REPORT

TO
KARIMBLA CONSTRUCTION SERVICES (NSW) PTY LTD

ON
PRELIMINARY GEOTECHNICAL ASSESSMENT

FOR
PROPOSED RESIDENTIAL DEVELOPMENT

AT
100 BENNELONG PARKWAY
SYDNEY OLYMPIC PARK, NSW

22 April 2014
Ref: 27007ZTrptRev1

JK Geotechnics
GEOTECHNICAL & ENVIRONMENTAL ENGINEERS

PO Box 976, North Ryde BC NSW 1670
Tel: 02 9888 5000 Fax: 02 9888 5003
www.jkgeotechnics.com.au

Jeffery & Katauskas Pty Ltd, trading as
JK Geotechnics ABN 17 003 550 801

Date: 22 April 2014
Report No: 27007ZTrptRev
Revision No: 1

Report prepared by:

Tony Walker
Associate

Report reviewed by:

Agi Zenon
Principal Geotechnical Engineer

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

This Report (which includes all attachments and annexures) has been prepared by JK Geotechnics (JK) for its Client, and is intended for the use only by that Client.

This Report has been prepared pursuant to a contract between JK and its Client and is therefore subject to:

a) JK's proposal in respect of the work covered by the Report;

b) the limitations defined in the Client's brief to JK;

c) the terms of contract between JK and the Client, including terms limiting the liability of JK.

If the Client, or any person, provides a copy of this Report to any third party, such third party must not rely on this Report, except with the express written consent of JK which, if given, will be deemed to be upon the same terms, conditions, restrictions and limitations as apply by virtue of (a), (b), and (c) above.

Any third party who seeks to rely on this Report without the express written consent of JK does so entirely at their own risk and to the fullest extent permitted by law, JK accepts no liability whatsoever, in respect of any loss or damage suffered by any such third party.

TABLE OF CONTENTS

1 INTRODUCTION

This report presents the results of our preliminary geotechnical assessment of the site for the proposed residential development at 100 Bennelong Parkway, Sydney Olympic Park, NSW. The Director for Planning and Development, Walter Gordon, commissioned the assessment on behalf of Karimbla Construction Services (NSW) Pty Ltd by returning the signed Acceptance of Proposal form dated 4 November 2013. The assessment was carried out on the basis of our proposal (Ref P37886ZN) dated 4 November 2013. This revised report incorporates revisions to the architectural drawings previously provided to us.

We have been provided with architectural drawings (Project No. 13056 Drawing Nos A-DA-100-000E, A-DA-100-001E, A-DA-110-000R, A-DA-110-001O, A-DA-110-801P, A-DA-110-802O, A-DA-110-803M, A-DA-310-001E and A-DA-310-002E) prepared by Turner (Architects). From these drawings, we understand that the proposed development, following demolition of existing site improvements, will comprise three multi-level buildings separated by a new road. Each building is to be constructed over an in-ground basement. Buildings A and B will be adjacent to the rail corridor, with two common carparking basement levels below the entire building footprints. A third basement level (Parking Level 3) is proposed below the northern portion of the common basement. Hence the basement will have a floor level which will step from a reduced level (RL) of 1.48m AHD down to RL-2.58m. The third building (Building C), which is to be in the eastern portion of the property, will be U-shaped in plan with a pool in its south-east corner and a courtyard infill on its eastern side extending to the street frontage. The car parking ground floor level and Parking Level 1, which is to be generally at RL4.8m, will extend below the building and courtyard areas. The lower basement level, which will step down from RL2.0m on its western side to RL0.6m on its eastern side, will generally run below the western arm of the building. We estimate that bulk excavations would be of the order of 9-12m for construction of the western basement and would grade down to around 7m (maximum) at the eastern (Building C) basement. Locally deeper excavation would be required for footing and service trenches in the basements. We have assumed that typical structural loads for this type of development apply.

The purpose of the assessment was to conduct a desktop study and brief site visit, as a basis for preliminary comments and recommendations on excavation conditions, shoring, retaining walls, footings and on-grade floor slabs.

2 ASSESSMENT PROCEDURE

This preliminary assessment comprised a review and research of relevant geotechnical and geological information contained in our database of previous geotechnical investigations we have carried out in the general site area. A brief site visit was also carried out to assess the topographic and drainage setting, and the interface with adjoining buildings and structures. No subsurface investigation on the site itself has been carried out as part of this preliminary assessment.

3 SITE DESCRIPTION

The surrounding topography was characterised by undulating terrain with low hills to the west, low lying areas and hills to the east. The site itself was located within the lower slopes of a hill which sloped moderately towards the east to the Bennelong Parkway frontage. Previous earthworks have been carried out on and adjacent to the site, forming terraces down the hillside slope to accommodate the existing developments.

At the time of our assessment, the site contained a two storey concrete framed office building with a narrow undercroft parking area over the southern portion of its eastern side, and an attached warehouse on its western side. The warehouse was of steel portal frame construction with concrete panel and masonry block walls. The office building was also bounded to the south by a two level carpark comprising a suspended upper concrete slab supported on concrete columns and masonry block walls. The adjacent concrete hardstand area to the west was at a similar level to the upper level of the carpark and the floor of the warehouse. There was a narrow strip of land between the hardstand area and the boundary comprising a shallow batter which graded in height from its southern end to around 1.5m near the warehouse. The flat area at the crest of the batter was lined with small trees. The area to the west of the warehouse was flat with a few trees and storage tanks. The northern side of the warehouse and the office building was also a relatively flat and concrete paved. However, the area immediately adjacent to the office building was brick paved. This brick paving continued across part of its eastern side to nearby steps leading down to a lower level of the site. There were gardens adjacent to the brick paving retained by an approximately 3m high, tiered timber log wall.

A driveway, surfaced with asphaltic concrete (AC), was located on the northern side of the site providing access from the eastern side of the northern concrete hardstand area down to the street frontage and to two at-grade carparking levels. One other road ran from the driveway past the tiered garden and steps towards the south past the undercroft to access to the lower level of the suspended carpark. This road was bounded by the upper at-grade carpark to the east. Both the road and the carpark were surfaced with AC. There was a concrete ramp at the southern end of this road leading up to the upper suspended carpark level. The southern edge of the ramp was bounded by a concrete wall which retained the ground to the south. The eastern edge of the at-grade carpark was at the crest of a moderate to steep batter that fell around 2m in level down to the lower at-grade, AC paved carpark. The southern end of this lower carpark was located at the toe of a steep batter, partly retained by a small timber plank and post wall.

The neighbouring property to the north was occupied by two office buildings with an upper level AC carpark opposite the western end of the site between the upper level building, and the boundary. We estimate that the carpark to be around up to 1.5m higher than the site. There was also an AC driveway and carpark between the lower building and the boundary. This appeared to be around 1-1.5m lower than the northern hardstand area on the subject site. There were trees along the common boundary.

The area to the east of the lower level carpark on the subject site was covered with numerous medium sized trees and sloped moderately to steeply falling around 2-3m in level down to the footpath and grassed nature strip along Bennelong Parkway. Opposite the site, the road was relatively flat. The western side of the site was bounded by the rail corridor, which ran from the south towards the north, then turned to the north-west near the north-west corner of the warehouse. The boundary was lined by an approximately 2.8m high concrete panel wall opposite much of the southern hardstand area continuing into the property further to the south. The northern end was bounded by a steel soldier pile wall with concrete infill panels that continued along the remainder of the boundary and further to the north-west. There was a multi-level building under construction uphill to the west of the rail corridor. The neighbouring site to the south contained a detention pond, several metres lower than the site and bounded by moderately steep grassed batters.

