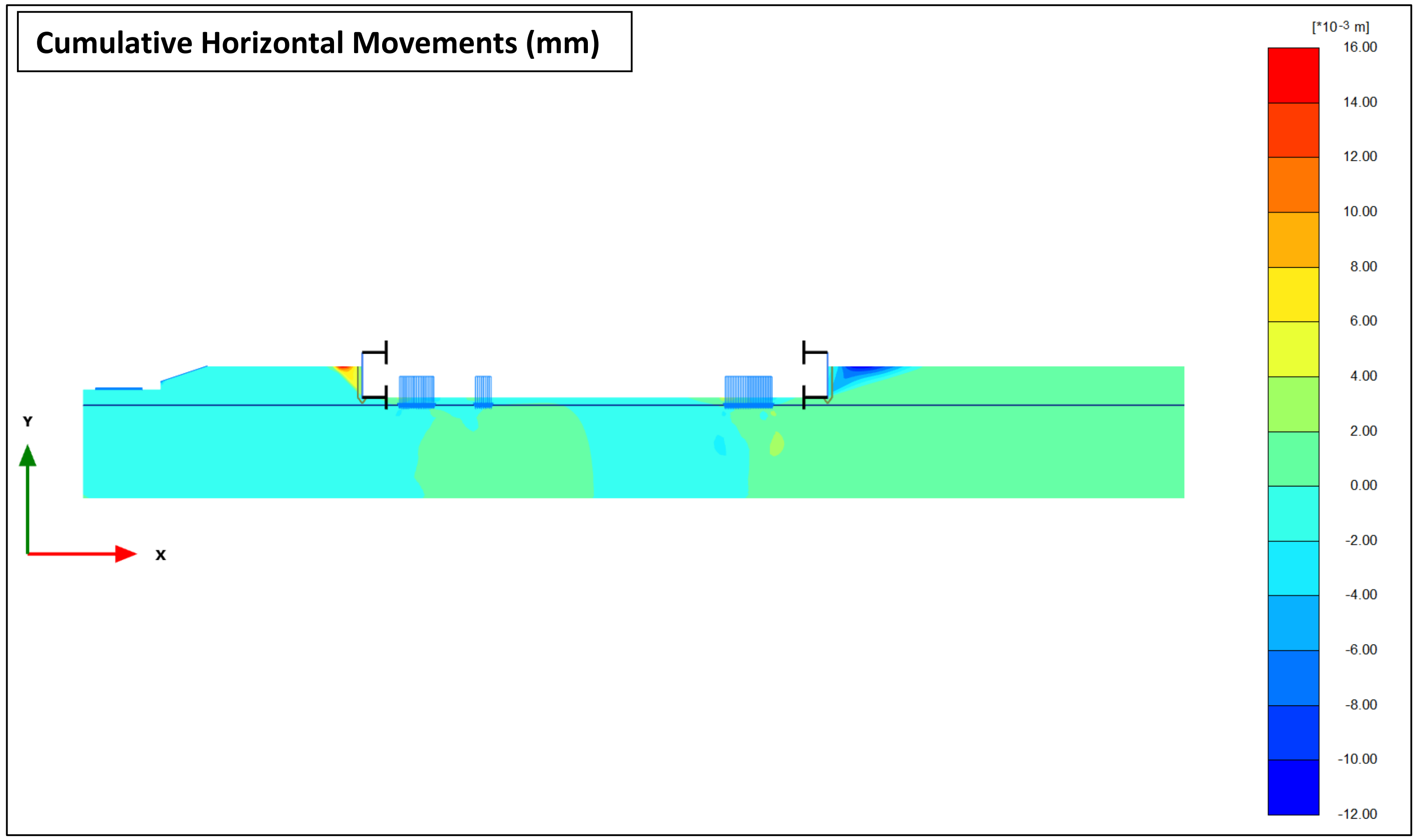
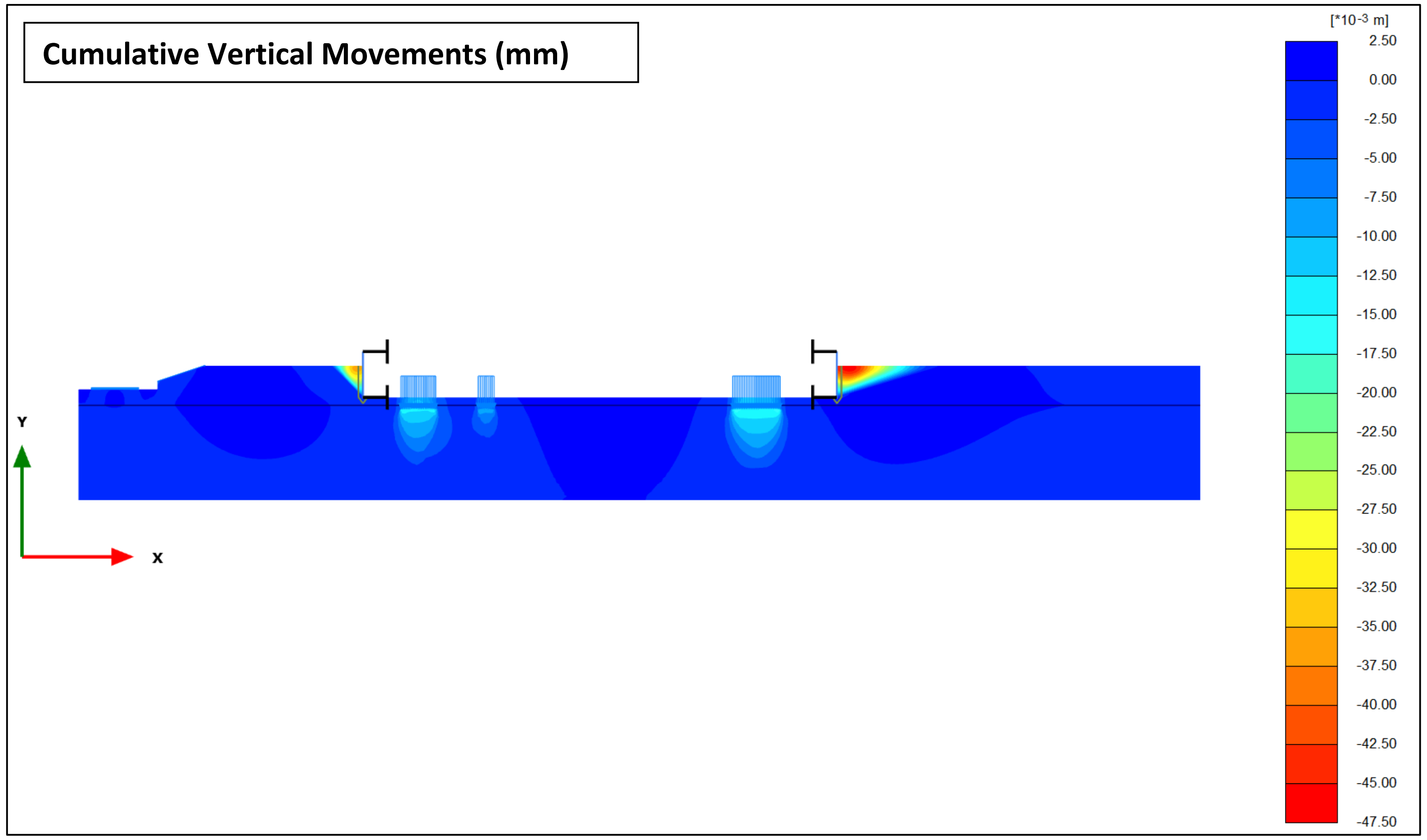
Figure No. 4





Cumulative Horizontal and Vertical Movements (mm)
Stage 6: Backfill

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Report No 33079YC

Figure No. 5

APPENDIX A

Geotechnical Report Prepared by JK Geotechnics

(Ref:33079SCrpt, dated 7 May 2020)

REPORT TO

FRASERS PROPERTY TELOPEA DEVELOPER LIMITED

ON

STAGE 1A - GEOTECHNICAL INVESTIGATION

FOR

PROPOSED HOUSING DEVELOPMENT

AT

POLDING PLACE, TELOPEA, NSW

Date: 25 August 2020
Ref: 33079SCrptRev1

JKGeotechnics
www.jkgeotechnics.com.au

T: +61 2 9888 5000
JK Geotechnics Pty Ltd
ABN 17 003 550 801

Report prepared by:

Thomas Clent
Senior Engineering Geologist

Report reviewed by:

Paul Stubbs
Principal | Geotechnical Engineer

For and on behalf of
JK GEOTECHNICS
PO BOX 976
NORTH RYDE BC NSW 1670

DOCUMENT REVISION RECORD

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33079SCrpt	Final Report	7 May 2020
33079SCrptRev1	Additional Information	25 August 2020

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STS Table B: Moisture Content Test Report
Envirolab Services Certificate of Analysis No. 241381
Borehole Logs 1 to 8 Inclusive (With Core Photographs)
Figure 1: Site Location Plan
Figure 2: Borehole Location Plan
Vibration Emission Design Goals
Report Explanation Notes

1 INTRODUCTION

This report presents the results of a geotechnical investigation for the proposed Stage 1A housing renewal project at Polding Place, Telopea, NSW. The location of the site is shown in Figure 1. The investigation commissioned by Cameron Jackson of Frasers Property Australia and was carried out in accordance with our proposal dated 21 February 2020, Ref: P51260S.

At the time of the fieldwork, as shown on the supplied preliminary masterplan basement car parking plan (20181205, no date) provided by Frasers Property Group, we understood the existing buildings on site would be demolished and a residential development constructed with 1-2 levels of basement car parking.

Since completing the fieldwork, we have been provided with the preliminary architectural plans by Plus Architects Pty Ltd (Job No. 20320, Revision 03, dated 28 April 2020) and the survey plan by Craig & Rhodes (Ref:191-19, Amend No.01, dated 20 February 2020). From the drawings we understand the proposed residential development will comprise two residential towers with up to 10 above ground levels over two to three basement levels. The lowest basement is proposed at RL49.4m and RL49.9m which will require excavation to depths ranging from about 5m to 9m depth below existing ground surface levels.

The purpose of the investigation was to obtain geotechnical information on the subsurface conditions as a basis for comments and recommendations on excavation, groundwater, retention and footings.

2 INVESTIGATION PROCEDURE

Boreholes BH1 to BH8 were drilled using our track mounted JK308 drilling rig to total depths ranging from 6.90m to 10.65m below the existing ground surface. The boreholes were auger drilled to depths ranging from 4.10m to 7.60m and were then continued using diamond coring techniques using an NMLC core barrel with water flush to depths between 6.9m and 10.65m.

The borehole locations, as shown on Figure 2, were set out by taped measurements from existing surface features. The approximate surface levels, as shown on the borehole logs, were estimated by interpolation between the spot levels and contours shown on the supplied survey plan by Craig & Rhodes (Ref:191-19, Amend No.01, dated 20 February 2020. The datum of the levels is Australian Height Datum (AHD).

The strength of the residual soils was assessed from Standard Penetration Test (SPT) ‘N’ values, augmented by hand penetrometer readings on cohesive samples returned by the SPT split tube sampler. Within the augered portions of the boreholes the strength of the underlying weathered bedrock was assessed by observation of the drilling resistance with a Tungsten Carbide (TC) bit attached to the augers, together with examination of the recovered rock cuttings and subsequent correlation with laboratory moisture content test results. The strength of the cored rock was assessed with reference to laboratory Point load Strength Index ($I_{S(50)}$) test results carried out on the recovered rock core. The results of the point load strength index tests are summarised in the attached STS Table A and on the cored borehole logs.

Groundwater observations were made during and on completion of drilling. Groundwater monitoring wells were installed within BH1 and BH4 on completion of drilling and a return visit was made to the site to measure the groundwater levels on 29 April 2020. No longer-term monitoring of groundwater levels was carried out.

Our geotechnical engineer, Mr Warren Smith, set out the borehole locations, nominated the sampling and testing locations, and prepared logs of the strata encountered. The borehole logs are attached to this report together with a set of explanatory notes, which describe the investigation techniques, and their limitations, and define the logging terms and symbols used.

Selected samples were returned to Soil Test Services Pty Ltd (STS) and Envirolab Services Pty Ltd, both NATA accredited laboratories, for testing to determine moisture contents, point load strength index values, pH, sulphate content, chloride content and resistivity. The results of the laboratory testing are summarised in the attached STS Tables A and B and Envirolab Certificate of Analysis 241381.

3 RESULTS OF INVESTIGATION

3.1 Site Description

The site lies within undulating topography on the upper reaches of a south-easterly facing hillside which slopes at about 5° to 10°. The site is accessed by Polding Place which is a cul-de-sac road running off Sturt Street which lies to the north-east.

At the time of the fieldwork the site contained five 3-storey brick apartment buildings. Each building contained a ground level asphaltic concrete surfaced car park with concrete footpaths leading to the apartment buildings. The footpaths, buildings and car parks all looked in fair condition, based on a cursory inspection. Grass and low to medium height trees were interspersed between the buildings and footpaths. A 1m high stone retaining wall laid back at about 45° was positioned within the south-eastern portion of the site and retained the garden areas of the apartment building. An asphaltic concrete surfaced car park was positioned at the toe of the retaining wall.

To the south-east of the site are residential properties which adjoin the common boundary; where the boundary conditions could be observed, the surface levels were predominantly similar and followed the gradual slope of the hillside. However, some of the adjacent buildings appeared to have at least one level of basement which was observed from their Manson Street frontage.

To the north of the site is a railway track which is between about 1m to 2m lower than the site, with a gently sloping batter along the site boundary.

To the west of the site are more residential buildings ranging from one to two storeys; the ground surface levels across the boundaries are predominantly similar.

3.2 Subsurface Conditions

The 1:100,000 geological map of Sydney indicated the site is underlain by Ashfield Shale of the Wianamatta Group, but close to the boundary with the underlying Hawkesbury Sandstone.

In summary, the boreholes encountered surface fill covering residual silty clay that graded into weathered siltstone and then sandstone bedrock at shallow to moderate depths. Further comments on the subsurface conditions encountered are provided below. Reference should be made to the attached borehole logs for detailed descriptions of the subsurface conditions encountered.

Fill

Fill was encountered in all boreholes to depths ranging from 0.2m to 1.2m. The fill comprised silty clay, with inclusions of ash, and sandstone/ironstone gravel.

Residual Silty Clay

Residual silty clay was encountered below the fill in all boreholes apart from BH3, which encountered extremely weathered siltstone. The residual silty clay was assessed to be of medium to high plasticity and generally hard strength.

Weathered Bedrock

Weathered sandstone and siltstone bedrock were encountered at depths ranging from 1.7m to 4.2m, with the level of the surface of the rock falling towards the south and west from about RL58.5m in BH3 to about RL52.6m in BH7. The siltstone was initially assessed from auger drilling to be extremely weathered to distinctly weathered and of hard (soil strength) to very low strength, increasing to low to medium strength and then generally medium to high strength shortly thereafter. The upper rock core within In BH1, BH3, BH5, and BH6, was assessed to be moderately weathered to slightly weathered and of at least medium strength. In BH7 the cored siltstone was initially extremely weathered to moderately weathered and of very low strength, improving to fresh and high strength sandstone at a depth of 7.8m. Medium to high strength bedrock was encountered in all boreholes at depths ranging from 4.1m (~RL54.4m) to 7.8m (~RL47.3m).

Some bands of extremely weathered siltstone were also present within the core. Defects within the core comprised sub-horizontal bedding partings, crushed seams, joints inclined predominantly between 30° to 90° and extremely weathered and clay seams of up to 200mm thickness.

Groundwater

Groundwater seepage was encountered within BH1 and BH7 at depths of 1.2m and 4.4m during auger drilling, the remaining boreholes measured as dry on completion of auger drilling. Thereafter, the use of water for core drilling limited further meaningful measurements of groundwater levels. The groundwater levels were measured within the monitoring wells installed within BH1 and BH4 on the 29 April 2020 and showed groundwater at depths of 2.6m and 3m, respectively.

3.3 Laboratory Test Results

The moisture content and point load strength index test results showed reasonably good correlation with our field assessment of rock strength. The Unconfined Compressive Strength (UCS) of the rock core, estimated from the point load strength index test results, generally ranged from 1MPa to 66MPa with some higher results of up to 78MPa.

The pH values on samples of the residual silty clay and weathered bedrock were between 5.4 and 6.2, indicating slightly acidic soil conditions. The sulphate contents ranged from 93mg/kg to 260mg/kg, the chloride contents ranging from 10mg/kg to 170mg/kg, and the resistivity ranged from 8,300ohm.cm to 62,000ohm.cm. Based on these results, the residual silty clay and weathered bedrock lie within the 'mild' exposure classification for concrete piles in accordance with Table 6.4.2(C) of AS2159-2009 'Piling – Design and Installation' and 'non-aggressive' exposure classification for steel piles in accordance with Table 6.5.2(C) of AS2159-2009.

4 COMMENTS AND RECOMMENDATIONS

4.1 Excavation

Prior to the start of excavation, we recommend that dilapidation surveys be completed on structures located within a horizontal distance from the excavation perimeter of at least twice the excavation depth. The dilapidation surveys should comprise detailed inspections of the adjoining buildings, both externally and internally, with all defects rigorously described, i.e. defect location, defect type, crack width, crack length, etc. The respective owners of the adjoining properties should be asked to confirm that the dilapidation reports represent a fair record of actual conditions. The preparation of the dilapidation reports will also help to guard against opportunistic claims for damage that was present prior to the start of excavation.

Excavation to the required depths of about 5m to 9m will encounter fill, residual silty clay and for the most part weathered siltstone ranging in strength from very low to high.

Excavation of the soils will be achievable using conventional excavation equipment, such as the buckets of hydraulic excavators. Some of the upper weathered siltstone may also be able to be excavated using such equipment.

Excavation of the rock of low strength or higher strength will require assistance using rock excavation equipment, such as hydraulic rock hammers, ripping hooks, rotary grinders or rock saws. It may be found that such rock excavation equipment will be required to break through bands of higher strength rock and then the weaker bands being able to be removed using the excavator bucket.

Hydraulic rock hammers must be used with care due to the risk of damage to the adjacent structures from the vibrations generated by such equipment. If hydraulic rock hammers are used the vibrations transmitted to the adjoining properties to the south and north should be quantitatively monitored at all times during rock hammer use. The monitors should be attached to flashing warning lights, or other suitable devices, to warn the operator when acceptable limits have been reached so that excavation works can cease. Reference should be made to the attached Vibration Emission Design Goals sheet for acceptable limits of transmitted vibrations.

Where the transmitted vibrations are excessive it would be necessary to change to alternative excavation equipment, such as smaller rock breakers, ripping hooks, rotary grinders or rock saws. A rock saw could be used to cut a slot around the perimeter of the excavation prior to the use of a hydraulic hammer to break the rock from between the saw cuts in order to limit the transmitted vibrations. However, the effectiveness of this must be confirmed by the results of vibration monitoring.

4.2 Groundwater

Groundwater was encountered within the wells installed in BH1 and BH4 at levels between RL53.5m and RL51.8m, which is above the proposed lowest basement at RL49.4m. Due to the variability in levels within the wells we expect that the groundwater measured comprises seepage flowing above and through the weathered rock and collecting within the wells. Therefore, during construction we expect that any seepage that does occur within the excavation may occur at various locations within the site and may emerge at variable depths within the rock profile. The seepage would tend to occur along the soil/rock interface and through bedding partings and joints within the rock profile.

