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REPORT

TO

KIRELA PTY LTD

ON

PRELIMINARY GEOTECHNICAL ASSESSMENT

FOR

PROPOSED REDEVELOPMENT OF THE SOUTH-EASTERN PRECINCT OF THE BAKEHOUSE QUARTER

AT

CORNER OF GEORGE STREET AND PARRAMATTA ROAD, NORTH STRATHFIELD, NSW

13 May 2010

Ref: 23767LBrpt

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1 INTRODUCTION

This report presents the results of a preliminary geotechnical assessment for the proposed redevelopment of the south-eastern precinct of the Bakehouse Quarter, located on the corner of George Street and Parramatta Road, North Strathfield, NSW. The investigation was commissioned by Mr David Wilcox of Urbis, on behalf of Kirela Pty Ltd, and was carried out in accordance with our proposal, Ref: P31684LB.

The development is currently at the application stage and the final extent of the proposed development is still being refined. Based on the available DA plans prepared by WAH Architects Pty Ltd (Job No. 21010SE, Drawing No. DA01 to DA17, dated 23 April 2010) we understand the proposed redevelopment will include the following:

- Alterations to the existing Building G2 to form a tavern in the north-western corner of the site. Part of the existing building on its northern and eastern sides will be demolished to allow construction of a new roadway.
- Construction of Building F in the north-eastern corner to comprise a drama theatre on the lower ground floor level, retail tenancies on the lower and upper ground floor levels, six basement car parking levels, two parking levels on Levels 1 and 2, and office space on Levels 3 to 12. Essentially the building will have seven below ground levels and thirteen above ground levels. The lowest basement will be a split level at RL-10.6m on the eastern side and RL-12.1m on the western side. Excavations for the proposed basement will be required to depths of about 20m to 22m. Part of the existing building on the northern end of the site will be demolished to allow construction of Building F.
- Construction of Building O on the corner of Parramatta Road and George Street (south-western corner of the site) comprising a restaurant and outdoor terrace on the ground floor and rehearsal rooms above the restaurant on the first floor.

- Construction of Building P on the southern side of the site comprising a two level theatre building.
- Construction of Building Q in the south-eastern corner comprising a hotel and function centre with a total of thirteen above ground levels. The exiting substation in this area of the site will need to be demolished to allow construction of Building Q.
- No basement levels are proposed below Buildings O, P or Q, with the lowest level of Building Q proposed at RL10.9m and the lowest level of Buildings P and Q proposed at RL7.4m. This will require excavations ranging from about 0.5m adjacent to George Street to a maximum of about 2.5m at the eastern end of Building P. Building Q will be up to about 1.5m higher than the existing ground surface.
- The external on grade car parking and roadways (George Lane and Railway Street) below the M4 motorway will remain unchanged with no development planed in that area. The only new pavements will be that for the new road on the northern and eastern sides of Building G2.

The purpose of this preliminary geotechnical assessment was to provide preliminary comments and recommendations on geotechnical issues for the proposed development to assist with planning and DA approval, in particular to provide comments on the risk of geotechnical failure and the measures that will need to be taken to provide stability during construction and in the long term. Preliminary comments are provided on likely RailCorp requirements, excavation, retention, footings, subgrade preparation, pavements and further geotechnical investigations. To aid in our assessment we have also completed limited subsurface investigations, in order to assess the nature of the upper soils and weathered rock. We also have reviewed results of previous geotechnical investigations for Building A and L at the western end the Bakehouse Quarter precinct, about 200m to 300m to the west and south-west of the subject site.

2 INVESTIGATION PROCEDURE

The scope of this investigation was limited to one day of fieldwork, where boreholes BH1 to BH6 were auger drilled using our truck mounted JK350 drilling rig to depths of 6m below the existing ground surface. The scope of the fieldwork is not intended to be sufficient for detailed design, but was carried out to obtain preliminary information on rock levels within the one day allowed for the fieldwork. The extent of the proposed basements below Building F was not advised until after the fieldwork had been completed. Since the basement for Building F are proposed to extend to depths significantly greater than the depth of our investigation, more detailed geotechnical investigations will be required to allow detailed design as recommended in Section 4.7 of this report.

The borehole locations, as shown on the Figure 1, were set out by taped measurements from existing surface features and inferred site boundaries. The approximate surface levels of the boreholes were estimated by interpolation between spot levels shown on the supplied unreferenced survey plan. The datum of the levels is not shown on the survey and so has been taken as an 'assumed' site datum. Prior to drilling, the borehole locations were checked for underground services using electronic service detection equipment.

The apparent compaction of the fill and the strength of the natural soils was assessed from Standard Penetration Test (SPT) 'N' values, supplemented by hand penetrometer tests carried out on cohesive samples recovered from the SPT split spoon sampler. The strength of the underlying weathered shale bedrock was assessed by observation of the drilling resistance of a tungsten carbide (TC) bit attached to the auger, together with examination of the recovered rock chips and subsequent correlation with laboratory moisture content test results. It must be noted that this method of rock strength assessment is approximate and variations of one strength order should not be unexpected. As part of the detailed design further geotechnical investigations, including cored boreholes, will be required to more thoroughly assess the rock strength.

Groundwater observations were made during and on completion of drilling. Further measurements of groundwater levels within the boreholes were not possible as they had to be backfilled following completion since they were located within a public car park.

Our engineering geologist, Mr Adam Mitchell, set out the borehole locations, nominated the sampling and testing locations, and prepared logs of the strata encountered. The borehole logs, which include field test results and groundwater observations, are attached to this report together with a set of Report Explanation Notes, which describe the investigation techniques, and their limitations, and define the logging terms and symbols used.

Selected samples were tested by Soil Test Services Pty Ltd (STS), a NATA registered laboratory, to determine moisture contents, Atterberg limits linear shrinkages and soil pH. The laboratory results are summarised in the attached Table A. Contamination testing of the site soils was outside the scope of this geotechnical investigation as we understand that contamination testing has already been carried out by others.

3 RESULTS OF INVESTIGATION

3.1 Site Description

The site is located within gently undulating topography and on the side of west facing hillside. The site itself undulates somewhat, with a slight depression in the centre of the site beneath the M4 motorway. Overall, the site slopes down towards the west at about 3° to 4°. The M4 motorway crosses the site diagonally, as shown on Figure 1, and is elevated above the surrounding ground surface.