4 GENERAL GEOLOGY AND INFERRED SUBSURFACE CONDITIONS

Reference to the 1:100,000 Geological Map of the Sydney Region indicates that the site is underlain by Ashfield Shale (black to dark grey shale and laminite) of the Wianamatta Group. This subsurface profile does not take into account the soils derived from insitu weathering of the shale or earthworks that have previously been undertaken at the site. The map boundary with man-made fill/Quaternary stream alluvial and estuarine deposits (consisting of silty to peaty quartz sand, silt and clay) is shown on the map to run along the eastern boundary.

Our review of the available geotechnical information obtained from our investigation along the rail corridor at the uphill end of the subject site revealed shallow fill overlying residual clays of high plasticity, grading into weathered shale bedrock. The shale was around 2m depth below the ground surface, which has since been modified by the rail construction and the site development. The top of the shale appeared to follow the ground surface contours along the line of the rail corridor at that time. The shale was variable in weathering and initially in the extremely low to very low strength range. The shale improved in quality with depth. Medium strength or better shale was encountered after about 3m penetration of the shale. We surmise that the buried shale surface steps down towards the east.

We have also witnessed the drilling of numerous piled footings in 1999 on a nearby building site about 100-150m to the north (fronting Parkview Drive). The shale was encountered in piled footing excavations typically at depths between 2.0m and 3.5m, that is, between about RL5.0m and RL7.0m at that time.

In April 2000, JK Geotechnics carried out an investigation at Bicentennial Park Visitors Centre, which is located on the opposite side of Bennelong Parkway about 100m or so to the north-east. This investigation revealed fill, very soft to soft, then firm to stiff organic clays, over an interbedded sequence of medium dense to very dense silty sands, sands and clayey sands (probably alluvial deposits) and very stiff to hard silty clays above the inferred bedrock profile at depths between 4.3m and 4.7m.

5 PRELIMINARY COMMENTS AND RECOMMENDATIONS

The proposed development will involve substantial changes to the site involving excavations of substantial volumes of the inferred fill, residual silty clays, and weathered shale bedrock. We consider that the site would be suitable for the proposed construction provided good engineering design, construction and maintenance practices are be adopted to maintain stability during excavation and in the long term. We recommend that the following design and construction safeguards be included in this development. These recommendations are of a preliminary nature suitable only for preliminary design and planning purposes and should be reviewed prior to Construction Certificate after completing site specific subsurface investigations.

5.1 Dilapidation Survey and Adjoining Infrastructure

Prior to commencement of the bulk excavation, we recommend that dilapidation survey reports be carried out on the boundary structures in the neighbouring rail property to the west. The report would provide a record of existing conditions prior to commencement of the work. The neighbouring property owner should be provided with copy of the dilapidation report for the property and should also be asked to confirm that it represents a fair assessment of existing conditions. The report may be used as a benchmark against which potential claims for damage as a result of the works may be assessed. The reports should be carefully reviewed by the structural engineer prior to excavation commencing as there may specific issues to be taken into account in design of the excavation support systems.

Excavations and retention systems will need to be carefully planned and scheduled so as not to have any adverse effects on the nearby structures adjoining or above the excavation. During the excavation, every care should be taken to not undermine or render unstable the footings of adjoining structures and to maintain stability in the long term.

5.2 Excavation and Groundwater

5.2.1 Excavation Methods

Following demolition and stripping, we infer that the materials to be excavated will comprise existing fill, residual silty clays and weathered shale bedrock. The underlying fill and soils should be readily excavated by conventional earthworks plant (e.g. small to medium excavator, dozer blade, etc). Excavation in extremely low to very low strength shale could easily be achieved using a Caterpillar D9 tractor or equivalent, probably with some light to medium ripping. Much of this material can probably also be excavated with a large excavator bucket. Localised stronger bands/zones may require heavy ripping or the use hydraulic rock hammers.

We expect that excavation of the low to medium strength or stronger rock will present hard ripping or “hard rock” excavation conditions. Our recommendation would be to excavate within the low to medium strength or stronger rock with rock saws, then by ripping tynes on medium to heavy excavators or dozers. Ripping may only just be possible with a Caterpillar D10 or D11 dozer and a very generous allowance should be made for rock hammer assistance to the ripping. The use of an impact ripper is recommended. Excavation production rates are likely to be very low and shoe wear rates high. Alternatively, hydraulic rock breaking equipment will be required for productive effective excavation. This equipment would also be required for detailed excavations such as footings or services, for which rotary grinders would also be useful. The ease with which excavation of rock is achieved depends upon the equipment used, the skill and experience of the operator and the characteristics of the rock. The contractor must make his own judgement on all of these factors.

5.2.2 Potential Vibration Risks

If a hydraulic impact hammer is used, considerable caution should be taken during rock excavation, as there will likely be direct transmission of ground vibrations to adjoining structures and buildings.

Guideline levels of vibration velocity for evaluating the effects of vibration in structures are given in the attached Vibration Emission Design Goals sheet. We recommend that the acceptable limit for transmitted vibrations be set once more definite details of the contractor's excavation methods and sequence and development staging are known to confirm that they are still suitable. Given the close proximity of the neighbouring rail corridor, a vibration monitoring management plan should be prepared for the site.

Rock hammers should only be used under close supervision and with instrument vibration monitoring. If large rock hammers are to be used, we recommend that the initial excavation in rock should preferably be commenced away from likely critical areas and instrument vibration monitoring undertaken to confirm whether the vibration limits are likely to be exceeded and to provide guidance on how far the rock hammer should be kept away from the site boundaries and boundary structures. By monitoring vibrations in this way, it will allow some freedom to the excavation contractor in the equipment he adopts, so that a balance can be made between productivity and vibration reduction.

If it is found that transmitted vibrations are unacceptable, which may occur with a 1000kg hammer (Krupp 960) within about 10m to 15m of existing structures, it may be necessary to change to a small to medium sized excavator fitted with a light rock hammer no larger than 600kg (e.g. Krupp 350 size), or to a rotary grinder.

Vibrations induced by excavations can be reduced by alternative methods such as the following:

- Start the rock excavation away from likely critical areas.
- Maintain rock hammer orientation into the face and enlarge excavation by breaking small wedges off faces.
- Operate one hammer at a time and in short bursts only, to reduce amplification of vibrations.
- Use smaller equipment (offset by a loss in productivity and economy and greater duration of the nuisance).

- Use line drilling, especially along excavation boundaries, to aid breaking and trimming.
- Sawing or excavating a trench (with low energy equipment) along the sides of the excavation using a rock saw and then carry out bulk excavation with a large (Caterpillar D11) dozer or impact ripper.

In addition, we recommend that only excavation contractors with appropriate insurances and experience on similar projects be used.

