REPORT

TO
LORETO KIRRIBILLI LIMITED

ON
GEOTECHNICAL ASSESSMENT

FOR
PROPOSED MASTERPLAN

AT
LORETO KIRRIBILLI
85 CARABELLA STREET
KIRRIBILLI NSW

10 July 2017
Ref: 30067Srpt2rev2

JK Geotechnics
GEOTECHNICAL & ENVIRONMENTAL ENGINEERS

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1 INTRODUCTION

This report presents an information overview for the masterplan at Loreto Kirribilli, 85 Carabella Street, Kirribilli, NSW. The report was commissioned by Mr Richard Schilling of Loreto Kirribilli Limited by return email. The commission was on the basis of our proposal, Ref: P43538Srev1, dated 28 November 2016.

This report has been compiled using information from the following previous investigations:

- Geotechnical Investigation for New School Development by Peter J. Burgess & Associates dated August 1990, Ref: 2757.
- Geotechnical Investigation and Design for Retaining Wall Replacement by Jeffery & Katauskas (currently trading as JK Geotechnics) dated December 2009, Ref: 23613SP.
- Geotechnical Investigation for Proposed Alterations and Additions by Jeffery & Katauskas dated June 2010, Ref: 23903SY.
- Geotechnical Opinion for Failed Slope and Retaining Walls by Jeffery & Katauskas dated April 2011, Ref: 24708SY.
- Geotechnical Investigation for Proposed Lift by JK Geotechnics dated July 2012, Ref: 25882P.
- Geotechnical Investigation for Proposed Development SSD by JK Geotechnics dated January 2017, Ref: 30067S.

Based on the supplied concept plan drawings prepared by Francis-Jones Morehen Thorp Pty Ltd dated July 2017, we understand the proposed concept development comprises of the following:

- Western Precinct
    - Demolition of the existing B-Block and then construction of proposed Innovation Centre and Gymnasium Extension.
    - Partial demolition of external stairs, landings, walkways and planters in between the Gymnasium, Centenary Hall and the Junior School. Following demolition, construction of external covered walkways and extension of the Junior School Play Terrace.
- Northern Precinct
    - Partial demolition of external stairs, landings, walkways and planters in between Science and Centenary Hall. Following demolition, construction of a six storey vertical connector pod.
- Eastern Precinct

- Partial demolition of external stairs, landings, walkways and planters in between Science, Elamang, Performing Arts and Mary Ward. Following demolition, construction of a six story vertical connector pod.
    - Demolition of Music and Performing Arts and Mary Ward and construction of a new four storey building and two storey car park.
- Southern Precinct
    - Partial demolition of external stairs, landings, walkways and planters in between the Chapel and J-Block. Following demolition, construction of a five storey vertical connector pod.
    - In addition, demolition of the Junior School, excavation to Centenary Hall level and construction of a new five storey building.

The purpose of the masterplan report was to compile geotechnical information on the subsurface conditions from previous investigations as a basis for providing comments and recommendations of a general nature on excavation conditions, retention, footings and other geotechnical aspects of the project.

2 ASSESSMENT METHODOLOGY

The assessment is based upon a detailed inspection of the topographic, surface drainage and geological conditions of the site and its immediate environs. A summary of our observations is presented in Section 3 below. Furthermore, we compiled information generated from our database search of the site and have included all previous information with this report. Our general recommendations regarding the proposed works involved within the masterplan are discussed in Section 4 below.

The attached Figure 1 identifies the site within the local region. Figures 2, 3A, 3B, 3C and 3D present a geotechnical sketch plan of each area showing the principal geotechnical features present at the site. Figures 2 and 3A to 3D are based on the survey plan prepared by Hammond Smeallie & Co Pty Ltd (Dwg. No. 6551_2016, Rev A dated 18/04/16). Additional features on Figures 3A to 3D have been measured by hand held clinometer and tape measure techniques and hence are approximate only. Should any of the features be critical to the proposed alterations and additions, we recommend they be located more accurately using instrument survey techniques. Additionally, three cross-sectional sketches, Figures 4, 5 and 6 have been provided showing the cross-sectional profile across the site.

