REPORT

TO
JOHN R BROGAN & ASSOCIATES PTY LTD

ON
PRELIMINARY GEOTECHNICAL INVESTIGATION

FOR
DUE DILLIGENCE OF PROSPECTIVE DEVELOPMENT

AT
CNR. OF ELIZABETH DRIVE & BONNYRIGG AVENUE
BONNYRIGG, NSW

29 January 2015
Ref: 27813Vrpt.rev.1-Bonnyrigg

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1 INTRODUCTION

JK GEOTECHNICS have been commissioned by John R Brogan and Associates Pty Ltd to carry out a preliminary geotechnical investigation to assist with the Due Diligence process for a proposed warehouse development at a site on the corner of Elizabeth Drive and Bonnyrigg Avenue, Bonnyrigg, NSW. The commission was by Official Order Ref.: RCO: RCO:am:39045, based on our fee proposal Ref. P38912V-Bonnyrigg.

A summary of the principal geotechnical issues, based on the findings of this investigation, is provided on Section 4.1.

This report presents the investigation procedures and findings and goes on to make comments and preliminary recommendations on the principal geotechnical aspects of the proposed development to assist the architects and structural engineers with the due diligence process, preliminary planning and design, based on the results of seven test boreholes. The report provides information and preliminary recommendations on:

- Detailed logs of the boreholes with penetration test results and groundwater observations;
- Interpretation of Subsurface Profile including bedrock;
- AS2870 site classification;
- Main Geotechnical Issues for the Development;
- Earthworks;
- Retention;
- Suitable Footings Systems and Options;
- Foundation strata and depth;
- Allowable Bearing Pressures;
- Allowable Shaft Adhesions;

We also provide requirements for a detailed geotechnical subsurface investigation of the site. The recommendations provided herein must be reviewed once further geotechnical work has been completed, after demolition and at DA and CC stages, and after the development details such as layout drawings, floor levels, footing system and structural loads are decided upon and determined.

A preliminary Stage 1 Environmental Site Assessment (ESA), including groundwater and acid sulphate, was undertaken by Environmental Investigation Services (EIS) in conjunction with this preliminary geotechnical investigation. The results of ESA are reported in Ref.E27813KGrpt.

1.1 Prospective Development

The prospective development was at Due Diligence stage. From the latest provided drawings, and an email from John R Brogan and Associates Pty Ltd describing the favoured development option, we understand that the development will comprise a large warehouse over an undercroft car park level. Adjoining the warehouse will be timber sales yard, landscape yard, outdoor nursery, bagged goods canopy area and service driveway.

The (trading) floor of the warehouse and much of the service driveway is proposed to be part of a suspended structure over the customer (undercroft) car park. t is envisaged that a system of piling would be carried through the existing thick concrete pavement of the bus depot down to a competent foundation stratum underneath.

Only shallow and limited extent excavations are expected to be required to form the car parking undercroft level, which is proposed to be at RL45.3m AHD. The bulk earthwork level is proposed to be at RL45.0m. Existing bus depot concrete pavements higher than RL45.0m (mostly near the Bonnyrigg Avenue eastern portion of the site) would be demolished and proposed to be crushed and reused as fill material under the new car park pavements. The undercroft level is proposed to be designed to a have a gradual fall towards the western boundary to avoid having to strength or reconstruct the perimeter retaining walls.

At this concept stage, other details of the development such as floor, pavement and earthwork levels and structural loads had not been determined or supplied. Structural loads had not been determined at this DD stage, but we have assumed moderate to high loads may apply.

We note that the existing Bunnings warehouse in the adjoining property to the north-east will not form part of the prospective development. The intention is for the existing warehouse to remain in place but used for other tenancy shop.

In preparing this report we have been provided with the following relevant information:

- Proposed Bunnings Warehouse - Elevations prepared by John R Brogan & Associates Pty. Ltd. (Project No. 1062 Drawing No: 130, AMD No. P1 dated 19 January 2015).

2 INVESTIGATION PROCEDURE

The fieldwork for the investigation was carried out on 10 October 2014 and comprised seven geotechnical test boreholes (BHs 1 to 7). The borehole locations were set out by tape measurements from the discernible site boundaries and were recorded using a handheld GPS. The borehole locations are shown approximately on the attached Figure 1, which is based on a satellite image of the site. Surveying the borehole locations and determination of surface levels were outside the limited scope of this preliminary investigation. We recommend that this be completed later by a registered surveyor based on the locations shown on Figure 1.

Prior to commencement of the fieldwork the investigation locations were electromagnetically scanned by a specialist subcontractor so that all borehole locations could be located clear of buried services.

The boreholes drilled with our truck mounted JK350 drilling rig to depths between 6.0m to 6.2m below existing surface levels using spiral auger techniques and a Tungsten Carbide ('TC') drill bit. All boreholes proved bedrock.

The apparent compaction of the fill and strength of the subsurface natural clay soils were assessed from Standard Penetration Test (SPT) 'N' values augmented by hand penetrometer tests on the SPT split tube samples and observations during drilling. The strength of the bedrock was assessed by observation of the auger penetration resistance using a 'TC' drill bit, together with examination of the recovered rock cuttings and from correlations with subsequent moisture content test results on recovered rock chips. It should be noted that strengths assessed in this way are approximate and variances of one strength order should not be unexpected.

Selected samples were tested by Soil Test Services (STS), a NATA registered laboratory, to determine standard compaction properties, four day soaked CBR values, moisture contents, Atterberg Limits, and Linear Shrinkage. The results are summarised in the attached STS report Tables A and B.

Groundwater observations were made in the boreholes during drilling and shortly after completion of drilling. Slotted PVC standpipes were installed in three boreholes for monitoring of the water levels over an extended period of time as well as to allow EIS to obtain samples of the groundwater. Further groundwater monitoring was carried out by EIS on 16 October 2014.

Environmental samples were obtained from the geotechnical boreholes for the Environmental Site Assessment by EIS.

The fieldwork was completed in the full-time presence of our geotechnical engineer, Ms Michelle Watson, who set out the borehole locations, nominated the testing and sampling, and prepared the attached borehole logs. The borehole logs, with field test results, are attached to this report, together with a glossary of the terms and symbols used in the logs. For more details of the investigation procedures and their limitations, reference should be made to the attached Report Explanation Notes.

3 RESULTS OF INVESTIGATION

3.2 Site Description

The subject site for the prospective development is shown on Figure 1, which is a satellite image of the site. The site is located in a region of gently undulating topography. At the time of the investigation, the site comprised two main areas: the Westbus Depot in the north-east and a vacant area to the south-west.

The site is bound by Bonnyrigg Avenue to the south-east, by Elizabeth Drive to the south-west, by an existing Bunnings Warehouse to the north-east and by Clear Paddock Creek to the north-west. We note that the existing Bunnings warehouse in the adjoining property to the north-east will not form part of the prospective development on the subject site. The intention is for the existing warehouse to remain in place but used for other tenancy shop.

The Westbus Depot is roughly 130m by 135m in plan area and slopes to the west at 2° to 3°. The depot is primarily covered by concrete pavement in variable condition (fair to good). Large areas of cracking were observed in the concrete pavement in the eastern portion near to Bonnyrigg Avenue boundary. The Depot contained three main structures: a two storey concrete and metal construction building in the north-eastern portion of the site, a steel awning structure covering two diesel tanks in the south-western portion of the site and a double height steel bus wash shed in the north-western portion of the site. All structures appeared to be in good condition.