5.2.3 Groundwater Seepage

The proposed basement excavations would be expected to encounter at least some seepage, especially during or following rainfall periods. We expect that localised seepage may possibly occur into the excavation along the soil/bedrock boundary and along existing defects, such as bedding planes and joints, which exist in the rock. Given the expected low permeability of the bedrock profile, we consider that construction of a drained basement design would be feasible and appropriate. Groundwater seepage into the basement excavations would be expected to reduce as the excavation progresses, and the surrounding profile is drained of water. Locally higher inflows could be expected to occur through open joints or bedding planes during and following heavy rainfall events. Such a process would not be expected to cause any adverse effects on any surrounding structures or improvements. Seepage volumes into the excavations are expected to be controlled by the use of surface drains and sump and pump systems at basement level during construction.

An assessment of likely seepage, its quality, and required pumping capacity would best be made during and following completion of the bulk excavation, when seepage could be observed.

We recommend that complete and permanent drainage be provided behind the basement walls. In addition, drainage should be provided below the basement floor slabs to safeguard against the possibility of groundwater pressure causing an uplift pressure and to drain water charged rock defects. If under-floor drainage is not installed, then the basement floor slab will be subjected to uplift pressures from the groundwater; this may require additional mass or ground anchors. The inferred relatively minor to modest groundwater flows would be able to drain through a free draining gravel bed or perimeter and cross drains below the floor slabs. The piped drains should be graded to sumps with an automatic fail-safe pump system for discharge of collected seepage to the stormwater system. Appropriate damp-proofing will also be required for the permanent walls in contact with the excavated areas.

5.3 Temporary Excavation Batters

During the excavation, every care should be taken to not undermine or render unstable the footings of any adjoining structure.

We recommend temporary batters in the fill, silty clay and low strength or weaker shale should generally be cut temporarily to a safe batter no steeper than 1 Vertical (V) in 1 Horizontal (H) provided surcharge loadings (footings, vehicles, stockpiles etc) are not located within the zone of influence of the excavations. As a guide, surcharge loadings should be no closer than 2H from the top of any batter or the face of any excavation (including footing excavations), where H is the vertical height of the batter or depth of the excavation in the fill, silty clay and very low strength or weaker rock. Flatter batters or shoring would be required where groundwater is encountered; further geotechnical advice should be requested if such a case exists before proceeding deeper with the excavation.

Where the batter slopes cannot be accommodated, or are not preferred, then the vertical excavation in the fill, clays and shale should be supported by appropriate shoring systems or properly engineered retaining walls, (e.g. soldier pile walls or contiguous pile walls), with due allowance for the slope of the ground behind the walls. These necessary vertical support systems will need to be installed prior to excavation. The piles of the shoring walls should be suitably embedded below the base of the excavations. Props or anchors may also be needed to restrain the upper sections of the walls and these must be installed progressively and immediately once the propping point has been uncovered, and prior to excavation adjacent to neighbouring structures and sensitive services which are located within the 2H zone of influence of the excavation perimeter.

The western basement excavation below Buildings A and B is proposed not to have substantial set backs from the rail corridor and hence there will not be enough space to accommodate and complete the excavation with battered sides within the soils and the poorer quality (extremely low to low strength) shale profile, which appears to extend to around 4-5m depth below the upper portion of the site. As anchors would extend beyond the site boundary, propping of the shoring system will be required. It is unlikely that RailCorp will give permission to install anchor or rock-bolts, even if for only temporary short term construction period.

It is likely that good quality shale of at least medium strength between the shoring piles may be cut vertically and the face left temporarily unsupported. However, we forewarn that the shales in

Sydney do at times contain adversely orientated continuous joints which can cause instability of rock cuttings in shale, particularly where the joints are smooth, clay smeared or slickensided. These joints can be as flat as 35° to 55° and run in north-west/south-east or north-east/south-west or slight variations to these directions. These joints, if they exist at this site, are not readily identified from vertical boreholes, and are usually only observed once the shale faces are exposed by the excavations. Several inclined cored boreholes, drilled along each face of the excavation, may possibly establish the design requirements to support the cut faces above these persistent joints should they exist.

Should these joints exist, the shoring would need to be redesigned to resist an additional load generated by a wedge of rock. This may require the retrofit of significant anchors and possibly larger shoring piles to stabilise the rock wedge. One option would be to design and install the shoring piles for the additional load from a design wedge of rock, carry out progressive face inspections, and if necessary install the larger capacity anchors. As a preliminary guide, the design wedge of rock may be defined by the following:

- The wedge may be present anywhere in the vertical shale faces at the perimeter of the excavation;
- the top of the wedge would occur at the bedding in the shale and would be horizontal;
- the sides of the wedge would be vertical, assumed to be formed at vertical joints;
- the wedge exposed in the face would be at least 3m high and would extend at least 5-6m along the excavation perimeter;
- the persistent joint would radiate up from the base of the wedge at any angle between 35° to 55° above the horizontal; and
- the persistent joint is clay coated and smooth with a friction angle of 30°.

Allowance must be made for groundwater seepage flowing from the rock faces and permanent and effective drainage provisions should be incorporated into all areas to be covered with shotcrete or the permanent walls to discharge the seepage (e.g. geotextile enclosed strip drains). The drains should target defects from which seepage could flow (e.g. joints or bedding separations).

To manage the potential risks associated with these continuous joints and other defects in the shale faces, we recommend that the excavation should not be allowed to advance more than 1.5m vertically between geotechnical inspections and the excavation should be staged or stepped so that a whole face is not excavated 1.5m vertically between visits. These geotechnical inspections would establish whether inclined joints or other defects are present that would lead to horizontal loads on the shoring walls to be much higher than allowed for in the design of the shoring system. If adverse defects are identified by the geotechnical engineer during the inspections, then stabilisation or flatter batters will be required.

Where there appears to be only occasional defects in the fresh, medium to high strength rock, the face may only require to be protected by dowels, mesh and shotcrete or the permanent basement walls. The extent of shotcrete to temporarily protect the rock faces prior to construction of the permanent walls should be confirmed during the geotechnical inspections. Stabilisation may also require the use of additional temporary anchors or higher long term strut load capacity, rock bolts, mesh and/or shotcrete protection to support the rock. It would be unusual to complete such an excavation without some form of support being required to the rock faces, though this may take forms other than rock bolting.

Permanent batters in fill and soils should be no steeper than 1V in 2H, but flatter batters (of about 1V in 3H) may be preferred to allow access for maintenance of vegetation. Permanent batters should be covered with topsoil and planted with a quick growing grass to provide early protection, with a hardy, perennial grass for longer term protection. The grass cover would reduce erosion. All stormwater runoff should be directed away from all slopes to also reduce erosion.

Our recommendations of excavation and retention provided in this report should be complemented by reference to the Code of Practice - Excavation Work by Safe Work Australia issued in July 2012.

5.4 <u>Retaining Walls</u>

Shoring systems and retaining walls should be engineered to suit the site conditions, with the wall footings preferably founded into the underlying strata of adequate bearing capacity (see Section 5.5 below).

Design of cantilever retaining walls may be on the basis of triangular stress distribution and an ‘active’ lateral pressure coefficient, K_a, of at least 0.35 for the fill, the residual clays, and extremely low strength rock, provided some deflection is tolerable. The K value may be reduced to 0.2 for the rock of very low strength. A bulk unit weight of 20kN/m^3 for the soils and 22kN/m^3 for any extremely low to very low strength rock may be adopted. Walls which are to be subsequently propped by the permanent structure (e.g. by suspended floor slabs) should be designed based on a trapezoidal pressure distribution using higher lateral pressure coefficient, K, of at least 0.55 for the fill and clays or 0.4 for very low strength rock.