For further details of the investigation techniques adopted, reference should be made to the attached Report Explanation Notes. For detailed information regarding previous investigations, reference should be made to the relevant geotechnical reports issued as part of the work. The borehole logs have been supplied in the attached Appendices A, B, C, D and E.

3 RESULTS OF INVESTIGATION

3.1 Site Description

The site is situated within relatively hilly coastal topography with slopes generally in a north-eastern direction towards Careening Cove. The site itself is located roughly mid-slope of north-eastern facing hill sloping down at approximately 8° to 10°.

The site has north-eastern and south-western street frontages onto Elamang Avenue and Carabella Street, respectively. Elamang Avenue slopes down towards the north-west generally at 1° to 2° and comprises an asphaltic concrete (AC) paved road with sandstone block kerbs that appear in moderate condition with some cracking up to 3mm wide observed along the boundary length. A vegetated verge with small to large trees and a concrete footpath lie between the road and the site boundary except along the central portion of the common boundary. Over this portion the footpath is elevated and supported on a sandstone block retaining wall up to 4m high that typically appears in good condition. The sandstone block wall was observed to be founded on a vertical sandstone cut face that was assessed as at least medium strength based upon examination using a geopick.

Carabella Street slopes down towards the north-west variably at 1° to 3° and comprises an AC paved road with sandstone block kerbs. The road typically appears in good condition with no defects observed based upon a cursory inspection from the roadside. Towards the south-eastern end of the site on the opposing side of the road, sandstone outcrops and vertically cut sandstone faces were observed and assessed to be at least medium strength. A concrete footpath lies between the road and the site boundary with large sized trees intermittently along the footpath. The subject site varies from being at footpath level to up to 2m below the footpath level.

The south-eastern end of the site is L-shaped and is bounded by a number of neighbouring properties typically comprising two to three storey brick and stone masonry buildings that generally appear in moderate to good condition based upon a brief cursory inspection, where possible, from within the subject site. The rear of the neighbouring properties are mostly retained above the subject site via sandstone block retaining walls up to about 4.9m high that typically appear in good condition. The neighbouring building at No. 10 Elamang Avenue either abuts the boundary or is setback about 2m. A vertical sandstone cut face was observed from within the subject site at the rear of No. 69 Carabella Street on the far side of the existing tennis court.

The north-western boundary is a reasonably straight boundary with two neighbouring properties setback about 1.2m from the common boundary at the closest point. The neighbouring buildings comprise of three to four storey brick buildings that appear in moderate condition based upon a cursory inspection from the street frontage and within the subject site. Whilst difficult to observe on-site due to access constraints and vegetation, it appears generally that the neighbouring properties maintain relatively similar levels to the subject site. Towards the Carabella Street frontage the subject site is retained up to 2m via a concrete retaining wall that appears in moderate condition with some minor overturning evident into the neighbouring property.

At the time of this assessment, whilst some areas appear to follow the natural topography of the hillside, typically the site steps down the hill via a series of retaining walls and buildings. The site contains regular landscaping vegetation as well as small to large trees. The site contains numerous brick, concrete, stone masonry and cement rendered buildings that typically appear in good condition based upon a cursory external inspection. Retaining walls over the site typically ranged in height between 1m and 3m and mostly comprised of brick or sandstone block walls that were in moderate to good condition with some minor cracking evident. A large criblock wall up to about 7m high is located at the western corner of the gymnasium founded on a vertical sandstone cut face. The criblock wall and sandstone cut face extend to the east below the grassed area, following the line of the gymnasium. The sandstone cut face varies in height from 2m at the eastern end increasing to about 4.5m at the western end. The criblock walls appear in good condition with no obvious signs of movement based upon a cursory inspection from the base of the sandstone cut face. Below the gym on its north-eastern side, a steep batter slope leads down towards Elamang Avenue which is at about RL7m to 9m. No evidence of slope instability was evident although external concrete slabs along the gym perimeter were observed to have moved up to 35mm out from the building.

