REPORT TO

ONE GC CHATSWOOD PTY LTD

ON

GEOTECHNICAL ASSESSMENT

FOR

PROPOSED MIXED USE DEVELOPMENT

AT

641-655A PACIFIC HIGHWAY, CHATSWOOD, NSW

Date: 18 March 2022

Ref: 34837LCrpt

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:

Linton Speechley
Principal | 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
34837LCrpt	Final Report	18 March 2022

Table of Contents

1 INTRODUCTION

This report presents the results of a geotechnical desktop assessment for a proposed mixed-use development at 641 to 655 Pacific Highway, Chatswood, NSW. The location of the site is shown in Figure 1. The assessment was commissioned by William Lam of One GC Chatswood Pty Ltd by signed acceptance of proposal form dated 18 February 2022. The works have been carried out in accordance with our proposal dated 14 February 2022, Ref: P55917L.

In order to prepare our geotechnical assessment, we have been provided with unreferenced draft architectural drawings by Architectus within a document dated 20 January 2022. Based on these draft architectural drawings, we understand that the proposed development will include demolition of all existing site structures and then construction of two residential tower blocks with ground floor retail/commercial spaces. Tower 1 in the northern part of the site will be 27 storeys high, and Tower 2 in the southern part of the site will be 25 Storeys high. Both tower blocks will have ground floor retail/commercial spaces. At this stage four basement car parking levels are proposed, and for the purposes of this desktop report we have assumed excavation to a depth of about 12m will be required.

The purpose of our assessment is to review the available geotechnical information for the site and for nearby sites to assess the likely subsurface conditions, and provide our preliminary comments and recommendations on geotechnical issues for the proposed development to assist with planning and preliminary concept design.

2 ASSESSMENT PROCEDURE

The assessment comprised an inspection of the site and its immediate surrounds by our Senior Engineering Geologist, Mr Thomas Clent, on 3 March 2022. Observations made during the inspection are summarised in Section 3.1.

A search of our project database was also carried out to find previous geotechnical investigations carried out on nearby sites to assess the likely subsurface conditions. The results of our previous investigations are summarised in Section 3.2.

3 RESULTS OF INVESTIGATION

3.1 Site Description

The subject site at 641 to 655 Pacific Highway is located in an area of gently sloping to relatively level terrain, with an overall slope of approximately 3° down to the north. The site has street frontages with Pacific Highway and Gordon Avenue on the western and southern site boundaries, respectively.

At the time of the walkover visit, the site contained two residential buildings. The northern residential building comprised a two-storey brick apartment building over one level of basement car parking. The basement car park level is approximately 1.8m below the surrounding pavement level. An undecroft car park structure is present along the eastern side of the building with terraced garden areas above. Raised garden beds and lawn areas are present on the western side of the building retained by low height (up to about 0.7m high) brick walls.

The southern building comprises a three-storey brick apartment building with a ground floor undercroft car park level. Garden areas are present on all sides of the building, containing grass and small to medium sized bushes and trees. Both residential buildings appear to be in good condition, based on our cursory inspection.

A concrete surfaced laneway, connecting to Gordon Avenue, is present along the eastern side of the site. The southern portion of the laneway is outside the site boundaries and the northern portion of the laneway is within the site boundaries (Hammond Lane). This is shown on the attached Figure 1.

As mentioned above the site has street frontages on the southern and western site boundaries. On the western Pacific Highway site boundary, the surface levels within the site are slightly above the adjacent footpaths (0.5m to 0.75m) where the garden areas have been raised up. These raised areas are supported by brick retaining walls on the site boundaries. Gordon Avenue has a similar ground level to the subject site along the southern boundary.

The southern portion of the eastern boundary contains a concrete paved laneway, the laneway is approximately 6m wide with a three-storey residential building further to the east. The neighbouring residential building has at least one basement level. Further north along the eastern site boundary, is a single storey brick building (club house) and bowling green. Both the bowling green and club house have a similar surface level which is approximately 1m above the subject site. The neighbouring site is retained by a brick wall. On the eastern side of the bowling green is the Chatswood Railway line approximately 60m east of the subject site.