We advise that cantilevered walls are considered suitable for supporting retained soil heights of up to 4m to 5m and only where some lateral and vertical movements of adjoining ground can be tolerated. If greater height walls are proposed or where reduced movements are desired, then propped or anchored walls should preferably be used.

The recommended lateral earth pressure coefficients and pressures assume almost horizontal ground surfaces behind the crest of the walls. If inclined backfill surfaces are to be designed, then the above factors would have to be increased or the inclined section of backfill should be taken as a surcharge load in the design.

The good quality shale of at least medium strength can be taken to be self-supporting. However, we recommend that the retention system should be designed by assuming a nominal uniform lateral pressure of 10kPa to account for small-scale wedges of rock which could be isolated by inclined joints and horizontal bedding planes. The quality of the shale should be confirmed during geotechnical inspections during construction. Note that RailCorp may also require the shoring system to be designed to resist an additional load generated by a 45° planar failure of the rock. On the other shale faces, it would be prudent to design the shoring piles to resist wedge failure outlined in Section 5.3.

Applicable hydrostatic pressures should be added to the lateral earth pressures, unless specific measures are taken to introduce complete and permanent drainage of the ground behind the walls. Any surcharge affecting the walls (e.g. footings, retaining walls and their backfill, the ground slope behind the wall, etc.) should also be taken into account in the wall design.

The retaining walls should be designed as drained and allowance made for complete and permanent drainage of the ground behind the walls. Our preference for wall drainage, where walls are constructed within temporary safe batters, would be to use a proper subsoil drainage system incorporating a slotted pipe surrounded by a free-draining single sized crushed aggregate or alternatively, a proprietary drainage system. The aggregate should be appropriately protected by a non-woven, geotextile, filter fabric. Backfill behind retaining walls formed within temporary batters should be placed and compacted in a controlled manner to an engineering specification as soon as the wall is capable of providing support, to reduce the risk of slumps of the batter slopes and the complications that would cause. Caution will be required during backfilling to prevent over-compaction adjacent to the retaining wall and thereby causing excessive forces on the wall.

5.5 Footings

As shale bedrock will be exposed over all or part of the basement floors, we recommend that all footings for the buildings and basement walls be founded within the shale, preferably of the same quality, to provide uniform support and reduce the potential for differential footing settlements. Given the likely quality of the shale, strip and pad footings or bored piles or augered, grout inject (CFA) piles may tentatively be designed for a maximum allowable working bearing pressure of 3500kPa for the shale, assumed to be of at least low strength (Class III) shale as defined in the guidelines provided in “Foundations on Sandstone and Shale in the Sydney Region” by PJN Pells, G Mostyn and BF Walker published by the Australian Geomechanics in 1998. However, this bearing value must be confirmed by drilling diamond cored boreholes into the shale foundation below the founding levels of the footings. This testing is to confirm that no adverse seams or defects are present below the founding levels. The presence of such seams would require a reduction in allowable bearing capacity or an increase in footing depth.

Rock sockets in piled footings below the indicative founding level specified above may be designed for a safe adhesion value of 10% of the appropriate safe bearing pressure under compressive vertical loading. One half of the adhesion value may be used for uplift. These parameters assume excavation is not carried out within the zone of influence of the footing and sockets are free of smeared material (a special roughening tool is normally required to achieve this in bored piers).

Where footings are founded close to the top of a rock face, the allowable bearing pressure below these footings will need to be carefully assessed. The safe bearing pressure would need to take into account rock strength, the inclination of the rock face, jointing and the influence of clay seams as well as the magnitude and inclination of the applied loadings.

A lateral stress of one third of the rock bearing value may be adopted for sockets in the rock below basement floor level provided footing or service trenches or lift wells or other excavated faces are not located within the zone of influence of the footing and the rock does not contain unfavourable defects etc. The upper 0.3m depth of the socket should not be taken into account to allow for disturbance effects during excavation.

The bearing pressure given above is based on a serviceability criteria of deflections at the footing base/pile toe of less than 1% of the least footing dimension (or pile diameter).

If piles are to be socketed into the rock then we recommend that heavy drilling rigs with rock augers be used to drill the piles through medium strength or stronger rock or through the iron indurated bands. Groundwater inflow would be expected into some of the bored pile excavations and we expect that this will be controllable by conventional pumping methods. However, some contingency for pouring concrete by tremie methods should be allowed. Bored pile footings should be drilled, cleaned, inspected and poured with minimal delay, on the same day. 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.

The initial stages of footing excavation/drilling, particularly if bored piles are adopted, should be inspected by a geotechnical engineer/engineering geologist to ascertain that the recommended foundation material has been reached and to check initial assumptions about foundation conditions and possible variations that may occur between borehole locations. The need for further inspections can be assessed following the initial visit.

All footing excavations should be free from all loose or softened materials prior to placement of concrete. We recommend that all footing excavations be checked and approved prior to concrete being poured.

5.6 On-Grade Floor Slabs

Shale is anticipated to be exposed over the proposed lowest basement levels and no special treatment is required other than the removal of loose and softened material. Areas, which have to be built-up to infill low points in the excavation, should be filled with properly compacted sub-base material. Although we do not expect that very significant under-floor drainage will be required, this should be reviewed following further monitoring of groundwater seepage during and on completion of the excavations. The under-floor drainage (such as perimeter and cross drains, ‘rock-saw’ slots cut into the rock floor and/or a free draining gravel bed) should be installed with sumps for automatic pumped discharge of groundwater. If under-floor drainage is not installed, then the on-ground floor slab will be subjected to uplift pressures from the groundwater; this may require additional mass or ground anchors.

The basement floor slabs, where subject to traffic loadings, should have a sub-base layer of at least 100mm thickness of crushed rock to RTA QA specification 3051 unbound base material (or equivalent good quality durable fine crushed rock) which is compacted to at least 100% of Standard Maximum Dry Density (SMDD).

Sections of the basement below the eastern building may expose fill or residual silty clays at bulk excavation level. The slabs over relatively shallow fill or over the residual silty clays may be constructed as slab-on-grade provided adequate sub-grade preparation is carried out. Where the remaining fill is deeper than about 1m, it may require removal and replacement with engineering fill otherwise the basement slab should be designed as fully suspended.

Following completion of excavation, the exposed fill and clay subgrade should, if deemed necessary after inspection by the geotechnical engineer, be proof-rolled. Proof-rolling should be carried out with a minimum 5 tonne dead weight smooth drum vibratory roller. The final pass of the proof rolling should be carried out without vibration and within the presence of a geotechnical engineer or experienced geotechnican. The purpose of the proof rolling should be to improve the compaction of the near surface soils and to detect any weak or unstable areas. During proof rolling care should be taken to avoid damage to nearby structures and buried services by vibrations transmitted by the roller. If necessary, the vibrations should be reduced or ceased.