Based on our site walkover, sandstone bedrock is exposed over the site at the following locations:

- Adjacent to the eastern portion of B-Block below the external brick retaining wall adjacent to the car park area.
- The south-eastern side of J-Block a brick retaining wall is founded on a vertical sandstone cut face. The sandstone cut face then extends along a roughly north-south line behind the lower ground floor of J-Block.
- The full length of the rear wall of the underground carpark adjacent to Elamang Avenue.

- Near the boundary adjacent to the south-western side of the existing Performing Arts building.

We note that some areas of exposed sandstone may not have been identified during our site walkover due to access or other constraints. Generally, based upon examination using a geopick, the exposed sandstone across the site was considered to be at least medium strength with relatively few defects other than some sub-horizontal bedding partings and relatively thin extremely weathered seams. Minor groundwater seepage was observed along the majority of the sandstone cut faces. Details of the site are shown on the attached Figures 3A to 3D.

3.2 General Subsurface Conditions

The 1:100,000 Geological Map of Sydney indicates the site to be underlain by Hawkesbury Sandstone of the Wianamatta Group comprising medium to coarse grained quartz sandstone, very minor shale and laminite lenses. This has been confirmed both by the core drilling and observation around the site and surrounds generally.

The site typically contains variable depths of fill with relatively thin bands of residual soils before sandstone bedrock is encountered which is generally initially extremely weathered to distinctly weathered before quickly improving to medium to high strength sandstone. Reference should be made to the attached borehole logs within the Appendices for detailed subsurface descriptions at specific locations. A summary of the subsoil conditions, as encountered, is presented below:

Fill

Fill appears to be quite variable across the site based on the previous investigations ranging from 0.5m to 3m depth. The fill typically comprised a sandy material containing varying amounts of clay, silt fines, fine to coarse grained sandstone, basalt and ironstone gravel, slag, ash, root fibres and roots.

As expected, fill has generally been encountered behind the majority of retaining walls present on-site. The fill behind walls appeared to extend a similar depth as the wall height, as the walls generally appeared to be founded on the underlying sandstone bedrock.

Residual Soils

Generally only very minor bands of residual soils were encountered as typically the profile comprised of fill directly overlying sandstone bedrock. The residual soils comprised fine to coarse grained clayey sand and sandy clay soils of low to medium plasticity.

Bedrock

As expected, the sandstone bedrock appears to gradually step down the hillside from Carabella Street to Elamang Avenue. The exposed sandstone bedrock observed on-site was assessed as at least medium strength or better with relatively few defects evident other than some sub-horizontal bedding partings and the occasional thin extremely weathered seam. Based on the previous investigations, once better quality sandstone is encountered, it appears to be relatively consistent with only occasional defects such as extremely weathered seams, generally no greater than 100mm thick and steeply sloping joints between 5° and 70°, with rare sub-vertical joints.

Groundwater

The groundwater table was not encountered within any of the investigations but groundwater seepage was observed to be occurring along the soil-rock interface and through defects within the rock mass, such as joints and bedding partings. At the time of the walkover, the groundwater seepage rates appeared to be very low.

3.3 Laboratory Test Results

The point load strength index tests generally showed good correlation with the field logging of rock strength; the results are shown in Table A and on the borehole logs. The Unconfined Compressive Strength (UCS) of the distinctly weathered to fresh sandstone typically ranged from 4MPa up to 36MPa, although lower and higher strength sandstone outside this range was occasionally encountered.

