REPORT

TO
GREENGATE DEVELOPMENT PTY LTD

ON
GEOTECHNICAL ASSESSMENT

FOR
PROPOSED RESIDENTIAL DEVELOPMENT

AT
42-48 CHAPEL STREET, KOGARAH, NSW

21 January 2013
Ref: 26258Zrpt

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FIGURE 1: SITE LAYOUT PLAN

FIGURE 2: PROPOSED BOREHOLE LOCATIONS

1 INTRODUCTION

This report presents the results of our geotechnical assessment of the site for the proposed residential development at 42-48 Chapel Street, Kogarah, NSW. The assessment was commissioned by Mr Stuart Scott of Greengate Development Pty Ltd, by signed 'Acceptance of Proposal' form dated 8 January 2013. The commission was in accordance with our proposal (Ref P36409Z Kogarah) dated 18 December 2012.

We understand from the provided architectural drawings (001 to 07, Revision D, and SK13, Revision A) prepared by Greengate Design Pty Ltd, that following demolition of existing buildings on site, the proposed development envisages three buildings (three levels, four levels and 12 levels in height), set out in 'U' fashion around a central 'village green', all over two common basement levels. Excavation to a maximum depth of approximately 6.5m will be required to achieve the finished lower basement (Basement 2) floor reduced level (RL) at 16.7m, with deeper excavations locally for the proposed lift overrun pit. The basement will be set back a minimum of about 2m from the eastern (street) and southern site boundaries, and 10m and 15m from the western and northern site boundaries, respectively. We have assumed that typical structural loads for this type of development apply.

The purpose of the assessment was to obtain geotechnical information on subsurface conditions from a desktop study, as a basis for preliminary comments and recommendations on excavation conditions, excavation support, shoring/retaining walls, footings, on-grade floor slabs, as well as for a geotechnical investigation.

2 ASSESSMENT METHODOLOGY

We carried out a visual survey of the site from the street frontages to obtain an appreciation of the interface between the site and neighbouring buildings, and of the topographic and drainage setting.

A desktop study of previous geotechnical investigations we have carried out in the immediate vicinity was also conducted. The sites for which relevant geotechnical information was available included:

- St Patrick's Primary School (immediately to the north of the subject site).
- Proposed new Integrated Oncology Centre (over the north east of the subject site).
- Proposed Rose Cottage Extension (within the southern portion of St George Hospital to the north east).
- Moorefield Girls' High School (across Princes Highway to the east).
- Investigation for several new buildings within St George Hospital (some distance east to the north).

The boreholes and test pit locations for the above initial four investigations are indicated on the attached Site Layout Plan (Figure 1).

A summary of our observations is presented in Section 3 below. The anticipated subsurface conditions are presented in Section 4, and our preliminary comments and recommendations regarding the proposed development are discussed in Section 5.

3 SITE DESCRIPTION

The site is located over the southern portion of the St Patrick's Primary School in topography which slopes locally gently down to the south-east and extends between Princes Highway along the east and Chapel Street along the west.

At the time of the assessment, a two and three storey framed brick school building occupied the western portion of the site, a two storey brick building with a basement over its eastern end occupied the southern portion, and playing courts and garden terraces occupied the north-eastern portion. Asphaltic concrete (AC) and grassed areas occupied the remainder of the site. Trees up to 10m high were located along the site boundaries and within the garden terraces. Sandstone

bedrock outcropped in the garden terraced area and was assessed to be distinctly weathered and of medium strength.

A brick wall supported the eastern end of the site up to about 2m above Princes Highway. A school Hall and a two storey Presbytery were located beyond the eastern and western end of the northern site boundary, respectively. Three storey unit buildings were located beyond the southern and south-western site boundaries.

4 SUBSURFACE CONDITIONS

The 1:100,000 geological map of Sydney indicates that the site is underlain by Hawkesbury Sandstone. The subsurface conditions established during previous investigations, of the subject site and the surrounding area, comprised surficial fill over residual silty clays, then shale and/or sandstone bedrock from relatively shallow depth.

