REPORT TO

BRIDGE42

ON

GEOTECHNICAL ASSESSMENT

FOR

PROPOSED RESIDENTIAL DEVELOPMENT

AT

22-24 RAGLAN STREET, MANLY, NSW

Date: 25 November 2022

Ref: 35612SFrpt

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DOCUMENT REVISION RECORD

Report Reference	Report Status	Report Date
35612SFrpt	Final Report	25 November 2022

Table of Contents

1 INTRODUCTION

This report presents the results of a geotechnical desktop assessment for a proposed residential development at 22 to 24 Raglan Street, Manly, NSW. A site location plan is presented as Figure 1. The assessment was commissioned by Mr Guillaume Gauthier of Bridge42 by email dated 7 November 2022 on behalf of Leftfield Group. The assessment was carried out in accordance with our proposal, Ref: P57639YF, dated 31 October 2022.

From review of the architectural drawings prepared by Carlisle Architects (Job No. 21-02, Dwg. DA-02, Rev 5 dated 10 November 2022), we understand the development includes the following:

- Demolition of existing site structure
- Construction of a single basement parking level with finished floor level at approximately RL2.87mAHD. The basement will extend to the eastern, southern and western boundaries however will be set back about 2.4m from the northern boundary. Excavation to about 3.7m depth below existing surface levels will be required to achieve bulk excavation level assuming at least a 0.5m thick basement slab.
- Construction of a ground floor level plus three residential stories above.

The purpose of the assessment was to obtain geotechnical information on likely subsurface conditions as a basis for comments and recommendations on excavation, retention, groundwater, footings, slabs on grade and site specific geotechnical investigation which will be required for detailed design following the DA stage.

2 ASSESSMENT PROCEDURE

The assessment involved the following procedure:

- A desk top study of our nearby geotechnical investigations,
- Review of the published information including geological maps
- A walkover of the site and surrounds by our Associate Geotechnical Engineer on 11 November 2022.

No subsurface investigations were carried out as part of this assessment.

3 RESULTS OF INVESTIGATION

3.1 Site Description

The site is in a relatively flat, low lying coastal plan area about 100m east of the toe of the hillside (down which Raglan Street runs) and is about 200m west of Manly Beach.

The site is approximately rectangular shaped with dimensions of about 20m by 30m with ground surface levels between RL5.7m and RL6.1m. It is currently occupied by a three storey brick and rendered hostel over partial ground floor parking. The existing structure covers the majority of the site except for narrow strips about 1.5m wide on the northern and southern sides of the site.

The eastern neighbouring property contains a two to three storey cement rendered mixed use development that abuts the common boundary along the full boundary length. The building appears in good condition based upon a cursory external inspection from the street frontage. It was unclear whether a basement level was present.

The neighbouring western property also partially wraps around the north-western corner of the site for a length of about 5m. The site contains a two to three storey brick and cement rendered building that appears to abut the common boundary along the full length. The building appeared in good condition based upon a cursory external inspection from the street front. It appears that a basement level is not present.

The remaining length of the neighbouring northern property contains a three storey brick building that is set back about 4m from the common boundary. The external areas in proximity to the subject site appear to be paved and landscaped patio and veranda areas.

3.2 Likely Subsurface Conditions

The 1:100,000 Sydney geological map indicates the site to be underlain by a channel of Quaternary period medium to fine marine sand. The hillside to the west of the site is underlain by Hawkesbury Sandstone.

We have completed several deep geotechnical investigations at sites within the same geology and within an area stretching about 500m to the north and 150m west and south-west of the site. Investigation techniques included Cone Penetrometer Tests (CPTs), dilatometer testing, boreholes with Standard Penetration Tests (SPTs) and coring of bedrock, and long term groundwater level monitoring. We have also completed shallow investigations with augered boreholes and SPT closer to the site including to the north and south.

In summary, a deep sandy soil profile was encountered comprising mostly sands and silty sands over sandstone bedrock, with groundwater one to two metres above 'sea level'.

Beneath a limited depth of fill, silty sandy soils were initially very loose to loose. The relative density from about 3m was variable, often increasing with depth to medium dense or denser but at some locations very loose sand extended to greater depth (to 9m at one test location on Manly Oval). Some silty clay and clayey silt bands were interpreted to be present.

West of the site, sandstone bedrock was inferred to be present from CPT tests at depths ranging from 20m to 34m below ground surface levels. To the north rock was also inferred to be present from depths of 30m to 32m. To the south-west of the site (Cnr of West Promenade and Sydney Road), rock was in the range of 12m to 22m depth. Where it was encountered in boreholes, the upper few metres of sandstone were highly variably weathered with strengths ranging from extremely low to medium strength. Some units of typically extremely weathered interbedded sandstone and shale were also encountered. Although the sandstone to the west of the site is Hawkesbury Sandstone, where rock is deeper it may be the underlying Newport Formation, which is weaker, interbedded sandstone and shale.