Where unstable areas are encountered, the area should be locally excavated down to a sound base and replaced with properly compacted engineered fill; allowance for this treatment should be made in design of shoring systems. Engineered fill should preferably comprise well-graded granular material (such as ripped or crushed sandstone), free of deleterious substances and having a maximum particle size of 75mm. The existing clay materials may be reused as engineered fill provided over-wet and over-size materials are removed. The engineered fill should be placed in 150mm loose thickness layers and compacted to between 98% and 102% of SMDD and if clays are used within 2% of Standard Optimum Moisture Content (SOMC). For backfilling confined excavations such as service trenches, a similar compaction to engineered fill should be adhered to, but if light compaction equipment is used then the layer thickness and maximum particle size should be limited to 100mm loose thickness and 40mm respectively. The use of clay soils for engineered fill will be more time consuming, weather dependent and will entail more rigorous earthworks supervision and compaction control than granular fill.

Density tests should be regularly carried out on the fill at not less than the frequencies given in AS3798 to confirm that the recommended compaction densities are achieved. At least Level 2 testing (or Level 1 if footings and floor slabs are to be founded in engineered fill) of earthworks should be carried out in accordance with AS3798. Preferably, the geotechnical testing authority should be engaged directly on behalf of the client and not by the earthworks subcontractor. Earthworks recommendations provided in this report should be complemented by reference to AS3798.

Due to the presence of the reactive clays, it will be important to pay careful attention to drainage during construction. The site may become untrafficable when wet and appropriate cross-falls should be maintained at all times. We recommend that if soil softening occurs, the subgrade be over-excavated to below the depth of moisture softening and that the excavated material be replaced with engineered compacted as specified above. Desirably, the earthworks should be completed rapidly and the surface sealed as soon as possible. It is preferred that a granular working platform be provided to cover and protect the underlying clay and provide a trafficable surface during construction. The earthworks should be carefully planned and scheduled to maintain cross-falls during construction.

5.7 Further Geotechnical Work

Our recommendations are of a preliminary nature and are based on subsurface information for the general site area. These need to be confirmed by completing geotechnical boreholes or insitu testing to clearly prove and test the subsurface profile prior to finalising the structural design for Construction Certificate and commencing construction. The boreholes and insitu testing are to confirm our inferences on the surface profile and to reduce the potential risk of design changes if adverse conditions are exposed and subsequent delays that it could cause. The investigation should comprise 10 boreholes taken to at least 3m below the basement floors but not less than that required for design of the shoring piles or piles to support the buildings. Standpipes should also be installed in three of these boreholes to allow for monitoring and sampling of the groundwater.

The following summarises the scope of other further geotechnical work recommended within this report. For specific details reference should be made to the relevant sections of this report.

- Complete dilapidation survey of the boundary structures along the rail corridor and assess its vulnerability of the structures to excavation-induced movement or vibration.
- Quantitative monitoring of transmitted vibrations during the use of rock hammers or vibratory rollers.
- Assessment of groundwater inflow to confirm drainage requirements following excavation.
- Inspection of the excavation to confirm rock face treatment for cuts in the shale.
- Witness proof loading of rock anchors installed in shoring walls.

- Inspection of footing excavations to ascertain that the recommended foundation has been reached and to check initial assumptions regarding foundation conditions and possible variations that may occur.
- Inspect the clay subgrade following excavation and witness the proof rolling of clay subgrade to detect soft spots requiring treatment.
- Density tests to control compaction of any pavement or engineered fill layers.

We recommend that advice be sought from RailCorp, who are likely impose some site specific design, construction or monitoring procedures as rail infrastructure is located within the zone of influence and above the proposed excavations. For example, RailCorp will most likely require numerical modelling to confirm the impact of the proposed excavation, dilapidation survey of rail assets, and survey monitoring of excavation induced movements during construction, etc.

We also recommend that JK Geotechnics view the proposed earthworks and structural drawings and section details in order to confirm they are within the guidelines of this report.

6 GENERAL COMMENTS

The preliminary recommendations presented in this report include specific issues to be addressed during detailed design and the construction phase of the project. In the event that site subsurface conditions are not confirmed or 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 may be prepared based on the results of the site specific investigations and 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 site specific investigations and advice has been obtained. If required, we could be commissioned to carry out the investigations and 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.

Ref: 27007ZT Figure 1

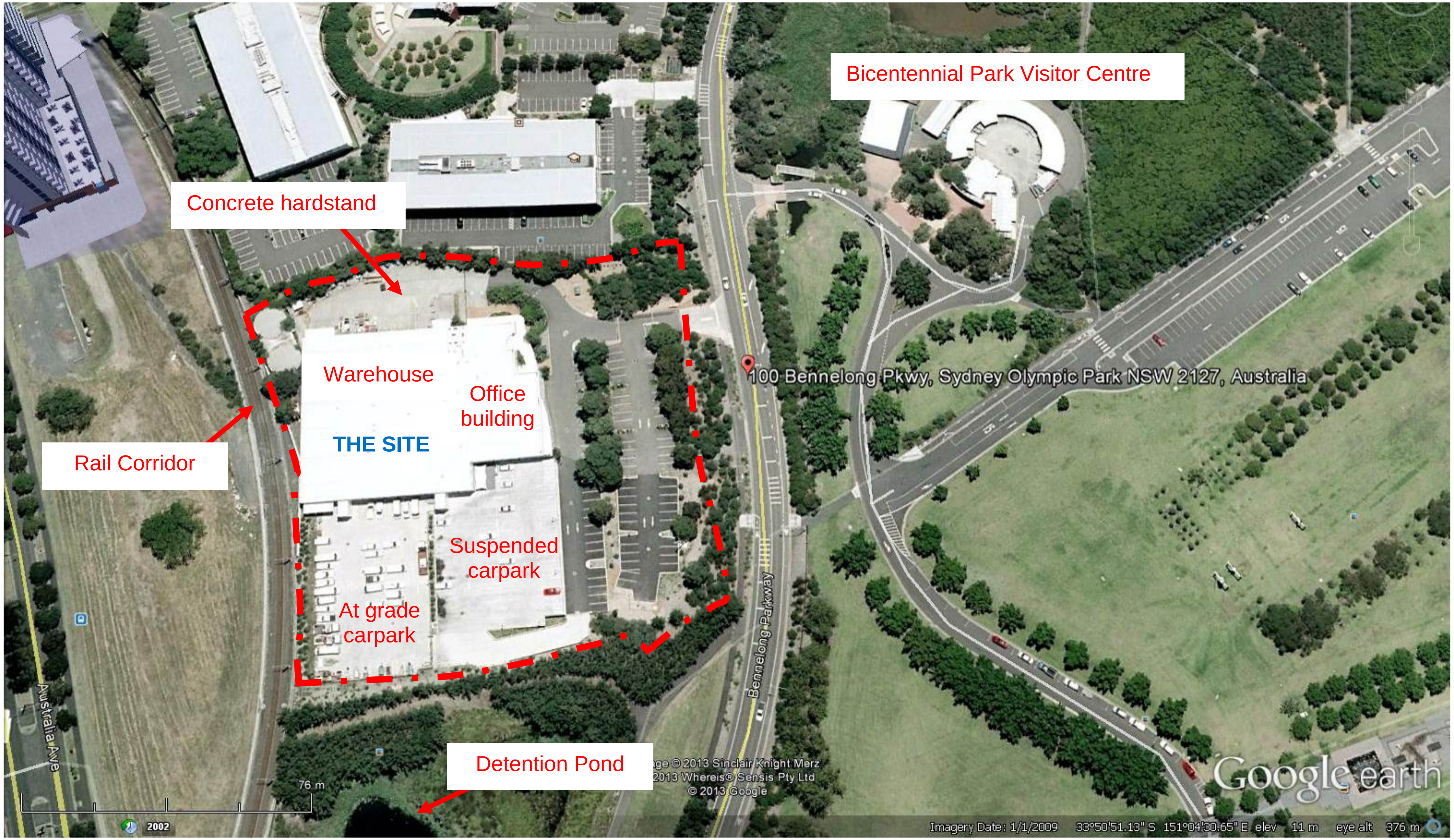


Image sourced from *'Google earth'*

Site Locality Plan

To be read in conjunction with text of report.