4 COMMENTS AND RECOMMENDATIONS

4.1 Geotechnical Issues

The provided masterplan drawings are of a general nature only by providing an overview of the proposed development. As a result, the following comments and recommendations are similarly of a general nature only. We recommend the following recommendations are reviewed in detail for each individual development.

The proposed developments within each precinct are reasonably variable but generally involve demolition of existing buildings and retaining walls, excavation of materials ranging from 1m to 13m depth and construction of new buildings comprising vertical pod connectors, four to five storey buildings with up to two levels of basement and extensions to existing buildings. We anticipate sandstone bedrock at relatively shallow depths across the majority of the site except where retaining walls have been constructed and subsequently backfilled.

There are a number of geotechnical issues which may affect the proposed masterplan development which can be inferred from the concept plans prepared to date and further issues may be identified as the project becomes more advanced.

1. Excavations will be substantial for a number of the developments and they will predominantly encounter sandstone bedrock of medium to high strength. These are considered to require 'hard rock' excavation techniques which may result in noise and vibration issues due to the close proximity of adjacent buildings within the site and also neighbouring properties.
2. Excavation in close proximity to existing structures must consider both vibration and stability effects.
3. We consider the sandstone bedrock to be the most competent founding stratum and we would expect it to be exposed at the base of most excavations.
4. Groundwater seepage is likely to occur from the excavated rock faces but this is not expected to affect regional groundwater behaviour and the volumes likely to occur will be quite low. Very occasionally more concentrated flows can occur from particular defects in the rock mass but such events cannot be predicted and simply have to be dealt with should they arise.

4.2 Excavation Conditions

All excavation recommendations should be complemented by reference to latest version of Safe Work Australia's 'Excavation Work Code of Practice'.

4.2.1 Dilapidation Surveys

Prior to the commencement of demolition and excavation, we recommend that detailed dilapidation surveys be completed on any structures near the site within a distance from the excavation of twice the excavation depth. For the more distant structures, of say 20m or more, a reduction in the detail of the dilapidation reports could be considered.

The dilapidation surveys should include internal and external inspections of the buildings, where all defects including defect location, type, length and width are described and photographed.

The respective owners of the neighbouring buildings should be asked to confirm that the dilapidation survey reports present a fair record of existing conditions. The dilapidation survey reports may be used as a benchmark against which to assess possible future claims for damage arising from the works.

4.2.2 Excavation Methods

We expect excavation depths of up to 13m which will probably encounter pavements, fill, residual soils and sandstone bedrock. Excavation of soils and up to very low strength sandstone will be achievable using conventional excavation equipment, such as the buckets of hydraulic excavators. Excavation of the sandstone will require the use of rock excavation equipment, such as hydraulic rock hammers, rotary grinders, ripping hooks and rock saws. Sandstone of high strength will represent 'hard rock' excavation conditions. The excavator contractor should be made aware of this by being supplied with all geotechnical information; low productivity and high equipment wear should be expected due to the rock strength.

Rock excavations using hydraulic rock hammers will need to be strictly controlled as there could be direct transmission of ground vibrations to nearby structures and buried services. We recommend that electronic quantitative vibration monitoring be carried when out using hydraulic rock hammers to determine if the transmitted vibrations are within an acceptable limit for the nearby structures and services. 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 methods, such as smaller rock hammers, rotary grinders, ripping hooks or rock saws.

The following procedures are recommended to reduce vibrations if rock hammers are used and transmitted vibrations are found to be excessive:

- Rock saw the perimeter faces. This will effectively reduce ground borne vibrations provided the base of the rock saw slot is maintained at a lower level than the adjacent excavation level at all times. Rock sawing would also improve the stability and aesthetics of the completed rock cut faces and is strongly recommended.
- Maintain rock hammer orientation towards the face and enlarge the excavation by breaking small wedges off the face.
- Operate the rock hammer in short bursts only, to reduce amplification of vibrations.
- Use excavation contractors with appropriate experience and a competent supervisor who is aware of vibration damage risks, etc. The contractor should have all appropriate statutory and public liability insurances.