During construction any such seepage that does occur should be able to be controlled using conventional sump and pumps techniques.

In the long term, drainage should be provided behind all retaining walls and possibly below the basement slab. The completed excavation should also be inspected by the hydraulic consultant to confirm that the designed drainage system is adequate for the actual seepage flows.

4.3 Retention

Where space permits temporary batters through the clayey soils and poor-quality siltstone and sandstone bedrock (such as over the northern sides of the excavation) may be formed at no steeper than 1 Vertical (V): 1 Horizontal (H). Where adopted all surcharge loads such as stockpiles, traffic loads etc must be kept well clear of the crest of the batters (ie below a 45° line drawn upwards from the toe of the batter) Where permanent batters are adopted they should be formed at no steeper than 1Vertical (V) : 2 Horizontal (H) and should be protected from erosion by vegetation, shotcrete and mesh or similar. For maintenance purposes it may be more practical to from permanent batters at no stepper than 1V:3H or 4H.

Where space does not allow for the formation of batters and excavation will extend below adjoining properties a retention system will need to be installed prior to the commencement of excavation. Such a retention system may comprise soldier pile walls with shotcrete infill panels. From experience the construction of such shoring systems has become very cost effective and we do not expect that creation of temporary batters, stockpiling of materials for use as back fill, export of surplus materials to tip, import of expensive drainage gravel and construction of "conventional" retaining walls will be an economical option in any circumstance.

Bored piers would be appropriate for the piled walls, but some groundwater seepage may be encountered requiring the use of pumps and tremie concreting techniques. The piers should be founded at least 1m below the base of the excavation, including excavations for footings and services, but more as required for stability design.

Temporary lateral restraint of the retention system would be required in the form of external anchors or internal props, with each restraining point progressively installed as it is exposed during excavation. Long term lateral support would be provided by the floor slabs within the excavation and the toe sockets of the piles. If anchors are to locally extend below neighbouring properties, permission would need to be obtained from the owners of the adjoining properties before the installation of the anchors below their properties. Such permission can take some time to obtain, which should be allowed for within the project program. The use of anchors will need to take into account the location of any basements, such as to the south-east, and services within the adjoining streets so that these can be avoided. For the south-eastern side of the excavation, anchors are unlikely to be feasible due to the adjoining basement and the use of internal props for this side of the basement are likely to be required. However, this will be subject to the final basement set-back plan.

Propped or anchored retaining walls may be designed based on a trapezoidal earth pressure distribution of magnitude 6H kPa, where H is the retained height in metres, where structures or movement sensitive services are located beyond a horizontal distance of 2H from the wall. Where structures or movement sensitive services are located within 2H of the wall, a trapezoidal earth pressure distribution of 8H kPa should be used. These pressures should be constant over the central 50% of the trapezoidal pressure distribution. In addition to these pressures, the retention wall design should be checked and designed to accommodate a wedge formed by a joint inclined at 45° intersecting the excavation face at the base of the cut.

The above pressures assume horizontal backfill behind the walls and any inclined backfill should be taken as a surcharge load. All surcharge loads should be allowed for in the design, plus full hydrostatic pressures, unless measures are undertaken to provide complete and permanent drainage behind the wall.

Anchors should have their bond formed within rock outside a line drawn up at 45° from the base of the excavation, with a minimum bond length of 3m and a minimum free length of 3m. Provisional design of the anchors may be based on a bond stress of 100kPa for rock of at least very low to low strength. All anchors should be proof loaded to at least 1.3 times their design working load before locking off at about 80% of their working load. Lift-off tests should be carried out on at least 10% of the anchors 24 to 48 hours following locking off to confirm that the anchors are holding their load. Generally, anchors are installed on a design and construct basis so that optimisation of the bond stresses does not become a contractual issue in the event of anchors failing to hold their test loads.

Passive toe resistance of the retention system below the base of the bulk excavation may be estimated based on an allowable lateral resistance of 200kPa for rock of at least very low to low strength. The passive resistance should be ignored for at least 0.5m below the base of the excavation, including footing and service excavations.

4.4 Footings

Since siltstone or sandstone bedrock will be encountered within the excavation the use of pad or strip footings founded within the rock will be appropriate. If any above ground portions of the building extend outside of the basement footprint these portions should be supported on piles founded within the rock below a line drawn up at 45° from the base of the excavation so that additional loads are not placed on the basement walls, unless the walls have been designed for such loads.

The rock encountered within the boreholes was found to be variable with bands of very low to low strength rock in amongst rock of medium to high strength. Therefore, the footings will need to be designed for an allowable bearing pressure appropriate for the lower strength rock and not the medium to high strength sandstone, unless a closer spaced grid of boreholes enables the rock quality to be more accurately characterised. We recommend that the design of the footings be based on an allowable end bearing pressure (AEBP) of no more than 1200kPa.

The table below shows the depth and level of rock adequate for a higher (AEBP) of 3500kPa which was encountered at variable depths within some of the boreholes. However, due to the variability and absence of such adequate bedrock from two of the boreholes (BH2 and BH6) If higher bearing pressures were required a closer grid of additional cored boreholes would need to be drilled to greater depths to assess where such consistent rock is present. Also, piles would need to be adopted in order to found within consistently higher strength rock which is likely to be absent at bulk excavation level over at least part of the site.

Borehole	**Depth and Level of Rock Adequate for Allowable End Bearing Pressures (AEBP)**					
	600kPa		**1200kPa**		**3500kPa**	
	Depth	**Approx Level (AHD)**	**Depth**	**Approx Level (AHD)**	**Depth**	**Approx Level (AHD)**
1	2.8m	53.3m	3.5m	52.6m	8.0m	48.1m
2	1.5m	57.4m	2.0m	56.9m	Not Proven	
3	Not encountered		0.5m	58.7m	4.5m	54.2m
4	Not encountered		1.3m	53.5m	6.2m	48.6m
5	1.4m	57.0m	1.8m	56.6m	4.5m	53.9m
6	1.2m	57.3m	2.1m	56.4m	Not Proven	
7	2.5m	52.6m	6.5m	48.6m	7.8m	47.3m
8	1.0m	55.6m	1.6m	55.6m	7.8m	48.8m

Where piles are used, allowable shaft adhesions equivalent to 10% of the allowable end bearing pressure may be used for the design of piles in compression, below a nominal 0.3m socket and provided socket roughness and cleanliness is maintained.

The footing excavations should be inspected by a geotechnical engineer to confirm that the appropriate foundation material has been encountered.

4.5 Basement Slab

The subgrade at bulk excavation level will comprise weathered siltstone. As recommended above, drainage will need to be provided below the basement slab either as a closely spaced grid of subsoil drains or a gravel blanket. The drainage will need to be connected to a permanent fail-safe pump out system, which is fitted with automatic level controls to avoid flooding.

The basement slab should be designed with a subbase layer of at least 100mm thickness of crushed rock to RMS QA specification 3051 (2013) unbound base material (or other approved good quality and durable fine crushed rock), which is compacted to at least 100% of Standard Maximum Dry Density (SMDD) if a continuous drainage blanket is not adopted. This subbase layer will provide a separation between the siltstone/sandstone subgrade and the slab and provide a uniform base for the slab.

4.6 Nearby Railway Line

As shown on the both the Annexure N-Staging Plan and survey plan by Craig & Rhodes (Ref:191-19, Amendment No.01, dated 20 February 2020) the railway line (Carlingford Line) is located on the north-western and western sides of the site. We understand the current works being carried out on the rail line are part of the upgrade to the Telopea Light Rail scheme.

Application will need to be given to the asset owners (Sydney Trains) for any development which is in proximity to the rail corridor. Sydney Trains may require finite element analysis of the possible movements affecting the rail infrastructure where parts of the development may be positioned within 25m of the rail corridor. Sydney Trains may also require monitoring to be carried out during construction, bur the extent of this will be dependent on the results of the modelling.

4.7 Further Geotechnical Work/Construction Inspections

- Groundwater seepage/level monitoring or assessment.
- Shoring pile inspections.
- Progressive inspections of the excavation to check for the presence of adversely orientated defects.
- Site Inspection at bulk excavation level to refine areas which may be appropriate for high bearing pressures
- Geotechnical inspection of all footing excavations and pile drilling.
- Additional cored boreholes where bearing pressures are to be more than 1,200kPa.

5 GENERAL COMMENTS

The recommendations presented in this report include specific issues to be addressed during the construction phase of the project. As an example, special treatment of soft spots may be required as a result of their discovery during proof-rolling, etc. In the event that any of the construction phase recommendations presented in this report are not implemented, the general recommendations may become inapplicable and JK Geotechnics accept no responsibility whatsoever for the performance of the structure where recommendations are not implemented in full and properly tested, inspected and documented.

Occasionally, the subsurface conditions between the completed boreholes may be found to be different (or may be interpreted to be different) from those expected. Variation can also occur with groundwater conditions, especially after climatic changes. If such differences appear to exist, we recommend that you immediately contact this office.

This report provides advice on geotechnical aspects for the proposed civil and structural design. As part of the documentation stage of this project, Contract Documents and Specifications may be prepared based on our report. However, there may be design features we are not aware of or have not commented on for a variety of reasons. The designers should satisfy themselves that all the necessary advice has been obtained. If required, we could be commissioned to review the geotechnical aspects of contract documents to confirm the intent of our recommendations has been correctly implemented.

A waste classification will need to be assigned to any soil excavated from the site prior to offsite disposal. Subject to the appropriate testing, material can be classified as Virgin Excavated Natural Material (VENM), General Solid, Restricted Solid or Hazardous Waste. Analysis takes seven to 10 working days to complete, therefore, an adequate allowance should be included in the construction program unless testing is completed prior to construction. If contamination is encountered, then substantial further testing (and associated delays) should be expected. We strongly recommend that this issue is addressed prior to the commencement of excavation on site.

This report has been prepared for the particular project described and no responsibility is accepted for the use of any part of this report in any other context or for any other purpose. If there is any change in the proposed development described in this report then all recommendations should be reviewed. Copyright in this report is the property of JK Geotechnics. We have used a degree of care, skill and diligence normally exercised by consulting engineers in similar circumstances and locality. No other warranty expressed or implied is made or intended. Subject to payment of all fees due for the investigation, the client alone shall have a licence to use this report. The report shall not be reproduced except in full.

115 Wicks Road
Macquarie Park NSW 2113
Telephone: 02 9888 5000
Facsimile: 02 9888 5001

SOIL TEST SERVICES

TABLE A
POINT LOAD STRENGTH INDEX TEST REPORT

Client:	JK Geotechnics	**Ref No:**	33079SC
Project:	Proposed Housing Renewal	**Report:**	A
Location:	Polding Place, Telopea, NSW	**Report Date:**	1/05/2020
		Page 1 of 2	

BOREHOLE NUMBER	DEPTH m	$I_{S\ (50)}$ MPa	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)
1	7.75 - 7.79	0.8	16
	8.02 - 8.05	1.4	28
	8.19 - 8.21	1.0	20
	8.66 - 8.69	3.3	66
	9.08 - 9.12	3.3	66
	9.50 - 9.54	1.4	28
	9.88 - 9.91	1.5	30
	10.07 - 10.11	2.2	44
	10.47 - 10.50	2.3	46
2	6.40 - 6.43	0.4	8
	6.85 - 6.88	0.2	4
	7.19 - 7.22	0.2	4
	7.70 - 7.73	0.3	6
3	4.67 - 4.70	2.3	46
	4.91 - 4.94	3.0	60
	5.30 - 5.34	2.0	40
	5.87 - 5.91	2.2	44
	6.21 - 6.25	2.2	44
	6.60 - 6.64	1.9	38
	6.92 - 6.96	1.7	34
4	5.96 - 6.00	0.6	12
	6.26 - 6.29	0.9	18
	6.72 - 6.77	0.9	18
5	4.90 - 4.94	1.7	34
	5.28 - 5.32	1.8	36

NOTES: See Page 2 of 2

115 Wicks Road
Macquarie Park NSW 2113
Telephone: 02 9888 5000
Facsimile: 02 9888 5001

SOIL TEST SERVICES

TABLE A
POINT LOAD STRENGTH INDEX TEST REPORT

Client:	JK Geotechnics	**Ref No:**	33079SC
Project:	Proposed Housing Renewal	**Report:**	A
Location:	Polding Place, Telopea, NSW	**Report Date:**	1/05/2020
		Page 2 of 2	

BOREHOLE NUMBER	DEPTH m	$I_{S\ (50)}$ MPa	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)
5	5.82 - 5.87	1.9	38
	6.06 - 6.11	2.6	52
	6.47 - 6.52	2.4	48
	6.80 - 6.84	2.3	46
6	4.13 - 4.15	0.5	10
	4.59 - 4.63	0.5	10
	5.22 - 5.25	0.3	6
	5.97 - 6.00	0.4	8
	6.17 - 6.20	0.7	14
	6.88 - 6.92	0.7	14
7	6.24 - 6.27	0.1	2
	7.32 - 7.35	0.03	1
	7.64 - 7.67	0.6	12
	8.10 - 8.14	2.9	58
	8.57 - 8.61	3.9	78
8	6.50 - 6.54	0.5	10
	6.90 - 6.93	0.2	4
	7.88 - 7.91	1.1	22
	8.07 - 8.11	0.9	18
	8.40 - 8.45	0.7	14

NOTES:

1. In the above table testing was completed in the Axial direction.
2. The above strength tests were completed at the 'as received' moisture content.
3. Test Method: RMS T223.
4. For reporting purposes, the $I_{S(50)}$ has been rounded to the nearest 0.1MPa, or to one significant figure if less than 0.1MPa
5. The Estimated Unconfined Compressive Strength was calculated from the Point Load Strength Index by the following approximate relationship and rounded off to the nearest whole number :
   U.C.S. = 20 $I_{S\ (50)}$

115 Wicks Road
Macquarie Park, NSW 2113
PO Box 976
North Ryde, Bc 1670
Telephone: 02 9888 5000
Facsimile: 02 9888 5001

SOIL TEST SERVICES
ABN 43 002 145 173

TABLE B

MOISTURE CONTENT TEST REPORT

Client: JK Geotechnics
Project: Proposed Housing Renewal
Location: 14 Sturt Street, Telopea, NSW

Ref No: 33079SC
Report: B
Report Date: 28/04/2020
Page 1 of 1

AS 1289	TEST METHOD	2.1.1
BOREHOLE NUMBER	DEPTH m	MOISTURE CONTENT %
1	4.00 - 4.50	16.4
2	2.00 - 2.50	7.0
3	2.50 - 3.00	6.3
4	1.50 - 2.00	8.4
5	2.50 - 3.00	5.2
6	2.50 - 3.00	7.8
7	4.00 - 4.50	6.6
8	2.50 - 3.00	6.7

Notes:

- The test sample for liquid limit was air-dried & dry-sieved
- The linear shrinkage mould was 125mm
- Refer to appropriate notes for soil descriptions
- Date of receipt of sample: 24/04/2020.
- Sampled and supplied by client. Samples tested as received.

NATA

NATA Accredited Laboratory Number:1327

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the items tested or sampled.

28/04/2020
Authorised Signature / Date
(D. Treweek)

Envirolab Services Pty Ltd
ABN 37 112 535 645
12 Ashley St Chatswood NSW 2067
ph 02 9910 6200   fax 02 9910 6201
customerservice@envirolab.com.au
www.envirolab.com.au

CERTIFICATE OF ANALYSIS 241381

Client Details

Client	JK Geotechnics
Attention	Warren Smith
Address	PO Box 976, North Ryde BC, NSW, 1670

Sample Details

Your Reference	**33079SC, Telopea**
Number of Samples	6 Soil
Date samples received	23/04/2020
Date completed instructions received	23/04/2020

Analysis Details

Please refer to the following pages for results, methodology summary and quality control data.

Samples were analysed as received from the client. Results relate specifically to the samples as received.

Results are reported on a dry weight basis for solids and on an as received basis for other matrices.

Please refer to the last page of this report for any comments relating to the results.

Report Details

Date results requested by	30/04/2020
Date of Issue	28/04/2020

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Results Approved By
Priya Samarawickrama, Senior Chemist

Authorised By

Nancy Zhang, Laboratory Manager

Misc Inorg - Soil						
Our Reference		241381-1	241381-2	241381-3	241381-4	241381-5
Your Reference	UNITS	BH2	BH4	BH5	BH7	BH7
Depth		5-5.5	0.5-0.95	3.5-4.0	5.0-5.5	6.3-6.4
Date Sampled		14/04/2020	15/04/2020	16/04/2020	14/04/2020	14/04/2020
Type of sample		Soil	Soil	Soil	Soil	Soil
Date prepared	-	24/04/2020	24/04/2020	24/04/2020	24/04/2020	24/04/2020
Date analysed	-	24/04/2020	24/04/2020	24/04/2020	24/04/2020	24/04/2020
pH 1:5 soil:water	pH Units	5.9	5.4	6.2	5.5	5.5
Chloride, Cl 1:5 soil:water	mg/kg	<10	21	<10	<10	10
Sulphate, SO4 1:5 soil:water	mg/kg	20	170	10	31	29
Resistivity in soil*	ohm m	490	83	620	240	250

Misc Inorg - Soil		
Our Reference		241381-6
Your Reference	UNITS	BH8
Depth		6.0-6.1
Date Sampled		16/04/2020
Type of sample		Soil
Date prepared	-	24/04/2020
Date analysed	-	24/04/2020
pH 1:5 soil:water	pH Units	5.6
Chloride, Cl 1:5 soil:water	mg/kg	<10
Sulphate, SO4 1:5 soil:water	mg/kg	24
Resistivity in soil*	ohm m	270

Method ID	Methodology Summary
Inorg-001	pH - Measured using pH meter and electrode in accordance with APHA latest edition, 4500-H+. Please note that the results for water analyses are indicative only, as analysis outside of the APHA storage times.
Inorg-002	Conductivity and Salinity - measured using a conductivity cell at 25oC in accordance with APHA 22nd ED 2510 and Rayment & Lyons. Resistivity is calculated from Conductivity (non NATA). Resistivity (calculated) may not correlate with results otherwise obtained using Resistivity-Current method, depending on the nature of the soil being analysed.
Inorg-081	Anions - a range of Anions are determined by Ion Chromatography, in accordance with APHA latest edition, 4110-B. Waters samples are filtered on receipt prior to analysis. Alternatively determined by colourimetry/turbidity using Discrete Analyser.

QUALITY CONTROL: Misc Inorg - Soil					Duplicate				Spike Recovery %	
Test Description	Units	PQL	Method	Blank	#	Base	Dup.	RPD	LCS-1	241381-5
Date prepared	-			24/04/2020	2	24/04/2020	24/04/2020		24/04/2020	27/04/2020
Date analysed	-			24/04/2020	2	24/04/2020	24/04/2020		24/04/2020	27/04/2020
pH 1:5 soil:water	pH Units		Inorg-001	[NT]	2	5.4	5.4	0	102	[NT]
Chloride, Cl 1:5 soil:water	mg/kg	10	Inorg-081	<10	2	21	20	5	83	83
Sulphate, SO4 1:5 soil:water	mg/kg	10	Inorg-081	<10	2	170	140	19	85	101
Resistivity in soil*	ohm m	1	Inorg-002	<1	2	83	97	16	[NT]	[NT]

Result Definitions	
NT	Not tested
NA	Test not required
INS	Insufficient sample for this test
PQL	Practical Quantitation Limit
<	Less than
>	Greater than
RPD	Relative Percent Difference
LCS	Laboratory Control Sample
NS	Not specified
NEPM	National Environmental Protection Measure
NR	Not Reported

Quality Control Definitions

Blank	This is the component of the analytical signal which is not derived from the sample but from reagents, glassware etc, can be determined by processing solvents and reagents in exactly the same manner as for samples.
Duplicate	This is the complete duplicate analysis of a sample from the process batch. If possible, the sample selected should be one where the analyte concentration is easily measurable.
Matrix Spike	A portion of the sample is spiked with a known concentration of target analyte. The purpose of the matrix spike is to monitor the performance of the analytical method used and to determine whether matrix interferences exist.
LCS (Laboratory Control Sample)	This comprises either a standard reference material or a control matrix (such as a blank sand or water) fortified with analytes representative of the analyte class. It is simply a check sample.
Surrogate Spike	Surrogates are known additions to each sample, blank, matrix spike and LCS in a batch, of compounds which are similar to the analyte of interest, however are not expected to be found in real samples.