27007ZT • FIGURE 1

VIBRATION EMISSION DESIGN GOALS

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

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

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

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

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

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

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

REPORT EXPLANATION NOTES

INTRODUCTION

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

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

DESCRIPTION AND CLASSIFICATION METHODS

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

Soil types are described according to the predominating particle size and behaviour as set out in the attached Unified Soil Classification Table qualified by the grading of other particles present (e.g. sandy clay) as set out below:

Soil Classification	Particle Size
Clay	less than 0.002mm
Silt	0.002 to 0.075mm
Sand	0.075 to 2mm
Gravel	2 to 60mm

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

Relative Density	SPT 'N' Value (blows/300mm)
Very loose	less than 4
Loose	4 – 10
Medium dense	10 – 30
Dense	30 – 50
Very Dense	greater than 50

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

Classification	Unconfined Compressive Strength kPa
Very Soft	less than 25
Soft	25 – 50
Firm	50 – 100
Stiff	100 – 200
Very Stiff	200 – 400
Hard	Greater than 400
Friable	Strength not attainable – soil crumbles

Rock types are classified by their geological names, together with descriptive terms regarding weathering, strength, defects, etc. Where relevant, further information regarding rock classification is given in the text of the report. In the Sydney Basin, 'Shale' is used to describe thinly bedded to laminated siltstone.

SAMPLING

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

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

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

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

INVESTIGATION METHODS

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

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

Hand Auger Drilling: A borehole of 50mm to 100mm diameter is advanced by manually operated equipment. Premature refusal of the hand augers can occur on a variety of materials such as hard clay, gravel or ironstone, and does not necessarily indicate rock level.

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

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

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

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

Continuous Core Drilling: A continuous core sample is obtained using a diamond tipped core barrel. Provided full core recovery is achieved (which is not always possible in very low strength rocks and granular soils), this technique provides a very reliable (but relatively expensive) method of investigation. In rocks, an NMLC triple tube core barrel, which gives a core of about 50mm diameter, is usually used with water flush. The length of core recovered is compared to the length drilled and any length not recovered is shown as CORE LOSS. The location of losses are determined on site by the supervising engineer; where the location is uncertain, the loss is placed at the top end of the drill run.

Standard Penetration Tests: Standard Penetration Tests (SPT) are used mainly in non-cohesive soils, but can also be used in cohesive soils as a means of indicating density or strength and also of obtaining a relatively undisturbed sample. The test procedure is described in Australian Standard 1289, “Methods of Testing Soils for Engineering Purposes” – Test F3.1.

The test is carried out in a borehole by driving a 50mm diameter split sample tube with a tapered shoe, under the impact of a 63kg hammer with a free fall of 760mm. It is normal for the tube to be driven in three successive 150mm increments and the ‘N’ value is taken as the number of blows for the last 300mm. In dense sands, very hard clays or weak rock, the full 450mm penetration may not be practicable and the test is discontinued.

The test results are reported in the following form:

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

  N = 13
  4, 6, 7

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

  N>30
  15, 30/40mm

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

Occasionally, the drop hammer is used to drive 50mm diameter thin walled sample tubes (U50) in clays. In such circumstances, the test results are shown on the borehole logs in brackets.

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

Static Cone Penetrometer Testing and Interpretation: Cone penetrometer testing (sometimes referred to as a Dutch Cone) described in this report has been carried out using an Electronic Friction Cone Penetrometer (EFCP). The test is described in Australian Standard 1289, Test F5.1.

In the tests, a 35mm diameter rod with a conical tip is pushed continuously into the soil, the reaction being provided by a specially designed truck or rig which is fitted with an hydraulic ram system. Measurements are made of the end bearing resistance on the cone and the frictional resistance on a separate 134mm long sleeve, immediately behind the cone. Transducers in the tip of the assembly are electrically connected by wires passing through the centre of the push rods to an amplifier and recorder unit mounted on the control truck.

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

The information provided on the charts comprise:

- Cone resistance – the actual end bearing force divided by the cross sectional area of the cone – expressed in MPa.
- Sleeve friction – the frictional force on the sleeve divided by the surface area – expressed in kPa.
- Friction ratio – the ratio of sleeve friction to cone resistance, expressed as a percentage.

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

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

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

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

Portable Dynamic Cone Penetrometers: Portable Dynamic Cone Penetrometer (DCP) tests are carried out by driving a rod into the ground with a sliding hammer and counting the blows for successive 100mm increments of penetration.

Two relatively similar tests are used:

- Cone penetrometer (commonly known as the Scala Penetrometer) – a 16mm rod with a 20mm diameter cone end is driven with a 9kg hammer dropping 510mm (AS1289, Test F3.2). The test was developed initially for pavement subgrade investigations, and correlations of the test results with California Bearing Ratio have been published by various Road Authorities.
- Perth sand penetrometer – a 16mm diameter flat ended rod is driven with a 9kg hammer, dropping 600mm (AS1289, Test F3.3). This test was developed for testing the density of sands (originating in Perth) and is mainly used in granular soils and filling.

LOGS

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

The attached explanatory notes define the terms and symbols used in preparation of the logs.

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

GROUNDWATER

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

- Although groundwater may be present, in low permeability soils it may enter the hole slowly or perhaps not at all during the time it is left open.
- A localised perched water table may lead to an erroneous indication of the true water table.
- Water table levels will vary from time to time with seasons or recent weather changes and may not be the same at the time of construction.
- The use of water or mud as a drilling fluid will mask any groundwater inflow. Water has to be blown out of the hole and drilling mud must be washed out of the hole or 'reverted' chemically if water observations are to be made.

More reliable measurements can be made by installing standpipes which are read after stabilising at intervals ranging from several days to perhaps weeks for low permeability soils. Piezometers, sealed in a particular stratum, may be advisable in low permeability soils or where there may be interference from perched water tables or surface water.

FILL

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

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

LABORATORY TESTING

Laboratory testing is normally carried out in accordance with Australian Standard 1289 *'Methods of Testing Soil for Engineering Purposes'*. Details of the test procedure used are given on the individual report forms.

ENGINEERING REPORTS

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

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

- Unexpected variations in ground conditions – the potential for this will be partially dependent on borehole spacing and sampling frequency as well as investigation technique.
- Changes in policy or interpretation of policy by statutory authorities.
- The actions of persons or contractors responding to commercial pressures.