We recommend that a copy of the relevant report be provided to the excavation contractor so that they can make their own assessment of excavation conditions.

Once the appropriate usage of rock hammer equipment is established then full time monitors should be established on the nearest buildings with alarms set up to warn site staff if tolerable thresholds are exceeded. Should the alarm be triggered it is of course imperative that the cause of the exceedance be established and measures taken to ensure repetition does not occur.

There is a possibility that the rock could be quarried for use as dimension stone in which case the majority of the excavation would involve saw cutting.

4.3 Retention Systems

Given the topography of the site, inevitably construction of additional retaining systems will be required. As mentioned previously, we expect excavation depths to vary across the proposed masterplan development but possibly up to 13m depth. Excavations within the soils and up to low strength sandstone will not be self-supporting and retention systems will need to be installed prior to the start of excavation.

Based on the previous investigations, it appears that the depth of soil will most likely be limited except behind already constructed retaining walls that have been backfilled. Additional investigations within the subject areas are crucial to allow for detailed design prior to construction. For relatively shallow excavations, it may be feasible to construct temporary batters if the space allows. Temporary batters can be formed in clayey soil at about 1 Vertical (V) to 1 Horizontal (H) or at 1V:2H in sandy soil provided there are no surcharge loads. Locally, deeper soil pockets may exist and we recommend that as a preliminary phase of construction, a test trench or series of test pits near the proposed sides of the excavation should be excavated and then inspected by a geotechnical engineer to determine whether any localised areas require special treatment.

The proposed developments generally are located adjacent to or within close proximity of existing structures and as such prior to any excavation or demolition, a detailed evaluation must be undertaken on any adjacent footings that may be affected by the works. Based upon the further works, the need for any stabilisation works will be assessed and advice provided accordingly.

Where temporary batters are not feasible, two suitable retention systems may be used. Where buildings are not present within the zone of influence of the excavation (defined as 2H, where H is the height of retained material), due to the possible shallow depth to sandstone bedrock of medium strength or better, the most cost effective retention system might be the construction of a concrete retaining wall. This wall would be constructed by excavating trenches in short sections (say no greater than 2m long) down to the underlying sandstone bedrock of medium strength or greater, installing dowels into the underlying sandstone bedrock of at least medium strength and then pouring a reinforced or unreinforced concrete retaining wall. The soils on this site may be sandy and may not allow stable trenches to be formed in which case this system should not be pursued. Should such collapses occur the collapse could extend over the boundary and this may result in damage to pavements and services.

Where soils are unstable or more than about 1m deep, we consider that anchored contiguous or soldier pile walls will provide an economic means of retention for the site. It is unlikely to be practical for the pile wall to be founded with sufficient embedment below bulk excavation level to satisfy stability and founding considerations as most of the excavation will be in good quality rock and the sockets would be difficult and expensive to construct. The piles can instead be terminated no less than 0.3m in at least medium strength or stronger sandstone bedrock above bulk excavation level, though we recommend that permanent structural loads on the piles be minimised and transferred instead to below bulk excavation level. Toe restraint for the piles may be achieved by using an additional row of temporary anchors or toe bolts, which must be installed prior to excavating in front of the pile toe. A vertical face in the medium strength or stronger sandstone may be excavated below the toe of the piles, but must not undermine the pile toes. Sawing of the cut faces is considered essential below the pile toes.

The sandstone bedrock at this site, including the sandstone below the toe of the soldier piles can be cut vertically, but must be progressively inspected by a geotechnical engineer at no more than 1.5m depth increments to assess the need for temporary support (e.g. rock bolts, dowels, shotcrete etc.) of potentially unstable rock wedges or extremely weathered bands. Although the limited borehole information across the site shows relatively few defects in the medium to high strength sandstone once encountered, we expect some stabilisation measures; particularly treatment of extremely weathered bands will be required. A provision should be made in the contract documents (budget and program) for the above inspections and stabilisation measures including rock bolts.