The depot also contained an underground sedimentation tank in the northern corner of the site that was around 7m wide in the north-west to south-east direction and 47m long in the north-east to south-west direction. The tank was observed to contain water at a depth of 0.55m below the concrete pavement.

The vacant lot comprising the south-west portion of the site was approximately 130m by 45m in plan area and sloped to the north-west at 3°. A large stockpile approximately 3m to 4m high was located at the western end of the vacant area. The stockpile was covered in dense vegetation and appeared to comprise mostly silty clay fill but with an undermined volume of inclusions such as brick fragments, plastic fragments, fibre cement fragments, roots and ash.

The adjacent site to the north-east was at a level 0.9m below the subject site, separated from the site by a gabion type (wire baskets with rockfill) retaining wall that appeared the be in good condition. The gabion retaining wall reached a maximum height of around 0.9m at the centre of the common boundary and reduced in height to the east and west until the adjacent site is level with the subject site at the eastern and western extremes of the common boundary. The adjacent site to the north-east contained a Bunnings Warehouse set back approximately 20m from the common boundary that appeared to be in good condition. The remainder of the site comprised asphalt pavement in good condition and a number of small garden areas.

Clear Paddock Creek to the north-west was at a level below the subject site, separated from the site by a gabion retaining wall that appeared to be in good condition. The retaining wall was around 1.5m high at the northern end and increased in height to 3.5m at the southern end. The creek appeared to be set back from the boundary by around 40m and the surrounding area was heavily vegetated.

3.3 Geology and Subsurface Conditions

The 1:100,000 Geological Map of Penrith indicates the site to be underlain by Bringelly Shale of the Wianamatta Group, and close to the intersection of fluvial sand, clay and silt deposits associated with the nearby creek. The presence of the Bringelly Shale was confirmed by the boreholes, which disclosed a subsurface profile generally comprising fill over a profile of silty clays over weathered shale. A graphical summary of the borehole information is presented in Figure 2 attached. Reference should be made to the attached borehole logs for detailed subsurface descriptions at specific locations. A summary of the subsoil conditions, as encountered, is presented below:

Concrete Pavements

Concrete pavement was encountered at the surface of all boreholes within the Westbus Depot (BH3 to BH7). The pavement ranged in thickness from 0.28m to 0.34m.

Fill

Fill was encountered beneath the concrete pavement in boreholes within the Westbus Depot (BH3 to BH7) with the exception of BH4. The fill comprised a brown, light grey and red brown silty clay containing inclusions such as slag, ash, root fibres, shale gravel, igneous gravel, ironstone gravel and sandstone gravel. Based on the SPT 'N' values and hand penetrometer readings the fill was found to be of variable compaction, with most areas being poorly or moderately compacted. The fill extended to depths between 1.6m and 3.0m below existing surface levels.

Fill was encountered beneath the surface in BH1 and BH2 within the vacant lot. The fill comprised a sandy gravel containing inclusions such as ceramic fragments and root fibres. The fill extended to depths between 0.35m and 0.85m. A large stockpile approximately 3m to 4m high was located at the western end of the vacant area. The stockpile was covered in dense vegetation and appeared to comprise mostly silty clay fill but with an undermined volume of inclusions such as brick fragments, plastic fragments, fibre cement fragments, roots and ash.

Residual Silty Clays

Residual silty clay was encountered below the pavement in BH4 and below the fill in all other locations except BH6. The silty clay was generally assessed to be of high plasticity and very stiff to hard strength and contained minor portions of root fibres and ironstone gravel. The residual clays extended to depths ranging between 1.5m (BH1) and 4.6m (BH7).

A layer of interbedded residual clay and extremely weathered shale was encountered below the residual silty clays in BH1 and BH7 extending to depths of 3.2m and 4.6m respectively.

Shale Bedrock

The shale bedrock was encountered at depths between 1.8m (BH2) and 4.6m (BH7) below existing surface levels. The shale was typically initially distinctly weathered and very low to low strength, improving to medium strength with depth in some locations.

Groundwater

Groundwater was not encountered during or on completion of drilling in any of the boreholes. Additional groundwater readings were undertaken by EIS on 16 October 2014. On this date groundwater was measured at a level of 5.23m below existing surface levels in the monitoring well installed in BH5, while the monitoring wells installed in BH2 and BH6 were dry.

3.4 Laboratory Test Results

The results of the moisture content tests generally correlate well with the field logging assessments of rock strength.

The Atterberg Limits and Linear Shrinkage test results confirmed the residual silty clays to be of high plasticity and indicated the clays have a high potential (Class H1 in terms of AS2870-2011) for shrink-swell reactive movements with changes in moisture content. The tests on the silty clay fill from BH5 indicated the clay fill to be of medium plasticity and of a lesser reactivity potential: medium-probably Class M. This range of reactivity potentials are corroborated by the range of swells measured during the soaked CBR testing of between 3%-4%.

The four-day soaked CBR tests on samples of residual silty clay and of clay fill subgrade similarly resulted in very low CBR values of 1%-1.5%. All samples were compacted to 98%-99% of Standard Maximum Dry Density (SMDD).

4 COMMENTS AND RECOMMENDATIONS

4.1 Summary of Principal Geotechnical Findings and Issues and Further Work

The boreholes disclosed significantly variable subsurface conditions comprising fill and residual clays grading into shale bedrock at depths in the range of 1.8m-4.6m (refer to Figure 2). The depth variations of the fill are quite remarkable in the range of 0.35m to 3m. All boreholes were dry on completion of drilling and remained dry including monitoring of the standpipe wells with exception of BH5, which recorded a groundwater level at 5.23m at 6 days after completion of drilling and standpipe installation.

The most important anomaly, which will affect the proposed undercroft customer parking pavement, was found in the 1.6m-3m deep fill of BHs 3, 5, 6, and 7. The fill is likely associated with rising of the site levels by the current development and the gabion type retaining walls along the north-eastern and western boundaries. The fill, which is mostly silty clay with gravel in composition, was assessed to be variably compacted, most being poorly to moderately compacted, on the basis of the SPT N values and our observations during drilling. This assessment does not give a precise determination of in situ densities since they are affected by friction during driving/pushing (SPT), the presence of gravel within the fill and the moisture content of the clay fill. Nonetheless, they provide a qualitative guide. We do not know the origin of the fill materials or its placement control.

The following is a summary of principal geotechnical issues to be taken into consideration for the Due Diligence process, and for the preliminary design and planning for an assumed typical warehouse development:

1. The presence of fill and its compaction control is clearly the main issue. For the most of the site the fill is relatively deep 1.6m or greater. We are unaware of records that document the manner of placement, compaction specification and control of the fill. Accordingly, we consider this existing material to be ‘uncontrolled’ fill. Because of this fill, the site is considered to be Class P (‘problem’) in accordance with AS2870-2011.
2. In addition to the variability in compaction, the fill subgrade has very low CBR value of 1.5% and also it is shrink/swell reactive.
3. The fill is deemed unsuitable as a bearing stratum for warehouse footings and trading floor slab. Unless penetration testing (e.g. via borehole SPTs and a closely spaced grid of continuous Electronic Cone Penetration tests) of the fill area is carried out and/or compaction records are provided for our review, to assess with more confidence the load carrying capacity and potential settlements of the fill mass, we recommend that the fill is not used as a bearing stratum for slabs and footings.
4. As mentioned in Section 1.1, the proposal is to suspend the entire trading floor slab and the warehouse structure, as well as the service roadway, on piled footings taken down to a competent foundation stratum. We concur with this fully suspended structure and driveway proposal. The most competent foundation stratum at the site for the proposed development is the bedrock, which for most of the site was found at significantly variable depths (1.8m to 4.6m). Supporting structures on hybrid foundations (e.g. partly on engineered fill/residual clay and partly on rock) must be avoided and is not recommended.
5. The fill (together with the cover of existing thick concrete pavement) may be used to support the proposed undercroft customer parking pavement. If the traffic loads on the new undercroft pavement are equal or less than the current bus depot usage then there is little reason to judge that it would not continue to perform adequately as in the past usage. However, if the new undercroft pavement is subjected to greater loads or imposes its new pavement structure imposes greater loads than current on the existing uncontrolled fill mass then it is considered a ‘moderate to high risk’ (of poor performance) as a supporting subgrade under the pavements.
6. It is also important to note that the fill is a variable material from unknown origins that may contain large inclusions and obstacles, which may not have been picked by our small diameter boreholes (100mm) and which could affect future construction. Variations in fill quality/nature should be anticipated. There is a possibility that some of the fill may contain contaminants and reference to the EIS report is recommended.
7. When the fill is removed and/or replaced with controlled, engineered fill then the site can be upgraded to Class ‘H1’ due to the moderate reactivity of the underlying residual clay

profile, although we note that abnormal moisture conditions could exist after removal of the current concrete pavement cover resulting once again in a more severe Class P site classification. We advice that in the strict sense AS2870-2011 site classification does not apply to this development but it is a useful guide in estimating foundation as well as shrink/swell movements that have the potential to occur at this site.

8. The residual clays beneath the fill at the site were also determined to have very low soaked CBR (1%) and hence, this clay subgrade is considered to be "poor" subgrade for the pavements and slabs. The use of thick pavements and/or treating of the subgrade with lime would be required.
9. As mentioned in Section 1.1, the current option for the proposed development is for the undercroft level to be designed to a have a gradual fall towards the western boundary to avoid having to strength or reconstruct the perimeter retaining walls. Additional loads greater than currently being exerted should not be imposed on existing gabion retaining walls.

Further comments on these issues and geotechnical design parameters are provided in the subsequent sections of this report. The preliminary recommendations provided in this report may be used for preliminary design and construction planning purposes only; they would need to be confirmed by further geotechnical borehole investigation as discussed further below.

4.1.1 Further Geotechnical Work

At the time of our investigation, details of the development such as floor and pavement levels and structural loads were unknown or determined at the time of this Due Diligence investigation. The subsequent earthworks and footing recommendations are, therefore, provided in general and preliminary terms only, which will require revision once exact development details, such as earthwork levels, final floor levels, structural loads etc. are determined.

Given the variability in subsurface conditions, we consider that the number of boreholes and tests employed in this investigation provides only a broad general coverage of the site. We recommend that further boreholes be drilled to test the soils and sample the bedrock using diamond coring methods to assess for higher bearing values. Penetration testing (e.g. via boreholes SPTs and also a closely spaced grid of continuous Electronic Cone Penetration) of the deep fill areas is recommended to assess with more confidence the load carrying capacity and potential settlements of the existing fill subgrade. A meeting of the design team, once the design has been further advanced, would be of benefit to discuss the geotechnical issues in more detailed and determine the scope of the further detailed investigations

Furthermore, it will be essential during earthworks and construction that regular geotechnical inspections and testing be commissioned to check initial assumptions about earthworks and foundation conditions and likely variations that may occur between borehole/test locations and to provide further relevant geotechnical advice. Irregular or 'milestone' inspections by a geotechnical engineer are often not adequate for such variations in subsurface conditions and for excavation and foundation works. It is recommended that the Client be made aware of the need to commission a geotechnical engineer for regular frequent inspections.

The preliminary recommendations provided in this report should be reviewed following the additional geotechnical investigation as well as after these inspections. Furthermore, the recommendations provided herein should also be reviewed once exact development details, such structural layout, earthwork levels, floor levels, structural loads etc., are determined.

It is likely that further advice/input will be required during the structural design to address issues that may not have been addressed in this report. To some degree, this is an "iterative" process between evaluation of the geotechnical site conditions and the structural design. For the earthworks, piling and other foundation works, we strongly recommend that only competent contractors be considered, and that they are provided with a full copy of this report.

4.2 <u>Site Classification and Foundations</u>

The unaltered site as seen is classified as Class 'P' in accordance with AS2870-2011 due to the presence of the uncontrolled fill. Where the fill is stripped and/or replaced with engineered controlled fill then the site can be upgraded to Class H1, although we note that abnormal moisture conditions could exist after removal of existing concrete pavements, resulting once again in a more severe Class P site classification. We advice that in the strict sense AS2870-2011 site classification does not apply to this development but it is a useful guide in estimating foundation as well as shrink/swell movements (40mm-60mm for Class H1) that have the potential to occur at this site.

The most competent foundation stratum beneath the site for the development, as identified by the boreholes, is the shale bedrock. The shale bedrock is the recommended foundation stratum for structure with movement sensitive finishes or items and/or for structures that have columns or walls with moderate to heavy loads. In view of the presence of uncontrolled and deep fill and high reactivity of the residual clays and the relatively shallow depths to the bedrock, we recommend that the warehouse, and its slab, is supported on footings uniformly founded into the shale bedrock. As discussed in Section 1.1, we understand that this is the proposed development to option to be adopted; the service roadway will also be suspended on piles. Since it is proposed to keep the existing thick concrete pavement for portion of the site it is proposed to break holes though the concrete to allow the installation of piles down to the bedrock.

The following table provides our recommended geotechnical parameters for design of footings bearing on the bedrock. Below the table we provide further comments relating to footing design.

RECOMMEND GEOTECHNICAL PARAMETERS
FOR DESIGN OF FOOTINGS BEARING ON SHALE BEDROCK

SHALE/SILTSONE STRENGTHS	ALLOWABLE BEARING PRESSURE (KPa)	ALLOWABLE SHAFT ADHESION FOR PILE SOCKETS (KPa)	TYPICAL FIELD ELASTIC MODULUS E (MPa)
EL-VL	700	70	70
VL-L	1000	100	100
L or higher strength	1500	150	300

The allowable bearing pressures and shaft adhesions have been estimated on the basis of our augered borehole data. It should be noted that strengths assessed in this way are approximate and variances of one strength order should not be unexpected. It is probable the higher bearing pressures may be used in the 'L or higher strength' strata but this would have to be confirmed during the recommended detailed investigation stage by completing diamond coring of the bedrock with strength testing of recovered rock cores. Settlements may be estimated using the above typical values for the field elastic modulus under static loading conditions.

The minimum embedment of footings into the sandstone/shale/siltstone should be around 0.3m. The shaft adhesion values are recommended on condition that cleanliness and roughness of pier sockets and bases are achieved. The above shaft adhesions are applicable for compressive loads only. For uplift or tension loads, the above adhesion values should be halved. Refer to the borehole logs for depths to the various rock strengths.

Depending on levels adopted for the development (floor, pavement and earthwork levels) then it is likley piles would have to be employed. Suitable pile types are considered to be conventional bored piers and augered, grout injected (CFA) piles. We recommend that only high capacity drilling rigs, equipped with rock drilling equipment, be brought to site. It is also important to note that the fill is a highly variable material from unknown origins that contain large inclusions and obstacles which could affect pier or pile construction.