Australian Drinking Water Guidelines recommend that Thermotolerant Coliform, Faecal Enterococci, & E.Coli levels are less than 1cfu/100mL. The recommended maximums are taken from "Australian Drinking Water Guidelines", published by NHMRC & ARMC 2011.

The recommended maximums for analytes in urine are taken from “2018 TLVs and BEIs”, as published by ACGIH (where available). Limit provided for Nickel is a precautionary guideline as per Position Paper prepared by AIOH Exposure Standards Committee, 2016.

Guideline limits for Rinse Water Quality reported as per analytical requirements and specifications of AS 4187, Amdt 2 2019, Table 7.2

Laboratory Acceptance Criteria

Duplicate sample and matrix spike recoveries may not be reported on smaller jobs, however, were analysed at a frequency to meet or exceed NEPM requirements. All samples are tested in batches of 20. The duplicate sample RPD and matrix spike recoveries for the batch were within the laboratory acceptance criteria.

Filters, swabs, wipes, tubes and badges will not have duplicate data as the whole sample is generally extracted during sample extraction.

Spikes for Physical and Aggregate Tests are not applicable.

For VOCs in water samples, three vials are required for duplicate or spike analysis.

Duplicates: >10xPQL - RPD acceptance criteria will vary depending on the analytes and the analytical techniques but is typically in the range 20%-50% – see ELN-P05 QA/QC tables for details; <10xPQL - RPD are higher as the results approach PQL and the estimated measurement uncertainty will statistically increase.

Matrix Spikes, LCS and Surrogate recoveries: Generally 70-130% for inorganics/metals (not SPOCAS); 60-140% for organics/SPOCAS (+/-50% surrogates) and 10-140% for labile SVOCs (including labile surrogates), ultra trace organics and speciated phenols is acceptable.

In circumstances where no duplicate and/or sample spike has been reported at 1 in 10 and/or 1 in 20 samples respectively, the sample volume submitted was insufficient in order to satisfy laboratory QA/QC protocols.

When samples are received where certain analytes are outside of recommended technical holding times (THTs), the analysis has proceeded. Where analytes are on the verge of breaching THTs, every effort will be made to analyse within the THT or as soon as practicable.

Where sampling dates are not provided, Envirolab are not in a position to comment on the validity of the analysis where recommended technical holding times may have been breached.

Measurement Uncertainty estimates are available for most tests upon request.

Analysis of aqueous samples typically involves the extraction/digestion and/or analysis of the liquid phase only (i.e. NOT any settled sediment phase but inclusive of suspended particles if present), unless stipulated on the Envirolab COC and/or by correspondence. Notable exceptions include certain Physical Tests (pH/EC/BOD/COD/Apparent Colour etc.), Solids testing, total recoverable metals and PFAS where solids are included by default.

Samples for Microbiological analysis (not Amoeba forms) received outside of the 2-8°C temperature range do not meet the ideal cooling conditions as stated in AS2031-2012.

Report Comments

pH/EC

Samples were out of the recommended holding time for this analysis except #3 and #6.

JKGeotechnics

BOREHOLE LOG

Borehole No.
BH1
1 / 3

Client: FRASERS PROPERTY AUSTRALIA
Project: PROPOSED HOUSING RENEWAL
Location: 14 POLDING PLACE, TELOPEA, NSW

Job No.: 33079SC **Method:** SPIRAL AUGER **R.L. Surface:** ~56.1 m
Date: 14/4/20 **Datum:** AHD
Plant Type: JK308 **Logged/Checked By:** W.S./T.C.

Groundwater Record	SAMPLES ES	SAMPLES U50	SAMPLES DB	SAMPLES DS	Field Tests	RL (m AHD)	Depth (m)	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
						56			FILL: Silty clay, medium plasticity, dark brown, trace of ash and root fibres.	w<PL			GRASS COVER
				■	N = 5 3,3,2				FILL: Silty clay, medium plasticity, dark brown and red brown, trace of ash, root fibres and fine to medium grained ironstone gravel.	w>PL			APPEARS POORLY COMPACTED
▶						55	1						
				■	N=0 0,0,0			CI	Silty CLAY: medium plasticity, orange brown and red brown, trace of fine to medium grained ironstone gravel.	w~LL	Hd	400 500	RESIDUAL SPT SUNK UNDER SELF WEIGHT
ON 29/4/20 ▽						54	2						
				■	N=SPT 15/ 150mm REFUSAL	53	3	-	Extremely Weathered siltstone: silty CLAY, medium plasticity, light grey, orange brown and red brown, with fine to medium grained ironstone gravel.	XW	Hd		ASHFIELD SHALE
				■		52	4		SILTSTONE: grey, with iron indurated bands.	DW	VL - L		VERY LOW TO LOW 'TC' BIT RESISTANCE
						51	5						
						50	6						

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BOREHOLE LOG

Borehole No.
BH1
2 / 3

Client: FRASERS PROPERTY AUSTRALIA
Project: PROPOSED HOUSING RENEWAL
Location: 14 POLDING PLACE, TELOPEA, NSW

Job No.: 33079SC
Method: SPIRAL AUGER
R.L. Surface: ~56.1 m
Date: 14/4/20
Datum: AHD
Plant Type: JK308
Logged/Checked By: W.S./T.C.

Groundwater Record	SAMPLES				Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
	ES	U50	DB	DS										
ON COMPLETION OF AUGERING				■		49			-	SILTSTONE: grey, with iron indurated bands. *(continued)*	DW	VL - L		
						48 47 46 45 44 43	8 9 10 11 12 13			REFER TO CORED BOREHOLE LOG				GROUNDWATER MONITORING WELL INSTALLED TO 10.65m. CLASS 18 MACHINE SLOTTED 50mm DIA. PVC STANDPIPE 7.65m TO 10.65m. CASING 0.1m TO 7.65m. 2mm SAND FILTER PACK 7m TO 10.65m. BENTONITE SEAL 6.0m TO 7.0m. 2mm SAND FILTER PACK 1.0m TO 6.0m. BENTONITE SEAL 0.1m TO 1.0m. COMPLETED WITH A CONCRETED GATIC COVER.

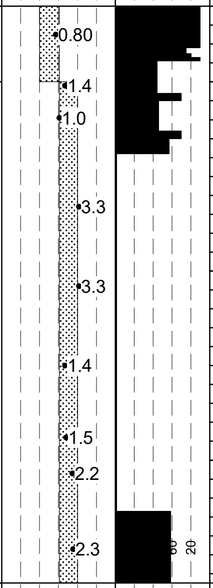
CORED BOREHOLE LOG

Borehole No.
BH1
3 / 3

Client: FRASERS PROPERTY AUSTRALIA
Project: PROPOSED HOUSING RENEWAL
Location: 14 POLDING PLACE, TELOPEA, NSW

Job No.: 33079SC
Core Size: NMLC
R.L. Surface: ~56.1 m
Date: 14/4/20
Inclination: VERTICAL
Datum: AHD
Plant Type: JK308
Bearing: N/A
Logged/Checked By: W.S./T.C.

Water Loss\Level	Barrel Lift	RL (m AHD)	Depth (m)	Graphic Log	CORE DESCRIPTION Rock Type, grain characteristics, colour, texture and fabric, features, inclusions and minor components	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$ VL -0.1, L -0.3, M -1, H -3, VH -10, EH	DEFECT DETAILS SPACING (mm) 600, 200, 60, 20	DEFECT DETAILS DESCRIPTION Type, orientation, defect shape and roughness, defect coatings and seams, openness and thickness Specific	General	Formation
		49			START CORING AT 7.60m							
100% RETURN		48	8		SILTSTONE: grey and brown, with iron indurated bands, bedded up to 15°.	MW	M H	0.80 1.4 1.0		(7.85m) Be, 7°, Un, R, Fe Sn (8.10m) Be, 11°, P, R, Fe Sn		Ashfield Shale
		47 46	9 10		SILTSTONE: light and dark grey, with sandstone laminae, bedded up to 10°.	FR		3.3 3.3 1.4 1.5 2.2 2.3		(10.27m) Be, 7°, P, R, Clay Ct (10.34m) Be, 6°, P, S, Clay Ct		
		45 44 43	11 12 13		END OF BOREHOLE AT 10.65 m							



FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS

JKGeotechnics

Job No: 33097SC
Borehole No: BH1
Depth: 7.60m - 10.65m

BOREHOLE LOG

Borehole No.
BH2
1 / 2

Client: FRASERS PROPERTY AUSTRALIA
Project: PROPOSED HOUSING RENEWAL
Location: 14 POLDING PLACE, TELOPEA, NSW

Job No.: 33079SC **Method:** SPIRAL AUGER **R.L. Surface:** ~58.9 m
Date: 14/4/20 **Datum:** AHD
Plant Type: JK308 **Logged/Checked By:** W.S./T.C.

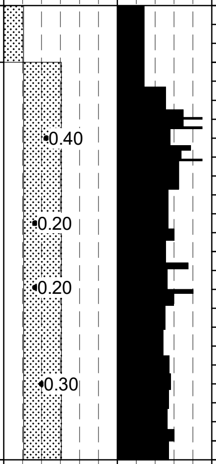
Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	RL (m AHD)	Depth (m)	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
DRY ON COMPLETION OF AUGERING									FILL: Silty clay, medium plasticity, dark brown, trace of root fibres. FILL: Silty clay, medium plasticity, dark brown, trace of medium to coarse grained igneous gravel.	w>PL			GRASS COVER
				■	N = 11 5,4,7	58	1	CH	Silty CLAY: high plasticity, orange brown and red brown, trace of root fibres, ash and fine to medium grained ironstone gravel.	w>PL	Hd	>600 >600	RESIDUAL
				■	N = 22 6,8,14	57	2	-	Extremely Weathered siltstone: silty CLAY, high plasticity, light grey and red brown, trace of fine to medium grained ironstone gravel.	XW	Hd	>600 >600 >600	ASHFIELD SHALE
ON COMPLETION OF CORING ▽				■ ■ ■		56 55 54	3 4 5		SILTSTONE: dark grey and brown, with iron indurated bands.	DW	L - M		LOW 'TC' BIT RESISTANCE WITH MODERATE BANDS
						53 52	6		REFER TO CORED BOREHOLE LOG				

CORED BOREHOLE LOG

Borehole No.
BH2
2 / 2

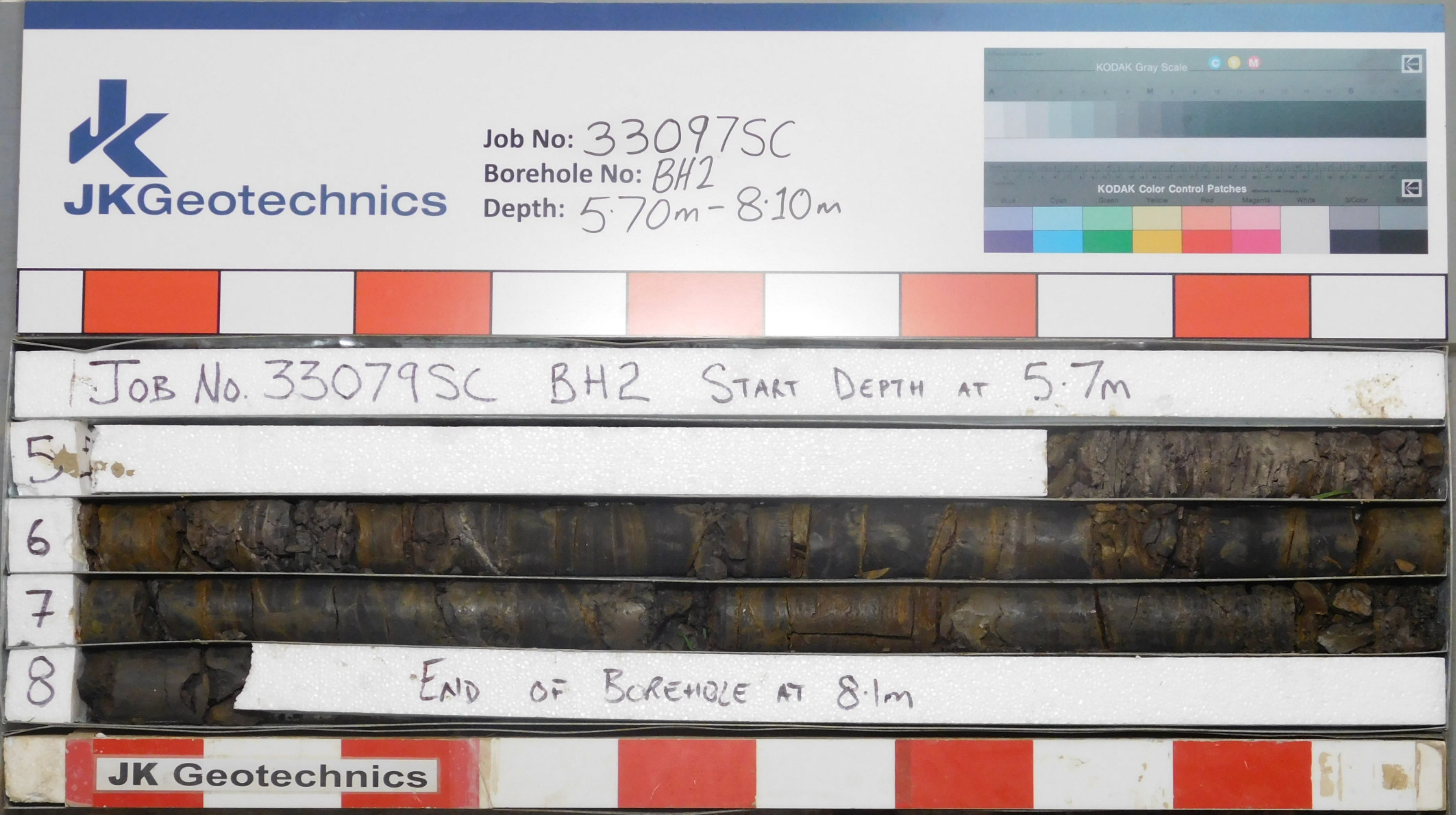
Client: FRASERS PROPERTY AUSTRALIA
Project: PROPOSED HOUSING RENEWAL
Location: 14 POLDING PLACE, TELOPEA, NSW

Job No.: 33079SC
Core Size: NMLC
R.L. Surface: ~58.9 m
Date: 14/4/20
Inclination: VERTICAL
Datum: AHD
Plant Type: JK308
Bearing: N/A
Logged/Checked By: W.S./T.C.



Water Loss\Level	Barrel Lift	RL (m AHD)	Depth (m)	Graphic Log	CORE DESCRIPTION: Rock Type, grain characteristics, colour, texture and fabric, features, inclusions and minor components	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$ (VL -0.1, L -0.3, M -1, H -3, VH -10, EH)	DEFECT DETAILS: SPACING (mm) (600, 200, 60, 20)	DEFECT DETAILS: DESCRIPTION: Type, orientation, defect shape and roughness, defect coatings and seams, openness and thickness — Specific	General	Formation
					START CORING AT 5.70m							
100% RETURN		53 52 51	6 7 8		SILTSTONE: dark grey and brown, with iron indurated bands, bedded up to 15°.	HW MW	VL L - M	0.40 0.20 0.20 0.30		(6.00m) J, 90°, Ir, R, Vn (6.25m) J, 35°, Cu, R, Clay Vn (6.29m) J, 30°, P, R, Clay Ct (6.45-6.48m) Cr, 0°, 30 mm.t (6.51m) Be, 3°, P, R, Fe Sn (6.62m) J, 22°, Un, R, Clay Ct (6.66-6.68m) CS, 0°, 20 mm.t (6.73-6.82m) Cr, 0°, 90 mm.t (6.94m) J, 18°, P, R, Fe Sn (7.25-7.32m) XWS, 0°, 70 mm.t (7.36-7.46m) XWS, 0°, 100 mm.t (7.55m) J, 80 - 90°, Un, R, Fe Sn (7.63-7.66m) XWS, 0°, 30 mm.t (7.73m) Be, 11°, Un, R, Fe Sn (7.83m) J, 32°, Un, R, Fe Sn (7.88-8.00m) Cr, 0°, 120 mm.t (8.00-8.08m) XWS, 0°, 80 mm.t		Ashfield Shale
		50 49 48 47	9 10 11		END OF BOREHOLE AT 8.10 m				600 200 60 20 600 200 60 20			

FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS



JKGeotechnics

Job No: 33097SC

Borehole No: BH2

Depth: 5.70m - 8.10m

BOREHOLE LOG

Borehole No.
BH3
1 / 2

Client: FRASERS PROPERTY AUSTRALIA
Project: PROPOSED HOUSING RENEWAL
Location: 14 POLDING PLACE, TELOPEA, NSW

Job No.: 33079SC
Method: SPIRAL AUGER
R.L. Surface: ~58.7 m
Date: 15/4/20
Datum: AHD
Plant Type: JK308
Logged/Checked By: W.S./T.C.