The site is predominantly asphaltic concrete covered, with some concrete areas and grassed areas along the existing roadways. The site is predominantly being used as a car park for the Bakehouse Quarter Precinct, with access along George Lane and Railway Street. The pavements are generally in poor condition. In the north-eastern corner is a single storey brick building that is being used as part of the parking areas, adjacent to the on-grade parking area where BH1 and BH2 are located. In the north-western corner is Building G2, which is a single storey brick building that appeared to be in good external condition. In the south-eastern corner is a single storey brick sub-station building, which also appeared to be in good external condition.

The site is bound to the south by Parramatta Road, to the west by George Street, to the north by one and two storey brick buildings currently part of the Bakehouse Quarter, and to the east by Railway Street, which runs parallel with the adjacent railway line.

The railway line to the east is up to about 1.2m higher than the existing site level. It is retained by a brick wall, in fair condition, adjacent to the parking area to the north of George Lane, and then a graded embankment at about 20° to 30° along the length of Railway Street. The site is retained on the western and southern sides by a combination of brick and concrete retaining walls. These retaining walls range from about 0.4m high in the south-eastern corner of the site, to about 1.25m high in the south-western corner, with the walls along the George Street boundary about 0.9m high. The walls are of variable condition, generally being in good condition along most of the southern boundary, but poor in the south-western corner and along the western boundary. These poor quality walls are cracked, leaning over at the top by up to about 0.1m and bulging in parts.

3.2 Subsurface Conditions

Reference to the 1:100,000 Geological Map of the Sydney Region indicates that the site is underlain by Ashfield Shale which consists of dark grey shale with laminites. The boreholes encountered fill covering residual silty clays that graded into the shale bedrock. Further comments on the subsurface conditions encountered are provided below. A graphical summary of the borehole information is presented as Figure 2. Reference should be made to the borehole logs for detailed descriptions of the subsurface conditions encountered.

Pavements

Asphaltic concrete was initially penetrated in all boreholes of 50mm to 180mm thickness.

Fill

Fill was encountered in all boreholes to depths ranging from 0.6m to 2.2m. The fill comprised sandy gravel, clayey sandy gravel, gravelly silty clay and silty clay, with the gravel comprising sandstone, igneous, concrete, brick and tile. Based on the SPT 'N' values the fill was generally assessed to be moderately compacted. Concrete was encountered within the fill in BH6, at a depth of 1.3m and was 100mm thick.

Residual Silty Clay

The residual silty clay was assessed to be of medium or high plasticity and of very stiff to hard strength.

Weathered Shale

Shale was encountered in all boreholes at depths ranging from 2.6m to 5.1m. Within BH1, the shale was initially extremely weathered and of extremely low strength, becoming distinctly weathered and of low strength below a depth of 3.8m.

In the remaining boreholes, the shale was initially distinctly weathered and of very low to low strength, which improved to at least low strength with depth.

Groundwater

No groundwater seepage was encountered during drilling. Groundwater was measured on completion of BH1 and BH2 at depths of 4.9m and 4.1m, respectively. All other boreholes were dry on completion.

3.3 Laboratory Test Results

Based on the Atterberg limits and linear shrinkage test results, the silty clays tested are of medium or high plasticity and are assessed to have a high potential for shrink/swell movements with changes in moisture content. The ph of the soil tested ranged from 5.7 to 5.1, indicating that the soils are moderately acidic.

The moisture content test results on samples of the shale showed reasonably good correlation with our field assessments of rock strength.

4 COMMENTS AND RECOMMENDATIONS

4.1 Geotechnical Issues and Further Geotechnical Investigations

Based on the results of this preliminary geotechnical investigation the main geotechnical issues for the proposed development are as follows:

- The site is located adjacent to the rail corridor and the design and construction of the development will need to take into account the requirements of RailCorp. We expect that design of the deep basement, particularly the retention design, will need to be reviewed by RailCorp as part of the approval process. To allow such design to be completed, detailed geotechnical investigations will be required, and should include cored boreholes taken to depths below the proposed excavations and detailed geotechnical analysis. Further comments on possible RailCorp requirements are provided in Section 4.2 below.

- Excavation for the proposed basement below Building F will be required to depths of about 20m to 22m. Excavation is also required for Building P to a maximum depth of about 2.5m. The shallow excavation for Building P may be formed at temporary batters, where space permits, but the deeper excavation will need to be supported by retention systems installed prior to the start of excavation. This is particularly the case for the Building F basement as it is proposed adjacent to the existing buildings to the north and west and the boundary with the railway line to the east. The retention system will need to extend to below the base of the proposed excavation even though shale bedrock will be encountered, due to the likelihood of adversely inclined joints within the shale.
- During excavation of the shale within the Building F basement the use of rock excavation equipment will be required. If hydraulic rock hammers are used they must be used with care due to the risk of damage to the adjacent buildings from vibrations generated by such equipment. The vibrations transmitted to the adjacent buildings must be monitored during the works and contingency plans put in place in case the vibrations are excessive.
- Groundwater seepage is expected to occur into the basement excavation, but given the low permeability of the residual clays and shale such seepage should be controllable using conventional sump and pump techniques. Areas of more concentrated seepage may occur at joints and bedding partings within the shale and these may require local treatment. Drainage will need to be provided in the long term behind the basement walls and below the basement slab.
- Since the proposed Building Q will be higher than the existing levels, the building will either need to be suspended above the ground surface or the areas filled. However, given the size of the proposed building of thirteen stories, it will need to be supported on piles founded within the shale bedrock. The floor slab for Building Q will extend over the lowest level of Building P, so we

recommend that the slabs be designed as fully suspended slabs supported on the piles founded within the shale. This will eliminate the need for excavation of the existing fill and placement of fill as engineered fill. Any fill may then be placed as form fill with a lower compaction specification and level of quality control and testing.

- We assume that the alterations to Building G2 will not place any additional loads on the existing building footings. If additional loads are envisaged further investigations may be required to determine the existing footings of the building and the foundation material to assess if underpinning of the footings is required.

Further comments on the above issues are provided within the following sections of this report. The comments provided herein are only of a preliminary nature and must be amplified to assist with detailed design following more detailed geotechnical investigations. The scope of the more detailed geotechnical investigations is discussed within this report and summarised in Section 4.7.

As can be seen from our comments above the major geotechnical issues are associated with design and construction of the deep basement for Building F. We consider that provided the preliminary recommendations provided within this report are followed and further more detailed geotechnical investigations are carried out, the risk of geotechnical failure and instability would be within acceptable levels and would be no higher than other similar developments.