As a minimum requirement, the initial stages of footing excavation should be inspected by a Geotechnical Engineer to ascertain that the recommended foundation 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. We can assist with the future geotechnical inspections if you wish to commission us at the appropriate time. For grout piles, during installation of piles it is recommended that the initial piles be installed as close as practical to our borehole locations to calibrate the equipment and operator to the subsurface conditions by direction comparison of the installation performance and readings to the borehole results. These initial readings can then be used to assist with installation of piles away from the borehole locations to assess that the appropriate foundation material has been reached.

However, it is possible that the fill may be found suitable (unlikely for most of the site) as a foundation bearing stratum subject to finding and reviewing compaction records and completing further penetration testing (refer to Section 4.1.1) to assess with more confidence the load carrying capacity and potential settlements of the existing fill subgrade; notwithstanding, it is most likely that low allowable bearing pressures would have to be accepted (e.g.100kPa or less).

4.3 Earthworks and Retaining Walls

As mentioned in Section 1.1, the proposal is to suspend the entire trading floor slab and the warehouse structure, as well as the service roadway, on piled footings taken down to a competent foundation stratum (i.e. the bedrock). We concur with this fully suspended structure and driveway proposal. In addition, it is proposed to keep the existing thick bus depot concrete pavement where it is not higher than the proposed RL45m. The undercroft level is proposed to be designed to a have a gradual fall towards the western boundary to avoid having to strength or reconstruct the perimeter retaining walls.

Hence, the only earthworks required would be in the areas where existing bus depot concrete pavements are higher than RL45.0m (mostly near the Bonnyrigg Avenue eastern portion of the site). In these areas the proposal is to demolish and crush the existing concrete for reused as fill material under the new car park pavements. The main geotechnical issues with earthworks, including subgrade preparation, under floor slab (and pavement areas) are to do with the existing fill that appears to be variably compacted based on SPT and HP results and our observations during drilling. Another issue is to do with the reactivity and very low soaked CBR values of both the clay fill and underlying residual clay subgrade. We suggest that generous time and budget allowances be provided for subgrade improvement works. In these areas prior to placement of new fill for the new pavements we recommend the following subgrade preparation:

1. After removal of the concrete pavement and reaching the bulk excavation level of RL45m for the undercroft car parking level the exposed subgrade at the base of the excavation should be proof rolled with at least 8 passes of a heavy (not less than 12 tonne) smooth drum vibratory roller. The purpose of the proof rolling is to detect any soft or heaving areas. Caution is required when proof rolling near any neighbouring improvements and buried services.
2. The final pass should be undertaken in the presence of a geotechnician or geotechnical engineer, to detect any unstable or soft subgrade areas, and to allow for some further improvement in strength/compaction.
3. If dry conditions prevail at the time of construction then any exposed residual clay subgrade may become desiccated or have shrinkage cracks prior to pouring any concrete slabs. If this occurs then the subgrade must be watered and rolled until the cracks disappear.
4. Unstable subgrade detected during proof rolling should be locally excavated down to a stiff or sound base and replaced with engineered fill or further advice should be sought. Allowance should be made for either, tyning, aerating and drying the subgrade, or removal and replacement with a select imported fill, or lime/cement stabilisation. Such an excavation if deeper than 1.2m would have to be completed with battered sides of not steeper than 1 Vertical to 1.5 Horizontal. The earthworks contractor must ensure that during the backfilling earthworks that the engineered fill is well 'keyed' into the side batters of the excavation.
5. It is important to provide good and effective site drainage both during construction and for long-term site maintenance. The principle aim of the drainage is to promote run-off and reduce ponding. A poorly drained clay subgrade may become untraffickable when wet. The earthworks should be carefully planned and scheduled to maintain good cross-falls during construction.

Unless penetration testing (e.g. via borehole SPTs and a closely spaced grid of continuous Electronic Cone Penetration tests) of the deep fill areas is carried out and/or compaction records are provided for our review, to assess with more confidence the load carrying capacity and potential settlements of the fill mass, we recommend that the fill is not used as a bearing stratum for slabs and footings. If the slab is non-suspended then our preference and recommendation is that the existing uncontrolled fill be fully replaced with engineered fill. Earthworks recommendations provided in this report should be complemented by reference to AS3798.

Notwithstanding, in addition to potential settlement issues, the warehouse ground floor slab must be designed and constructed to withstand potential shrink/swell movements of the clay subgrade, which may be subject to free surface movements equivalent to those quoted in AS2870-2011, 40mm-60mm, for Class H1 clay subgrades.

The undercroft level is proposed to be designed to a have a gradual fall towards the western boundary to avoid having to strength or reconstruct the perimeter retaining walls. Additional loads should not be imposed on the existing gabion retaining walls.

However, if the existing gabion retaining walls are to be replaced by more substantial engineered retaining walls that can cope with additional loads or if the existing walls have to be checked by the structural engineers then the following geotechnical parameters may be employed.

Shoring systems or permanent cantilevered retaining walls may be designed based on a triangular earth pressure distribution using an active earth pressure coefficient, K_a, of at least 0.35 and a bulk unit weight of $20kN/m^3$. Where walls are restrained from some lateral movements, such as those propped by other structural elements in front of the wall, a higher earth pressure coefficient, K, of at least 0.6 should be used. These coefficients assume horizontal backfill surfaces and where inclined backfill is proposed the coefficients would need to be increased or the inclined backfill taken as a surcharge load. All surcharge loads should be allowed for in the design. Full hydrostatic pressures should be considered unless measures are undertaken to provide complete and permanent drainage of the ground behind the wall. Caution will be required not to overcompact and cause excessive lateral pressures on the retaining walls. Only small rollers or hand compaction should be used for fill compaction adjacent to any retaining wall.

4.3.1 Engineered Fill Specifications

Any fill used to backfill unstable subgrade areas, raise surface levels or backfill service trenches should be engineered fill. Materials preferred for use as engineered fill are well-graded granular materials, such as ripped or crushed sandstone, free of deleterious substances and having a maximum particle size not exceeding 75mm. Such fill should be compacted in layers not greater than 200mm loose thickness, to a minimum density of 98% of Standard Maximum Dry Density (SMDD).

We do not recommend the reuse of clay fill or residual clays at this site as engineered fill due to their very low soaked CBR values, high plasticity and reactivity.

The demolished concrete pavements may be re-used as engineered fill provided that they are suitably crushed to particle sizes of 75mm or less, and provided that there are no deleterious materials included.

Density tests should be regularly carried out on the fill to confirm the above specifications are achieved. The frequency of density testing should be at least one test per layer per $500m^2$ or three tests per visit, whichever requires the most tests. We recommend that full time Level 1 control of fill compaction, as defined in AS3798-2007, be adhered to on this site. Preferably, the geotechnical testing authority (GTA) should be engaged directly on behalf of the client and not by the earthworks subcontractor.

During construction of the fill platform runoff should be enhanced by providing suitable falls to reduce ponding of water on the surface of the fill. Ponding of water may lead to softening of the fill and subsequent delays in the earthworks program.

4.4 Pavement Design

The design of new pavements will depend on subgrade preparation, subgrade drainage, the nature and composition of fill excavated or imported to the site, as well as vehicle loadings and use. Refer to Section 4.3 on subgrade preparation and other earthwork procedures including engineered fill specifications and compaction control. It is important to understand that there is a chance that some settlement may still occur under pavements bearing on the existing fill, even after it is treated by proof rolling, especially in areas where deep fill was found. Increased maintenance and repair costs would be anticipated.

Various alternative types of construction could be used for the pavements. Concrete construction would undoubtedly be the best in areas where heavy vehicles manoeuvre such as truck turning and manoeuvring. Flexible pavements may have a lower initial cost but maintenance will be higher. These factors should be considered when making the final choice.