Groundwater Record	SAMPLES ES	SAMPLES U50	SAMPLES DB	SAMPLES DS	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
DRY ON COMPLETION OF AUGERING								▨	-	FILL: Silty clay, medium plasticity, dark brown, trace of root fibres.	w>PL			GRASS COVER
									-	Extremely Weathered siltstone: silty CLAY, high plasticity, light grey and red brown, trace of fine to medium grained ironstone gravel.	XW	Hd		ASHFIELD SHALE
				■	N=SPT 30/ 150mm REFUSAL	58				SILTSTONE: dark grey and brown, with iron indurated bands.	DW	L - M		LOW 'TC' BIT RESISTANCE WITH MODERATE BANDS
				■		57	1							
							2					M		MODERATE RESISTANCE WITH LOW BANDS
				■		56	3							
				■		55	4							
						54	5			REFER TO CORED BOREHOLE LOG				
						53	6							
						52								

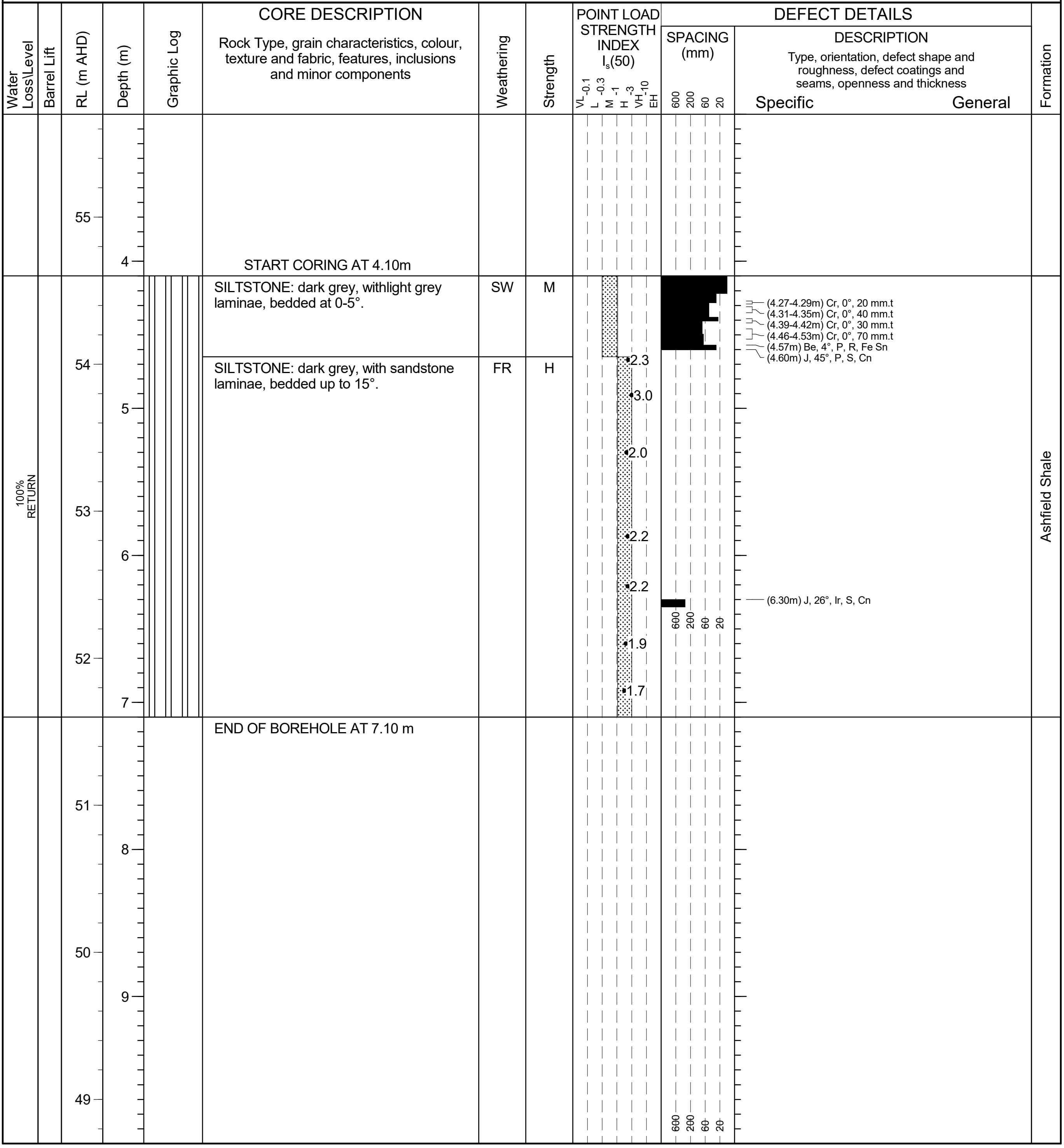
JKGeotechnics

CORED BOREHOLE LOG

Borehole No.
BH3
2 / 2

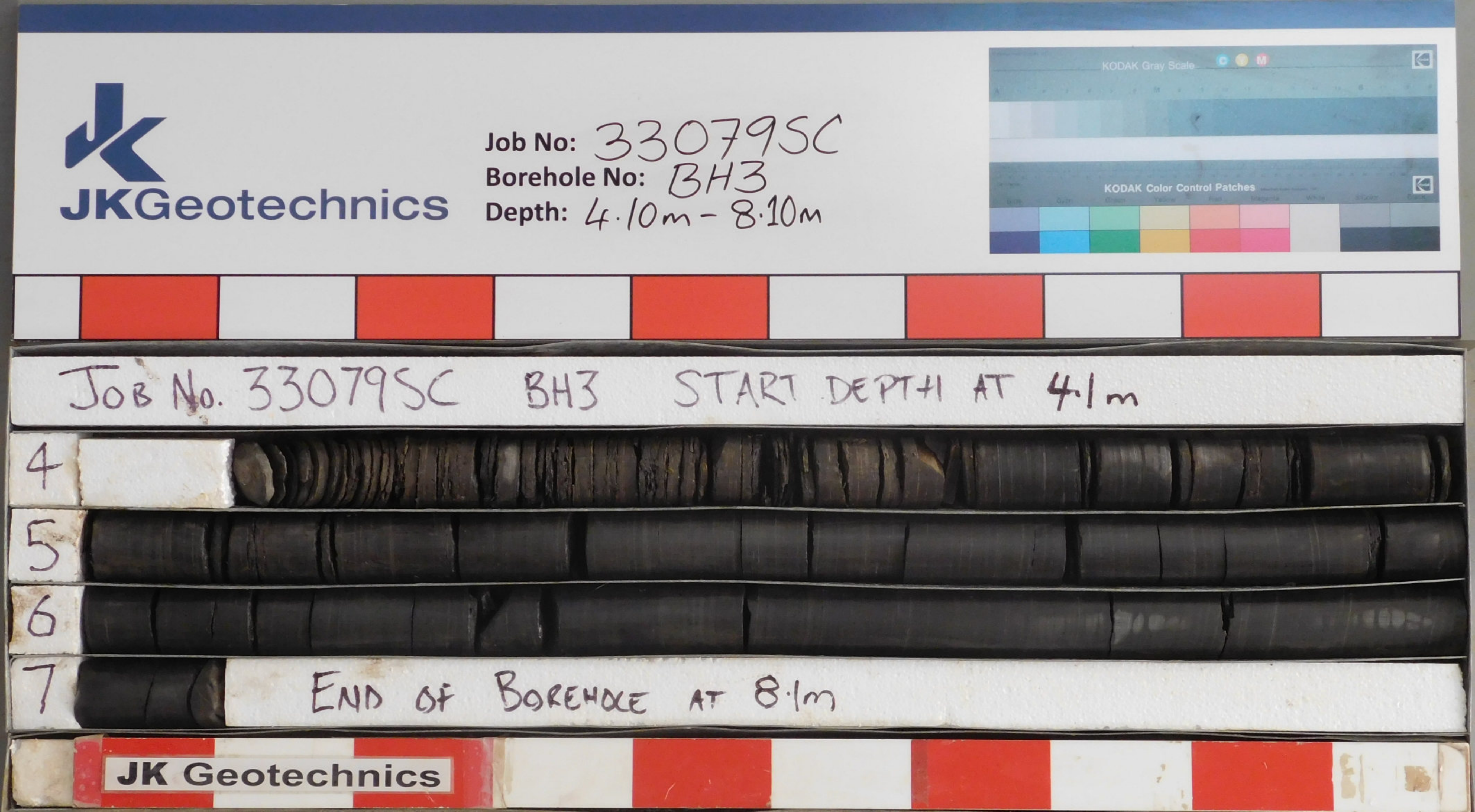
Client: FRASERS PROPERTY AUSTRALIA
Project: PROPOSED HOUSING RENEWAL
Location: 14 POLDING PLACE, TELOPEA, NSW

Job No.: 33079SC
Core Size: NMLC
R.L. Surface: ~58.7 m
Date: 15/4/20
Inclination: VERTICAL
Datum: AHD
Plant Type: JK308
Bearing: N/A
Logged/Checked By: W.S./T.C.



Water Loss\Level	Barrel Lift	RL (m AHD)	Depth (m)	Graphic Log	CORE DESCRIPTION Rock Type, grain characteristics, colour, texture and fabric, features, inclusions and minor components	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$	DEFECT DETAILS DESCRIPTION Type, orientation, defect shape and roughness, defect coatings and seams, openness and thickness Specific	General	Formation
		55	4		START CORING AT 4.10m						
100% RETURN		54			SILTSTONE: dark grey, withlight grey laminae, bedded at 0-5°.	SW	M		(4.27-4.29m) Cr, 0°, 20 mm.t (4.31-4.35m) Cr, 0°, 40 mm.t (4.39-4.42m) Cr, 0°, 30 mm.t (4.46-4.53m) Cr, 0°, 70 mm.t (4.57m) Be, 4°, P, R, Fe Sn (4.60m) J, 45°, P, S, Cn		Ashfield Shale
		53	5 6		SILTSTONE: dark grey, with sandstone laminae, bedded up to 15°.	FR	H	2.3 3.0 2.0 2.2 2.2 1.9 1.7	(6.30m) J, 26°, Ir, S, Cn		
		52	7								
		51 50 49	8 9		END OF BOREHOLE AT 7.10 m						

FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS



JKGeotechnics

Job No: 33079SC
Borehole No: BH3
Depth: 4.10m – 8.10m

JOB No. 33079SC BH3 START DEPTH AT 4.1m

END OF BOREHOLE AT 8.1m

JKGeotechnics

BOREHOLE LOG

Borehole No.
BH4
1 / 2

Client: FRASERS PROPERTY AUSTRALIA
Project: PROPOSED HOUSING RENEWAL
Location: 14 POLDING PLACE, TELOPEA, NSW

Job No.: 33079SC
Method: SPIRAL AUGER
R.L. Surface: ~54.8 m
Date: 15/4/20
Datum: AHD
Plant Type: JK308
Logged/Checked By: W.S./T.C.

Groundwater Record	SAMPLES ES	SAMPLES U50	SAMPLES DB	SAMPLES DS	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
DRY ON COMPLETION OF AUGERING										ASPHALTIC CONCRETE: 200mm.t				
									-	FILL: Silty clay, medium plasticity, dark brown, trace of root fibres and fine to medium grained igneous gravel.	w>PL			
				■	N = 10 4,5,5	54	1		CH	Silty CLAY: high plasticity, orange brown and red brown, trace fine to medium grained ironstone gravel and root fibres.	w~PL	Hd	550 >600 >600	RESIDUAL
				■		53			-	SILTSTONE: dark grey and brown, with iron indurated bands.	DW	L - M		ASHFIELD SHALE LOW 'TC' BIT RESISTANCE WITH MODERATE BANDS
ON 29/4/20				■ ■ ■		52 51	2 3 4					M		MODERATE RESISTANCE
						50 49 48	5 6			REFER TO CORED BOREHOLE LOG				GROUNDWATER MONITORING WELL INSTALLED TO 7.1m. CLASS 18 MACHINE SLOTTED 50mm DIA. PVC STANDPIPE 4.1m TO 7.1m. CASING 0.1m TO 4.1m. 2mm SAND FILTER PACK 3m TO 7.1m. BENTONITE SEAL 2.0m TO 3.0m. 2mm SAND FILTER PACK 1.0m TO 2.0m. BENTONITE SEAL 0.1m TO 1.0m. COMPLETED WITH A CONCRETED GATIC COVER.

JKGeotechnics

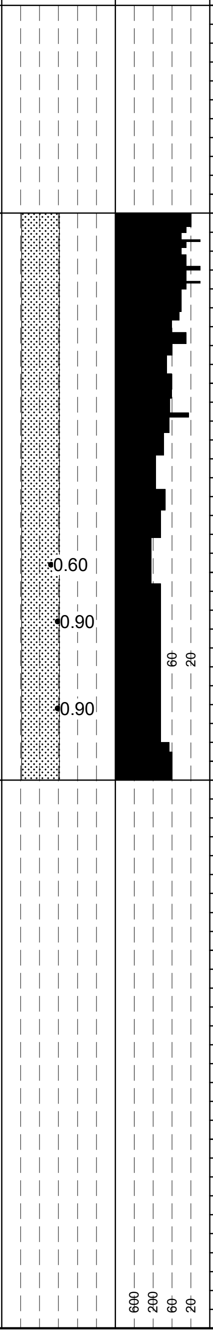
CORED BOREHOLE LOG

Borehole No.
BH4
2 / 2

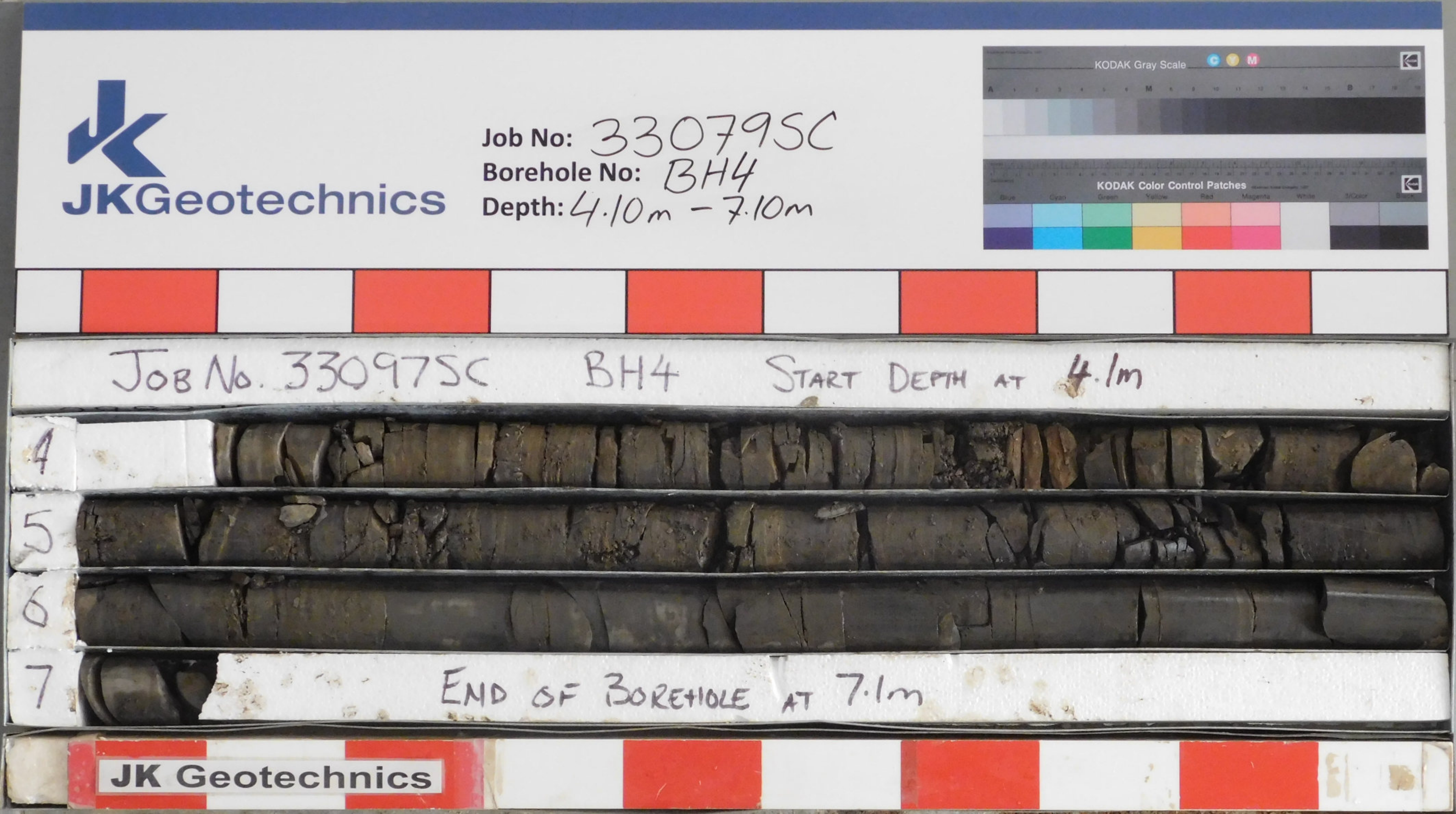
Client: FRASERS PROPERTY AUSTRALIA
Project: PROPOSED HOUSING RENEWAL
Location: 14 POLDING PLACE, TELOPEA, NSW

Job No.: 33079SC
Core Size: NMLC
R.L. Surface: ~54.8 m
Date: 15/4/20
Inclination: VERTICAL
Datum: AHD
Plant Type: JK308
Bearing: N/A
Logged/Checked By: W.S./T.C.

Water Loss\Level	Barrel Lift	RL (m AHD)	Depth (m)	Graphic Log	CORE DESCRIPTION Rock Type, grain characteristics, colour, texture and fabric, features, inclusions and minor components	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$ VL -0.1, L -0.3, M -1, H -3, VH -10, EH	DEFECT DETAILS SPACING (mm) 600 200 60 20	DEFECT DETAILS DESCRIPTION Type, orientation, defect shape and roughness, defect coatings and seams, openness and thickness Specific / General	Formation
ON 29/4/20		51	4		START CORING AT 4.10m						
100% RETURN		50 49 48	5 6 7		SILTSTONE: dark grey and brown, with iron indurated bands, bedded up to 15°.	SW	L - M			(4.20m) J, 80 - 90°, Ir, R, Cn (4.31-4.33m) CS, 0°, 20 mm.t (4.34-4.36m) XWS, 0°, 20 mm.t (4.63-4.70m) XWS, 0°, 70 mm.t (4.73m) Be, 4°, P, S, Fe Sn (4.85m) J, 35°, Ir, R, Cn (4.95m) J, 28°, Ir, R, Fe Sn (5.00-5.03m) XWS, 0°, 30 mm.t (5.15-5.16m) XWS, 0°, 10 mm.t (5.18m) J, 50 - 65°, C, R, Fe Sn (5.22-5.30m) XWS, 0°, 80 mm.t (5.35-5.41m) XWS, 0°, 60 mm.t (5.67m) J, 35°, Ir, R, Cn (5.74-5.89m) J, 85 - 90°, Ir, R, Cn	Ashfield Shale
						FR					
					SILTSTONE: dark grey, bedded up to 5°.			0.60 0.90 0.90	60 20	(6.20m) J, 19°, P, S, Cn (6.37m) J, 23°, P, S, Fe Sn (6.50m) J, 48°, Ir, R, Fe Sn (6.62m) J, 28°, P, S, Cn (6.78m) J, 16°, P, S, Fe Sn (6.90-7.00m) J, 60 - 90°, Ir, R, Fe Sn	
		47 46 45	8 9		END OF BOREHOLE AT 7.10 m				600 200 60 20		



FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS



JKGeotechnics

Job No: 33079SC

Borehole No: BH4

Depth: 4.10m – 7.10m

JKGeotechnics

BOREHOLE LOG

Borehole No.
BH5
1 / 2

Client: FRASERS PROPERTY AUSTRALIA
Project: PROPOSED HOUSING RENEWAL
Location: 14 POLDING PLACE, TELOPEA, NSW

Job No.: 33079SC **Method:** SPIRAL AUGER **R.L. Surface:** ~58.4 m
Date: 16/4/20 **Datum:** AHD
Plant Type: JK308 **Logged/Checked By:** W.S./T.C.