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

SITE ANOMALIES

In the event that conditions encountered on site during construction appear to vary from those which were expected from the information contained in the report, the company requests that it immediately be notified. Most problems are much more readily resolved when conditions are exposed that at some later stage, well after the event.

REPRODUCTION OF INFORMATION FOR CONTRACTUAL PURPOSES

Attention is drawn to the document *'Guidelines for the Provision of Geotechnical Information in Tender Documents'*, published by the Institution of Engineers, Australia. Where information obtained from this investigation is provided for tendering purposes, it is recommended that all information, including the written report and discussion, be made available. In circumstances where the discussion or comments section is not relevant to the contractual situation, it may be appropriate to prepare a specially edited document. The company would be pleased to assist in this regard and/or to make additional report copies available for contract purposes at a nominal charge.

Copyright in all documents (such as drawings, borehole or test pit logs, reports and specifications) provided by the Company shall remain the property of Jeffery and Katauskas Pty Ltd. Subject to the payment of all fees due, the Client alone shall have a licence to use the documents provided for the sole purpose of completing the project to which they relate. License to use the documents may be revoked without notice if the Client is in breach of any objection to make a payment to us.

REVIEW OF DESIGN

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

SITE INSPECTION

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

Requirements could range from:

i) a site visit to confirm that conditions exposed are no worse than those interpreted, to
ii) a visit to assist the contractor or other site personnel in identifying various soil/rock types such as appropriate footing or pier founding depths, or
iii) full time engineering presence on site.

GRAPHIC LOG SYMBOLS FOR SOILS AND ROCKS

SOIL

- FILL
- TOPSOIL
- CLAY (CL, CH)
- SILT (ML, MH)
- SAND (SP, SW)
- GRAVEL (GP, GW)
- SANDY CLAY (CL, CH)
- SILTY CLAY (CL, CH)
- CLAYEY SAND (SC)
- SILTY SAND (SM)
- GRAVELLY CLAY (CL, CH)
- CLAYEY GRAVEL (GC)
- SANDY SILT (ML)
- PEAT AND ORGANIC SOILS

ROCK

- CONGLOMERATE
- SANDSTONE
- SHALE
- SILTSTONE, MUDSTONE, CLAYSTONE
- LIMESTONE
- PHYLLITE, SCHIST
- TUFF
- GRANITE, GABBRO
- DOLERITE, DIORITE
- BASALT, ANDESITE
- QUARTZITE

DEFECTS AND INCLUSIONS

- CLAY SEAM
- SHEARED OR CRUSHED SEAM
- BRECCIATED OR SHATTERED SEAM/ZONE
- IRONSTONE GRAVEL
- ORGANIC MATERIAL

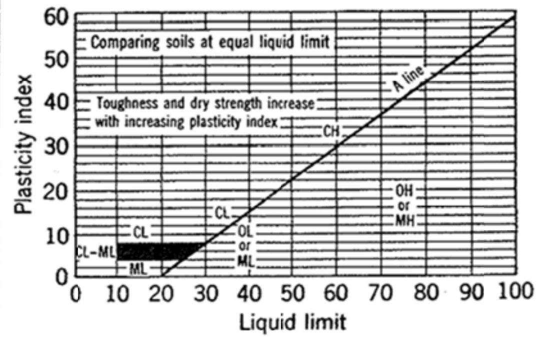
OTHER MATERIALS

- CONCRETE
- BITUMINOUS CONCRETE, COAL
- COLLUVIUM

Major divisions			Field Identification Procedures (Excluding particles larger than 75 μm and basing fractions on estimated weights)	Group Symbols [a]	Typical Names	Information Required for Describing Soils	Laboratory Classification Criteria
Coarse-grained soils. More than half of material is *larger* than 75 μm sieve size[b] (The 75 μm sieve size is about the smallest particle visible to naked eye)	Gravels. More than half of coarse fraction is larger than 4 mm sieve size	Clean gravels (little or no fines)	Wide range in grain size and substantial amounts of all intermediate particle sizes	*GW*	Well graded gravels, gravel-sand mixtures, little or no fines	Give typical name; indicate approximate percentages of sand and gravel; maximum size; angularity, surface condition, and hardness of the coarse grains; local or geologic name and other pertinent descriptive information; and symbols in parentheses. For undisturbed soils add information on stratification, degree of compactness, cementation, moisture conditions and drainage characteristics. Example: *Silty sand*, gravelly; about 20% hard, angular gravel particles 12 mm maximum size; rounded and subangular sand grains coarse to fine, about 15% non-plastic fines with low dry strength; well compacted and moist in place; alluvial sand; (*SM*)	$C_U = \frac{D_{60}}{D_{10}}$ Greater than 4; $C_C = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ Between 1 and 3
			Predominantly one size or a range of sizes with some intermediate sizes missing	*GP*	Poorly graded gravels, gravel-sand mixtures, little or no fines		Not meeting all gradation requirements for *GW*
		Gravels with fines (appreciable amount of fines)	Nonplastic fines (for identification procedures see *ML* below)	*GM*	Silty gravels, poorly graded gravel-sand-silt mixtures		Atterberg limits below "A" line, or *PI* less than 4. Above "A" line with *PI* between 4 and 7 are *borderline* cases requiring use of dual symbols
			Plastic fines (for identification procedures, see *CL* below)	*GC*	Clayey gravels, poorly graded gravel-sand-clay mixtures		Atterberg limits above "A" line, with *PI* greater than 7
	Sands. More than half of coarse fraction is smaller than 4 mm sieve size	Clean sands (little or no fines)	Wide range in grain sizes and substantial amounts of all intermediate particle sizes	*SW*	Well graded sands, gravelly sands, little or no fines		$C_U = \frac{D_{60}}{D_{10}}$ Greater than 6; $C_C = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ Between 1 and 3
			Predominantly one size or a range of sizes with some intermediate sizes missing	*SP*	Poorly graded sands, gravelly sands, little or no fines		Not meeting all gradation requirements for *SW*
		Sands with fines (appreciable amount of fines)	Nonplastic fines (for identification procedures, see *ML* below)	*SM*	Silty sands, poorly graded sand-silt mixtures		Atterberg limits below "A" line or *PI* less than 5. Above "A" line with *PI* between 4 and 7 are *borderline* cases requiring use of dual symbols
			Plastic fines (for identification procedures, see *CL* below)	*SC*	Clayey sands, poorly graded sand-clay mixtures		Atterberg limits below "A" line with *PI* greater than 7

Laboratory classification criteria (coarse-grained soils): Use grain size curve in identifying the fractions as given under field identification. Determine percentages of gravel and sand from grain size curve. Depending on percentage of fines (fraction smaller than 75 μm sieve size) coarse grained soils are classified as follows: Less than 5% *GW, GP, SW, SP*; More than 12% *GM, GC, SM, SC*; 5% to 12% *Borderline* cases requiring use of dual symbols.