4.2 RailCorp Requirements

A deep basement is proposed below Building F, which will be adjacent to the rail corridor. From our experience RailCorp will require detailed geotechnical investigations and analysis to be completed; followed by review of the geotechnical investigation and analysis results and the retention system design. We expect that RailCorp will require deep cored boreholes drilled to below the depth of the proposed excavations and possible insitu stress testing of the rock. As part of the detailed design of the retention systems it is likely that at least 2D, but possibly 3D finite element analysis may be required to assess the movements that may affect the rail corridor. Such analysis will need to be carried out in consultation with the structural design of the retention system.

The detailed geotechnical investigations and analysis will take quite some time to complete and time will then be required for RailCorp to review the geotechnical investigations and analysis and the retention design. Allowance must be made within the project time table for completion of these works and you may wish to commence such works early to avoid delays. We recommend that RailCorp be contacted as soon as possible to determine their requirements so that the scope of the future geotechnical investigations can determined.

It is likely that RailCorp will also require significant monitoring during construction to assess the actual movements that affect the rail corridor. Survey monitoring is likely to be required of the retaining wall themselves, at the site boundary and within the rail corridor. In addition, a dilapidation survey of the rail corridor will probably be required prior to the start of the construction works.

4.3 Excavation

We understand that the buildings that will remain adjacent to the proposed excavation and the subject site are owned by the same party. Therefore, the need for dilapidation surveys on the adjoining buildings will depend on contractual arrangements between the site owner and the excavation contractor, and the requirements of the building owner. If the adjacent buildings were owned by others we would recommend that dilapidation surveys be completed prior to the start of the excavation. Nevertheless, we recommend that if possible the renovations works proposed for Building G2 (to the west of the proposed excavation) be delayed until after the basement has been completed so that if any damage is caused by the excavation it can be repaired as part of the renovation works. As detailed above, it is likely that RailCorp will require a dilapidation survey of the rail corridor to be prepared prior to the start of the excavation.

Excavation for the proposed Building F basement will be required to depths of about 20m to 22m and will encounter surface fill, residual silty clays and weathered shale below depths of about 2.5m to 4m. The excavation for Building P to depths of about 2.5m will encounter fill and residual silty clays, and possibly the upper layers of the shale within the deepest area of excavation.

Excavation of the soils will be achievable using conventional earthmoving equipment, such as the buckets of hydraulic excavators. The upper shale of extremely low to very low strength will also be able to be excavated using such equipment. Excavation of shale of low or higher strength will require assistance with rock excavation equipment, such as hydraulic rock hammers, ripping hooks, rotary grinder, or rock saws.

From our previous geotechnical investigations of other areas at the western end of the Bakehouse Quarter, it is likely that the deep excavations for the Building F basement will encounter shale of medium to high strength and this may represent hard rock excavations and productivity may be limited. The excavation contractor must make their own assessment of the excavation equipment required based on the results of the more detailed geotechnical investigation involving cored boreholes drilled to below the depth of the proposed excavations.

Hydraulic rock hammers must be used with care due to the risk of the vibrations generated by such equipment damaging the adjacent buildings or affecting the adjacent railway. If hydraulic rock hammers are used we recommend that vibration monitors be set up to quantitatively monitor the transmitted vibrations to the buildings to the north and west during the works. Reference should be made to the attached Vibration Emission Design Goals sheet for acceptable vibrations limits for buildings. The railway to the east may also have vibration limits and the asset owner (RailCorp) should be contacted to determine their acceptable limits and the need for vibration monitoring.

Where the transmitted vibrations are excessive it would be necessary to change to alternate excavation equipment, such as ripping hooks, rotary grinders or rock saws. A rock saw could be used to cut a slot around the excavation perimeter before excavation using a rock hammer, to reduce the transmitted vibrations. However, the effectiveness of such an approach must be confirmed by vibration monitoring.

4.4 Groundwater

Groundwater was measured on completion of BH1 and BH2 at depths of 4.9m and 4.1m and groundwater seepage will occur into the basement excavation for Building F. We do not expect that significant seepage will occur into the excavation for Building P, but some seepage may occur at the soil/rock interval (if exposed) particularly during and following rainfall.

Given the low permeability of the silty clays and weathered shale we expect that the groundwater seepage into the Building F basement excavation should be adequately controlled using conventional sump and pump techniques. However, the seepage into the excavation should be monitored and adjustments made as necessary for the drainage system to accommodate the actual flows. Seepage flows may increase into the excavation during and following rainfall.

In the long term, drainage should be provided behind the basement walls and below the basement slab. The drainage system should lead to sumps containing automatic and fail proof pumps to reduce the risk of basement flooding. Hydrostatic relief valves should also be installed. The long term groundwater levels should be determined as part of the detailed geotechnical investigations. Standpipes should be installed on site and data loggers installed into the standpipes to obtain longer term readings of groundwater levels. The standpipes should be installed early in the investigation/design process so that as much data as possible on groundwater levels can be obtained.

4.5 Retention

Where space permits, such as for the shallower excavation on the southern side of the site, temporary batters may be adopted, and these should be no stepper than 1 Vertical in 1 Horizontal (1V:1H). Such batters should remain stable in the short term provided all surcharge loads, including construction loads, are kept well clear of the crest of the batters. Permanent batters, if required, should be no steeper than 1V:2H, but flatter batters of the order of 1V:3H may be preferred to allow access for maintenance of vegetation. All permanent batters should be covered with topsoil and planted with a deep rooted runner grass, or other suitable coverings, to reduce erosion. All stormwater run-off should be directed away from all temporary and permanent slopes to also reduce erosion.

Permanent small height (say less than 3m) cantilevered retaining walls, where adjacent ground movements can be tolerated, may be designed based on a triangular earth pressure distribution using an active earth pressure coefficient, K_a, of 0.33 and a bulk unit weight of $20kN/m^3$.

Where batters cannot be accommodated, or for the Building F basement excavation, full depth retention systems should be installed prior to the start of the excavation. Although shale will be encountered within the deep basement excavation, we do not recommend leaving the shale unsupported, even if it is of medium to high strength; as there is the risk of adversely inclined joints within the shale being present. Such adversely inclined joints are known to exist within the shales, and if the cut faces are unsupported and the joints are encountered it could result in unstable wedges falling from the cut faces. Stabilisation of such wedges during excavation is not practical, as when the base of such joints are exposed it is often too late to install additional support. Therefore, we recommend that the retention system for Building F extends for the full depth of the excavation.