Groundwater was encountered at 4.7m depth at the corner of Raglan and Whistler Streets (approximately RL1.3m) and was more accurately recorded over a longer period of time in the range between RL1.1m and RL1.3m under the eastern side of Manly Oval.

4 COMMENTS AND RECOMMENDATIONS

4.1 Principal Geotechnical Considerations

All comments and recommendations are based on an assumed subsurface profile from information beyond the site and therefore should be reviewed by JK Geotechnics once geotechnical investigations are completed at the site. Further details on geotechnical investigation for detailed design are discussed below.

We expect about 3.7m of excavation is required for the proposed basement which is within the zone of influence of existing buildings of various scale, construction type and period. Some of them may be founded in very loose sands, and their footings may protrude onto the existing site. The principal geotechnical considerations will be how to maintain stability to neighbouring structures and infrastructure during demolition of existing structures and excavation. Careful demolition, completion of dilapidation surveys, consideration given to underpinning or grouting and installation of suitable shoring prior to excavation will be required.

Groundwater is expected to be about 1.5m below the bulk excavation level but long term monitoring is advised from as early a stage as possible to determine the magnitude of fluctuations with changes of rainfall. Groundwater monitoring will also likely be required to satisfy WaterNSW to prove that the basement will not intersect the groundwater table.

Given the expected very deep sandy profile, the assumed high column loads will have to be transferred to a suitable bearing stratum by grout injected continuous flight auger (CFA) piles or perhaps CSM barrette footings. Detailed geotechnical investigation will be required to identify such a stratum which is likely to be a medium dense or dense unit of sand or bedrock.

4.2 Dilapidation Surveys

Dilapidation surveys should be completed on the adjacent properties, and perhaps infrastructure, prior to commencement of excavation or even demolition.

Dilapidation surveys should comprise a detailed inspection of the adjoining properties, both externally and internally, with all defects rigorously described, i.e. defect location, defect type, crack width, crack length, orientation etc. The owners of the adjoining properties should be asked to confirm that the reports represent a fair record of actual conditions. The dilapidation reports may then be used as a benchmark against which to assess possible future claims for damage arising from the works.

4.3 Demolition and Working Platforms

Demolition should be carefully planned and executed in accordance with a sequenced methodology prepared by the structural engineer and with consideration to keeping the concrete pavement which may provide a good base for a working platform (or perhaps prevent the need for one being constructed at all). A working platform assessment should be completed once the preferred tracked plant for footings and shoring are known.

Working platforms for large tracked plant are required where the subgrade is of insufficient bearing capacity. Very loose upper sands such as is expected on this site often have insufficient bearing capacity. Contractors often assume (in their contracts) that working platforms will be provided for them and this can be a significant cost and time item for developers. Geotechnical investigation for a working platform assessment will often require a number of DCP tests and shallow boreholes. Any test pits, holes from removal of pad footings, or trenches should be backfilled with cement stabilised sand or well compacted granular material to avoid soft spots which would present a serious instability hazard.

There is potential for transmission of vibrations from demolition works to impact on the neighbouring structures some of which may be on shallow footings on very loose to loose sand.

Vibrations emitted during excavation should be minimised to prevent potential settlement of loose sands beneath footings. We therefore recommend that existing site building footings and floor slabs are saw cut or otherwise broken into smaller manageable pieces rather than to be demolished by use of rock breakers, particularly where in close proximity to buildings on shallow footings.

Monitoring should be completed on the neighbouring buildings targeting 'as low as reasonably practical' vibrations, say not greater than 3mm/s peak particle velocity (PPV). If this vibration limit is repeatedly reached, lower impact techniques should be adopted. The impact of large masonry or concrete having been dropped to the ground, or even into trucks, can cause damaging vibrations.

4.4 Underpinning/Soil Improvement

As discussed above, structures on shallow footings founded on very loose to loose sands are susceptible to settlement from vibrations during some demolition activities, movement of large plant and trucks, and soil decompression from shoring and pile installation. We therefore recommend that test pits are completed to investigate the footing system of the adjacent structures. If such shallow footing conditions are confirmed then consideration should be given to monitoring and 'underpinning'. Monitoring could be in the form of high accuracy surveying of prisms etc. Underpinning could be in the form of permeation grouting or chemical grouting to control settlement. Further advice should be sought in this regard once founding conditions are determined. Test pits should be inspected by a geotechnical engineer who may also recommend testing of the soil density by means of Dynamic Cone Penetration tests, or similar.