The new undercroft car parking pavement for most of the site is proposed to be constructed on top of the existing bus depot thick concrete pavement, which has performed reasonably well over most of its area under the use of heavy vehicles (buses). Hence, design of the new pavement may be checked on the basis that the existing concrete pavement will be acting like a concrete sub-base; hence, design may then proceed on subgrade CBR of 3%. Consideration should be given to providing a sand separation layer between the new concrete pavement and the underlying remaining bus depot concrete pavement layer.

In the areas higher than RL45m where the existing pavement is to be removed and some of the underlying fill is to be excavated to achieve the design levels, we advise the following options for new pavement design:

1. Design the pavements for a CBR value of 1% or an estimated subgrade reaction modulus (for concrete slabs or pavements) of 10kPa/mm (750mm diameter plate). We note that both the clay fill and residual clay fill subgrade have very low soaked CBR values (1%-1.5%).

<u>OR</u>

2. In the case of concrete pavements a bound sub-base of lean mix concrete of at least 200mm thickness may be considered; design may then proceed on subgrade CBR of 3%. Concrete pavements should be designed with an effective shear transmission of all joints by way of either doweled or keyed joints.

<u>OR</u>

3. Provide an appropriate selected fill layer as part of the overall pavement thickness. The selected fill may be well graded ripped or crushed sandstone with a minimum soaked CBR value of 10%. The pavement sections where imported fill is used to raise site levels may be designed taking into account the thickness and soaked CBR value of the imported fill material.

<u>OR</u>

4. Stabilise the subgrade to a depth of 200mm to 300mm by the addition of lime. When thoroughly mixed and recompacted to a minimum of 98% of SMDD, a reduction in reactivity along with substantial increase in strength will be achieved. As a guide, the addition of approximately 4% lime by dry weight of clay should result in a soaked CBR value of around 5% or an equivalent subgrade reaction modulus of 40kPa/mm. This should, however, be confirmed by laboratory testing. If lime stabilisation is undertaken, an experienced contractor with appropriate equipment should complete it. We note that use of lime close to existing neighbouring developments is generally not preferred unless an acceptable method of dust suppression can be adopted.

Further soaked CBR tests may be carried out on representative samples of the subgrade to obtain a large population of values to enable a proper statistical analysis to be performed and possibly an increase in the design CBR value. However, it should be borne in mind that even with more test values being obtained there will still be isolated pavement areas where the risk of potential failure and higher maintenance will occur due to the subgrade having a lower CBR value than the statistical characteristic value opted for design purposes. We recommend that in situ density tests be completed on the proof rolled and prepared subgrade to confirm that at least 98% STD has been achieved. If the existing fill is removed and replaced with imported fill, the CBR of the imported material may be taken into account. These design values should be confirmed by inspection and DCP testing of the subgrade following proof rolling.

All upper (base) course are recommended to be crushed rock to RTA QA specification 3051 (1994) unbound base and compacted to at least 100% of Standard Maximum Dry Density. All lower (sub-base) course are recommended to be crushed rock to RTA QA specification 3051 (1994) unbound base or ripped/crushed sandstone with CBR greater than 40%, maximum particle size of 60mm, well graded and Plastic Index less than 10. All lower course material should be compacted to an average of no less than 100% of SMDD, but with a minimum acceptance value of 98% of SMDD.

Concrete pavements are recommended to have a sub-base layer of at least 100mm thickness of crushed rock to RTA QA specification 3051 (1994) unbound base material (or equivalent good quality and durable fine crushed rock) which is compacted to at least 100% SMDD. Concrete pavements should be designed with an effective shear transmission of all joints by way of either doweled or keyed joints.

Careful attention to subsurface and surface drainage is required in view of the effect of moisture on the clay soils. Pavement levels will need to be graded to promote rapid removal of surface water so ponding does not occur on the surface of pavements.

5 GENERAL COMMENTS

The preliminary recommendations presented in this report include specific issues to be addressed during the DA, CC and construction phases of the project. In the event that any of the 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.

The long term successful performance of floor slabs and pavements is dependent on the satisfactory completion of the earthworks. In order to achieve this, the quality assurance program should not be limited to routine compaction density testing only. Other critical factors associated with the earthworks may include subgrade preparation, selection of fill materials, control of moisture content and drainage, etc. The satisfactory control and assessment of these items may require judgment from an experienced engineer. Such judgment often cannot be made by a technician who may not have formal engineering qualifications and experience. In order to identify potential problems, we recommend that a pre-construction meeting be held so that all parties involved understand the earthworks requirements and potential difficulties. This meeting should clearly define the lines of communication and responsibility.

Occasionally, the subsurface conditions between the completed boreholes may be found to be different (or may be interpreted to be different) from those expected. Variation can also occur with groundwater conditions, especially after climatic changes. If such differences appear to exist, we recommend that you immediately contact this office.

This report provides preliminary advice on geotechnical aspects for the proposed civil and structural design. As part of the documentation stage of this project, Draft Contract Documents and Specifications may be prepared based on our report. However, there may be design features we are not aware of or have not commented on for a variety of reasons. The designers should satisfy themselves that all the necessary advice has been obtained. If required, we could be commissioned to review the geotechnical aspects of contract documents to confirm the intent of our recommendations has been correctly implemented.

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.

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North Ryde, BC 1670
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SOIL TEST SERVICES

ABN 43 002 146 173

TABLE A
MOISTURE CONTENT, ATTERBERG LIMITS AND LINEAR SHRINKAGE TEST REPORT

Client:	JK Geotechnics	**Ref No:**	27813V
Project:	Due Dilligence Investigation	**Report:**	A
Location:	Corner of Elizabeth Drive and Bonnyrigg Avenue, Bonyrigg, NSW	**Report Date:**	23/10/2014
		Page 1 of 1	

AS 1289	TEST METHOD	2.1.1	3.1.2	3.2.1	3.3.1	3.4.1
BOREHOLE NUMBER	**DEPTH m**	**MOISTURE CONTENT %**	**LIQUID LIMIT %**	**PLASTIC LIMIT %**	**PLASTICITY INDEX %**	**LINEAR SHRINKAGE %**
1	1.50-1.95	12.0				
1	3.00-3.20	12.1				
1	4.10-4.50	7.7				
1	5.50-6.00	9.0				
2	0.90-2.00	20.8	57	20	37	15.0
2	1.80-02.00	7.2				
2	2.50-3.00	14.4				
2	3.50-4.00	8.8				
2	5.00-5.50	8.9				
2	5.70-6.00	7.1				
3	4.80-5.00	9.5				
3	5.50-6.00	9.5				
4	1.50-1.95	12.7				
4	4.00-4.50	7.7				
5	0.50-1.50	16.3	45	16	29	10.0
5	4.30-4.50	11.1				
5	5.50-6.00	10.7				
6	3.00-3.01	7.4				
6	4.00-4.50	6.8				
7	3.00-3.40	16.7				
7	5.70-6.00	6.1				

Notes:
- The test sample for liquid and plastic limit was air-dried & dry-sieved
- The linear shrinkage mould was 125mm
- Refer to appropriate notes for soil descriptions
- Date of receipt of sample: 13/10/2014

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SOIL TEST SERVICES
ABN 43 002 145 173

TABLE B
FOUR DAY SOAKED CALIFORNIA BEARING RATIO TEST REPORT

Client:	JK Geotechnics	**Ref No:**	27813V
Project:	Due Dilligence Investigation	**Report:**	B
Location:	Corner of Elizabeth Drive and Bonnyrigg Avenue, Bonnyrigg, NSW	**Report Date:**	23/10/2014
		Page 1 of 1	