Where buildings or movement sensitive services are present adjacent to the excavation, such as on the northern and western sides of Building F, anchored or propped contiguous or secant pile retaining walls would need to be adopted. Such rigid walls will also be required along the eastern boundary with the railway line as these walls will need to satisfy the stringent RailCorp requirements for ground movements. Along the southern side of the excavation, where no existing buildings are located, less rigid soldier pile walls with shotcrete infill panels may be considered.

Since we expect that the retention systems will extend into good quality shale of medium or high strength consideration could be given the adoption of contiguous or secant pile walls until good quality shale is encountered and then only extend selected piles to below the base of the excavation to form soldier pile walls within the good quality shale. The depth of the contiguous or secant piles will depend on the results of the detailed geotechnical investigation, but we expect that they would be required to depths of at least 7m to 8m. Walers and anchors would still be required though the lower soldier pile portion of the walls below the base of the contiguous portion.

Bored piers may be used for the retaining walls, but some difficulties with groundwater seepage should be expected and may require the use of temporary liners, pumps and tremie concreting. Alternatively, auger, grout injected (CFA) piles may be used to overcome such difficulties. The strength of the rock may also determine the pile type adopted.

Temporary lateral support should be provided by internal props or anchors with lateral support provided progressively as each restraining point is uncovered. Permission will need to be obtained from the owners of the adjoining properties before installation of anchors below those properties. Such permission can take time to obtain and we recommend that the permission be sought as early as possible to allow time for negotiation. Permanent lateral support would be provided by the floor slabs inside the basement.

Preliminary design of piled walls may be based on a trapezoidal earth pressure distribution of magnitude 8H kPa within the soils and extremely low strength shale (where H is the retained height of the soils and extremely low strength shale in metres) where adjacent buildings or movement sensitive services are located within 2H of the wall. Where buildings are located beyond 2H of the wall, the lateral pressure for the soils and extremely weathered shale may be reduced to 6H kPa. These maximum pressures should be held constant for the central 50% of the trapezoidal distribution. For the shale of low or greater strength, the walls should be designed to provide overall stability for a wedge formed by a joint inclined at 45°, plus allowance for a nominal pressure of 10kPa to allow for support of other small wedges. These lateral pressures will need to be confirmed as part of the detailed geotechnical investigation when cored boreholes are drilled to below the base of the proposed excavation.

The above lateral pressures and coefficients assume horizontal backfill surfaces and where inclined backfill is proposed the pressures or coefficients should be increased or the inclined backfill taken as a surcharge load. All surcharge loads, including traffic loads, adjacent structures, etc, should be allowed for in the design. Full hydrostatic pressures should be allowed unless measures are taken to provide complete and permanent drainage behind the walls.

Anchors should have their bond within shale of at least low strength, and may be preliminarily designed based on a maximum bond stress of 250kPa. Higher bond stresses may be appropriate within shale of medium or high strength, possibly of the order of 400kPa, subject to confirmation as part of the detailed geotechnical investigation. Anchors should have a minimum free length of 4m and minimum bond length of 4m, and should be formed outside a line drawn up at 45° from the bulk excavation level. All anchors should be proof loaded to at least 1.3 times their design working load before locking off at about 80% of the working load, with lift-off tests carried out following locking off to confirm that the anchors are holding the required loads. Additional lift-off tests should be carried out on at least 10% of the anchors 24 to 48 hours following lock off to confirm that the anchors are holding their load.

Passive toe resistance of the piled walls may be estimated based on a maximum allowable lateral resistance of 250kPa for shale of at least low strength. Higher passive resistance, say 350kPa, may be used within shale of medium to high strength, subject to the results of the detailed geotechnical investigations. The passive resistance should be ignored to at least 0.5m below the base of the excavation, including footing and service excavations.

4.6 Footings

For Building F the basement will be excavated into the shale bedrock so pad or strip footings founded within the shale would be appropriate. For the other buildings, the footing system adopted would depend on the size of the building, the excavations required and the earthworks that can be completed. For lightly loaded buildings of one or two stories, shallow footings founded within controlled, engineered fill or the residual silty clays may be appropriate. For larger buildings, such as Building Q, piles founded within the shale bedrock would be required.

Uniform footing systems should be adopted for all buildings, with either all footings founded within engineered fill/residual silty clays or all within the shale bedrock. If excavations for one building encounter the shale or the building is connected to another building that is supported on footings founded within the shale, all footings must be founded within the shale to provide uniform support and reduce the risk of differential settlements.

We are unaware of any records of placement or compaction control of the existing fill and as such it must be considered 'Uncontrolled' and is not suitable for support of footings or floor slabs. If footings or slab are to be supported on shallow footings founded within the fill, all existing uncontrolled fill would need to be fully excavated and replaced with controlled, engineered fill placed and compacted in thin layers. If such earthworks are required further geotechnical advice should be sought.

Engineered fill should preferably comprise well graded granular materials, such as ripped rock or crushed sandstone, free of deleterious substances and having a maximum particle size not exceeding 75mm. Such fill should be compacted in horizontal layers of not greater than 200mm loose thickness, to a density of at least 98% of Standard Maximum Dry Density (SMDD). Density testing of the fill should be carried out to confirm that the required density has been achieved.

Shallow footings founded within controlled, engineered fill may be designed based on an allowable bearing pressure of 100kPa. Footings founded within the residual silty clays of at least very stiff strength may be designed based on an allowable bearing pressure of 200kPa. Such shallow footings should be designed to accommodate shrink/swell movements similar to a Class H site in accordance with AS2870.

Footings founded within a nominal socket of at least 0.3m into shale of extremely low strength may be designed based on an allowable bearing pressure of 700kPa. Where footings are taken deeper to found within shale of at least low strength an allowable bearing pressure of 1000kPa may be used.

Higher bearing pressures would be appropriate within shale of medium or high strength, if encountered during the drilling of cored boreholes within the shale. As a guide, we expect that allowable bearing pressures in the order of 3500kPa to 6000kPa may be appropriate within the medium to high strength shale. The extent of the detailed geotechnical investigations and subsequent construction inspections and testing will depend on what bearing pressure is required for footing design and we recommend that advice be obtained from the structural engineer on what bearing pressure would be appropriate in order to determine the final extent of the geotechnical investigations and subsequent testing.