4.5 Shoring

Prior to excavation a shoring system must be installed to retain the soils and support the adjacent buildings. For a maximum 3m depth of excavation and without surcharges, a shoring wall sufficiently embedded to act in cantilever is feasible. Where surcharges are present such as buildings founded at shallow depth or live loads on roads, the wall may need to be anchored or propped, which will likely be required for the eastern and western sides. Anchors may not be feasible where there are adjacent basements, such as potentially on the neighbouring eastern property. Regardless, we recommend structural details of the neighbouring properties are confirmed as a matter of priority, particularly the presence of basement levels. Cantilever piles are normally of greater diameter than anchored piles and architectural design should allow sufficient space for the shoring required. Permission will be needed from property owners where anchors extend onto their property. It can be a lengthy process to achieve the permission so we encourage this be started without delay, if required.

Top down construction is also feasible, given the assumed sandy material will be easily excavated. Obviously footing piles would have to be drilled from the surface prior to the slab being constructed. Top down construction has the advantages of reducing the risk of shoring wall deflection and therefore reduces the risk of damaging neighbouring buildings, but also allows construction of above ground levels to commence at an earlier stage.

Assuming a sandy profile with groundwater about 1.5m below bulk excavation level, the following shoring systems would be suitable.

- A contiguous shoring pile wall drilled using cased cement grout injected CFA piles. Without the casing, there is a greater risk of soil decompression occurring thus potentially damaging neighbouring buildings. To prevent soil loss, gaps between piles should either be packed with grout, or shotcreted.
- Cutter Soil Mix (CSM) wall. This system mixes cement with the existing sand and water to form 'concrete' panels insitu, into which steel reinforcement (usually 'I' beams) is added. The site is relatively small compared to the space normally required for this equipment so contractors should be consulted regarding the feasibility prior to committing to design. This technique also has the potential for soil decompression so further consideration should be given to underpinning the adjacent structures, prior to shoring works. CSM walls may not necessarily have the same lifespans as CFA piled walls. Internal reinforced shotcrete finishes can be added, or perhaps since the basement is expected to be above the groundwater level, contractors may provide sufficient design life warranties.

Only experienced contractors with appropriate experience and insurances should be engaged.

Any surcharge loads affecting the walls (e.g. buildings, traffic loading, construction loads etc) should be taken into account in the wall design, and these are additional to the earth pressures. We assume that permanent lateral support of the retaining walls will be provided by the new structure.

Design parameters can be provided following detailed geotechnical investigation, but for preliminary concept design a 'worst case' of the typical conditions could be assumed and would comprise very loose sands and a groundwater level say just below bulk excavation level.

Localised shoring may also be required for construction of the lift pit which is in the centre of the site and depending on its depth may protrude below the groundwater level requiring dewatering. Interlocking driven sheet piles may be appropriate given the 9m to 16m offsets from the boundaries. If sheet pies are adopted, we recommend vibration monitoring be carried out during the installation. If vibrations are notable then lower vibration emitting shoring systems should be installed such as CFA secant pile walls.

If dewatering is anticipated the wall toe level must be designed following detailed seepage analysis to avoid a broader draw down profile which potentially may affect neighbouring structures.

4.6 Excavation Techniques

Excavation to about 3.7m in an assumed very loose to loose sandy profile should be readily achieved using buckets of hydraulic excavators and bobcats. Groundwater is expected to be about 1.5m below bulk excavation level.

Locally deeper excavations, such as for lift pits, may encounter groundwater which would require localised dewatering. Any dewatering should be carried out in accordance with a detailed methodology designed by an engineer to prevent 'boiling', and other issues (discussed in Section 4.4) and approved by a geotechnical engineer independent of the contractor.

4.7 Footings

Detailed geotechnical investigation is critical to the design for the footings. We expect that there may be a medium dense or dense layer within the expected deep soil profile that may be suitable for embedment of piles. We recommend a minimum of five Cone Penetration Tests be carried out within the site to reduce the risk of unidentified soil conditions. Dilatometer testing may assist in optimising soil parameters and therefore the pile and shoring design.

Footings will have to be cement grout injected CFA piles or perhaps, if CSM is used for shoring, then a CSM panel could be constructed (also known as a barrette) to save establishing a second large rig.

One advantage of CSM over CFA piles is that minimal spoil is generated for disposal, particularly if acid sulphate soils are present.

If loose soils are present and rock is not excessively deep (i.e. less than say 15m), then piled footings on rock could be an option. Cored boreholes would be required to provide detailed information on the rock strength and defects in order to optimise design bearing pressures.