To the north of the site is a two-storey residence with similar ground surface levels to the subject site.

3.2 Previous Nearby JK Geotechnics Investigations

Reference to the Sydney 1:100 000 Geological Series Sheet indicates that the site is mapped to be underlain by Ashfield Shale of the Wianamatta Group.

We have completed geotechnical investigations close to the subject site and the results of these previous investigations are summarised below.

4-6 Eddy Road

An investigation was carried out in 1988 within a property to the west of the subject site. The boreholes drilled encountered sandy fill to about 0.1m depth overlying residual silty clay with siltstone bedrock (shale bedrock) at depths ranging from about 2.8m to 5.6m. The clayey soils were assessed to be of stiff to hard strength. The siltstone bedrock was initially extremely weathered of hard (soil) strength improving to highly weathered and of very low strength at depths ranging from about 4.5m to 8.5m. Groundwater was found to be in the order of 1.5m below surface levels during the limited monitoring period.

1 to 9 Moriarty Road

1 to 9 Moriarty Road is located approximately 100m to the south west of the subject site. The boreholes drilled for that investigation encountered granular fill over residual silty clay with siltstone bedrock (shale) at depths ranging from 6.4m to 7.5m. The residual silty clays were assessed to be medium to high plasticity and of very stiff to hard strength. The siltstone bedrock was assessed to be extremely weathered and of hard (soil) strength improving to very low to low strength with depth. Groundwater levels ranging from approximately 3.1m to 7m below the existing grade were recorded over monitoring periods of 6.5 hours.

654 to 666 Pacific Highway

JK Geotechnics were engaged to complete a geotechnical investigation for a residential development at a site approximately 100m north-west of the subject site. The boreholes drilled encountered gravelly and clayey fill to depths of about 0.5m. Residual silty clay was encountered below the fill to depths ranging from 0.9m to 2.7m with siltstone bedrock (shale) at depths ranging from about 1.3m to 3m. The siltstone bedrock profile was generally extremely weathered of hard (soil) strength improving to distinctly weathered and very low to low strength at about 5m depth. Medium to high strength shale was encountered below depths of about 10m. Groundwater levels were recorded at depths ranging from 1.8m to 9m during the limited monitoring period.

4 COMMENTS AND RECOMMENDATIONS

4.1 Inferred Subsurface Conditions

Based on the results of the previous geotechnical investigations close to the subject site, we expect that the subsurface conditions below the site will comprise predominantly clayey soils over siltstone bedrock at depths ranging from 3m to 5m depth. We expect the siltstone bedrock to be initially extremely weathered and of hard (soil) strength grading to very low strength and improving with depth to low strength and then medium to high strength at depths below about 10m.

Groundwater is expected to be encountered at the soil/bedrock interface, possibly at relatively shallow depths of about 3m.

The above inferred subsurface profile may be used for planning purposes, but will need to be confirmed to allow detailed design. A detailed geotechnical investigation of the site must be carried out to determine the actual subsurface conditions and allow detailed design. The final scope of the geotechnical investigation should be determined once the final layout of the proposed buildings are known so the borehole locations can be targeted to suit the building layout.

Due to the expected size of the buildings all boreholes should involve the core drilling of the bedrock in order to optimise bearing pressures for the design of footings.

4.2 Geotechnical Issues

Based on the above inferred subsurface profile the main geotechnical issues for the proposed development are detailed below. Overall, we consider that the site is geotechnically suitable for the proposed development and will be comparable to other similar developments constructed within nearby properties.

The comments and recommendations provided herein are preliminary and should only be used for planning and concept design purposes. The comments and recommendations will need to be confirmed as part of the detailed geotechnical investigation of the site.

4.2.1 Dilapidation Surveys

Prior to the start of excavation, dilapidation surveys should be carried out on the adjoining properties to the north and east. Council and RMS may also require dilapidation surveys of their assets within the adjacent footpaths and roadways. The dilapidation surveys should comprise a detailed inspection of the adjoining properties and existing buildings, both externally and internally, with all defects rigorously described, i.e. defect location, defect type, crack width, crack length, etc. The 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 report will help to guard against opportunistic claims for damage that was present prior to the start of excavation.