The footing excavations and pile drilling should be inspected by a geotechnical engineer during construction. If high bearing pressures are adopted additional cored boreholes at close spacing or spoon testing of pad footing excavations may be required to confirm that the appropriate quality shale has been encountered.

The soil pH values indicate that the soils are moderately acidic at 5.7 to 5.1. These soils would be classified as 'mild' exposure classification for concrete piles in accordance with Table 6.4.2(C) of AS2159-2009 'Piling – Design and Installation'. For steel piles, the soils would be classified as 'non-aggressive' in accordance with Table 6.5.2(A) of AS2159-2009. Additional protection may be required for buried concrete in accordance with Table 6.4.3 of AS2159-2009.

4.7 Subgrade Preparation and Pavements

Where new pavements are required for the new roadway to the north and east of Building G2, the subgrade should be prepared by proof rolling within a smooth drum roller of at least 7 tonnes dead weight. The final pass of the proof rolling should be inspected by a geotechnical engineer to detect any weak subgrade areas. Any weak areas detected should be excavated to a sound base and the excavated material replaced with engineered fill.

The appropriate soaked CBR values for the design of the pavements should be determined as part of the detailed geotechnical investigations. However, as a guide, for the silty clay soils encountered within our boreholes we would expect CBR values of the order of 2% to 3%. Adequate drainage should be provided to prevent moisture ingress into the pavement and subgrade

Concrete pavements should have a subbase 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% of SMDD. Concrete pavements should be designed with an effective shear transmission at all joints by way of either doweled or keyed joints.

4.8 Further Geotechnical Investigations

As detailed within this report, detailed geotechnical investigations will be required to allow detailed design of the proposed development and to satisfy RailCorp requirements. In summary the detailed geotechnical investigations should include the following. However, the final scope of the investigations should be determined following discussion with the structural engineer on the bearing pressures required for footing design.

- Additional deep cored boreholes drilled in the footprint of the proposed Building F. These boreholes will need to be drilled to depths in excess of the proposed basement excavations and will require core drilling of the shale bedrock. The coring of the shale will also allow the use of higher allowable bearing pressures for the design of footings, which will be required given the size of the proposed building. Depending on RailCorp's requirements insitu stress testing of the rock may be required as part of the geotechnical investigation.
- Installation of standpipes within the deep boreholes for Building F, with data loggers to measure the longer term groundwater levels.
- Additional boreholes drilled in the footprint of the proposed Building Q. Given the size of this hotel building, cored boreholes should be drilled to allow the use of higher bearing pressures for the design of the piles to support this building.
- Additional boreholes within other areas of the site to provide a greater site coverage. This boreholes may either be auger drilled if only low bearing pressures are adopted, or core drilled if higher bearing pressures are required.
- Soaked CBR testing of the subgrade soils where new pavements are proposed.
- Detailed geotechnical analysis of the retention system for Building F basement to satisfy review by RailCorp requirements. Such analysis is likely to comprise at least 2D finite element analysis, but possible 3D analysis, to assess wall movements and movements within the rail corridor.

5 GENERAL COMMENTS

The recommendations presented in this report include specific issues to be addressed during the construction phase of the project. In the event that any of the construction phase recommendations presented in this report are not implemented, the general recommendations may become inapplicable and Jeffery and Katauskas Pty Ltd 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 advice on geotechnical aspects for the proposed civil and structural design. As part of the documentation stage of this project, Contract Documents and Specifications may be prepared based on our report. However, there may be design features we are not aware of or have not commented on for a variety of reasons. The designers should satisfy themselves that all the necessary advice has been obtained. If required, we could be commissioned to review the geotechnical aspects of contract documents to confirm the intent of our recommendations has been correctly implemented.

A waste classification will need to be assigned to any soil excavated from the site prior to offsite disposal. Subject to the appropriate testing, material can be classified as Virgin Excavated Natural Material (VENM), General Solid, Restricted Solid or Hazardous Waste. 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.

If there is any change in the proposed development described in this report then all recommendations should be reviewed.

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. Copyright in this report is the property of Jeffery and Katauskas Pty Ltd. 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.

Should you have any queries regarding this report, please do not hesitate to contact the undersigned.

For and on behalf of
JEFFERY AND KATAUSKAS PTY LTD.

Daniel Bliss
Senior Associate

Reviewed by:

Linton Speechley
Principal

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SOIL TEST SERVICES
ABN 43 002 145 173

TABLE A
SUMMARY OF LABORATORY TEST RESULTS

AS 1289	TEST METHOD	2.1.1	3.1.2	3.2.1	3.3.1	3.4.1	4.3.1
BOREHOLE NUMBER	DEPTH m	MOISTURE CONTENT %	LIQUID LIMIT %	PLASTIC LIMIT %	PLASTICITY INDEX %	LINEAR SHRINKAGE %	pH TEST
1	1.50-1.95	14.0	46	16	30	13.5	
1	3.00-3.45	12.3					
1	4.00-4.50	10.1					
1	5.50-6.00	8.6					
2	4.00-4.50	12.6					
2	5.50-6.00	10.6					
3	4.00-4.50	6.5					
3	5.50-6.00	8.5					
4	0.60-0.95						5.5
4	1.50-1.95	20.4	54	18	36	15.0	5.3
4	4.00-4.50	6.5					
4	5.50-6.00	3.2					
5	1.50-1.95						5.7
5	4.00-4.50	8.0					
5	5.50-6.00	9.0					
6	1.50-1.95						5.1
6	3.00-3.45	18.5	53	18	35	15.0	5.3
6	5.50-6.00	9.2					

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

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BOREHOLE LOG

Borehole No. **1**
1/1

Client: KIRELA PTY LTD
Project: REDEVELOPMENT OF SOUTH EASTERN PRECINCT OF BAKEHOUSE QUARTER
Location: CNR. GEORGE STREET & PARRAMATTA ROAD, NORTH STRATHFIELD, NSW

Job No. 23767LB
Date: 14-4-10
Method: SPIRAL AUGER JK500
Logged/Checked by: A.M./
R.L. Surface: ≈ 10.6m
Datum: ASSUMED