Groundwater Record	SAMPLES ES	SAMPLES U50	SAMPLES DB	SAMPLES DS	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
DRY ON COMPLETION OF AUGERING										FILL: Silty clay, medium plasticity, dark brown, trace of fine to medium grained igneous gravel, ash and root fibres.	w>PL			GRASS COVER
				■	N = 5 4,2,3	58	1		CH	Silty CLAY: high plasticity, orange brown and red brown, trace of root fibres, ash and fine to medium grained ironstone gravel.	w>PL	VSt - Hd	350 400 550	RESIDUAL
				■	N > 18 4,8,10/ 50mm REFUSAL	57			-	Extremely Weathered siltstone: silty CLAY, high plasticity, light grey, orange brown and red brown, trace of fine to medium grained ironstone gravel.	XW	Hd		ASHFIELD SHALE
						56	2			SILTSTONE: dark grey and brown, with iron indurated bands.	DW	L - M		MODERATE 'TC' BIT RESISTANCE WITH LOW BANDS
				■ ■		55	3 4					M		MODERATE RESISTANCE
						54 53 52	5 6			REFER TO CORED BOREHOLE LOG				

CORED BOREHOLE LOG

Borehole No.
BH5
2 / 2

Client: FRASERS PROPERTY AUSTRALIA
Project: PROPOSED HOUSING RENEWAL
Location: 14 POLDING PLACE, TELOPEA, NSW

Job No.: 33079SC
Date: 16/4/20
Plant Type: JK308
Core Size: NMLC
Inclination: VERTICAL
Bearing: N/A
R.L. Surface: ~58.4 m
Datum: AHD
Logged/Checked By: W.S./T.C.

Water Loss\Level	Barrel Lift	RL (m AHD)	Depth (m)	Graphic Log	CORE DESCRIPTION Rock Type, grain characteristics, colour, texture and fabric, features, inclusions and minor components	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$ VL-0.1 L-0.3 M-1 H-3 VH-10 EH	SPACING (mm) 600 200 60 20	DEFECT DETAILS DESCRIPTION Type, orientation, defect shape and roughness, defect coatings and seams, openness and thickness Specific	General	Formation
		55	4		START CORING AT 4.20m							
100% RETURN		54			SILTSTONE: dark grey and brown, bedded up to 10°.	SW	M - H	1.7		(4.24m) Be, 7°, P, R, Fe Sn (4.47m) Cr, 0°, 10 mm.t (4.60m) J, 80 - 90°, Ir, R, Fe Sn		Ashfield Shale
		53 52	5 6		SILTSTONE: dark grey, with sandstone laminae, bedded up to 10°.	SW FR	H	1.8 1.9 2.6 2.4 2.3		(5.36m) Be, 3°, P, S, Fe Sn (5.52m) Be, 3°, P, S, Fe Sn		
		51 50 49	7 8 9		END OF BOREHOLE AT 6.90 m							

FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS

JKGeotechnics

Job No: 33079SC
Borehole No: BH5
Depth: 4.20m – 6.90m

BOREHOLE LOG

Borehole No.
BH6
1 / 2

Client: FRASERS PROPERTY AUSTRALIA
Project: PROPOSED HOUSING RENEWAL
Location: 14 POLDING PLACE, TELOPEA, NSW

Job No.: 33079SC **Method:** SPIRAL AUGER **R.L. Surface:** ~58.5 m
Date: 15/4/20 **Datum:** AHD
Plant Type: JK308 **Logged/Checked By:** W.S./T.C.

Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
DRY ON COMPLETION OF AUGERING									-	FILL: Silty clay, medium plasticity, dark brown, trace of root fibres.	w>PL			GRASS COVER
					N = 13 4,5,8	58	1		CH	Silty CLAY: high plasticity, orange brown and red brown, trace fine to medium grained ironstone gravel and root fibres.	w>PL	Hd		RESIDUAL
					N > 26 8,11,15/ 100mm REFUSAL	57	2		-	Extremely Weathered siltstone: silty CLAY, high plasticity, light grey and red brown, trace of fine to medium grained ironstone gravel and root fibres.	XW	Hd		ASHFIELD SHALE
						56				SILTSTONE: dark grey and brown, with iron indurated bands.	DW	L - M		LOW 'TC' BIT RESISTANCE WITH MODERATE BANDS
						55	3 4					M		MODERATE RESISTANCE WITH LOW BANDS
						54 53 52	5 6			REFER TO CORED BOREHOLE LOG				

CORED BOREHOLE LOG

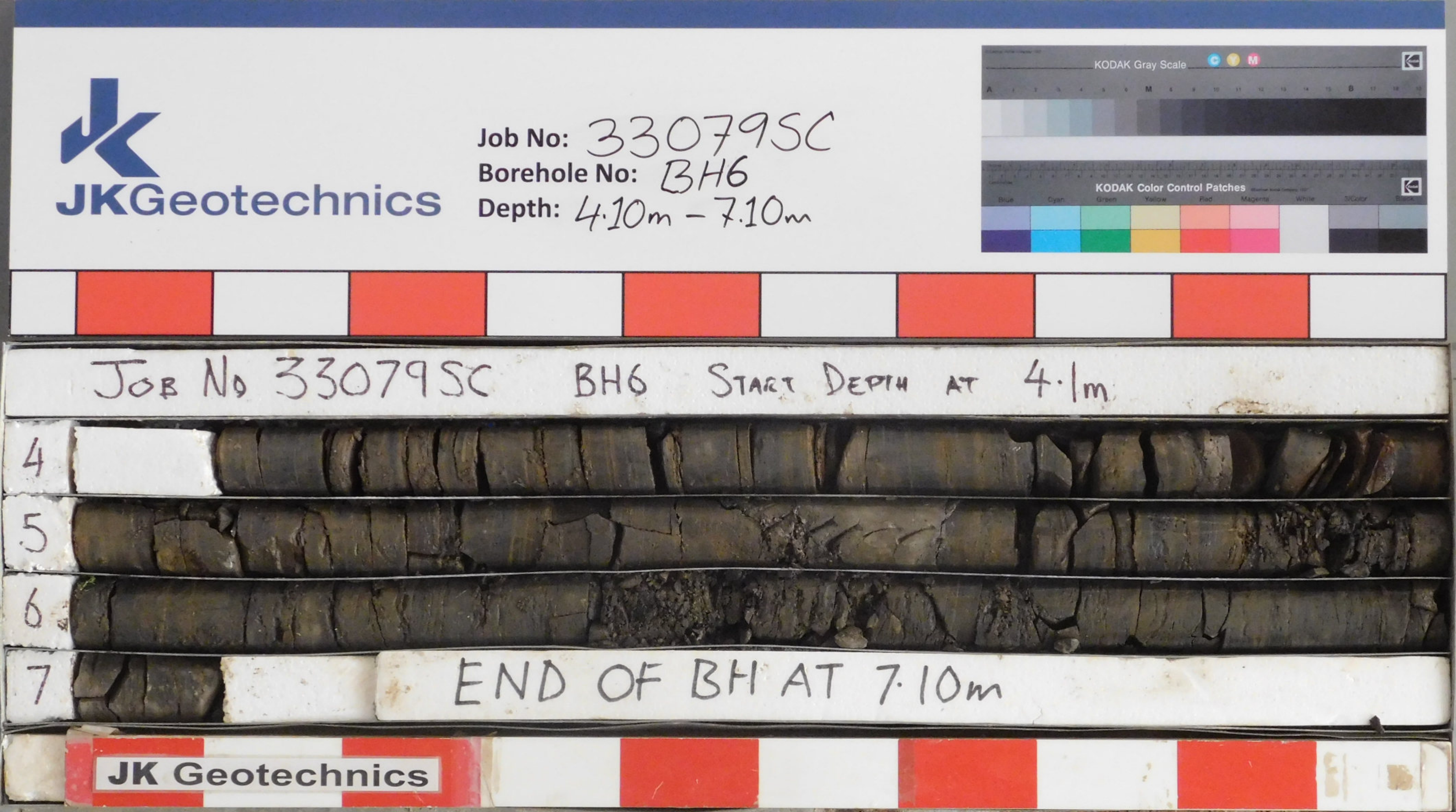
Borehole No.
BH6
2 / 2

Client: FRASERS PROPERTY AUSTRALIA
Project: PROPOSED HOUSING RENEWAL
Location: 14 POLDING PLACE, TELOPEA, NSW

Job No.: 33079SC
Core Size: NMLC
R.L. Surface: ~58.5 m
Date: 15/4/20
Inclination: VERTICAL
Datum: AHD
Plant Type: JK308
Bearing: N/A
Logged/Checked By: W.S./T.C.

Water Loss\Level	Barrel Lift	RL (m AHD)	Depth (m)	Graphic Log	CORE DESCRIPTION Rock Type, grain characteristics, colour, texture and fabric, features, inclusions and minor components	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$ VL -0.1, L -0.3, M -1, H -3, VH -10, EH	DEFECT DETAILS SPACING (mm) 600 200 60 20	DEFECT DETAILS DESCRIPTION Type, orientation, defect shape and roughness, defect coatings and seams, openness and thickness Specific	General	Formation
		55	4		START CORING AT 4.10m							
100% RETURN		54 53 52	5 6 7		SILTSTONE: dark grey and brown, with iron indurated bands, bedded up to 15°.	MW	L-M	0.50 0.50 0.30 0.40 0.70 0.70	600 200 60 20	(4.86m) Be, 3°, P, S, Fe Sn (4.92m) J, 22°, Ir, R, Fe Sn (5.15m) J, 80 - 90°, Ir, S, Vn (5.27-5.29m) XWS, 0°, 20 mm.t (5.40m) J, 81°, Ir, R, Fe Sn (5.50-5.64m) XWS, 0°, 140 mm.t (5.73m) J, 43°, Ir, R, Fe Sn (5.85-5.97m) XWS, 0°, 120 mm.t (6.40-6.60m) XWS, 0°, 200 mm.t (6.63m) J, 32°, Ir, R, Clay Vn (6.66-6.70m) XWS, 0°, 40 mm.t		Ashfield Shale
		51 50 49	8 9		END OF BOREHOLE AT 7.10 m				600 200 60 20			

FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS



JKGeotechnics

Job No: 33079SC
Borehole No: BH6
Depth: 4.10m – 7.10m

JOB NO 33079SC BH6 START DEPTH AT 4.1m

4

5

6

7

END OF BH AT 7.10m

JK Geotechnics

JKGeotechnics

BOREHOLE LOG

Borehole No.
BH7
1 / 2

Client: FRASERS PROPERTY AUSTRALIA
Project: PROPOSED HOUSING RENEWAL
Location: 14 POLDING PLACE, TELOPEA, NSW

Job No.: 33079SC **Method:** SPIRAL AUGER **R.L. Surface:** ~55.1 m
Date: 14/4/20 **Datum:** AHD
Plant Type: JK308 **Logged/Checked By:** W.S./T.C.

Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
				■	N = 7 3,3,4	55		▩		FILL: Silty clay, medium plasticity, dark brown, trace of medium to coarse grained igneous gravel and root fibres.	w>PL			GRASS COVER
				■	N = 8 3,3,5	54 53	1 2	▨	CH	Silty CLAY: high plasticity, orange brown and red brown, trace of root fibres, ash, and fine to medium grained ironstone gravel.	w>PL	Hd	450 450 500 450 470 480	RESIDUAL
				■		52	3	▥	-	Extremely Weathered siltstone: silty CLAY, high plasticity, light grey and red brown, trace of fine to medium grained ironstone gravel.	XW	Hd		ASHFIELD SHALE
ON COMPLETION OF AUGERING				■		51 50	4 5	▥		SILTSTONE: light grey and brown, with iron indurated bands.	DW	VL		VERY LOW 'TC' BIT RESISTANCE
						49	6			REFER TO CORED BOREHOLE LOG				

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CORED BOREHOLE LOG

Borehole No.
BH7
2 / 2

Client: FRASERS PROPERTY AUSTRALIA
Project: PROPOSED HOUSING RENEWAL
Location: 14 POLDING PLACE, TELOPEA, NSW

Job No.: 33079SC
Date: 14/4/20
Plant Type: JK308
Core Size: NMLC
Inclination: VERTICAL
Bearing: N/A
R.L. Surface: ~55.1 m
Datum: AHD
Logged/Checked By: W.S./T.C.

Water Loss\Level	Barrel Lift	RL (m AHD)	Depth (m)	Graphic Log	CORE DESCRIPTION Rock Type, grain characteristics, colour, texture and fabric, features, inclusions and minor components	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$ VL -0.1 L -0.3 M -1 H -3 VH -10 EH	SPACING (mm) 600 200 60 20	DEFECT DETAILS DESCRIPTION Type, orientation, defect shape and roughness, defect coatings and seams, openness and thickness Specific	General	Formation
		50			START CORING AT 5.70m							
100% RETURN		49	6		Extremely Weathered siltstone: silty CLAY, high plasticity, light an dark grey and red brown, trace of fine to medium grained ironstone gravel.	XW	Hd	0.10				Ashfield Shale
		48	7		SILTSTONE: dark grey.	MW	VL - L			(6.50m) Be, 0°, P, S, Fe Sn, 150 mm.t		
					Extremely Weathered siltstone: silty CLAY, high plasticity, light an dark grey and orange brown.	XW	Hd	0.030				
					SILTSTONE: dark grey, bedded up to 15°.	MW	VL	0.60				
					SANDSTONE: fine to medium grained, dark grey, bedded at 5-10°.	SW	M			(7.80m) Be, 7°, Ir, R, Fe Sn		
		47	8			FR	H	2.9 3.9				
		46 45 44	9 10 11		END OF BOREHOLE AT 8.70 m							

FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS

JKGeotechnics

Job No: 33097SC
Borehole No: BH7
Depth: 5·70m - 8·80m

JKGeotechnics

BOREHOLE LOG

Borehole No.
BH8
1 / 2

Client: FRASERS PROPERTY AUSTRALIA
Project: PROPOSED HOUSING RENEWAL
Location: 14 POLDING PLACE, TELOPEA, NSW

Job No.: 33079SC **Method:** SPIRAL AUGER **R.L. Surface:** ~56.6 m
Date: 16/4/20 **Datum:** AHD
Plant Type: JK308 **Logged/Checked By:** W.S./T.C.

Groundwater Record	SAMPLES ES	SAMPLES U50	SAMPLES DB	SAMPLES DS	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
										FILL: Silty clay, medium plasticity, dark brown, trace of medium to coarse grained igneous gravel, ash and root fibres.	w>PL			GRASS COVER
				■	N = 8 3,3,5	56			CH	Silty CLAY: high plasticity, orange brown and red brown, trace of root fibres, ash and fine to medium grained ironstone gravel.	w>PL	Hd	>600 >600 >600	RESIDUAL
				■	N=SPT 15/ 150mm REFUSAL	55	1		-	Extremely Weathered siltstone: silty CLAY, high plasticity, light grey and red brown, trace of fine to medium grained ironstone gravel.	XW	Hd		ASHFIELD SHALE
ON COMPLETION OF CORING				■ ■ ■		54 53 52 51	2 3 4 5			SILTSTONE: dark grey and brown, with iron indurated bands.	HW	L		LOW 'TC' BIT RESISTANCE WITH MODERATE BANDS
						50	6			REFER TO CORED BOREHOLE LOG				

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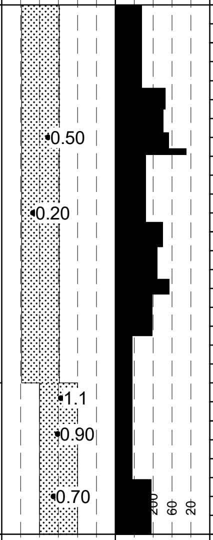
CORED BOREHOLE LOG

Borehole No.
BH8
2 / 2

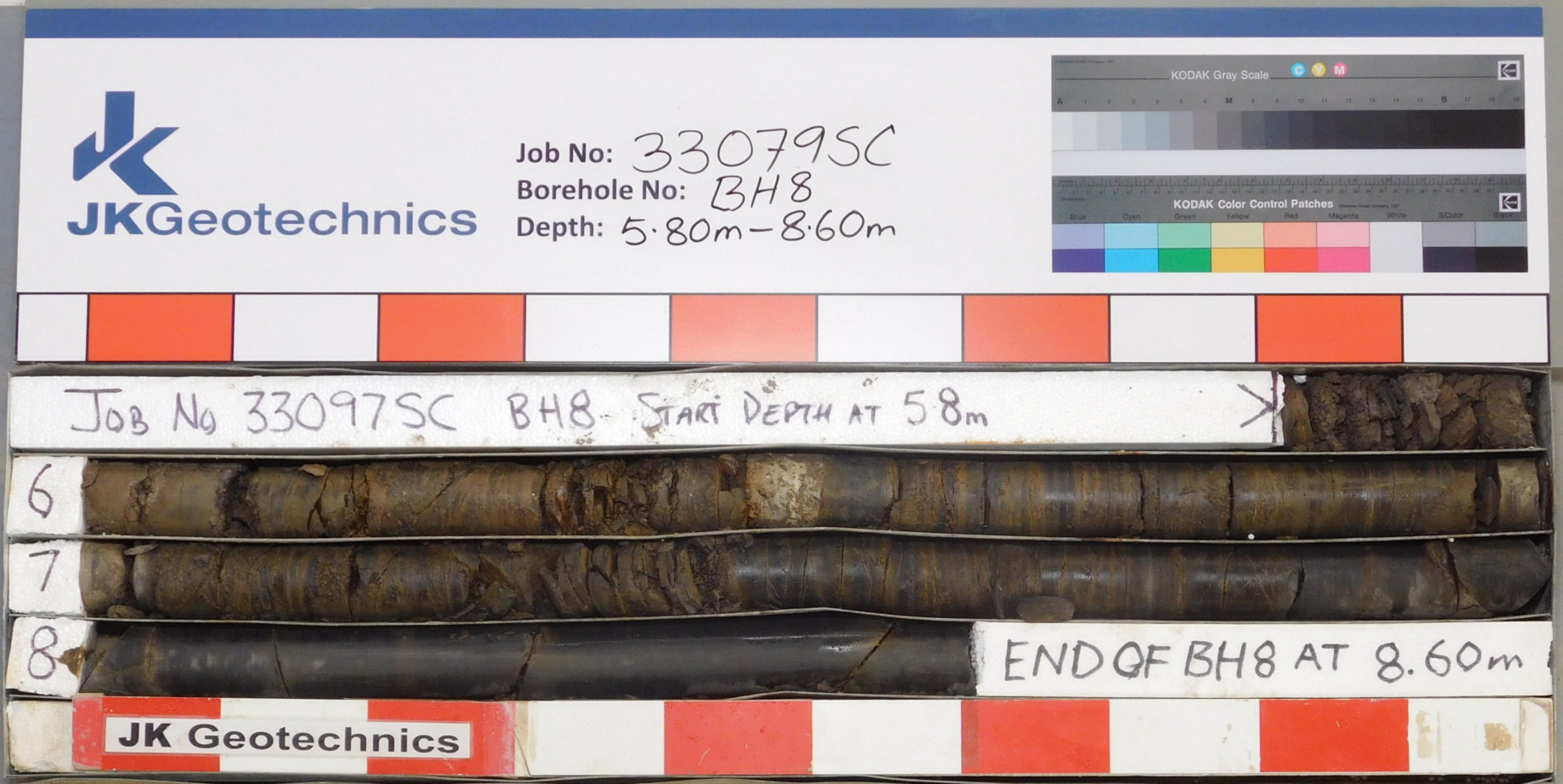
Client: FRASERS PROPERTY AUSTRALIA
Project: PROPOSED HOUSING RENEWAL
Location: 14 POLDING PLACE, TELOPEA, NSW

Job No.: 33079SC
Core Size: NMLC
R.L. Surface: ~56.6 m
Date: 16/4/20
Inclination: VERTICAL
Datum: AHD
Plant Type: JK308
Bearing: N/A
Logged/Checked By: W.S./T.C.