BOREHOLE NUMBER	2	5
DEPTH (m)	0.90 - 2.00	0.50 - 1.50
Surcharge (kg)	4.5	4.5
Maximum Dry Density (t/m^3)	1.66 STD	1.81 STD
Optimum Moisture Content (%)	16.3	16.8
Moulded Dry Density (t/m^3)	1.63	1.78
Sample Density Ratio (%)	98	98
Sample Moisture Ratio (%)	98	98
Moisture Contents		
Insitu (%)	19.3	22.9
Moulded (%)	16.0	16.4
After soaking and		
After Test, Top 30mm(%)	29.7	24.9
Remaining Depth (%)	21.4	20.7
Material Retained on 19mm Sieve (%)		
Swell (%)	4.0	3.0
C.B.R. value: @2.5mm penetration	1.0	1.5

NOTES:

- Refer to appropriate Borehole logs for soil descriptions
- Test Methods :
  - (a) Soaked C.B.R. : AS 1289 6.1.1
  - (b) Standard Compaction : AS 1289 5.1.1

NATA
NATA Accredited Laboratory Number:1327

Accredited for compliance with ISO/IEC 17025. This document shall not be reproduced except in full.

Authorised Signature / Date
(A. Tatikonda) 23/10/14

JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No.
1
1/1

Client: JOHN R. BROGAN & ASSOCIATES PTY LTD
Project: DUE DILIGENCE INVESTIGATION
Location: CNR. OF ELIZABETH DRIVE AND BONNYRIGG AVENUE, BONNYRIGG, NSW

Job No. 27813V
Date: 10-10-14
Method: SPIRAL AUGER JK350
R.L. Surface: N/A
Datum:
Logged/Checked by: M.W./F.V.

Groundwater Record	SAMPLES (ES, U50, DB, DS)	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DRY ON COMPLETION			0			FILL: Sandy gravel, fine to coarse grained grained concrete, igneous and shale, fine to medium sand, trace of ceramic fragments and root fibres.	D			GRASS COVER APPEARS POORLY COMPACTED
		N = 10 3,4,6	1		CH	SILTY CLAY: high plasticity, light grey mottled red brown, trace of root fibres and fine to medium grained ironstone gravel.	MC>PL	VSt	230 340 340	RESIDUAL
		N = 43 15,19,24	2		-	INTERBEDDED SILTY CLAY: high plasticity, light grey AND SHALE: light grey and grey.	MC<PL/ XW	H/ EL	590 >600 >600	
		N > 8 20,8/50mm REFUSAL	3		-	SHALE: grey.	DW	VL-L	>600 >600 >600	VERY LOW TO LOW 'TC' BIT RESISTANCE
			4			SHALE: grey and dark grey, trace of clay bands.				
			5							
			6			END OF BOREHOLE AT 6.0m				
			7							

JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No. **2**
1/1

Client: JOHN R. BROGAN & ASSOCIATES PTY LTD
Project: DUE DILIGENCE INVESTIGATION
Location: CNR. OF ELIZABETH DRIVE AND BONNYRIGG AVENUE, BONNYRIGG, NSW

Job No. 27813V **Method:** SPIRAL AUGER JK350 **R.L. Surface:** N/A
Date: 10-10-14 **Datum:**
Logged/Checked by: M.W./F.V.

Groundwater Record	Field Tests	Depth (m)	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DRY ON COMPLETION		0		FILL: Sandy gravel, fine to coarse grained sub rounded to angular igneous, with clay fines.	D			GRASS COVER APPEARS POORLY COMPACTED
	N = 7 3,4,3							
		1	CH	SILTY CLAY: high plasticity, light grey mottled red brown, trace of root fibres and fine to medium grained ironstone gravel.	MC≈PL	VSt-H	400 380	RESIDUAL
	N > 6 11,6/ 100mm REFUSAL						450 400 300	
		2	-	SHALE: grey.	DW	L		LOW 'TC' BIT RESISTANCE
					XW-DW	EL-VL		VERY LOW RESISTANCE
		3				L		LOW RESISTANCE
		4						PVC STANDPIPE INSTALLED TO 4m DEPTH, SLOTTED BETWEEN 4m & 1m, 2mm SAND FILTER PACK BETWEEN 4m & 1m, BENTONITE SEAL BETWEEN 1m & SURFACE, COMPLETED WITH GATIC COVER
				SHALE: grey and dark grey.				
		5				L-M		LOW TO MODERATE RESISTANCE
		6		END OF BOREHOLE AT 6.0m				
		7						

JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No.
3
1/1

Client: JOHN R. BROGAN & ASSOCIATES PTY LTD
Project: DUE DILIGENCE INVESTIGATION
Location: CNR. OF ELIZABETH DRIVE AND BONNYRIGG AVENUE, BONNYRIGG, NSW

Job No. 27813V
Date: 10-10-14
Method: SPIRAL AUGER JK350
R.L. Surface: N/A
Datum:
Logged/Checked by: M.W./F.V.

Groundwater Record	Field Tests	Depth (m)	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DRY ON COMPLETION		0		CONCRETE: 340mm.t				10mm DIA. REINFORCEMENT, 70mm TOP COVER
	N = 6 1,3,3		-	FILL: Silty clay, medium plasticity, brown, light grey and grey, trace of slag, ash, root fibres, fine to medium grained sand and fine to coarse grained shale and igneous gravel.	MC>PL		80 60 120 390	APPEARS POORLY COMPACTED
	N = 9 3,4,5	1, 2					90 150 70	
	N = 20 3,7,13	3	CH	SILTY CLAY: high plasticity, light grey mottled red brown and orange brown, trace of root fibres and fine to medium grained ironstone gravel.	MC>PL	VSt	250 270 250	RESIDUAL
	N > 14 4,14/75mm REFUSAL	4				VSt-H	380 520 >600	
		5	-	SHALE: dark grey, with clay bands.	DW	VL-L		VERY LOW TO LOW 'TC' BIT RESISTANCE
				SHALE: dark grey.				
		6		END OF BOREHOLE AT 6.0m				
		7						

JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No.
4
1/1

Client: JOHN R. BROGAN & ASSOCIATES PTY LTD
Project: DUE DILIGENCE INVESTIGATION
Location: CNR. OF ELIZABETH DRIVE AND BONNYRIGG AVENUE, BONNYRIGG, NSW

Job No. 27813V **Method:** SPIRAL AUGER JK350 **R.L. Surface:** N/A
Date: 10-10-14 **Datum:**
Logged/Checked by: M.W./F.V.

Groundwater Record	Field Tests	Depth (m)	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DRY ON COMPLETION		0		CONCRETE: 280mm.t				10mm DIA. REINFORCEMENT, 95mm TOP COVER
	N = 16 5,8,8		CH	SILTY CLAY: high plasticity, light grey mottled red brown, trace of root fibres and fine to medium grained ironstone gravel.	MC<PL	H	>600 >600 >600	RESIDUAL
	N = 42 16,20,22		-	SHALE: light grey and red brown.	XW	EL	>600 >600 >600	
		2		SHALE: dark grey, trace of red brown iron indurated bands and clay bands.	DW	VL		VERY LOW 'TC' BIT RESISTANCE
						L		LOW RESISTANCE
						M		MODERATE RESISTANCE
		6		END OF BOREHOLE AT 6.0m				

JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No. **5**

1/1

Client: JOHN R. BROGAN & ASSOCIATES PTY LTD

Project: DUE DILIGENCE INVESTIGATION

Location: CNR. OF ELIZABETH DRIVE AND BONNYRIGG AVENUE, BONNYRIGG, NSW

Job No. 27813V **Method:** SPIRAL AUGER JK350 **R.L. Surface:** N/A

Date: 10-10-14 **Datum:**

Logged/Checked by: M.W./F.V.