We also recommend that for the initial stages of excavation in front of the toes of any shoring piles, the excavation should extend to a depth of not more than 1.5m below the pile toes and not closer than 1.5m from the face of the pile, so that a geotechnical engineer can check for the possible presence of any adverse defects or weathered zones of rock which may destabilise the pile. An allowance should be made for temporary rock bolts below the pile toes to provide lateral restraint for the rock in these sensitive areas.

Inward movement of the excavation is to be expected as a result of stress relief within the vertically cut sandstone bedrock. However, as the dimensions of the excavation are not great, this movement will be somewhat reduced. From our experience the expected movement is not likely to have an adverse effect on the adjacent properties though some cracking cannot be ruled out.

4.4 Retaining Wall Design Parameters

Propped or anchored soldier pile walls which are not directly supporting structures may be designed based on a rectangular earth pressure distribution of 6H kPa, where H is the retained height of soils and extremely low to low strength sandstone. The design pressure should be increased to 8H kPa where there are movement sensitive structures within the zone of influence of the excavation. All surcharge loads, i.e. adjoining buildings, traffic, sloping backfill, etc, should be allowed for in the design, plus full hydrostatic pressures unless measures are undertaken to provide complete and permanent drainage behind the walls.

For the design of cantilevered retaining walls a triangular earth pressure distribution and a coefficient of active earth pressure (K_a) of 0.35 may be adopted where movement sensitive structures are not located within the zone of influence of the excavation. Where the walls are propped or movement sensitive structures are located within the zone of influence of the excavation a coefficient of lateral earth pressure of 0.55 should be adopted. All surcharge loads such as stockpiles, footing loads, traffic loads, etc. should be added to the above pressures. Appropriate hydrostatic pressures should also be added to the above pressures.

Anchors should have their bond formed within sandstone of at least medium strength and may be provisionally designed based on an allowable bond stress of 400kPa. Anchors should have a minimum free length of 3m and a minimum bond length of 3m. All anchors should be proof loaded to at least 1.3 times their 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. Generally anchors are installed on a design and construct contract so that optimisation of bond stresses does not become a contractual issue in the event of an anchor failing the test load.

If temporary anchors are to run below neighbouring properties, then permission from the owners must be obtained prior to installation. We recommend that requests for permission commence early in the construction process as our experience has shown that it can take significant time for such permission to be granted. If permission is not forthcoming, then the alternative is to provide lateral support by internal bracing or propping. We assume that permanent support of the shoring system will be provided by bracing from the proposed building.

4.5 Footings

Based on the limited results of the previous investigations, it appears reasonable to assume that sandstone of medium to high strength will be encountered at the bulk excavation levels with few significant defects present. The anticipated sandstone of high strength is suitable for an allowable bearing pressure of 3,500kPa. Given the limited data across the site. Additional investigations will be required for each development to confirm the subsurface conditions. We recommend that all footings are inspected by a geotechnical engineer to check that the rock is of satisfactory quality; spoon testing of at least some footings may be necessary.

Allowable bearing pressures of 6,000kPa (or greater) may be used within the high strength sandstone. If such a bearing pressure was to be considered, additional boreholes would be required and an increase in spoon testing, likely to comprise spoon testing at all footing locations.

Given the soil variability and relatively shallow bedrock, we do not recommend founding footings within the soils present on-site. If required, additional recommendations can be provided by a geotechnical investigation.

4.6 Hydrogeological Considerations

In general terms, groundwater flow through this area occurs essentially along the soil-rock interface and through the significant bedding partings and vertical joints and would not normally be considered significant. We anticipate the flows intercepted by the excavations would be collected by subsoil drains which would be required around the perimeter and below the floor generally within the proposed excavations and would drain to the stormwater system. Groundwater flows at greater depth below the site would continue and there would be no significant effect on downstream properties on the low side of Elamang Avenue.