Water Loss\Level	Barrel Lift	RL (m AHD)	Depth (m)	Graphic Log	CORE DESCRIPTION Rock Type, grain characteristics, colour, texture and fabric, inclusions and minor components	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$	DEFECT DETAILS: DESCRIPTION Type, orientation, defect shape and roughness, defect coatings and seams, openness and thickness — Specific	General	Formation
		51			START CORING AT 5.80m						
100% RETURN		50	6		SILTSTONE: dark grey with iron indurated bands, bedded up to 10°.	HW	L - M	0.50 0.20	(5.80-6.12m) XWS, 0°, 320 mm.t (6.24m) J, 45°, P, R, Fe Sn (6.30-6.40m) Cr, 0°, 100 mm.t (6.45-6.50m) XWS, 0°, 50 mm.t		Ashfield Shale
		49	7			MW			(6.56m) Be, 8°, P, R, Fe Sn (6.59m) J, 12°, P, R, Fe Sn (6.95m) Be, 7°, Ir, R, Fe Sn (7.08m) J, 40°, Ir, R, Fe Sn (7.25m) J, 48°, Ir, R, Fe Sn (7.33m) J, 38°, Ir, R, Fe Sn (7.55m) Be, 6°, P, R, Fe Sn		
		48	8		SILTSTONE: dark grey, bedded up to 5°.	FR	M - H	1.1 0.90 0.70	(8.31m) J, 32°, Ir, R, Clay Ct (8.55m) J, 38°, P, R, Clay Ct		
		47 46 45	9 10 11		END OF BOREHOLE AT 8.60 m						

FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS



JKGeotechnics

Job No: 33079SC

Borehole No: BH8

Depth: 5·80m – 8·60m

SOURCE: http://www.whereis.com/

AERIAL IMAGE SOURCE: MAPS.AU.NEARMAP.COM

This plan should be read in conjunction with the JK Geotechnics report.

Title: **SITE LOCATION PLAN**

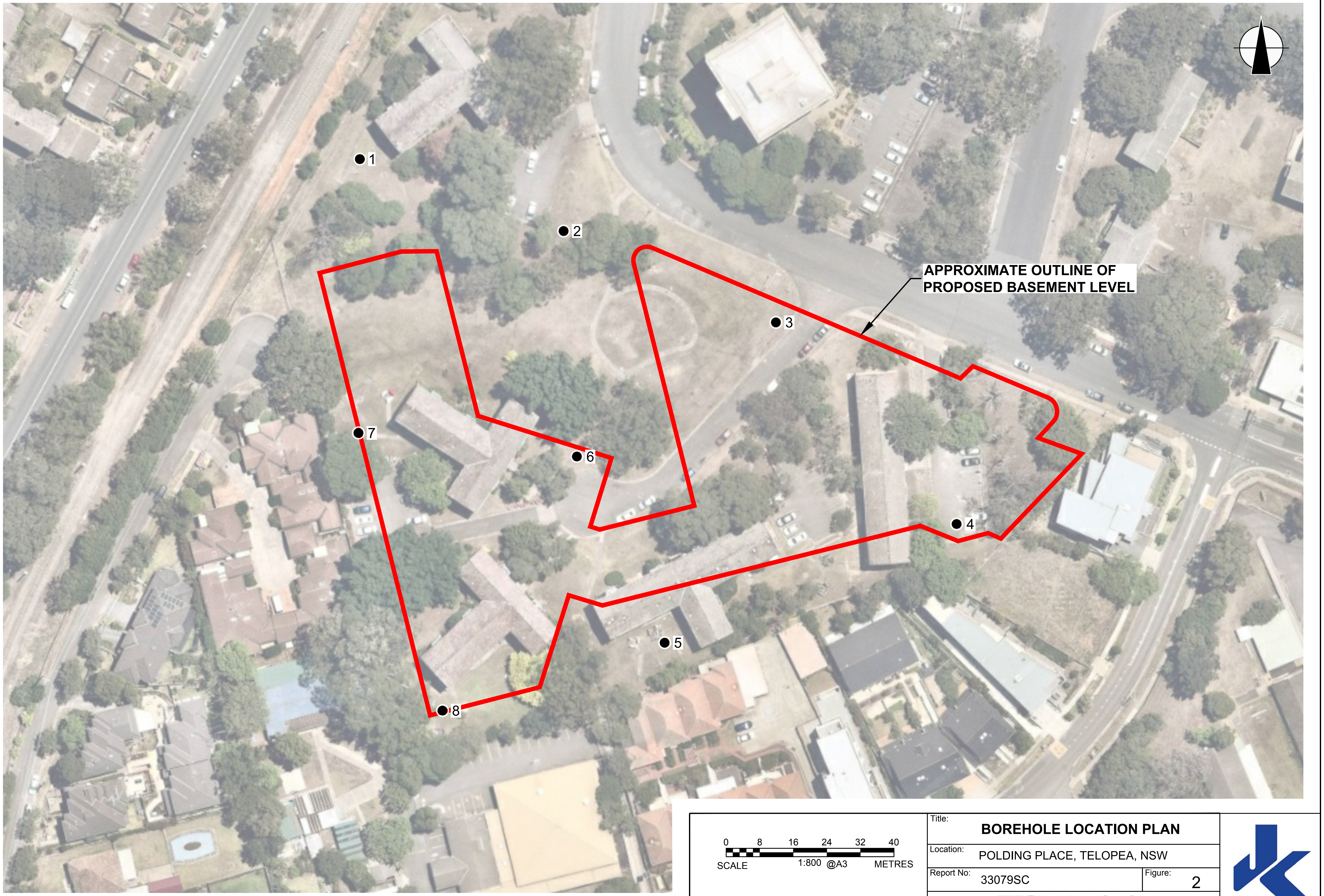
Location: POLDING PLACE, TELOPEA, NSW

Report No: 33079SC

Figure: 1

JKGeotechnics

PLOT DATE: 7/05/2020 3:11:06 PM DWG FILE: Y:\33000'S\33079SC TELOPEA\CAD\33079SC.DWG



This plan should be read in conjunction with the JK Geotechnics report.

Title: **BOREHOLE LOCATION PLAN**

Location: POLDING PLACE, TELOPEA, NSW

Report No: 33079SC

Figure: 2

JKGeotechnics

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VIBRATION EMISSION DESIGN GOALS

German Standard DIN 4150 – Part 3: 1999 provides guideline levels of vibration velocity for evaluating the effects of vibration in structures. The limits presented in this standard are generally recognised to be conservative.

The DIN 4150 values (maximum levels measured in any direction at the foundation, OR, maximum levels measured in (x) or (y) horizontal directions, in the plane of the uppermost floor), are summarised in Table 1 below.

It should be noted that peak vibration velocities higher than the minimum figures in Table 1 for low frequencies may be quite 'safe', depending on the frequency content of the vibration and the actual condition of the structure.

It should also be noted that these levels are 'safe limits', up to which no damage due to vibration effects has been observed for the particular class of building. 'Damage' is defined by DIN 4150 to include even minor non-structural effects such as superficial cracking in cement render, the enlargement of cracks already present, and the separation of partitions or intermediate walls from load bearing walls. Should damage be observed at vibration levels lower than the 'safe limits', then it may be attributed to other causes. DIN 4150 also states that when vibration levels higher than the 'safe limits' are present, it does not necessarily follow that damage will occur. Values given are only a broad guide.

Table 1: DIN 4150 – Structural Damage – Safe Limits for Building Vibration

Group	Type of Structure	Peak Vibration Velocity in mm/s			
		At Foundation Level at a Frequency of:			Plane of Floor of Uppermost Storey
		Less than 10Hz	10Hz to 50Hz	50Hz to 100Hz	All Frequencies
1	Buildings used for commercial purposes, industrial buildings and buildings of similar design.	20	20 to 40	40 to 50	40
2	Dwellings and buildings of similar design and/or use.	5	5 to 15	15 to 20	15
3	Structures that because of their particular sensitivity to vibration, do not correspond to those listed in Group 1 and 2 and have intrinsic value (eg. buildings that are under a preservation order).	3	3 to 8	8 to 10	8

Note: For frequencies above 100Hz, the higher values in the 50Hz to 100Hz column should be used.

Abbreviations Used in Defect Description

Cored Borehole Log Column		Symbol Abbreviation	Description
Point Load Strength Index		• 0.6	Axial point load strength index test result (MPa)
		x 0.6	Diametral point load strength index test result (MPa)
Defect Details	– Type	Be	Parting – bedding or cleavage
		CS	Clay seam
		Cr	Crushed/sheared seam or zone
		J	Joint
		Jh	Healed joint
		Ji	Incipient joint
		XWS	Extremely weathered seam
	– Orientation	Degrees	Defect orientation is measured relative to normal to the core axis (ie. relative to the horizontal for a vertical borehole)
	– Shape	P	Planar
		C	Curved
		Un	Undulating
		St	Stepped
		Ir	Irregular
	– Roughness	Vr	Very rough
		R	Rough
		S	Smooth
		Po	Polished
		Sl	Slickensided
	– Infill Material	Ca	Calcite
		Cb	Carbonaceous
		Clay	Clay
		Fe	Iron
		Qz	Quartz
		Py	Pyrite
	– Coatings	Cn	Clean
		Sn	Stained – no visible coating, surface is discoloured
		Vn	Veneer – visible, too thin to measure, may be patchy
		Ct	Coating ≤ 1mm thick
		Filled	Coating > 1mm thick
	– Thickness	mm.t	Defect thickness measured in millimetres

REPORT EXPLANATION NOTES

INTRODUCTION

These notes have been provided to amplify the geotechnical report in regard to classification methods, field procedures and certain matters relating to the Comments and Recommendations section. Not all notes are necessarily relevant to all reports.

The ground is a product of continuing natural and man-made processes and therefore exhibits a variety of characteristics and properties which vary from place to place and can change with time. Geotechnical engineering involves gathering and assimilating limited facts about these characteristics and properties in order to understand or predict the behaviour of the ground on a particular site under certain conditions. This report may contain such facts obtained by inspection, excavation, probing, sampling, testing or other means of investigation. If so, they are directly relevant only to the ground at the place where and time when the investigation was carried out.

DESCRIPTION AND CLASSIFICATION METHODS

The methods of description and classification of soils and rocks used in this report are based on Australian Standard 1726:2017 *'Geotechnical Site Investigations'*. In general, descriptions cover the following properties – soil or rock type, colour, structure, strength or density, and inclusions. Identification and classification of soil and rock involves judgement and the Company infers accuracy only to the extent that is common in current geotechnical practice.

Soil types are described according to the predominating particle size and behaviour as set out in the attached soil classification table qualified by the grading of other particles present (eg. sandy clay) as set out below:

Soil Classification	Particle Size
Clay	< 0.002mm
Silt	0.002 to 0.075mm
Sand	0.075 to 2.36mm
Gravel	2.36 to 63mm
Cobbles	63 to 200mm
Boulders	> 200mm

Non-cohesive soils are classified on the basis of relative density, generally from the results of Standard Penetration Test (SPT) as below:

Relative Density	SPT 'N' Value (blows/300mm)
Very loose (VL)	< 4
Loose (L)	4 to 10
Medium dense (MD)	10 to 30
Dense (D)	30 to 50
Very Dense (VD)	> 50

Cohesive soils are classified on the basis of strength (consistency) either by use of a hand penetrometer, vane shear, laboratory testing and/or tactile engineering examination. The strength terms are defined as follows.

Classification	Unconfined Compressive Strength (kPa)	Indicative Undrained Shear Strength (kPa)
Very Soft (VS)	≤ 25	≤ 12
Soft (S)	> 25 and ≤ 50	> 12 and ≤ 25
Firm (F)	> 50 and ≤ 100	> 25 and ≤ 50
Stiff (St)	> 100 and ≤ 200	> 50 and ≤ 100
Very Stiff (VSt)	> 200 and ≤ 400	> 100 and ≤ 200
Hard (Hd)	> 400	> 200
Friable (Fr)	Strength not attainable – soil crumbles	

Rock types are classified by their geological names, together with descriptive terms regarding weathering, strength, defects, etc. Where relevant, further information regarding rock classification is given in the text of the report. In the Sydney Basin, 'shale' is used to describe fissile mudstone, with a weakness parallel to bedding. Rocks with alternating inter-laminations of different grain size (eg. siltstone/claystone and siltstone/fine grained sandstone) is referred to as 'laminite'.

SAMPLING

Sampling is carried out during drilling or from other excavations to allow engineering examination (and laboratory testing where required) of the soil or rock.

Disturbed samples taken during drilling provide information on plasticity, grain size, colour, moisture content, minor constituents and, depending upon the degree of disturbance, some information on strength and structure. Bulk samples are similar but of greater volume required for some test procedures.

Undisturbed samples are taken by pushing a thin-walled sample tube, usually 50mm diameter (known as a U50), into the soil and withdrawing it with a sample of the soil contained in a relatively undisturbed state. Such samples yield information on structure and strength, and are necessary for laboratory determination of shrink-swell behaviour, strength and compressibility. Undisturbed sampling is generally effective only in cohesive soils.

Details of the type and method of sampling used are given on the attached logs.

INVESTIGATION METHODS

The following is a brief summary of investigation methods currently adopted by the Company and some comments on their use and application. All methods except test pits, hand auger drilling and portable Dynamic Cone Penetrometers require the use of a mechanical rig which is commonly mounted on a truck chassis or track base.

Test Pits: These are normally excavated with a backhoe or a tracked excavator, allowing close examination of the insitu soils and 'weaker' bedrock if it is safe to descend into the pit. The depth of penetration is limited to about 3m for a backhoe and up to 6m for a large excavator. Limitations of test pits are the problems associated with disturbance and difficulty of reinstatement and the consequent effects on close-by structures. Care must be taken if construction is to be carried out near test pit locations to either properly recompact the backfill during construction or to design and construct the structure so as not to be adversely affected by poorly compacted backfill at the test pit location.

Hand Auger Drilling: A borehole of 50mm to 100mm diameter is advanced by manually operated equipment. Refusal of the hand auger can occur on a variety of materials such as obstructions within any fill, tree roots, hard clay, gravel or ironstone, cobbles and boulders, and does not necessarily indicate rock level.

Continuous Spiral Flight Augers: The borehole is advanced using 75mm to 115mm diameter continuous spiral flight augers, which are withdrawn at intervals to allow sampling and insitu testing. This is a relatively economical means of drilling in clays and in sands above the water table. Samples are returned to the surface by the flights or may be collected after withdrawal of the auger flights, but they can be very disturbed and layers may become mixed. Information from the auger sampling (as distinct from specific sampling by SPTs or undisturbed samples) is of limited reliability due to mixing or softening of samples by groundwater, or uncertainties as to the original depth of the samples. Augering below the groundwater table is of even lesser reliability than augering above the water table.

Rock Augering: Use can be made of a Tungsten Carbide (TC) bit for auger drilling into rock to indicate rock quality and continuity by variation in drilling resistance and from examination of recovered rock cuttings. This method of investigation is quick and relatively inexpensive but provides only an indication of the likely rock strength and predicted values may be in error by a strength order. Where rock strengths may have a significant impact on construction feasibility or costs, then further investigation by means of cored boreholes may be warranted.

Wash Boring: The borehole is usually advanced by a rotary bit, with water being pumped down the drill rods and returned up the annulus, carrying the drill cuttings. Only major changes in stratification can be assessed from the cuttings, together with some information from "feel" and rate of penetration.

Mud Stabilised Drilling: Either Wash Boring or Continuous Core Drilling can use drilling mud as a circulating fluid to stabilise the borehole. The term 'mud' encompasses a range of products ranging from bentonite to polymers. The mud tends to mask the cuttings and reliable identification is only possible from intermittent intact sampling (eg. from SPT and U50 samples) or from rock coring, etc.

Continuous Core Drilling: A continuous core sample is obtained using a diamond tipped core barrel. Provided full core recovery is achieved (which is not always possible in very low strength rocks and granular soils), this technique provides a very reliable (but relatively expensive) method of investigation. In rocks, NMLC or HQ triple tube core barrels, which give a core of about 50mm and 61mm diameter, respectively, is usually used with water flush. The length of core recovered is compared to the length drilled and any length not recovered is shown as NO CORE. The location of NO CORE recovery is determined on site by the supervising engineer; where the location is uncertain, the loss is placed at the bottom of the drill run.

Standard Penetration Tests: Standard Penetration Tests (SPT) are used mainly in non-cohesive soils, but can also be used in cohesive soils, as a means of indicating density or strength and also of obtaining a relatively undisturbed sample. The test procedure is described in Australian Standard 1289.6.3.1–2004 (R2016) *'Methods of Testing Soils for Engineering Purposes, Soil Strength and Consolidation Tests – Determination of the Penetration Resistance of a Soil – Standard Penetration Test (SPT)'*.

The test is carried out in a borehole by driving a 50mm diameter split sample tube with a tapered shoe, under the impact of a 63.5kg hammer with a free fall of 760mm. It is normal for the tube to be driven in three successive 150mm increments and the 'N' value is taken as the number of blows for the last 300mm. In dense sands, very hard clays or weak rock, the full 450mm penetration may not be practicable and the test is discontinued.

The test results are reported in the following form:

- In the case where full penetration is obtained with successive blow counts for each 150mm of, say, 4, 6 and 7 blows, as

  N = 13
  4, 6, 7

- In a case where the test is discontinued short of full penetration, say after 15 blows for the first 150mm and 30 blows for the next 40mm, as

  N > 30
  15, 30/40mm

The results of the test can be related empirically to the engineering properties of the soil.

A modification to the SPT is where the same driving system is used with a solid 60° tipped steel cone of the same diameter as the SPT hollow sampler. The solid cone can be continuously driven for some distance in soft clays or loose sands, or may be used where damage would otherwise occur to the SPT. The results of this Solid Cone Penetration Test (SCPT) are shown as 'N_c' on the borehole logs, together with the number of blows per 150mm penetration.

Cone Penetrometer Testing (CPT) and Interpretation: The cone penetrometer is sometimes referred to as a Dutch Cone. The test is described in Australian Standard 1289.6.5.1–1999 (R2013) *'Methods of Testing Soils for Engineering Purposes, Soil Strength and Consolidation Tests – Determination of the Static Cone Penetration Resistance of a Soil – Field Test using a Mechanical and Electrical Cone or Friction-Cone Penetrometer'*.

In the tests, a 35mm or 44mm diameter rod with a conical tip is pushed continuously into the soil, the reaction being provided by a specially designed truck or rig which is fitted with a hydraulic ram system. Measurements are made of the end bearing resistance on the cone and the frictional resistance on a separate 134mm or 165mm long sleeve, immediately behind the cone. Transducers in the tip of the assembly are electrically connected by wires passing through the centre of the push rods to an amplifier and recorder unit mounted on the control truck. The CPT does not provide soil sample recovery.

As penetration occurs (at a rate of approximately 20mm per second), the information is output as incremental digital records every 10mm. The results given in this report have been plotted from the digital data.

The information provided on the charts comprise:

- Cone resistance – the actual end bearing force divided by the cross sectional area of the cone – expressed in MPa. There are two scales presented for the cone resistance. The lower scale has a range of 0 to 5MPa and the main scale has a range of 0 to 50MPa. For cone resistance values less than 5MPa, the plot will appear on both scales.
- Sleeve friction – the frictional force on the sleeve divided by the surface area – expressed in kPa.
- Friction ratio – the ratio of sleeve friction to cone resistance, expressed as a percentage.