Groundwater Record	Samples (ES, U50, DB, DS)	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DRY ON COMPLETION			0			CONCRETE: 270mm.t				8mm DIA. REINFORCEMENT, 75mm TOP COVER
		N = 10 3,5,5			-	FILL: Silty clay, medium plasticity, brown, light grey and red brown, trace of fine to medium grained sand, ash and fine to coarse grained sandstone, shale and ironstone gravel.	MC<PL	St- VSt	230 150 170	APPEARS MODERATELY COMPACTED
		N = 7 5,3,4	1–2					VSt	320 250 210	
		N = 11 5,4,7	3		CH	SILTY CLAY: high plasticity, light grey mottled orange brown.	MC>PL	VSt	330 230 290	
			4		-	SHALE: dark grey.	DW	VL		VERY LOW 'TC' BIT RESISTANCE
ON 16-10-14			5			as above, but with red brown iron indurated bands, trace of clay seams.		L		LOW RESISTANCE PVC STANDPIPE INSTALLED TO 6m DEPTH, SLOTTED FROM 6m TO 3m, 2mm SAND FILTER PACK FROM 6m TO 0.5m, BENTONITE SEAL FROM 0.5m TO SURFACE, COMPLETED WITH GATIC COVER
			6			END OF BOREHOLE AT 6.0m				
			7							

JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No.
6
1/1

Client: JOHN R. BROGAN & ASSOCIATES PTY LTD
Project: DUE DILIGENCE INVESTIGATION
Location: CNR. OF ELIZABETH DRIVE AND BONNYRIGG AVENUE, BONNYRIGG, NSW

Job No. 27813V
Date: 10-10-14
Method: SPIRAL AUGER JK350
R.L. Surface: N/A
Datum:
Logged/Checked by: M.W./F.V.

Groundwater Record	Field Tests	Depth (m)	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DRY ON COMPLETION		0		CONCRETE: 320mm.t				8mm DIA. REINFORCEMENT, 55mm TOP COVER
	N = 3 2,1,2		-	FILL: Silty clay, medium to high plasticity, brown, trace of ash and fine to medium grained igneous, shale and ironstone gravel.	MC>PL		20 200 400	APPEARS POORLY COMPACTED
	N = 14 4,5,9	1–2					200 150 380	APPEARS MODERATELY COMPACTED
	SPT 7/10mm REFUSAL	3	-	SHALE: dark grey.	DW	VL-L		VERY LOW TO LOW 'TC' BIT RESISTANCE
		4		as above, but trace of red brown iron indurated bands.		L-M		LOW TO MODERATE RESISTANCE
		5–6						MODERATE RESISTANCE PVC STANDPIPE INSTALLED TO 5.85m DEPTH, SLOTTED 5.85m TO 2.85m, 2mm SAND FILTER PACK FROM 5.85m TO 0.5m, BENTONITE SEAL FROM 0.5m TO SURFACE, COMPLETED WITH GATIC COVER
		7		END OF BOREHOLE AT 6.2m				

SAMPLES: ES, U50, DB, DS

JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No.
7
1/1

Client: JOHN R. BROGAN & ASSOCIATES PTY LTD

Project: DUE DILIGENCE INVESTIGATION

Location: CNR. OF ELIZABETH DRIVE AND BONNYRIGG AVENUE, BONNYRIGG, NSW

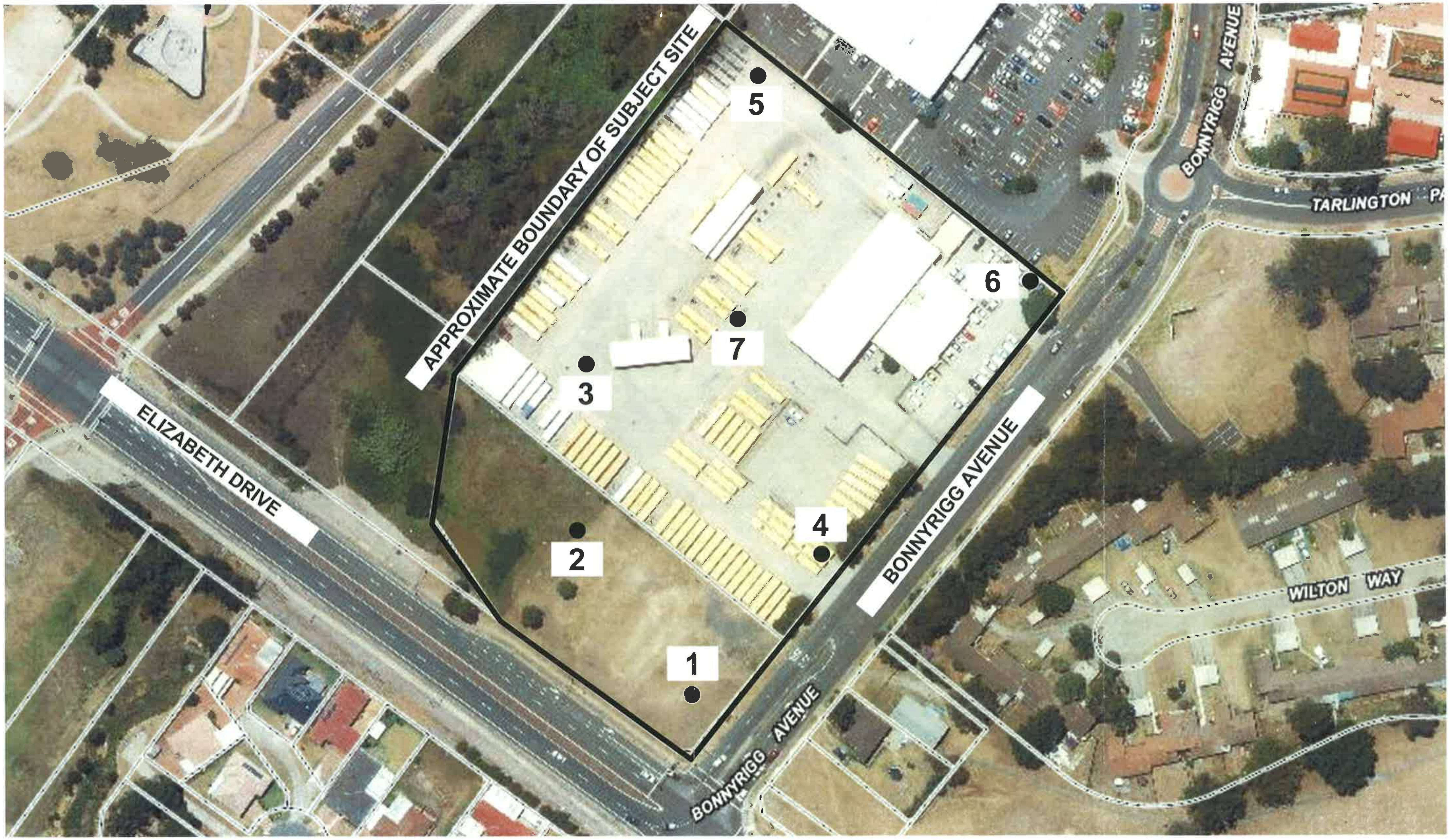
Job No. 27813V
Date: 10-10-14

Method: SPIRAL AUGER JK350

Logged/Checked by: M.W./F.V.