We recommend that the completed excavations be inspected by geotechnical and hydraulic engineers that any necessary subsoil drains could be located most effectively within the excavations. It may be necessary to install a pump-out pit below the lift overrun unless the overrun is completely waterproofed and can resist any uplift hydrostatic pressures.

4.7 Basement Slab

Based on the limited previous investigation results, we would typically expect the basement floor slabs for the proposed developments to directly overlie sandstone bedrock. If this is the case, we recommend that underfloor drainage blanket be provided. The underfloor drainage should comprise a strong, durable, single-sized washed aggregate such as ‘blue metal’ gravel. The underfloor drainage should connect with the perimeter drains and lead groundwater seepage to a sump for pumped disposal to the stormwater system unless gravity drainage is possible.

Joints in the basement concrete on-grade floor slabs should be designed to accommodate shear forces but not bending moments by using dowelled or keyed joints.

4.8 Quarrying Potential

The sandstone encountered in the borehole is of consistently good quality from an engineering perspective. For use as a building material the best Sydney Sandstone is known as yellow block and is characterised by being of a massive and homogeneous nature as opposed to the sandstone at this site which predominantly contains darker cross bedding which will form planes of weakness in thin slabs. The internal nature of the sandstone also varies with the best quality material being quartz rich while other sandstones have lower quartz content and higher clay content. The colour of sandstones can change considerably on exposure to the air in some cases. By inspection we consider the quarrying potential of the sandstone is worth investigating further provided it is recognised that the time required for quarrying is much greater than normal excavation. The next step in evaluating the material would be to have some thin sections analysed by a petrologist (which is relatively inexpensive) and then to contact one of the quarrying companies and invite them to look at the core samples and see the petrographic report.

4.9 Further Work

Given the general nature of the provided masterplan drawings, it is difficult to determine the extent of further work required for the development. We would consider the following as a minimum requirements although these should be reviewed once further details are available:

- Additional geotechnical investigations within the areas of concern, particularly where no previous information is available. The additional investigations would most likely comprise of cored boreholes and possibly test pits to assess existing footings. A geotechnical report would then be issued for the specific development in question.
- Dilapidation survey reports on nay neighbouring buildings prior to demolition and excavation.

- Vibration monitoring throughout demolition and bulk excavation.
- Monitoring of groundwater seepage into excavations to confirm drainage requirements.
- Progressive inspection of excavated cut faces to confirm if additional support or treatment is required.
- Footing inspections and testing.

Following completion of any investigations and detailed design, we also recommend a review of proposed earthworks and structural drawings and section details (once available) in order to confirm our recommendations are understood.

5 GENERAL COMMENTS

The recommendations presented in this report include specific issues to be addressed during the construction phase of the projects. 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.