A brief description of the subsurface conditions from the previous investigations is presented below:

- Sandy fill about 0.5m deep and up to 0.8m deep locally, appears to cover the area.
- Residual silty clays of medium and high plasticity and hard strength were encountered below the fill to depths between about 1m and 2.5m.
- Weathered shale bedrock of limited thickness was encountered below the residual silty clay over the higher lying areas to the north-west. The shale was generally extremely weathered and of extremely low and very low strength.
- Weathered sandstone bedrock was encountered below the shale at depths of about 3m to 4.5m over the high lying north-west areas and beneath the residual silty clay at depths between 1m and 2m over the lower lying south-eastern zones. The sandstone was generally of very low to low strength on first contact, improving to low to medium and medium to high strength with depth. Medium or higher strength sandstone was encountered at depths between 4.6m and 5.1m below the north-eastern portion of the site.
- Groundwater was not encountered in any of the boreholes drilled. We note, however, that the groundwater levels may not have stabilised in the limited observation period. Long term groundwater monitoring was not carried out.

5 PRELIMINARY COMMENTS AND RECOMMENDATIONS

The comments and recommendations which follow are of necessity preliminary in nature. Following completion of a geotechnical investigation (refer Section 5.6 below), these recommendations should be reviewed and revised as necessary to suit the specific site conditions encountered.

5.1 Excavation Conditions

5.1.1 Excavation Methods

The results of our assessment indicate that the site is characterised by gently sloping terrain which is covered with soil and shale/sandstone bedrock is inferred to be at relatively shallow depth.

The soil cover should be readily excavatable using conventional earthworks equipment (eg. hydraulic excavators). The underlying weathered shale and weathered sandstone of extremely and very low strength, may also be excavated by a large bucket excavator, possibly with some ripping. However, we expect excavation of low to medium and higher strength sandstone would be most effectively excavated using hydraulic impact rock hammers. This equipment would also be required for breaking up boulders or blocks, for trimming rock excavation side slopes, and for detailed rock excavations (such as for footings or buried services).

Based on our review of the available information, it appears that the bulk excavation over the lower south-eastern portion of the site falls into the category which can be excavated by a large bucket excavator, possibly with some ripping. However, over the higher lying north-western portion, the lower zones of the proposed bulk excavation may encounter sandstone bedrock which requires the use of hydraulic impact rock hammers for effective excavation.

5.1.2 Excavation Techniques

We recommend that considerable caution be taken during rock excavation on this site as there will likely be direct transmission of ground vibrations to adjoining buildings and structures. The proposed bulk excavation will be within approximately 10m from the residential unit buildings to the south and south-west, and within approximately 20m of the Presbytery and school Hall buildings to the north. Prior to excavation commencing, detailed dilapidation reports should be compiled on the buildings and structures to the north and south, and the owners asked to confirm

that the reports present a fair record of existing conditions. The dilapidation reports may then be used as a benchmark against which to assess possible future claims for damage arising from the works. The excavation procedures and the dilapidation reports should be carefully reviewed prior to excavation commencing, so that appropriate equipment is used. We recommend on ongoing vibration monitoring be carried out during rock excavations.

We recommend that excavations with hydraulic rock hammers be preceded by providing a vertical saw cut slot along the perimeter of the proposed excavation, and maintaining the base of the slot at a lower level than the adjoining rock excavation at all times. The excavation with hydraulic hammers should preferably commence over the central portion of the site (ie. away from likely critical areas) using a moderately sized excavator fitted with a relatively low energy hydraulic hammer no larger than a Krupp 900 size, or equivalent. Subject to review of the dilapidation reports, we recommend that vibrations, measured as Peak Particle Velocity(PPV) on the neighbouring buildings, be limited to no higher than 8mm/sec. If it is found that transmitted vibrations are excessive, then it would be necessary to change to a considerably smaller rock hammer or to use alternative excavation techniques. Alternative excavation techniques which will significantly reduce vibrations, include the use of a rotary grinder or grid sawing in conjunction with ripping and/or hammering. When using a rock saw or rotary grinder, the resulting dust must be suppressed by spraying with water.

The following procedures are recommended to reduce vibrations if rock hammers are used:

- Maintain rock hammer orientated towards the face and enlarge excavation by breaking small wedges off the face.
- Operate one hammer at a time and in short bursts only to reduce amplification of vibrations.
- Use excavation contractors with experience in confined work with a competent supervisor who is aware of vibration damage risks, possible rock face instability issues, etc. The contractor should be provided with a copy of this report and have all appropriate statutory and public liability insurances.