4.8 Groundwater and Permeability

About 100m inland from the subject site, groundwater levels were at about RL1.1m to RL1.3m. The level reduced at sites closer to Manly beach. We therefore expect similar groundwater levels will be encountered at the subject site i.e. about 1.5m below bulk proposed excavation level.

Continuous groundwater level monitoring should be carried out to determine the groundwater levels on site and also fluctuations following long periods of rainfall. There may also be minor tidal fluctuations being 200m from the ocean. Piezometers with electronic data loggers should be installed without delay since the assumptions made regarding groundwater will significantly change geotechnical design concepts if the groundwater level is above or close to bulk excavation level. Given the existing site structure, this will require the drilling of boreholes outside of the site boundaries, such as along the Raglan Street footpath and grassed area present north-east from the site along Whistler Street near the electricity substation. The aim is to achieve a piezometer layout to allow for a triangulation of the groundwater levels that also covers the site as best as possible.

Until infiltration testing within piezometers can be carried out, preliminary design of stormwater infiltration systems could be based on the typical hydraulic conductivity (permeability), K, for the expected natural soils. Based on past experience and published literature, permeability of sand to silty sands would typically be in the order of 10^{-4}m/s to 10^{-5}m/s but could range by a further one to two orders of magnitude depending on the silt fines content. Infiltration may also be affected by possible layers of clay and a varying groundwater level. Infiltration systems should also consider possible effects on adjacent basements. We recommend preliminary design values be revised following site specific testing when site access becomes available.

4.9 Subgrade Preparation and Slabs-on-Grade

We assume sandy soils will be present but layers of silt and clay may be present within the alluvial soil profile.

Slabs-on-grade are feasible above the groundwater level and would effectively be 'floating' independent of the superstructure. To confine the assumed sandy soils, a 100mm layer of crushed rock to RMS QA Specification 3051 (2013) unbound base material (or similar good quality and durable fine crushed rock) should be placed. The subgrade should then be prepared by rolling with a minimum 8 passes of a static smooth drum roller of not less than 7 tonnes to densify the near surface soils. No vibrations should be used due to the potential damage that could be caused to nearby structures. The final pass should be completed in the presence of a geotechnical engineer to check for the presence of any soft spots which usually indicates unsuitable soils. Should any soft spots be identified, they should be excavated and replaced with good quality granular material compacted in thin layers until no noticeable deflection is observed.

The subbase layer should be compacted to at least 100% of its Standard Maximum Dry Density.

Trafficable concrete pavements should be designed with effective shear transmission at all joints by way of either dowelled or keyed joints.

4.10 Sydney Water Assets

We noted the presence of a Sydney Water asset along Raglan Street and also at the northern end of the site. Our understanding is that if the development falls within 10m of any Sydney Water assets or the asset is within the proposed excavation zone of influence, then Sydney Water will likely request a Specialist Engineering Assessment (SEA) in accordance with Sydney Water Specialist Engineering Assessment document (Doc No. D0001870, Version 1 dated 19 February 2021). Reference should also be made to the Sydney Water Technical Guideline, Building Over and Adjacent (BOA) to Pipe Assets, which provides further guidance on the requirements that developments must comply with.

The SEA will require varying amounts of input from geotechnical, structural and civil engineers. The preparation of an SEA and obtaining approval from Sydney Water can be a lengthy process and therefore, if required, we recommend the process commences as soon as possible to avoid potential project delays. The engagement of a Water Services Coordinator will also be required to facilitate the process.

4.11 Detailed Geotechnical Investigation and Other Geotechnical Input

The following is a summary of the further geotechnical input required and has been detailed in the preceding sections of this report:

- Drilling of boreholes, installation of piezometers, groundwater level monitoring and infiltration testing.
- CPT testing of the site soils and perhaps subsequent dilatometer testing.
- Cored boreholes to prove the rock if the soil profile is of insufficient strength for piles, or if rock is shallower than anticipated.
- Investigation/survey of adjacent basements (by others).
- Test pits for adjacent building footings.
- Dilapidation surveys.
- Sydney Water Specialist Engineer Assessment, if required.
- Working platform assessment.
- Consideration of underpinning or completing ‘ground improvement’ under any adjacent shallow footings, prior to shoring works.
- Review of shoring and footing design.
- Inspection of initial shoring and footings.
- Proof roll inspection of subgrade.