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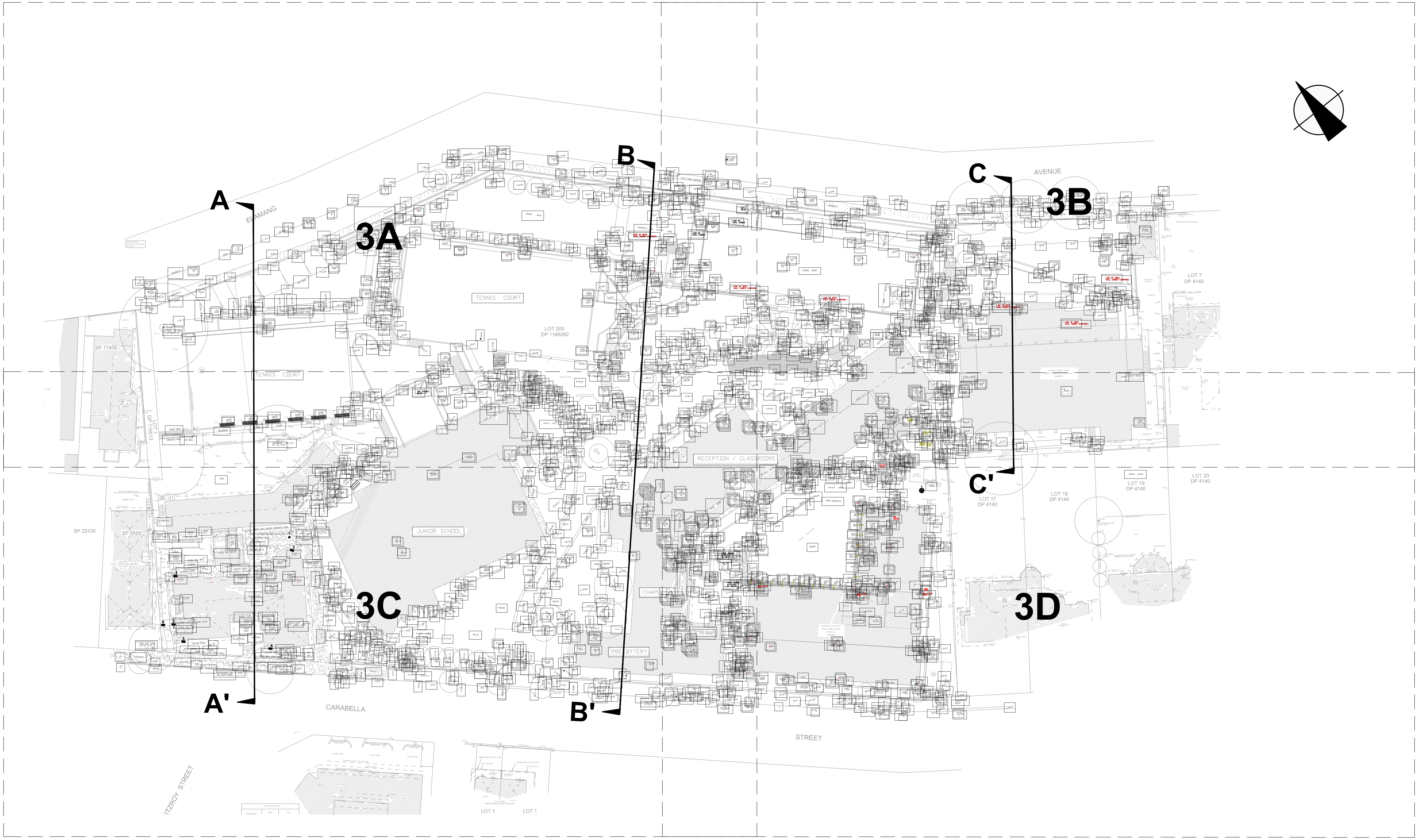
This plan should be read in conjunction with the JK Geotechnics report.

Title:	**SITE LOCATION PLAN**
Location:	LORETO, 85 CARABELLA STREET KIRRIBILLI, NSW
Report No: 30067S	Figure No: 1
	JK **Geotechnics**

PLOT DATE: 3/02/2017 3:12:09 PM DWG FILE: S:\6 GEOTECHNICAL\6F GEOTECHNICAL JOBS\30000'S\30067S KIRRIBILLI\CAD\30067S2.DWG



0 8 16 24 32 40

SCALE 1:800 @A3 METRES

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

Title: **OVERALL SITE PLAN**

Location: LORETO, 85 CARABELLA STREET KIRRIBILLI, NSW

Report No: 30067S

Figure No: 2

JK **Geotechnics**

PLOT DATE: 3/02/2017 3:10:04 PM DWG FILE: S:\6 GEOTECHNICAL\6F GEOTECHNICAL JOBS\30000'S\30067S KIRRIBILLI\CAD\30067S2.DWG