5.1.3 Seepage

Although groundwater was not encountered during any of the investigations carried out in the vicinity, we would expect some minor groundwater seepage flows may occur from the base of the fill, at the soil-rock interface and through joints and bedding planes within the completed cut faces, particularly after periods of heavy rain. Seepage, if any, during excavation is expected to be satisfactorily controlled by conventional sump pumping or gravity drainage systems.

We recommend that groundwater seepage into the excavation be monitored by site personnel and the results (ie volume, source, location etc) be provided to the hydraulic and geotechnical engineers so that any unexpected conditions can be timeously addressed.

5.2 Excavation Support

Where space permits, excavations in the soils and extremely weathered bedrock may be temporarily battered to a side slope no steeper than 1 Vertical (V) in 1 Horizontal (H). On the basis of the provided architected drawings and assessed subsurface profile, it would appear that temporary batters can probably only be accommodated over the northern excavation face and the northern end of the western excavation face. However, possible seepage may cause localised instability of the soil batters, and allowance should be made for sand bagging. Conventional retaining walls may then be constructed at the toe of the batters and subsequently backfilled.

Where temporary batters cannot be accommodated, or where they are not preferred, a retention system will be required and should be installed prior to excavation commencing. Given the assessed subsurface conditions, a suitable retention system will include a soldier pile wall (with shotcrete infill panels) which is progressively anchored as excavations proceed.

We expect that good quality sandstone of low or higher strength may be cut vertically. However, localised stabilisation measures may be necessary if adverse defects (such as inclined joints or bedding) are found. Treatment for zones requiring stabilisation may include rock bolting, shotcreting, underpinning, etc. Clay seams occurring in permanently exposed sandstone slopes may also require ‘dental’ treatment. We therefore recommend that the rock face be progressively inspected by a geotechnical engineer/engineering geologist as excavation proceeds (ie. at approximately 1.5m depth intervals) to identify adverse defects and to propose appropriate stabilisation measures. Provision should be made in the contract documents (budget and program) for the above inspections and stabilisation measures. The extent of the stabilisation measures which may be required cannot be determined from the available information.

The geotechnical investigation recommended in Section 5.6 below will provide a better indication, however, the extent of required stabilisation measures can only be finalised during the cut face inspections.

5.3 Shoring/Retaining Walls

As indicated in Section 5.2 above, a soldier pile wall with concrete infill panels may be used to support vertical cuts in soils and weathered bedrock. The piles can be used as load bearing piles for the building, if taken down to appropriate founding depths. Toe restraint may be provided by suitably socketing the piles below the base of the excavation. Lateral restraint of the piles within the excavation, in the form of rock anchors or internal props, will be required and must be installed progressively with depth. Construction of the soldier pile walls should be of high quality. Gaps or 'windows' between adjacent soldier piles should be infilled without delay so as to reduce shrinkage of clay soils immediately outside the excavation.

The major consideration in the selection of earth pressures for the design of the shoring/retaining walls is the need to limit deformations occurring outside the excavation. The following characteristic earth pressure coefficients and subsoil parameters may be adopted for a static design of temporary or permanent retention systems:

- For conventional free-standing cantilever walls supporting areas where movement is of little concern (such as landscape walls supporting garden or open areas), should be designed using a triangular lateral earth pressure distribution and an 'active' earth pressure coefficient, K_a, of 0.3, for the soil profile and extremely weathered bedrock, assuming a horizontal retained surface.
- Cantilever walls, the tops of which will be restrained by the floor slabs of the main structure or which support movement sensitive elements (possibly retaining the ground above the basement entry ramp) should be designed using a triangular lateral earth pressure distribution and an 'at rest' earth pressure coefficient, K_o, of 0.6, for the soil profile and extremely weathered bedrock, assuming a horizontal retained surface.
- A bulk unit weight of 20kN/m^3 should be adopted for the soil profile and extremely weathered bedrock profile.
- Anchored or internally propped walls where minor movements can be tolerated (ie. the major portion of the remaining excavation), should be designed using a trapezoidal earth pressure distribution of 6H kPa for soil profile and extremely weathered bedrock, where 'H'

is the retained height in metres. These pressures should be assumed to be uniform over the central 50% of the support system.