R.L. Surface: N/A
Datum:

Groundwater Record	Field Tests	Depth (m)	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DRY ON COMPLETION		0		CONCRETE: 300mm.t				10mm DIA. REINFORCEMENT, 65,75,95 &105mm TOP COVER
			-	FILL: Clayey sand, fine to coarse grained, light brown, with fine to coarse grained sandstone gravel.	M			
	N = 5 3,2,3			FILL: Silty clay, medium to high plasticity, brown and grey, trace of root fibres, slag, fine to coarse grained igneous and ironstone gravel.	MC>PL		340 240 170	APPEARS POORLY COMPACTED
	N = 8 1,4,4		CH	SILTY CLAY: high plasticity, light grey mottled red brown and orange brown, trace of root fibres.	MC>PL	VSt	270 240 350	RESIDUAL
	N > 43 8,11, 32/100mm REFUSAL	3				VSt-H	380 510 450	
			-	INTERBEDDED SILTY CLAY: high plasticity, light grey, AND SHALE: grey and dark grey.	H/ XW-DW	MC>PL / EL-VL		VERY LOW 'TC' BIT RESISTANCE
				SHALE: dark grey.	DW	L		LOW RESISTANCE
						L-M		LOW TO MODERATE RESISTANCE
		6		END OF BOREHOLE AT 6.0m				
		7						

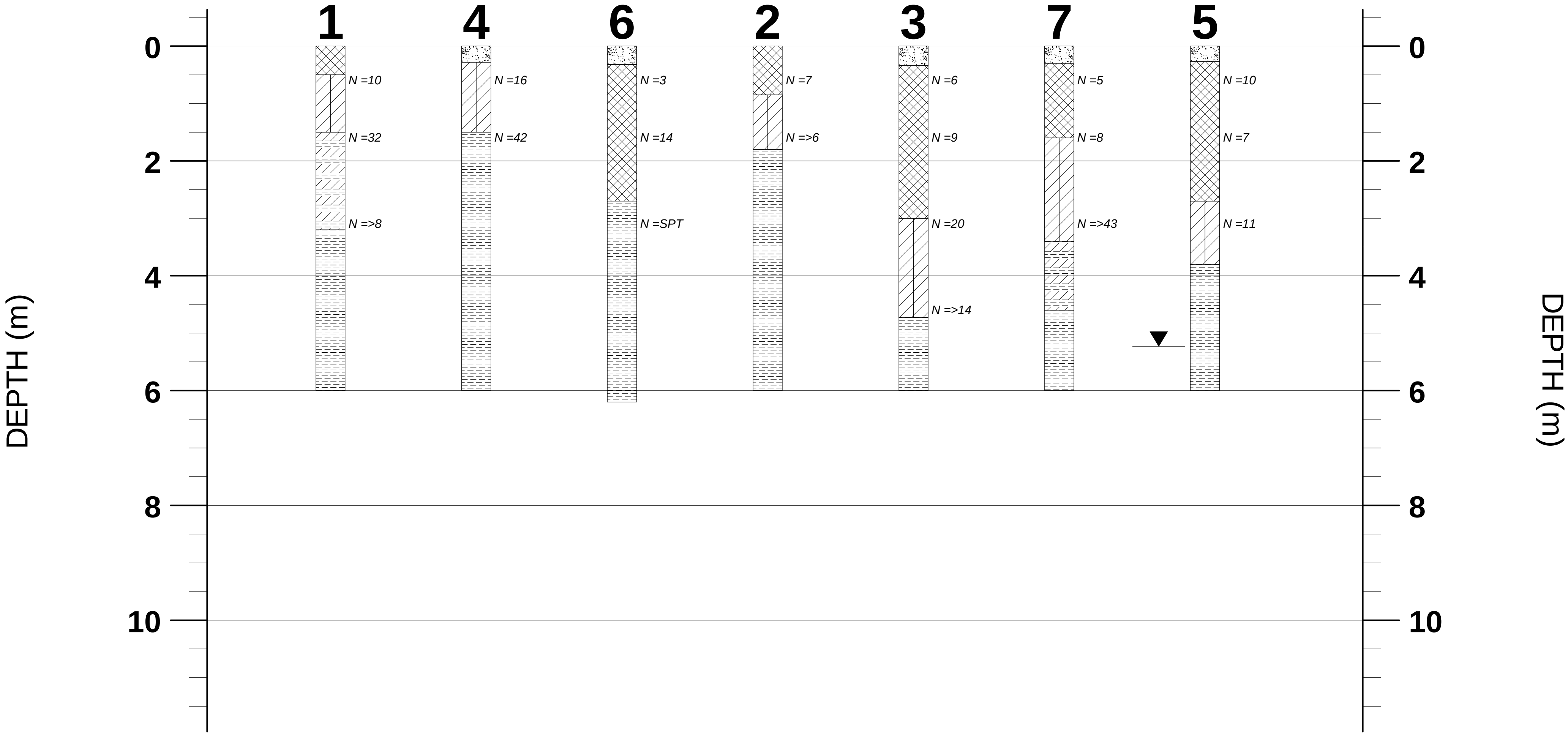


Scale (m): 0 60

JK Geotechnics
GEOTECHNICAL & ENVIRONMENTAL ENGINEERS

Title:	Report Number:	Figure Number:
BOREHOLE LOCATION PLAN	27813V	1

GRAPHICAL BOREHOLE SUMMARY



Fill

Silty Clay

Interbedded Shale and Clay

Shale

Concrete

Observed water level

N SPT "N" VALUE

Nc SOLID CONE BLOW COUNTS PER 150mm

NOTE: REFER TO BOREHOLE LOGS

Scale: 1 : 100 (vert) ; NTS (horiz)

JK Geotechnics

Job No.: 27813V

Figure No.: 2

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

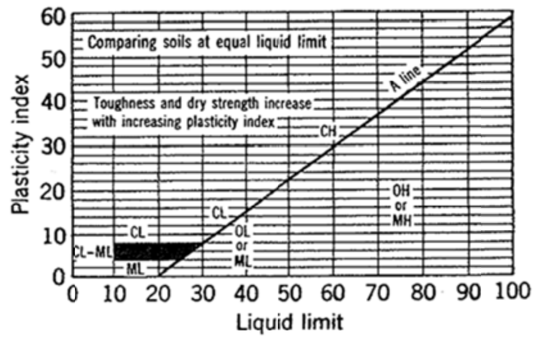
- CLAY SEAM
- SHEARED OR CRUSHED SEAM
- BRECCIATED OR SHATTERED SEAM/ZONE
- IRONSTONE GRAVEL
- ORGANIC MATERIAL

OTHER MATERIALS

- CONCRETE
- BITUMINOUS CONCRETE, COAL
- COLLUVIUM

Major Division	Subdivision	Type	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	$C_U = \frac{D_{60}}{D_{10}}$ 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	For undisturbed soils add information on stratification, degree of compactness, cementation, moisture conditions and drainage characteristics	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	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 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
			Identification Procedures on Fraction Smaller than 380 µm Sieve Size						
			Dry Strength (crushing characteristics)	Dilatancy (reaction to shaking)	Toughness (consistency near plastic limit)				
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	
			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	For undisturbed soils add information on structure, stratification, consistency in undisturbed and remoulded states, moisture and drainage conditions	
	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	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	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		

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.



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	