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
			0		-	ASPHALTIC CONCRETE: 50mm.t FILL: Clayey sandy gravel, fine to coarse grained sub angular sandstone, concrete and igneous gravel, red brown, dark grey, fine to medium grained sand.	M	-	-	APPEARS MODERATELY COMPACTED
		N = 11 5,5,6			CL	SILTY CLAY: medium plasticity, light grey mottled orange brown, with ironstone gravel bands.	MC>PL	H	>400 >400 >400	RESIDUAL
		N = 18 5,9,9	1 2						>400 >400 >400	
		N = 37 13,19,18	3		-	SHALE: brown and grey, with clay lenses.	XW	EL	-	VERY LOW 'TC' BIT RESISTANCE
ON COMPLETION			4 5			SHALE: dark grey.	DW	L	-	LOW RESISTANCE WITH MODERATE BANDS
			6 7			END OF BOREHOLE AT 6.0m				

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BOREHOLE LOG

Borehole No. 2
1/1

Client: KIRELA PTY LTD
Project: REDEVELOPMENT OF SOUTH EASTERN PRECINCT OF BAKEHOUSE QUARTER
Location: CNR. GEORGE STREET & PARRAMATTA ROAD, NORTH STRATHFIELD, NSW

Job No. 23767LB **Method:** SPIRAL AUGER JK500 **R.L. Surface:** ≈ 8.8m
Date: 14-4-10 **Datum:** ASSUMED

Logged/Checked by: A.M./

Groundwater Record	Field Tests	Depth (m)	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
		0		ASPHALTIC CONCRETE: 180mm.t				
	N > 15 8,7,8/ 75mm REFUSAL		-	FILL: Clayey sandy gravel, fine to coarse grained sandstone and igneous gravel, red brown and dark grey, fine to medium grained sand.	M	-	-	
	SPT 20/100mm REFUSAL	1		FILL: Gravelly silty clay, low plasticity, dark grey, fine to coarse grained sub angular sandstone, concrete, ironstone and igneous gravel.	MC>PL			APPEARS WELL COMPACTED
	N = 31 11,11,20	2 3	CH	SILTY CLAY: high plasticity, yellow brown mottled light grey.	MC≈PL	H	- >400 >400 >400	RESIDUAL
ON COMPLETION		4	-	SHALE: dark grey.	DW	VL	-	VERY LOW 'TC' BIT RESISTANCE WITH LOW BANDS
		5				L		LOW RESISTANCE
		6		END OF BOREHOLE AT 6.0m				
		7						

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BOREHOLE LOG

Borehole No. 3

1/1

Client: KIRELA PTY LTD

Project: REDEVELOPMENT OF SOUTH EASTERN PRECINCT OF BAKEHOUSE QUARTER

Location: CNR. GEORGE STREET & PARRAMATTA ROAD, NORTH STRATHFIELD, NSW

Job No. 23767LB

Date: 14-4-10

Method: SPIRAL AUGER JK500

R.L. Surface: ≈ 9.1m

Datum: ASSUMED

Logged/Checked by: A.M./

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		-	ASPHALTIC CONCRETE: 50mm.t				
						FILL: Clayey sandy gravel, fine to coarse grained sub angular concrete, sandstone and igneous gravel, red brown and dark grey, fine to medium grained sand.	M	-	-	APPEARS MODERATELY COMPACTED
	DS	N = 8 3,3,5				FILL: Sandy gravelly clay, low plasticity, dark grey and red brown, fine to coarse grained sub angular sandstone and igneous gravel and fine to coarse grained sand.	MC≈PL			
			1		CH	SILTY CLAY: high plasticity, light grey mottled yellow brown.	MC>PL	VSt	-	RESIDUAL
	DS	N = 10 3,5,5							290 300 280	
			2							
						as above, but with ironstone bands.		H		
	DS	N > 14 9,14/ 150mm REFUSAL	3						>400 >400 >400	
					-	SHALE: dark grey.	DW	L-M		LOW 'TC' BIT RESISTANCE WITH MODERATE BANDS
			4							
			5							
			6			END OF BOREHOLE AT 6.0m				
			7							

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BOREHOLE LOG

Borehole No. **4**
1/1

Client: KIRELA PTY LTD
Project: REDEVELOPMENT OF SOUTH EASTERN PRECINCT OF BAKEHOUSE QUARTER
Location: CNR. GEORGE STREET & PARRAMATTA ROAD, NORTH STRATHFIELD, NSW

Job No. 23767LB
Date: 14-4-10
Method: SPIRAL AUGER JK500
R.L. Surface: ≈ 10.1m
Datum: ASSUMED
Logged/Checked by: A.M./

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		-	ASPHALTIC CONCRETE: 60mm.t				
		N = 23 9,9,14				FILL: Sandy gravel, fine to coarse grained sub angular concrete, brick, tile and igneous gravel, brown and red brown, with fine to medium grained sand.	D	-	-	APPEARS WELL COMPACTED
			1			FILL: Silty clay, medium plasticity, brown, with fine to medium grained sub angular to sub rounded ironstone gravel.	MC≈PL		>400 >400 >400	
		N = 18 4,7,11	2		CH	SILTY CLAY: high plasticity, light grey mottled yellow brown, with ironstone bands.	MC>PL	VSt -H	- 210 310 300	RESIDUAL
			3		-	SHALE: dark grey, with clay bands.	DW	VL-L	-	VERY LOW 'TC' BIT RESISTANCE
			4 5			SHALE: dark grey.	DW	L-M		LOW RESISTANCE WITH MODERATE BANDS
								M		MODERATE RESISTANCE
			6			END OF BOREHOLE AT 6.0m				
			7							

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BOREHOLE LOG

Borehole No. **5**
1/1

Client: KIRELA PTY LTD
Project: REDEVELOPMENT OF SOUTH EASTERN PRECINCT OF BAKEHOUSE QUARTER
Location: CNR. GEORGE STREET & PARRAMATTA ROAD, NORTH STRATHFIELD, NSW

Job No. 23767LB
Date: 14-4-10
Method: SPIRAL AUGER JK500
Logged/Checked by: A.M./
R.L. Surface: ≈ 9.2m
Datum: ASSUMED

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		-	ASPHALTIC CONCRETE: 50mm.t	M	-	-	
						FILL: Clayey sandy gravel, fine to coarse grained sub angular brick, sandstone and igneous gravel, brown and grey, fine to coarse grained sand.				APPEARS POORLY TO MODERATELY COMPACTED
	DS	N = 7 1,2,5					MC>PL		320 320 240	
			1			FILL: Silty clay, medium plasticity, red brown, with fine to medium grained sub angular to sub rounded ironstone and igneous gravel.				
					CH	SILTY CLAY: high plasticity, light grey mottled yellow brown, with ironstone bands.	MC>PL	VSt	-	RESIDUAL
	DS	N = 9 3,4,5							210 200 200	
			2							
	DS	N > 18 9,18/ 150mm REFUSAL	3					H	>400 >400	
					-	SHALE: dark grey.	DW	VL-L	-	VERY LOW 'TC' BIT RESISTANCE
	DS		4							
								L		LOW RESISTANCE
	DS		5							
			6			END OF BOREHOLE AT 6.0m				
			7							