Major divisions		Identification Procedures on Fraction Smaller than 380 μm Sieve Size: Dry Strength (crushing characteristics)	Dilatancy (reaction to shaking)	Toughness (consistency near plastic limit)	Group Symbols	Typical Names	Information Required for Describing Soils
Fine-grained soils. More than half of material is *smaller* than 75 μm sieve size	Silts and clays liquid limit less than 50	None to slight	Quick to slow	None	*ML*	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands with slight plasticity	Give typical name; indicate degree and character of plasticity, amount and maximum size of coarse grains; colour in wet condition, odour if any, local or geologic name, and other pertinent descriptive information, and symbol in parentheses. For undisturbed soils add information on structure, stratification, consistency in undisturbed and remoulded states, moisture and drainage conditions. Example: *Clayey silt*, brown; slightly plastic; small percentage of fine sand; numerous vertical root holes; firm and dry in place; loess; (*ML*)
		Medium to high	None to very slow	Medium	*CL*	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays	
		Slight to medium	Slow	Slight	*OL*	Organic silts and organic silt-clays of low plasticity	
	Silts and clays liquid limit greater than 50	Slight to medium	Slow to none	Slight to medium	*MH*	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts	
		High to very high	None	High	*CH*	Inorganic clays of high plasticity, fat clays	
		Medium to high	None to very slow	Slight to medium	*OH*	Organic clays of medium to high plasticity	
Highly Organic Soils		Readily identified by colour, odour, spongy feel and frequently by fibrous texture			*Pt*	Peat and other highly organic soils	



Plasticity chart
for laboratory classification of fine grained soils

Note: 1 Soils possessing characteristics of two groups are designated by combinations of group symbols (eg. GW-GC, well graded gravel-sand mixture with clay fines).
2 Soils with liquid limits of the order of 35 to 50 may be visually classified as being of medium plasticity.

LOG SYMBOLS

LOG COLUMN	SYMBOL	DEFINITION
Groundwater Record	▼	Standing water level. Time delay following completion of drilling may be shown.
	C	Extent of borehole collapse shortly after drilling.
	►	Groundwater seepage into borehole or excavation noted during drilling or excavation.
Samples	ES U50 DB DS ASB ASS SAL	Soil sample taken over depth indicated, for environmental analysis. Undisturbed 50mm diameter tube sample taken over depth indicated. Bulk disturbed sample taken over depth indicated. Small disturbed bag sample taken over depth indicated. Soil sample taken over depth indicated, for asbestos screening. Soil sample taken over depth indicated, for acid sulfate soil analysis. Soil sample taken over depth indicated, for salinity analysis.
Field Tests	N = 17 4, 7, 10	Standard Penetration Test (SPT) performed between depths indicated by lines. Individual figures show blows per 150mm penetration. 'R' as noted below.
	N_c = 5 7 3R	Solid Cone Penetration Test (SCPT) performed between depths indicated by lines. Individual figures show blows per 150mm penetration for 60 degree solid cone driven by SPT hammer. 'R' refers to apparent hammer refusal within the corresponding 150mm depth increment.
	VNS = 25 PID = 100	Vane shear reading in kPa of Undrained Shear Strength. Photoionisation detector reading in ppm (Soil sample headspace test).
Moisture Condition (Cohesive Soils) (Cohesionless Soils)	MC>PL MC≈PL MC<PL D M W	Moisture content estimated to be greater than plastic limit. Moisture content estimated to be approximately equal to plastic limit. Moisture content estimated to be less than plastic limit. DRY – Runs freely through fingers. MOIST – Does not run freely but no free water visible on soil surface. WET – Free water visible on soil surface.
Strength (Consistency) Cohesive Soils	VS S F St VSt H ( )	VERY SOFT – Unconfined compressive strength less than 25kPa SOFT – Unconfined compressive strength 25-50kPa FIRM – Unconfined compressive strength 50-100kPa STIFF – Unconfined compressive strength 100-200kPa VERY STIFF – Unconfined compressive strength 200-400kPa HARD – Unconfined compressive strength greater than 400kPa Bracketed symbol indicates estimated consistency based on tactile examination or other tests.
Density Index/ Relative Density (Cohesionless Soils)	 VL L MD D VD ( )	**Density Index (I_D) Range (%)** — **SPT 'N' Value Range (Blows/300mm)** Very Loose <15 0-4 Loose 15-35 4-10 Medium Dense 35-65 10-30 Dense 65-85 30-50 Very Dense >85 >50 Bracketed symbol indicates estimated density based on ease of drilling or other tests.
Hand Penetrometer Readings	300 250	Numbers indicate individual test results in kPa on representative undisturbed material unless noted otherwise.
Remarks	'V' bit 'TC' bit T_{60}	Hardened steel 'V' shaped bit. Tungsten carbide wing bit. Penetration of auger string in mm under static load of rig applied by drill head hydraulics without rotation of augers.

LOG SYMBOLS continued

ROCK MATERIAL WEATHERING CLASSIFICATION

TERM	SYMBOL	DEFINITION
Residual Soil	RS	Soil developed on extremely weathered rock; the mass structure and substance fabric are no longer evident; there is a large change in volume but the soil has not been significantly transported.
Extremely weathered rock	XW	Rock is weathered to such an extent that it has "soil" properties, ie it either disintegrates or can be remoulded, in water.
Distinctly weathered rock	DW	Rock strength usually changed by weathering. The rock may be highly discoloured, usually by ironstaining. Porosity may be increased by leaching, or may be decreased due to deposition of weathering products in pores.
Slightly weathered rock	SW	Rock is slightly discoloured but shows little or no change of strength from fresh rock.
Fresh rock	FR	Rock shows no sign of decomposition or staining.

ROCK STRENGTH

Rock strength is defined by the Point Load Strength Index (Is 50) and refers to the strength of the rock substance in the direction normal to the bedding. The test procedure is described by the International Journal of Rock Mechanics, Mining, Science and Geomechanics. Abstract Volume 22, No 2, 1985.

TERM	SYMBOL	Is (50) MPa	FIELD GUIDE
Extremely Low:	EL		Easily remoulded by hand to a material with soil properties.
----------------------	-------------	0.03	
Very Low:	VL		May be crumbled in the hand. Sandstone is "sugary" and friable.
----------------------	-------------	0.1	
Low:	L		A piece of core 150mm long x 50mm dia. may be broken by hand and easily scored with a knife. Sharp edges of core may be friable and break during handling.
----------------------	-------------	0.3	
Medium Strength:	M		A piece of core 150mm long x 50mm dia. can be broken by hand with difficulty. Readily scored with knife.
----------------------	-------------	1	
High:	H		A piece of core 150mm long x 50mm dia. core cannot be broken by hand, can be slightly scratched or scored with knife; rock rings under hammer.
----------------------	--------------	3	
Very High:	VH		A piece of core 150mm long x 50mm dia. may be broken with hand-held pick after more than one blow. Cannot be scratched with pen knife; rock rings under hammer.
----------------------	--------------	10	
Extremely High:	EH		A piece of core 150mm long x 50mm dia. is very difficult to break with hand-held hammer. Rings when struck with a hammer.

ABBREVIATIONS USED IN DEFECT DESCRIPTION

ABBREVIATION	DESCRIPTION	NOTES
Be	Bedding Plane Parting	Defect orientations measured relative to the normal to the long core axis (ie relative to horizontal for vertical holes)
CS	Clay Seam	
J	Joint	
P	Planar	
Un	Undulating	
S	Smooth	
R	Rough	
IS	Ironstained	
XWS	Extremely Weathered Seam	
Cr	Crushed Seam	
60t	Thickness of defect in millimetres	