  The ratios of the sleeve resistance to cone resistance will vary with the type of soil encountered, with higher relative friction in clays than in sands. Friction ratios of 1% to 2% are commonly encountered in sands and occasionally very soft clays, rising to 4% to 10% in stiff clays and peats. Soil descriptions based on cone resistance and friction ratios are only inferred and must not be considered as exact.

Correlations between CPT and SPT values can be developed for both sands and clays but may be site specific.

Interpretation of CPT values can be made to empirically derive modulus or compressibility values to allow calculation of foundation settlements.

Stratification can be inferred from the cone and friction traces and from experience and information from nearby boreholes etc. Where shown, this information is presented for general guidance, but must be regarded as interpretive. The test method provides a continuous profile of engineering properties but, where precise information on soil classification is required, direct drilling and sampling may be preferable.

There are limitations when using the CPT in that it may not penetrate obstructions within any fill, thick layers of hard clay and very dense sand, gravel and weathered bedrock. Normally a 'dummy' cone is pushed through fill to protect the equipment. No information is recorded by the 'dummy' probe.

Flat Dilatometer Test: The flat dilatometer (DMT), also known as the Marchetti Dilometer comprises a stainless steel blade having a flat, circular steel membrane mounted flush on one side.

The blade is connected to a control unit at ground surface by a pneumatic-electrical tube running through the insertion rods. A gas tank, connected to the control unit by a pneumatic cable, supplies the gas pressure required to expand the membrane. The control unit is equipped with a pressure regulator, pressure gauges, an audio-visual signal and vent valves.

The blade is advanced into the ground using our CPT rig or one of our drilling rigs, and can be driven into the ground using an SPT hammer. As soon as the blade is in place, the membrane is inflated, and the pressure required to lift the membrane (approximately 0.1mm) is recorded. The pressure then required to lift the centre of the membrane by an additional 1mm is recorded. The membrane is then deflated before pushing to the next depth increment, usually 200mm down. The pressure readings are corrected for membrane stiffness.

The DMT is used to measure material index (I_D), horizontal stress index (K_D), and dilatometer modulus (E_D). Using established correlations, the DMT results can also be used to assess the 'at rest' earth pressure coefficient (K_o), over-consolidation ratio (OCR), undrained shear strength (C_u), friction angle (ϕ), coefficient of consolidation (C_h), coefficient of permeability (K_h), unit weight (γ), and vertical drained constrained modulus (M).

The seismic dilatometer (SDMT) is the combination of the DMT with an add-on seismic module for the measurement of shear wave velocity (V_s). Using established correlations, the SDMT results can also be used to assess the small strain modulus (G_o).

Portable Dynamic Cone Penetrometers: Portable Dynamic Cone Penetrometer (DCP) tests are carried out by driving a 16mm diameter rod with a 20mm diameter cone end with a 9kg hammer dropping 510mm. The test is described in Australian Standard 1289.6.3.2–1997 (R2013) *'Methods of Testing Soils for Engineering Purposes, Soil Strength and Consolidation Tests – Determination of the Penetration Resistance of a Soil – 9kg Dynamic Cone Penetrometer Test'*.

The results are used to assess the relative compaction of fill, the relative density of granular soils, and the strength of cohesive soils. Using established correlations, the DCP test results can also be used to assess California Bearing Ratio (CBR).

Refusal of the DCP can occur on a variety of materials such as obstructions within any fill, tree roots, hard clay, gravel or ironstone, cobbles and boulders, and does not necessarily indicate rock level.

Vane Shear Test: The vane shear test is used to measure the undrained shear strength (C_u) of typically very soft to firm fine grained cohesive soils. The vane shear is normally performed in the bottom of a borehole, but can be completed from surface level, the bottom and sides of test pits, and on recovered undisturbed tube samples (when using a hand vane).

The vane comprises four rectangular blades arranged in the form of a cross on the end of a thin rod, which is coupled to the bottom of a drill rod string when used in a borehole. The size of the vane is dependent on the strength of the fine grained cohesive soils; that is, larger vanes are normally used for very low strength soils. For borehole testing, the size of the vane can be limited by the size of the casing that is used.

For testing inside a borehole, a device is used at the top of the casing, which suspends the vane and rods so that they do not sink under self-weight into the 'soft' soils beyond the depth at which the test is to be carried out. A calibrated torque head is used to rotate the rods and vane and to measure the resistance of the vane to rotation.

With the vane in position, torque is applied to cause rotation of the vane at a constant rate. A rate of 6° per minute is the common rotation rate. Rotation is continued until the soil is sheared and the maximum torque has been recorded. This value is then used to calculate the undrained shear strength. The vane is then rotated rapidly a number of times and the operation repeated until a constant torque reading is obtained. This torque value is used to calculate the remoulded shear strength. Where appropriate, friction on the vane rods is measured and taken into account in the shear strength calculation.

LOGS

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

The terms and symbols used in preparation of the logs are defined in the following pages.

Interpretation of the information shown on the logs, and its application to design and construction, should therefore take into account the spacing of boreholes or test pits, the method of drilling or excavation, the frequency of sampling and testing and the possibility of other than 'straight line' variations between the boreholes or test pits. Subsurface conditions between boreholes or test pits may vary significantly from conditions encountered at the borehole or test pit locations.

GROUNDWATER

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

- Although groundwater may be present, in low permeability soils it may enter the hole slowly or perhaps not at all during the time it 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 and may not be the same at the time of construction.
- 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 be washed out of the hole or 'reverted' chemically if reliable water observations are to be made.

More reliable measurements can be made by installing standpipes which are read after the groundwater level has stabilised at intervals ranging from several days to 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 perched water tables or surface water.

FILL

The presence of fill materials can often be determined only by the inclusion of foreign objects (eg. bricks, steel, etc) or by distinctly unusual colour, texture or fabric. Identification of the extent of fill materials will also depend on investigation methods and frequency. Where natural soils similar to those at the site are used for fill, it may be difficult with limited testing and sampling to reliably assess the extent of the fill.

The presence of fill materials is usually regarded with caution as the possible variation in density, strength and material type is much greater than with natural soil deposits. Consequently, there is an increased risk of adverse engineering characteristics or behaviour. If the volume and quality of fill is of importance to a project, then frequent test pit excavations are preferable to boreholes.

LABORATORY TESTING

Laboratory testing is normally carried out in accordance with Australian Standard 1289 *'Methods of Testing Soils for Engineering Purposes'* or appropriate NSW Government Roads & Maritime Services (RMS) test methods. Details of the test procedure used are given on the individual report forms.

ENGINEERING REPORTS

Engineering reports are prepared by qualified personnel and are based on the information obtained and on current engineering standards of interpretation and analysis. Where the report has been prepared for a specific design proposal (eg. a three storey building) the information and interpretation may not be relevant if the design proposal is changed (eg. to a twenty storey building). If this happens, the Company will be pleased to review the report and the sufficiency of the investigation work.

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

- Unexpected variations in ground conditions – the potential for this will be partially dependent on borehole spacing and sampling frequency as well as investigation technique.
- Changes in policy or interpretation of policy by statutory authorities.
- The actions of persons or contractors responding to commercial pressures.
- Details of the development that the Company could not reasonably be expected to anticipate.

If these occur, the Company will be pleased to assist with investigation or advice to resolve any problems occurring.

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, the Company requests that it immediately be notified. Most problems are much more readily resolved when conditions are exposed rather than at some later stage, well after the event.

REPRODUCTION OF INFORMATION FOR CONTRACTUAL PURPOSES

Where information obtained from this investigation 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. The Company would be pleased to assist in this regard and/or to make additional report copies available for contract purposes at a nominal charge.

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REVIEW OF DESIGN

Where major civil or structural developments are proposed or where only a limited investigation has been completed or where the geotechnical conditions/constraints are quite complex, it is prudent to have a joint design review which involves an experienced geotechnical engineer/engineering geologist.

SITE INSPECTION

The Company will always be pleased to provide engineering inspection services for geotechnical aspects of work to which this report is related.

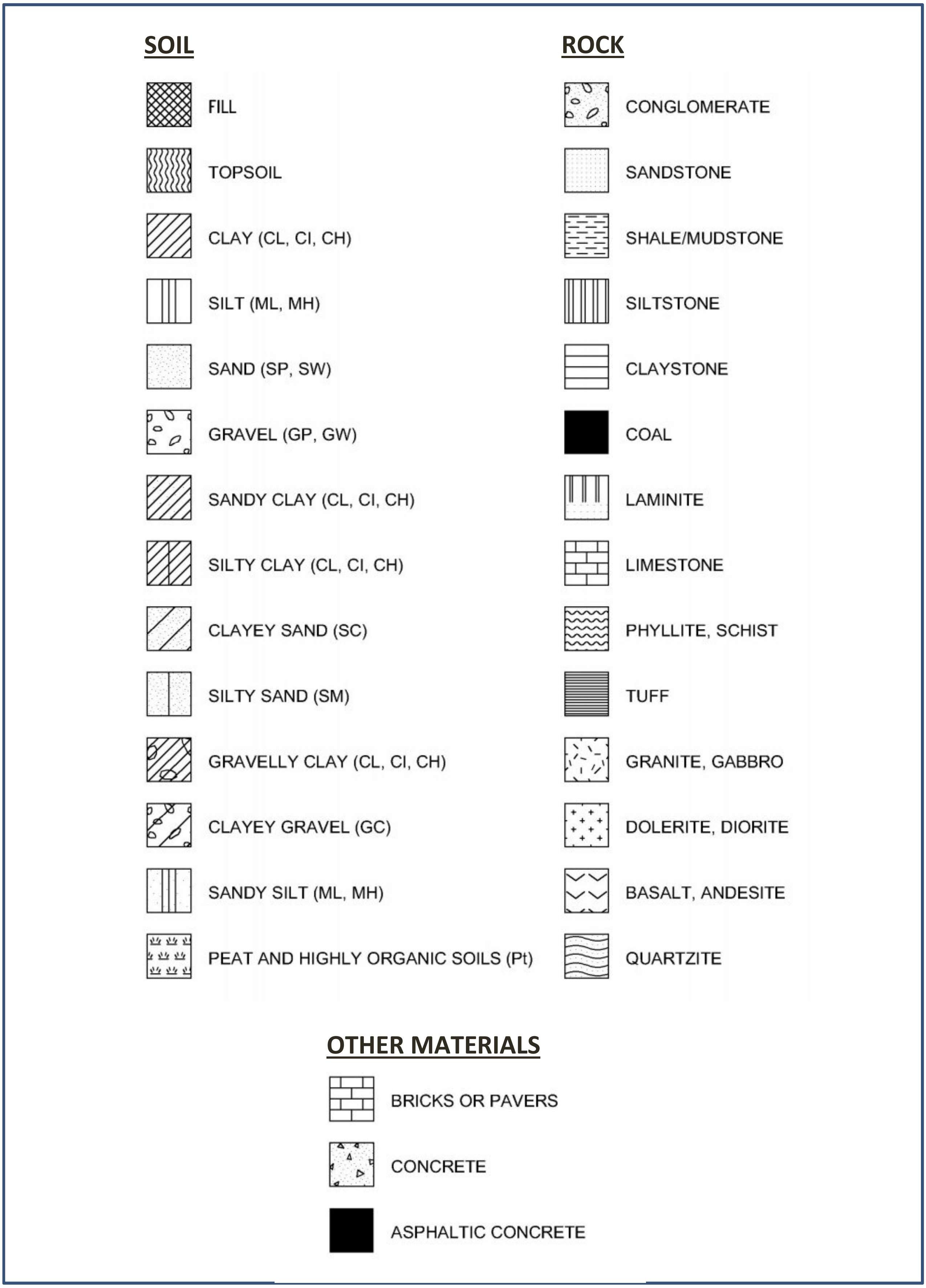
Requirements could range from:

i) a site visit to confirm that conditions exposed are no worse than those interpreted, to

ii) a visit to assist the contractor or other site personnel in identifying various soil/rock types and appropriate footing or pile founding depths, or

iii) full time engineering presence on site.

SYMBOL LEGENDS



CLASSIFICATION OF COARSE AND FINE GRAINED SOILS

Major Divisions		Group Symbol	Typical Names	Field Classification of Sand and Gravel	Laboratory Classification	
Coarse grained soil (more than 65% of soil excluding oversize fraction is greater than 0.075mm)	GRAVEL (more than half of coarse fraction is larger than 2.36mm	GW	Gravel and gravel-sand mixtures, little or no fines	Wide range in grain size and substantial amounts of all intermediate sizes, not enough fines to bind coarse grains, no dry strength	≤ 5% fines	$C_u > 4$ $1 < C_c < 3$
		GP	Gravel and gravel-sand mixtures, little or no fines, uniform gravels	Predominantly one size or range of sizes with some intermediate sizes missing, not enough fines to bind coarse grains, no dry strength	≤ 5% fines	Fails to comply with above
		GM	Gravel-silt mixtures and gravel-sand-silt mixtures	'Dirty' materials with excess of non-plastic fines, zero to medium dry strength	≥ 12% fines, fines are silty	Fines behave as silt
		GC	Gravel-clay mixtures and gravel-sand-clay mixtures	'Dirty' materials with excess of plastic fines, medium to high dry strength	≥ 12% fines, fines are clayey	Fines behave as clay
	SAND (more than half of coarse fraction is smaller than 2.36mm)	SW	Sand and gravel-sand mixtures, little or no fines	Wide range in grain size and substantial amounts of all intermediate sizes, not enough fines to bind coarse grains, no dry strength	≤ 5% fines	$C_u > 6$ $1 < C_c < 3$
		SP	Sand and gravel-sand mixtures, little or no fines	Predominantly one size or range of sizes with some intermediate sizes missing, not enough fines to bind coarse grains, no dry strength	≤ 5% fines	Fails to comply with above
		SM	Sand-silt mixtures	'Dirty' materials with excess of non-plastic fines, zero to medium dry strength	≥ 12% fines, fines are silty	N/A
		SC	Sand-clay mixtures	'Dirty' materials with excess of plastic fines, medium to high dry strength	≥ 12% fines, fines are clayey	

Major Divisions		Group Symbol	Typical Names	Field Classification of Silt and Clay			Laboratory Classification
				Dry Strength	Dilatancy	Toughness	% < 0.075mm
ine grained soils (more than 35% of soil excluding oversize fraction is less than 0.075mm)	SILT and CLAY (low to medium plasticity)	ML	Inorganic silt and very fine sand, rock flour, silty or clayey fine sand or silt with low plasticity	None to low	Slow to rapid	Low	Below A line
		CL, CI	Inorganic clay of low to medium plasticity, gravelly clay, sandy clay	Medium to high	None to slow	Medium	Above A line
		OL	Organic silt	Low to medium	Slow	Low	Below A line
	SILT and CLAY (high plasticity)	MH	Inorganic silt	Low to medium	None to slow	Low to medium	Below A line
		CH	Inorganic clay of high plasticity	High to very high	None	High	Above A line
		OH	Organic clay of medium to high plasticity, organic silt	Medium to high	None to very slow	Low to medium	Below A line
	Highly organic soil	Pt	Peat, highly organic soil	–	–	–	–

Laboratory **Classification** Criteria

A well graded coarse grained soil is one for which the coefficient of uniformity $Cu > 4$ and the coefficient of curvature $1 < C_c < 3$. Otherwise, the soil is poorly graded. These coefficients are given by:

$$C_U = \frac{D_{60}}{D_{10}} \quad and \quad C_C = \frac{(D_{30})^2}{D_{10}\,D_{60}}$$

Where D_{10}, D_{30} and D_{60} are those grain sizes for which 10%, 30% and 60% of the soil grains, respectively, are smaller.

NOTES:

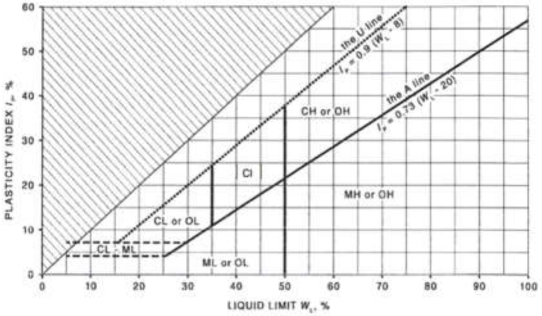
1 For a coarse grained soil with a fines content between 5% and 12%, the soil is given a dual classification comprising the two group symbols separated by a dash; for example, for a poorly graded gravel with between 5% and 12% silt fines, the classification is GP-GM.

2 Where the grading is determined from laboratory tests, it is defined by coefficients of curvature (C_c) and uniformity (C_u) derived from the particle size distribution curve.

3 Clay soils with liquid limits > 35% and ≤ 50% may be classified as being of medium plasticity.

4 The U line on the Modified Casagrande Chart is an approximate upper bound for most natural soils.

Modified Casagrande Chart for Classifying Silts and Clays according to their Behaviour



LOG SYMBOLS

Log Column	Symbol	Definition
Groundwater Record	▼	Standing water level. Time delay following completion of drilling/excavation may be shown.
	C	Extent of borehole/test pit collapse shortly after drilling/excavation.
	▶	Groundwater seepage into borehole or test pit noted during drilling or excavation.
Samples	ES	Sample taken over depth indicated, for environmental analysis.
	U50	Undisturbed 50mm diameter tube sample taken over depth indicated.
	DB	Bulk disturbed sample taken over depth indicated.
	DS	Small disturbed bag sample taken over depth indicated.
	ASB	Soil sample taken over depth indicated, for asbestos analysis.
	ASS	Soil sample taken over depth indicated, for acid sulfate soil analysis.
	SAL	Soil sample taken over depth indicated, for salinity analysis.
Field Tests	N = 17 4, 7, 10	Standard Penetration Test (SPT) performed between depths indicated by lines. Individual figures show blows per 150mm penetration. 'Refusal' refers to apparent hammer refusal within the corresponding 150mm depth increment.
	N_c = 5 7 3R	Solid Cone Penetration Test (SCPT) performed between depths indicated by lines. Individual figures show blows per 150mm penetration for 60° solid cone driven by SPT hammer. 'R' refers to apparent hammer refusal within the corresponding 150mm depth increment.
	VNS = 25	Vane shear reading in kPa of undrained shear strength.
	PID = 100	Photoionisation detector reading in ppm (soil sample headspace test).
Moisture Condition (Fine Grained Soils)	*w* > PL	Moisture content estimated to be greater than plastic limit.
	w ≈ PL	Moisture content estimated to be approximately equal to plastic limit.
	w < PL	Moisture content estimated to be less than plastic limit.
	w ≈ LL	Moisture content estimated to be near liquid limit.
	w > LL	Moisture content estimated to be wet of liquid limit.
(Coarse Grained Soils)	D	DRY – runs freely through fingers.
	M	MOIST – does not run freely but no free water visible on soil surface.
	W	WET – free water visible on soil surface.
Strength (Consistency) Cohesive Soils	VS	VERY SOFT – unconfined compressive strength ≤ 25kPa.
	S	SOFT – unconfined compressive strength > 25kPa and ≤ 50kPa.
	F	FIRM – unconfined compressive strength > 50kPa and ≤ 100kPa.
	St	STIFF – unconfined compressive strength > 100kPa and ≤ 200kPa.
	VSt	VERY STIFF – unconfined compressive strength > 200kPa and ≤ 400kPa.
	Hd	HARD – unconfined compressive strength > 400kPa.
	Fr	FRIABLE – strength not attainable, soil crumbles.
	( )	Bracketed symbol indicates estimated consistency based on tactile examination or other assessment.
Density Index/ Relative Density (Cohesionless Soils)		**Density Index (I_D) Range (%)** **SPT 'N' Value Range (Blows/300mm)**
	VL	VERY LOOSE ≤ 15 0 – 4
	L	LOOSE > 15 and ≤ 35 4 – 10
	MD	MEDIUM DENSE > 35 and ≤ 65 10 – 30
	D	DENSE > 65 and ≤ 85 30 – 50
	VD	VERY DENSE > 85 > 50
	( )	Bracketed symbol indicates estimated density based on ease of drilling or other assessment.
Hand Penetrometer Readings	300 250	Measures reading in kPa of unconfined compressive strength. Numbers indicate individual test results on representative undisturbed material unless noted otherwise.