- Anchored or internally propped walls which are supporting areas which are highly sensitive to lateral movement (such as where movement sensitive buried services are in close proximity) should be designed using a trapezoidal earth pressure distribution of 8H kPa for soil profile and weathered bedrock, where 'H' is the retained height in metres. These pressures should be assumed to be uniform over the central 50% of the support system.
- Any surcharge affecting the walls (eg. traffic loading, construction loads, etc) should be taken into account in the design using the appropriate earth pressure coefficients from above.
- The retaining walls should be provided with permanent and effective drainage of the ground behind the walls. Subsurface drains should incorporate a non-woven geotextile fabric (eg. Bidim A34) to act as a filter against subsoil erosion.
- For toe restraint design, an allowable lateral resistance of 200kPa for low or higher strength bedrock may be adopted, subject to inspection by a geotechnical engineer. The upper 0.3m depth of the pile socket or wall key should not be taken into account to allow for tolerance and disturbance effects during excavation.
- For rock anchors which extend beyond the site boundaries, the permission of the neighbours should be obtained before installation. Anchors bonded at least 3m into low or higher strength bedrock may be designed for an allowable bond stress of 200kPa. All anchors should be proof-tested to 1.3 times the working load under the direction of an experienced engineer. We assume that permanent lateral support of the basement walls will be provided by the new structure. If not, permanent anchors will be required which should be designed for corrosion resistance and for long term durability.

5.4 Footings

We expect bedrock will be exposed over the base of the bulk excavation. The proposed new building may therefore be supported using strip and/or pad footings founded on at least low strength bedrock, and may be designed using an allowable bearing pressure of 1,000kPa. Where the building footprint extends beyond the basement, piles will be required and should be founded below an imaginary line extending up 45^0 from the toe of the basement excavation. The above piles as well as the perimeter shoring piles founded on at least low strength bedrock may also be designed for an allowable end bearing pressure of 1,000kPa when socketed at least 0.3m into the

bedrock. In addition, an allowable shaft adhesion value of 100kPa (in compression) may be adopted over that length of rock socket in excess of 0.3m into bedrock.

The initial stages of footing excavation and pile drilling should be witnessed by a geotechnical engineer to confirm that an adequate founding material has been exposed.

We recommend that pad/strip footings be excavated, cleaned, inspected and poured with minimal delay to avoid deterioration. If delays in pouring concrete are anticipated, we recommend the base of the footings be protected with a blinding layer of concrete. Water should be prevented from ponding in the base of footings as this will tend to soften the foundation material, resulting in further excavation and cleaning being required. Groundwater inflow should be anticipated into bored pile excavations, however, we expect that this inflow would be controllable by conventional pumping methods. The bored piles should be drilled, cleaned, inspected and poured with minimal delay (ie. all on the same day).

5.5 On-Grade Floor Slabs

The lower basement floor slab will most probably directly overlie bedrock. Underfloor drainage is therefore required and should comprise a strong, durable, single size, washed aggregate (eg. 'blue metal' gravel). The underfloor drainage should connect with the wall drains and lead groundwater seepage to a sump for pumped disposal to the stormwater system. Joints in the on-grade concrete floor slab should be designed to accommodate shear forces but not bending moments by using dowelled or keyed joints.

5.6 Geotechnical Investigation

We recommend that a comprehensive geotechnical investigation of the site be carried out in order to establish the site specific subsurface conditions. The recommendations and comments detailed above should then be reviewed and revised as appropriate. We recommend that eight boreholes be drilled to depths of at least 2m below bulk excavation level. Four of the boreholes should be core drilled in order to obtain information on the bedrock quality (fracturing, jointing, etc) in addition to strength. A layout of the proposed borehole locations is presented in Figure 2.

5.7 Further Geotechnical Input

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

- Geotechnical investigation of the site.
- Dilapidation surveys of neighbouring buildings to the north and south.
- Quantitative vibration monitoring during rock excavations.
- Monitoring of groundwater seepage into excavation.
- Proof-testing of rock anchors.
- Geotechnical inspections during initial pile drilling and footing excavation.

6 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 preliminary advice on geotechnical aspects for the proposed civil and structural design. As part of the documentation stage of this project, Contract Documents and Specifications should not be prepared based on our report. Also, 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 preliminary 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.