5 GENERAL COMMENTS

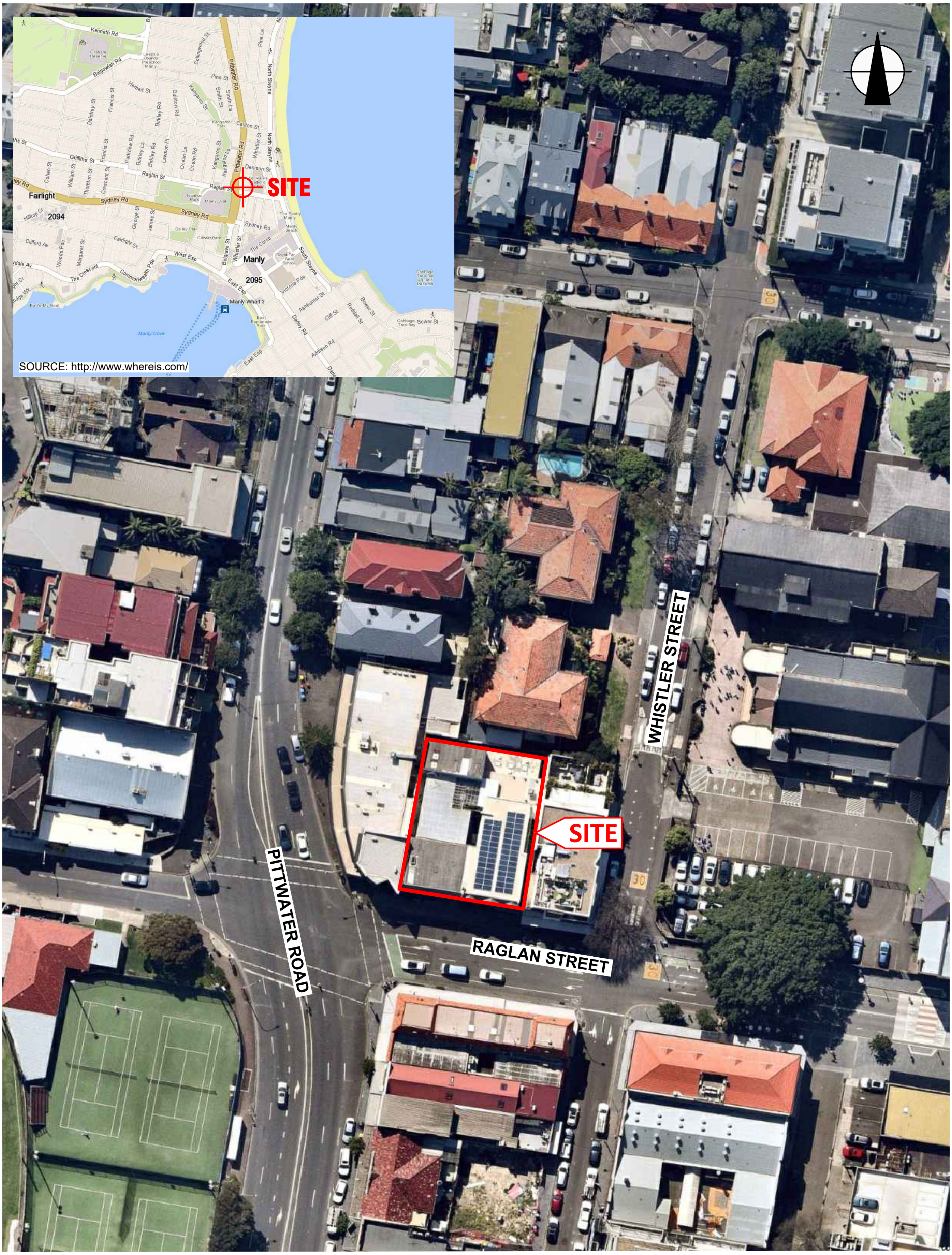
The recommendations presented in this report include specific issues to be addressed during the construction phase of the project. In the event that any of the construction phase recommendations presented in this report are not implemented, the general recommendations may become inapplicable and

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.

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 is required for any soil and/or bedrock excavated from the site prior to offsite disposal. Subject to the appropriate testing, material can be classified as Virgin Excavated Natural Material (VENM), Excavated Natural Material (ENM), General Solid, Restricted Solid or Hazardous Waste. Analysis can take up to seven to ten 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) could be expected. We strongly recommend that this requirement 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

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

Title:	**SITE LOCATION PLAN**
Location:	22-24 RAGLAN STREET, MANLY, NSW
Report No: 35612YF	Figure No: 1

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PLOT DATE: 16/11/2022 9:39:54 AM DWG FILE: J:\6F GEOTECHNICAL JOBS\35000'S\35612YF MANLY\CAD\35612YF.DWG