4.2.2 Excavation

Excavation to the proposed depths of about 12m is expected to encounter residual silty clay soils and siltstone bedrock (shale) of up to medium to high strength.

Excavation of the fill and residual soils, as well as any extremely weathered siltstone, will be readily achievable using conventional excavation equipment, such as the buckets of hydraulic excavators. Where very low strength siltstone is encountered it will require larger excavators with tiger teeth or ripping tynes, or a Dozer (say D7 size) with ripping tynes, where space permits for the economical use of such equipment.

Low, medium and high strength siltstone bedrock will require the use of rock excavation techniques, such as hydraulic impact hammers, rock saws and/or rock grinders. High strength siltstone will present ‘very hard’ rock excavation conditions.

During the use of hydraulic impact hammers, precautions must be made to reduce the risk of vibrational damage to adjoining structures. This will include quantitative monitoring of transmitted vibrations during rock hammer operation. Vibration monitoring should be carried out by an experienced vibration consultant or geotechnical engineer to check that vibrations are within acceptable limits.

If during excavation with hydraulic impact hammers, vibrations are found to be excessive or there is concern, then alternative lower vibration emitting equipment, such as rock saws, rock grinders or smaller hammers may need to be used. The use of a rotary grinder or rock sawing in conjunction with ripping presents an alternative low vibration excavation technique, however, productivity is likely to be slower. When using a rock saw or rotary grinder, the resulting dust must be suppressed by spraying with water.

We recommend that only excavation contractors with appropriate insurances and experience on similar projects be used.

The excavated material will need to be disposed off site and therefore will need to be suitably classified for waste disposal.

4.2.3 Groundwater

We expect groundwater seepage will occur at the soil/rock interface and through defects within the rock, and will increase during and immediately following rainfall periods. During construction we expect that any groundwater seepage will be able to be controlled by conventional sump and pump techniques.

Recent Information released by WaterNSW, '*Minimum Requirements for Building Site Groundwater Investigation and Reporting'* will require additional groundwater investigations and monitoring to assess likely groundwater inflows. Based on the scale of the basement, it is likely that Water NSW will not accept a drained basement in the long term and the basement will need to be a tanked structure with the basement designed for hydrostatic uplift pressures.

Information on groundwater levels will need to be obtained as part of the site specific subsurface geotechnical investigations. In this regard as the development includes substantial basement levels, in accordance with WaterNSW's *Minimum Requirements for Building Site Groundwater Investigation and Reporting'* additional groundwater investigations and monitoring will be required. That document requires at least three groundwater monitoring wells to be installed (one of which will need to extend down to a depth of at least 25m). Insitu permeability testing of the underlying strata, above and below the bulk excavation level, will be required so that an assessment of groundwater inflows during construction can be made. Groundwater monitoring will also be required for a minimum period of three months within a six month period immediately prior to the application for construction dewatering with Water NSW.

4.2.4 Retention

Since the basement may extend up to or close to the site boundaries, a full depth retention system will need to be installed prior to the start of excavation.

Such a retention systems may comprise soldier pile retaining walls with shotcrete infill panels, provided clayey soils are encountered that can stand vertically between the piles to allow placement of shotcrete. If deeper granular soils are encountered contiguous pile walls would be required. This would be further assessed as part of the detailed geotechnical subsurface investigations. Based on the expected subsurface conditions, piles will need to be socketed below the basement excavation level.

Retaining systems will require additional lateral support in the form of external anchors or internal props, which must be installed progressively as each restraining point is uncovered. Where anchors extend below adjoining properties permission will need to be obtained from the owners of the adjoining properties before installation of the anchors. The use of anchors below adjoining properties may also require easements and this should be investigated at an early stage of design planning and development. Anchors below the pacific Highway is unlikely to be approved by RMS. The presence of any buried services or adjoining basement levels outside of the basement excavation must also be considered in assessing the feasibility of external anchors.