Imaged sourced from *'Google Maps'*.

Site Layout Plan

○ Available Borehole and Test Pit Locations

To be read in conjunction with text of report.

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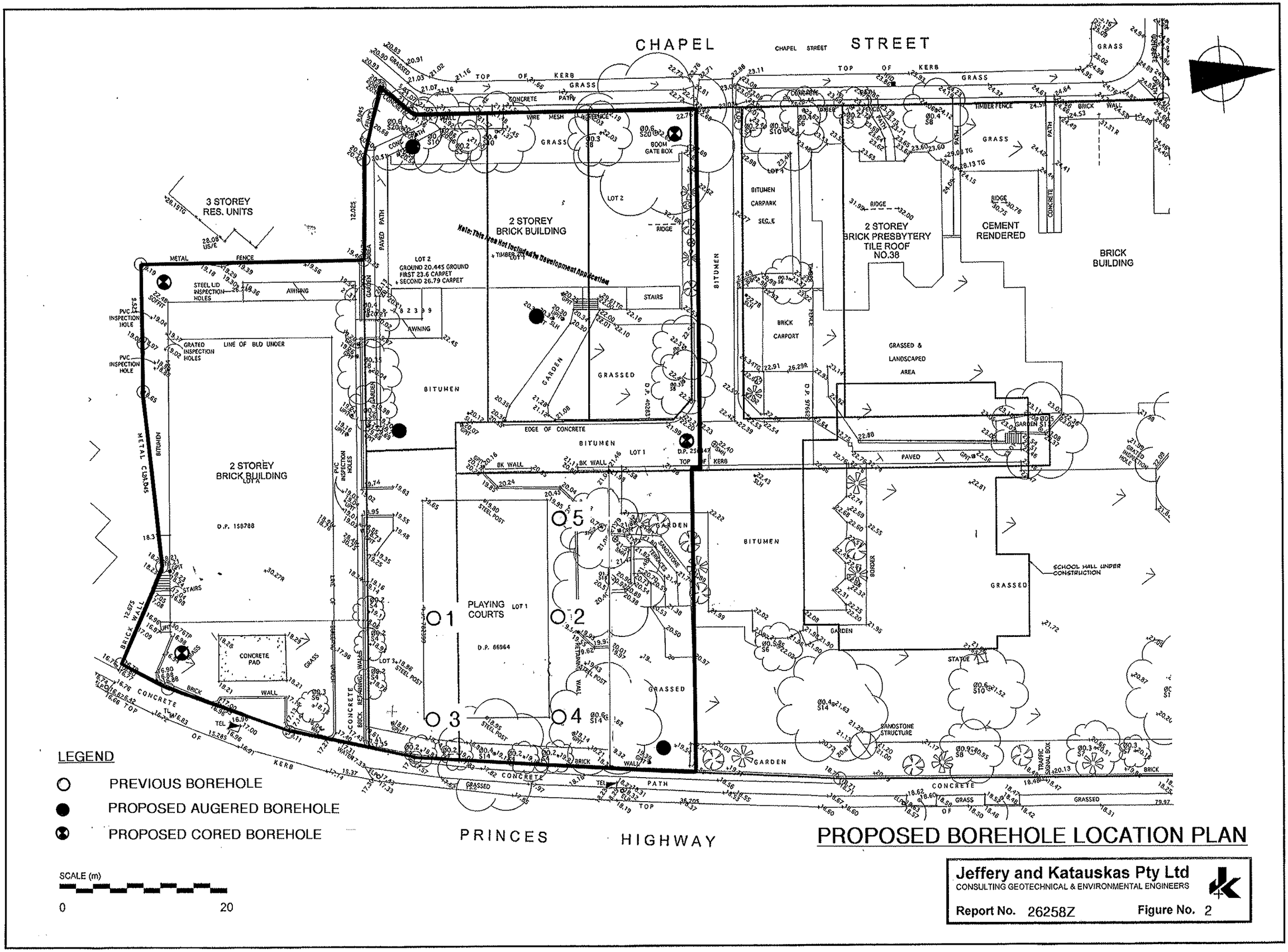

PROPOSED BOREHOLE LOCATION PLAN

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Report No. 26258Z Figure No. 2