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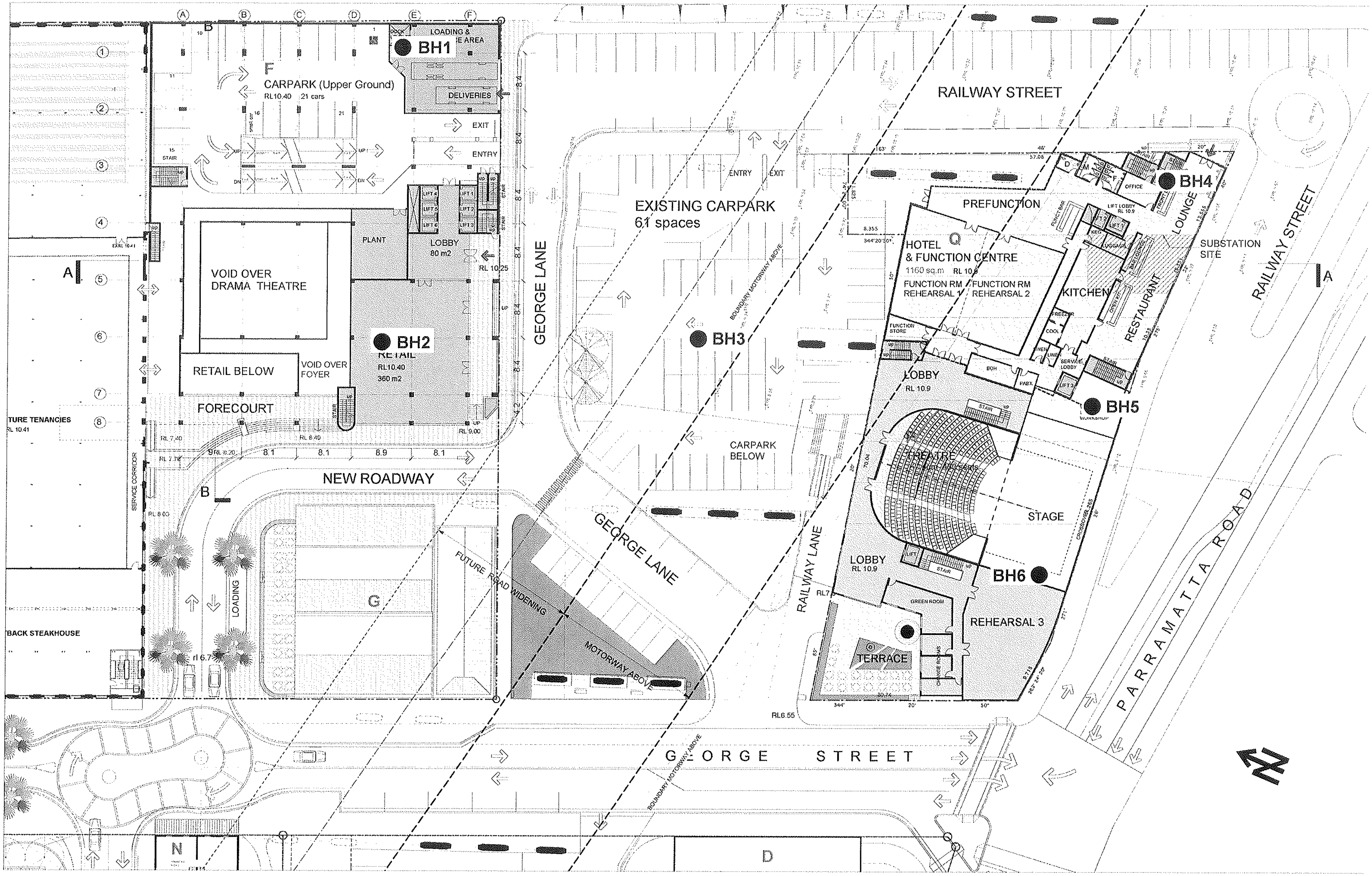
BOREHOLE LOG

Borehole No. **6**
1/1

Client: KIRELA PTY LTD
Project: REDEVELOPMENT OF SOUTH EASTERN PRECINCT OF BAKEHOUSE QUARTER
Location: CNR. GEORGE STREET & PARRAMATTA ROAD, NORTH STRATHFIELD, NSW

Job No. 23767LB **Method:** SPIRAL AUGER JK500 **R.L. Surface:** ≈ 8.9m
Date: 14-4-10 **Datum:** ASSUMED
Logged/Checked by: A.M./

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			ASPHALTIC CONCRETE: 60mm.t				
		N = 9 6,6,3	0–1.3		-	FILL: Sandy gravel, fine to coarse grained sub angular sandstone, concrete and igneous gravel, brown, fine to coarse grained sand.	D	-	-	APPEARS MODERATELY COMPACTED
			1.3		-	CONCRETE: 100mm.t	-	-	-	
		N = 8 3,3,5	1.4–2.0		-	FILL: Silty clay, medium plasticity, dark grey and dark brown, with fine to medium grained igneous and sandstone gravel.	MC>PL	-	200 150 170	ORGANIC/ HYDROCARBON ODOUR
		N = 23 6,11,12	2.0–4.2		CH	SILTY CLAY: high plasticity, light grey mottled yellow brown, with ironstone bands.	MC>PL	H	350 400 380	RESIDUAL
		N = 28 6,11,17	4.2–5.1			SILTY CLAY: high plasticity, grey.			>400 >400 >400	
			5.1–6.0		-	SHALE: dark grey.	DW	VL-L	-	VERY LOW 'TC' BIT RESISTANCE
			6			END OF BOREHOLE AT 6.0m				
			7							