Propped or anchored retaining walls may be provisionally designed based on a trapezoidal earth pressure distribution of magnitude 6H kPa (where H is the retained height in metres) where some resulting ground movements are tolerable and existing structures are located beyond a horizontal distance of 2H from the wall. Where movements are to be kept low and structures are located within a horizontal distance of 2H from the wall, a trapezoidal earth pressure distribution of 8H kPa should be used. These lateral pressures should be held constant for the central 50% of the pressure distribution. Design of retaining systems will also need to consider the potential for large continuous defects in the shale bedrock. These defects can be inclined at 40^{o} to 70^{o} and are often clay lined. The loads from such a sliding wedge will need to be considered in the retaining system design.

The above coefficients and lateral pressures assume horizontal backfill surfaces and where inclined backfill is proposed the coefficients/pressures would need to be increased or the inclined backfill taken as a surcharge load. All surcharge loads must 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 of at least low strength, with the bond formed beyond a line drawn up at 45° from the base of the excavation. Preliminary design of anchors may be based on an allowable bond stress of 100kPa for rock of very low strength or 200kPa for rock of low strength. All anchors should be proof loaded to at least 1.3 times the design working load before locking off at about 80% of the 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. Anchors are generally carried out on a design and construct basis so that failure of the anchors to hold their test load does not become a contractual issue.

Retaining systems for this site will be substantial structures and will require detailed geotechnical and structural design. Shoring designs using finite element software such as Plaxis will be required for all walls to assess the impact of the shoring wall design on their properties. This will particularly be required by Transport for NSW for the shoring system adjacent to Pacific Highway.

4.2.5 Footings

We anticipate siltstone bedrock of at least low strength to be exposed within the basement bulk excavation.

Allowable bearing pressures within the siltstone bedrock would start at 1000kPa for rock of low strength, increasing to 3500kPa for medium strength rock without significant defects. The drilling of cored boreholes will be required to allow the use of these higher bearing pressures, which should be carried out as part of the detailed geotechnical subsurface investigations. Advice should be obtained from the structural engineer on the likely bearing pressure required so that the boreholes can be extended to the appropriate quality rock during the geotechnical investigation.

4.2.6 Basement Floor

We expect weathered siltstone bedrock to be exposed at the bulk excavation level. 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 subgrade and the slab to provide a uniform base for the slab.

If a drained basement is found to be feasible and approved by Water NSW, 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 control pumps to avoid flooding.

However as mentioned in the previous sections, further groundwater investigations and monitoring will be required to satisfy Water NSW requirements. In our opinion and based on our recent experience with Water NSW, we consider that Water NSW is unlikely to provide approval for a drained basement and the basement will need to be a tanked structure designed for hydrostatic uplift pressures.

4.2.7 Geotechnical Subsurface Investigations

Geotechnical subsurface investigations will be required as part of the detailed design stages of the project and also as part of the Water NSW approval process. Investigations will include;

- At least 8 cored boreholes to depths in the order of 20m to 25m.
- Installation of a minimum of three, (but preferably more) groundwater monitoring wells with instrumented data loggers. Groundwater monitoring will need to be carried out for a period of at least three months prior to application for temporary construction dewatering.
- Boreholes will need insitu permeability testing (such as packer testing) to measure the permeability of the rock materials above and below the bulk excavation level.