Log Column	Symbol	Definition
Remarks	'V' bit	Hardened steel 'V' shaped bit.
	'TC' bit	Twin pronged tungsten carbide bit.
	T_{60}	Penetration of auger string in mm under static load of rig applied by drill head hydraulics without rotation of augers.
	Soil Origin	The geological origin of the soil can generally be described as:
		RESIDUAL – soil formed directly from insitu weathering of the underlying rock. No visible structure or fabric of the parent rock.
		EXTREMELY WEATHERED – soil formed directly from insitu weathering of the underlying rock. Material is of soil strength but retains the structure and/or fabric of the parent rock.
		ALLUVIAL – soil deposited by creeks and rivers.
		ESTUARINE – soil deposited in coastal estuaries, including sediments caused by inflowing creeks and rivers, and tidal currents.
		MARINE – soil deposited in a marine environment.
		AEOLIAN – soil carried and deposited by wind.
		COLLUVIAL – soil and rock debris transported downslope by gravity, with or without the assistance of flowing water. Colluvium is usually a thick deposit formed from a landslide. The description 'slopewash' is used for thinner surficial deposits.
		LITTORAL – beach deposited soil.

Classification of Material Weathering

Term		Abbreviation		Definition
Residual Soil		RS		Material is weathered to such an extent that it has soil properties. Mass structure and material texture and fabric of original rock are no longer visible, but the soil has not been significantly transported.
Extremely Weathered		XW		Material is weathered to such an extent that it has soil properties. Mass structure and material texture and fabric of original rock are still visible.
Highly Weathered	Distinctly Weathered (Note 1)	HW	DW	The whole of the rock material is discoloured, usually by iron staining or bleaching to the extent that the colour of the original rock is not recognisable. Rock strength is significantly changed by weathering. Some primary minerals have weathered to clay minerals. Porosity may be increased by leaching, or may be decreased due to deposition of weathering products in pores.
Moderately Weathered		MW		The whole of the rock material is discoloured, usually by iron staining or bleaching to the extent that the colour of the original rock is not recognisable, but shows little or no change of strength from fresh rock.
Slightly Weathered		SW		Rock is partially discoloured with staining or bleaching along joints but shows little or no change of strength from fresh rock.
Fresh		FR		Rock shows no sign of decomposition of individual minerals or colour changes.

NOTE 1: The term 'Distinctly Weathered' is used where it is not practicable to distinguish between 'Highly Weathered' and 'Moderately Weathered' rock. 'Distinctly Weathered' is defined as follows: *'Rock strength usually changed by weathering. The rock may be highly discoloured, usually by iron staining. Porosity may be increased by leaching, or may be decreased due to deposition of weathering products in pores'*. There is some change in rock strength.

Rock Material Strength Classification

Term	Abbreviation	Uniaxial Compressive Strength (MPa)	Guide to Strength	
			Point Load Strength Index Is$_{(50)}$ (MPa)	Field Assessment
Very Low Strength	VL	0.6 to 2	0.03 to 0.1	Material crumbles under firm blows with sharp end of pick; can be peeled with knife; too hard to cut a triaxial sample by hand. Pieces up to 30mm thick can be broken by finger pressure.
Low Strength	L	2 to 6	0.1 to 0.3	Easily scored with a knife; indentations 1mm to 3mm show in the specimen with firm blows of the pick point; has dull sound under hammer. A piece of core 150mm long by 50mm diameter may be broken by hand. Sharp edges of core may be friable and break during handling.
Medium Strength	M	6 to 20	0.3 to 1	Scored with a knife; a piece of core 150mm long by 50mm diameter can be broken by hand with difficulty.
High Strength	H	20 to 60	1 to 3	A piece of core 150mm long by 50mm diameter cannot be broken by hand but can be broken by a pick with a single firm blow; rock rings under hammer.
Very High Strength	VH	60 to 200	3 to 10	Hand specimen breaks with pick after more than one blow; rock rings under hammer.
Extremely High Strength	EH	> 200	> 10	Specimen requires many blows with geological pick to break through intact material; rock rings under hammer.

APPENDIX B

Information Supplied by Robert Bird Group-Emails and Drawings

Thomas Clent

From: Craig FURNESS <Craig.Furness@robertbird.com.au>
Sent: Friday, 14 May 2021 4:20 PM
To: Thomas Clent
Cc: Chris Koukoutaris
Subject: RE: Telopea SEARs-

Where we have temporary batters the permanent wall will be a 250mm thick concrete wall

Craig Furness
PRINCIPAL (CENTRAL REGION)

Level 6, 100 Pacific Highway, North Sydney, NSW, 2060, Australia
Phone: 02 82463200
Mobile: 0427 495 121
Website: www.robertbird.com

RobertBirdGroup
Member of the Surbana Jurong Group

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From: Thomas Clent <TClent@jkgeotechnics.com.au>
Sent: Friday, 14 May 2021 3:42 PM
To: Craig FURNESS <Craig.Furness@robertbird.com.au>
Cc: Chris Koukoutaris <Chris.Koukoutaris@frasersproperty.com.au>
Subject: Re: Telopea SEARs-

This Message Is From an External Sender
This message came from outside your organization.

Thanks Craig,

What type and thickness of basement wall is proposed, where you have the slope batters.

Sent from my iPhone

> On 14/05/2021, at 15:21, Craig FURNESS <Craig.Furness@robertbird.com.au> wrote:
>
> Hi Thomas,
> The anchor loads for the wall are:
> Lower anchor (RL52.0) = 550kN
> Upper anchor (RL54.1) = 310kN

The slab thickness are:
Lower Ground floor (RL 55.4) – 300mm
Basement 1 (RL 51.4) – 200mm
Basement 2 (RL 48.4) – 150mm

If you need any further information, please do not hesitate to contact me.

Regards

Craig Furness
PRINCIPAL (CENTRAL REGION)

Level 6, 100 Pacific Highway, North Sydney, NSW, 2060, Australia
Phone: 02 82463200
Mobile: 0427 495 121
Website: www.robertbird.com

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<image003.png><image004.png><image005.png><image006.png>

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Regards
Thomas Clent
Senior Engineering Geologist

Some of our staff are still working remotely, therefore, where possible, please contact staff on their mobile number.

T: +612 9888 5000
D: 0411 257 270
E: TClent@jkgeotechnics.com.au
www.jkgeotechnics.com.au

PO Box 976
NORTH RYDE BC NSW 1670
115 Wicks Road
MACQUARIE PARK NSW 2113

JKGeotechnics

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From: Chris Koukoutaris <Chris.Koukoutaris@frasersproperty.com.au>
Sent: Friday, 14 May 2021 6:48 AM
To: Craig FURNESS <Craig.Furness@robertbird.com.au>
Cc: 'Thomas Clent' <TClent@jkgeotechnics.com.au>
Subject: FW: Telopea SEARs-

Hi Craig,

Can you please provide proposed floor slab thickness as requested below?

thanks

Chris Koukoutaris
Senior Development Manager
Frasers Property Australia

T +61 2 9767 2223 **M** +61 434 034 371
E Chris.Koukoutaris@frasersproperty.com.au
Level 2, 1C Homebush Bay Drive, Rhodes NSW 2138 Australia
www.frasersproperty.com.au | LinkedIn | YouTube

<image007.png>

From: Thomas Clent <TClent@jkgeotechnics.com.au>
Sent: Friday, 14 May 2021 6:30 AM
To: Chris Koukoutaris <Chris.Koukoutaris@frasersproperty.com.au>
Subject: RE: Telopea SEARs-

EXTERNAL EMAIL: Do not click links or open attachments unless you recognise the sender and know the content is safe.

Further to my email below, could you also ask for the proposed floor slab thicknesses.

Thanks

Regards
Thomas Clent
Senior Engineering Geologist

Some of our staff are still working remotely, therefore, where possible, please contact staff on their mobile number.

<image008.jpg>
T: +612 9888 5000
D: 0411 257 270
E: TClent@jkgeotechnics.com.au
www.jkgeotechnics.com.au
<image009.jpg>

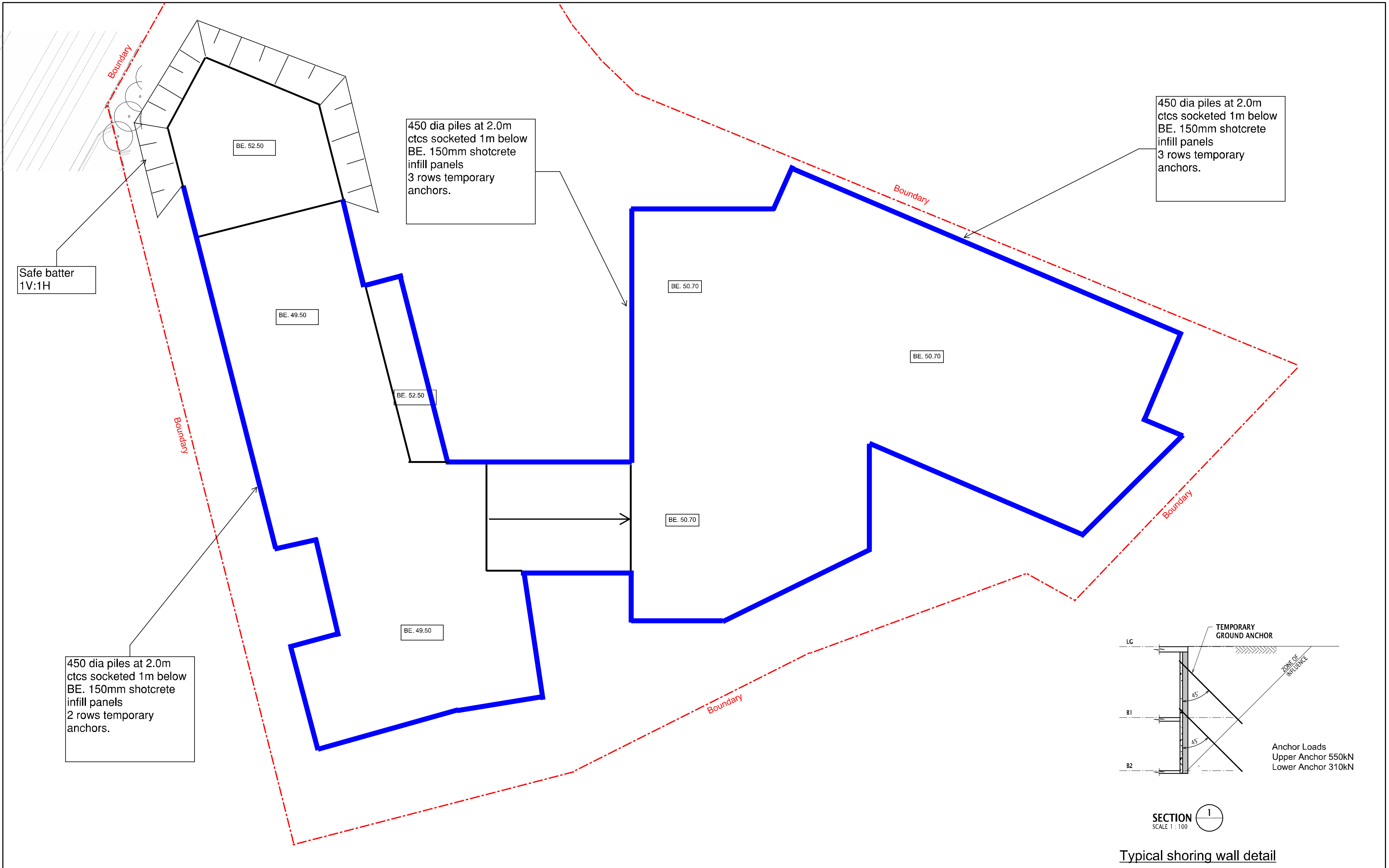
PO Box 976
NORTH RYDE BC NSW 1670
115 Wicks Road
MACQUARIE PARK NSW 2113

-----Original Message-----
From: Thomas Clent <TClent@jkgeotechnics.com.au>
Sent: Thursday, 13 May 2021 5:32 PM
To: Chris Koukoutaris <Chris.Koukoutaris@frasersproperty.com.au>
Subject: Re: Telopea SEARs

Hi Chris,

Would you be able to get the structural engineer to send me the proposed anchor loads for the soldier pile wall? I can't see any on the structural drawings.

Thanks



450 dia piles at 2.0m
ctcs socketed 1m below
BE. 150mm shotcrete
infill panels
3 rows temporary
anchors.

450 dia piles at 2.0m
ctcs socketed 1m below
BE. 150mm shotcrete
infill panels
3 rows temporary
anchors.

Safe batter
1V:1H

450 dia piles at 2.0m
ctcs socketed 1m below
BE. 150mm shotcrete
infill panels
2 rows temporary
anchors.

Anchor Loads
Upper Anchor 550kN
Lower Anchor 310kN

SECTION 1
SCALE 1 : 100

Typical shoring wall detail

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FRASERS PROPERTY

RobertBirdGroup
Member of the Surbana Jurong Group

Rev.	Revision Description	Date
A	For Information	07/07/21

Client: Frasers Property
Project/Job: Telopea Stage 1
Subject: Bulk Earthworks Plan
Designer: C. Furness
Checker:
Approved:
Job No: 20137
Sheet No: SK012
Rev: A

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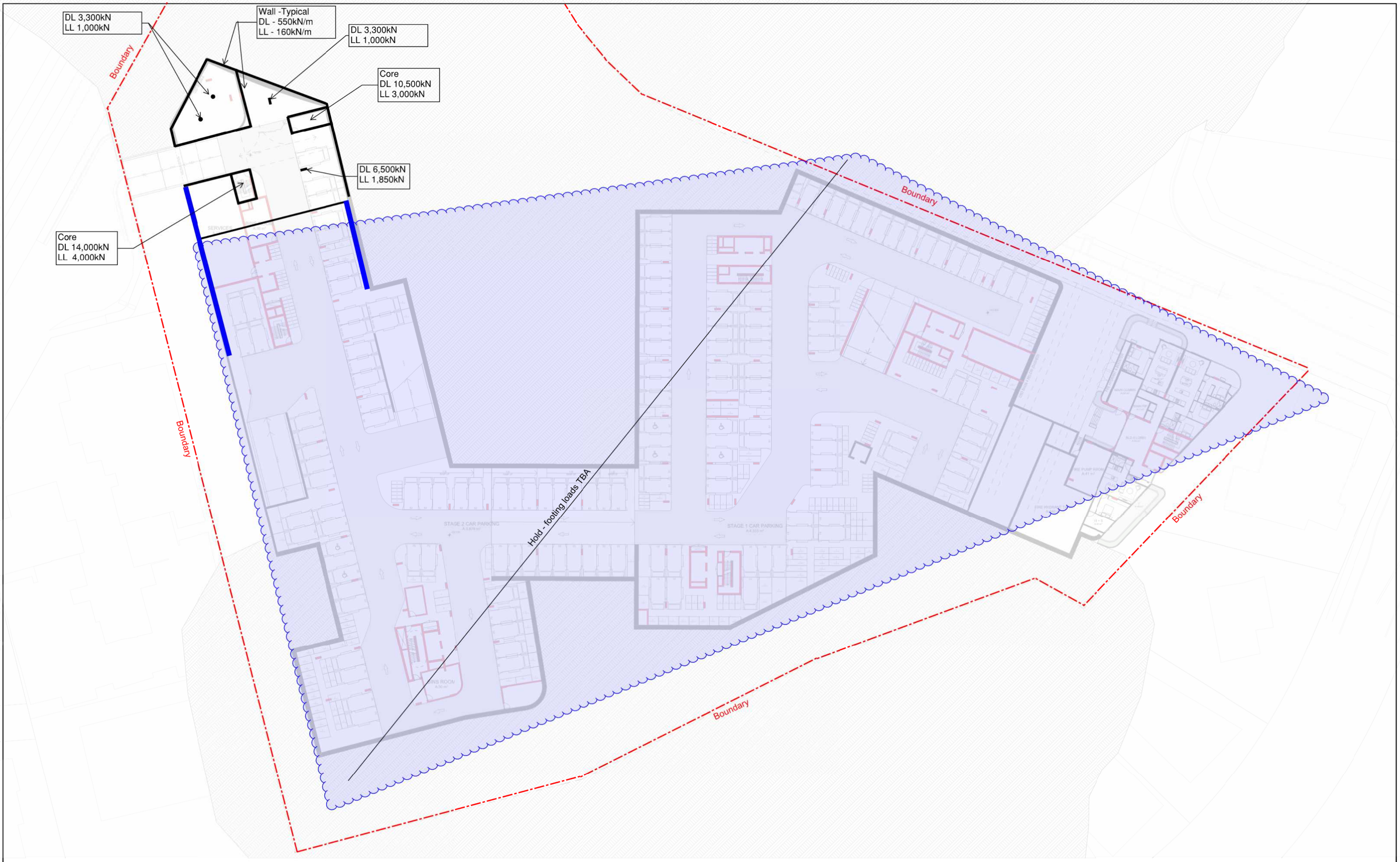

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A	For Information	07/07/21
Rev.	Revision Description	Date

Client: Frasers Property
Project/Job: Telopea Stage 1
Subject: Footing Loads
Designer: C. Furness
Checker:
Approved:
Job No: 20137
Sheet No: SK013
Rev: A

APPENDIX C

Survey Drawing by Craig and Rhodes (Project: 191-19, Amend No. 01, dated 20 February 2020)

AMEND. No.:	01	DATE:	20/02/2020	DETAILS:	FINAL ISSUE
AMEND. No.:	00	DATE:	10/02/2020	DETAILS:	PRELIMINARY ISSUE

DIAL 1100 BEFORE YOU DIG

APPROVED COMPANY AS/NZS 4801 Occupational Health & Safety Management Systems — QMS Certification Services

APPROVED COMPANY ISO 14001 Environmental Management Systems — QMS Certification Services

APPROVED COMPANY ISO 9001 Quality Management Systems — QMS Certification Services

Project:

DETAIL SURVEY OF STAGE 1 TELOPEA URBAN RENEWAL AREA
SHORTLAND STREET AND SURROUNDS, TELOPEA

Principal: FRASERS PROPERTY

Scale 1:200	Date 07/02/2020	Council Ref.	
	Datum A.H.D.	L.G.A. CITY OF PARAMATTA	
Calc's L.E.	Drawn. T.L.	Proj.Man. B.M.	Client Ref:

CRAIG& RHODES
TAKE THE LEAD

ABN 77 050 209 991 ACN 050 209 991
Suite 701, Level 7, 3 Rider Boulevard,
Rhodes, NSW, 2138
PO Box 3220, Rhodes NSW 2138
Tel. 9869-1855
reception@crhodes.com.au
www.craigandrhodes.com.au

Our Ref. 191-19

Dwg File Ref. [Rev] - Sheet Ref.

191-19G T02 [01]
- SHEET 11 OF 15