SCALE (m)
0 25

BOREHOLE LOCATION PLAN

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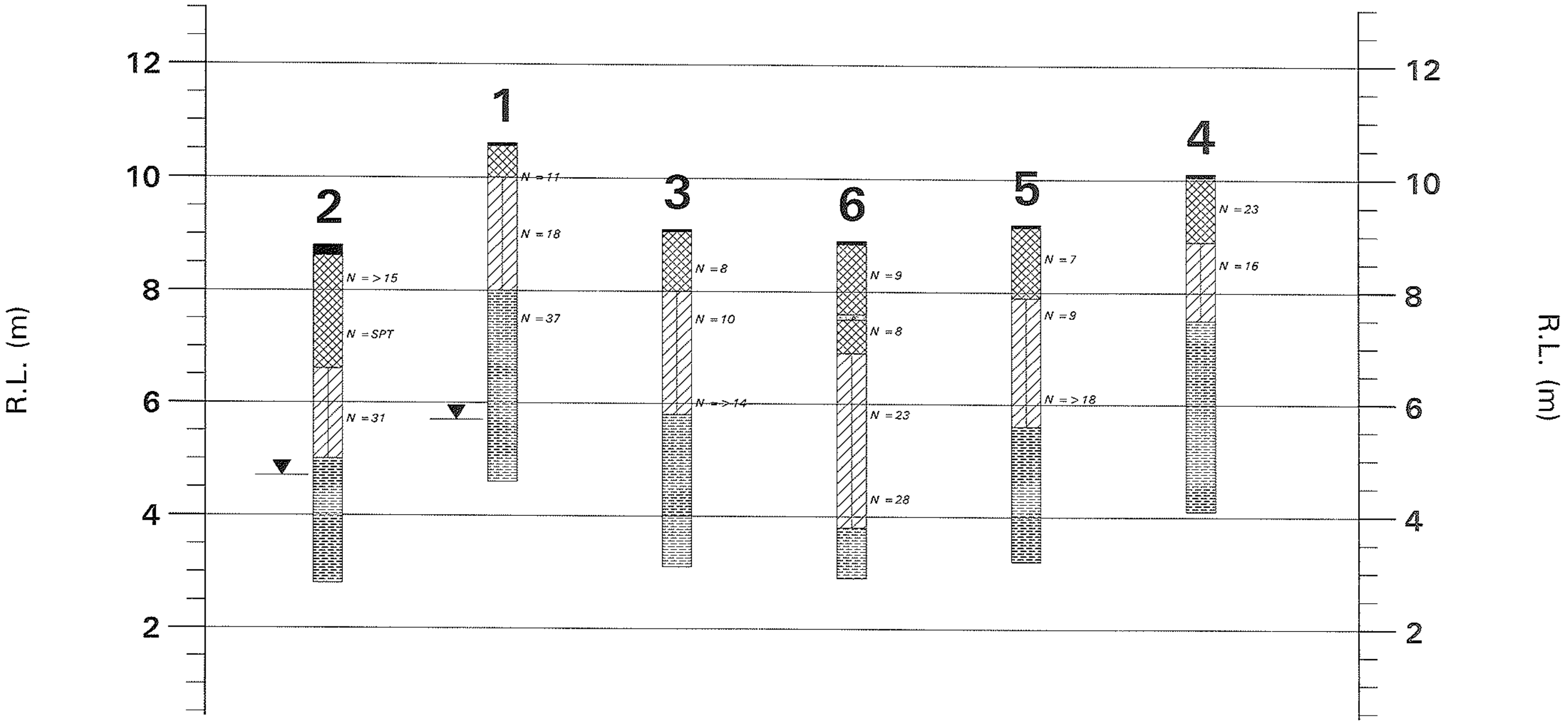
Report No. 23767LB Figure No. 1

GRAPHICAL BOREHOLE SUMMARY



Asphaltic/ Bituminous Paving or Coal

Fill

Silty 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)

Jeffery and Katauskas Pty Ltd

Job No.: 23767LB

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 (eg sandy clay) as set out below:

Soil Classification	Particle Size
Clay	less than 0.002mm
Silt	0.002 to 0.06mm
Sand	0.06 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

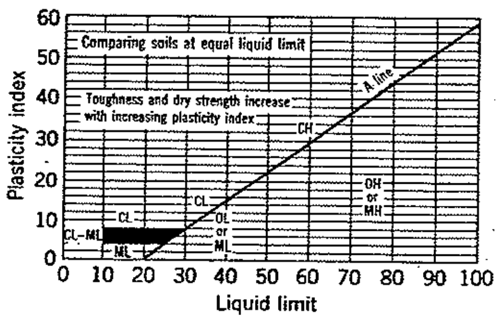
OTHER MATERIALS

- CONCRETE
- BITUMINOUS CONCRETE, COAL
- COLLUVIUM

UNIFIED SOIL CLASSIFICATION TABLE

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*)	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. $C_U = \frac{D_{60}}{D_{10}}$ 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

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 (e.g. 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.

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CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

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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	Soil sample taken over depth indicated, for environmental analysis.
	U50	Undisturbed 50mm diameter tube sample taken over depth indicated.
	DB	Bulk disturbed sample taken over depth indicated.
	DS	Small disturbed bag sample taken over depth indicated.
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	Vane shear reading in kPa of Undrained Shear Strength.
	PID = 100	Photoionisation detector reading in ppm (Soil sample headspace test).
Moisture Condition (Cohesive Soils)	MC>PL	Moisture content estimated to be greater than plastic limit.
	MC≈PL	Moisture content estimated to be approximately equal to plastic limit.
	MC<PL	Moisture content estimated to be less than plastic limit.
(Cohesionless Soils)	D	DRY - runs freely through fingers.
	M	MOIST - does not run freely but no free water visible on soil surface.
	W	WET - free water visible on soil surface.
Strength (Consistency) Cohesive Soils	VS	VERY SOFT - Unconfined compressive strength less than 25kPa
	S	SOFT - Unconfined compressive strength 25-50kPa
	F	FIRM - Unconfined compressive strength 50-100kPa
	St	STIFF - Unconfined compressive strength 100-200kPa
	VSt	VERY STIFF - Unconfined compressive strength 200-400kPa
	H	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)		**Density Index (I_D) Range (%)** **SPT 'N' Value Range (Blows/300mm)**
	VL	Very Loose <15 0-4
	L	Loose 15-35 4-10
	MD	Medium Dense 35-65 10-30
	D	Dense 65-85 30-50
	VD	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	Hardened steel 'V' shaped bit.
	'TC' bit	Tungsten carbide wing bit.
	60	Penetration of auger string in mm under static load of rig applied by drill head hydraulics without rotation of augers.

Ref: Standard Sheets Log Symbols
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LOG SYMBOLS

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	