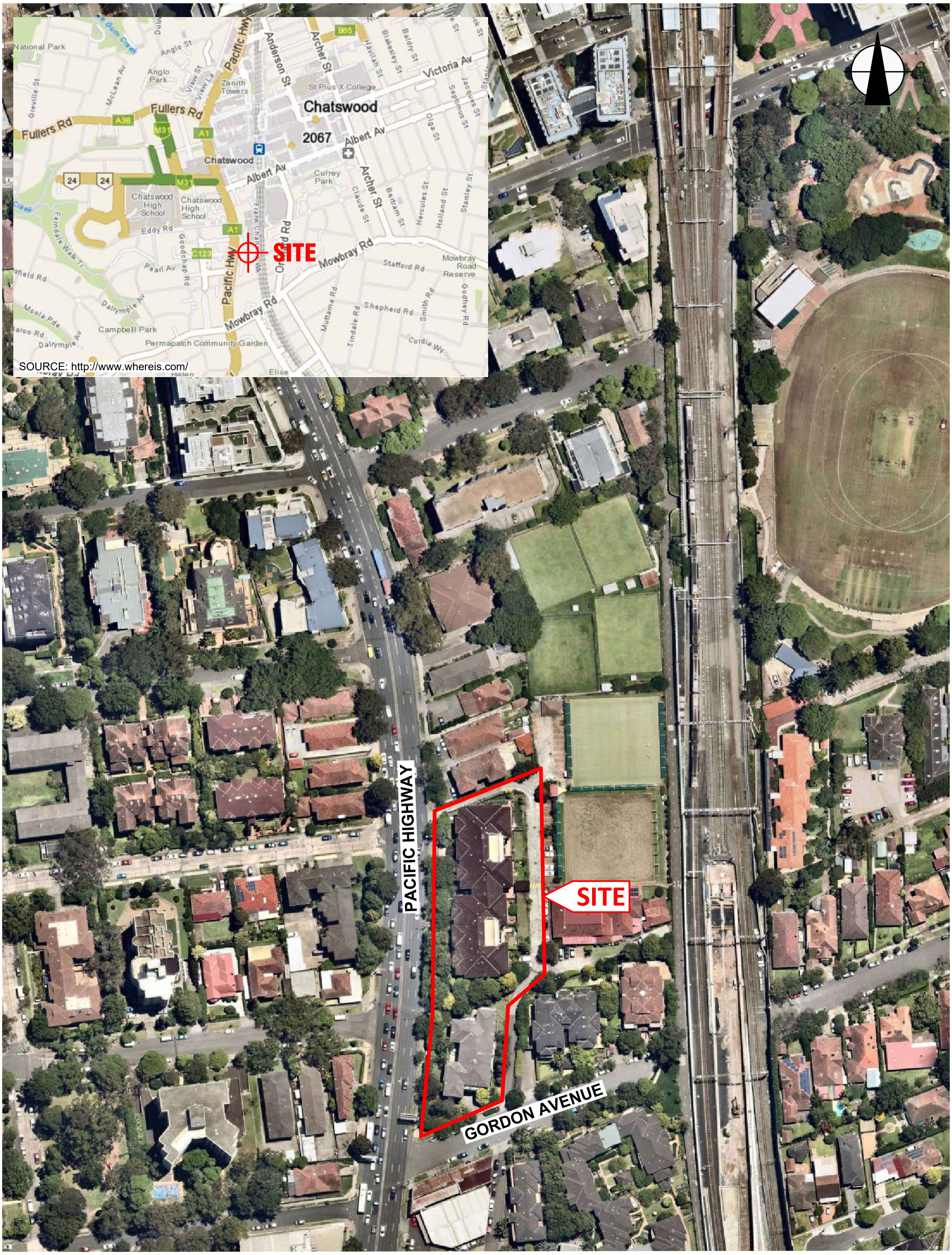
5 GENERAL COMMENTS

The preliminary recommendations presented in this report are based on an inferred subsurface profile based on previous geotechnical investigations carried out on nearby sites. A site specific geotechnical investigation will be required. The comments and recommendations provided herein must be confirmed and amplified as part of the detailed geotechnical investigation.

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. If the natural soil has been stockpiled, classification of this soil as Excavated Natural Material (ENM) can also be undertaken, if requested. However, the criteria for ENM are more stringent and the cost associated with attempting to meet these criteria may be significant. 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.



AERIAL IMAGE SOURCE: MAPS.AU.NEARMAP.COM	Title: **SITE LOCATION PLAN**	
	Location: 641-655a Pacific Highway, Chatswood, NSW	
	Report No: 34837LC	Figure No: 1
This plan should be read in conjunction with the JK Geotechnics report.	**JKGeotechnics**	

PLOT DATE: 17/03/2022 9:23:18 AM DWG FILE: J:\6F GEOTECHNICAL JOBS\34000'S\34837L CHATSWOOD\CAD\34837LC.DWG