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REPORT

TO

HANSEN YUNCKEN

ON

PRELIMINARY GEOTECHNICAL INVESTIGATION

FOR

PROPOSED INDUSTRIAL ESTATE

AT

CORNER OF THE HORSLEY DRIVE & COWPASTURE ROAD, WETHERILL PARK, NSW

9 December 2011

Ref: 25371ZRrpt

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1 INTRODUCTION

This report presents the results of our preliminary geotechnical investigation for the proposed industrial estate at the corner of Horsley drive and Cowpasture Road, Wetherill Park, NSW. The investigation was commissioned by Mr Chris Lykoudis of Hansen Yuncken Pty Ltd in an email dated 18 November 2011 and was carried out in accordance with our fee proposal (Ref: P34697ZR) dated 8 November 2011.

We have been provided with the following information:

- A sketch plan showing indicative borehole locations (Ref. Co11492.00, dated 16 November 2011) prepared by Costin Roe Consulting (CRC).
- A Preliminary Bulk Earthworks plan (Drawing Number Co11492.00-PC06, dated 24 November 2011) prepared by CRC.

Based on a review of the provided information we understand the proposed development will comprise a new industrial estate comprising twelve lots connected via curved access road extending into the site from Cowpasture Road. The new warehouse buildings will be constructed on level platforms within the lots which will be formed by cut and fill earthworks with a view to achieving a 'balance' between the cut and fill quantities. We have assumed that the new warehouse buildings will be of concrete tilt-up panel construction with paved surrounds. To achieve the proposed bulk excavation levels (which range between RL68m and RL82m) will require excavations to a maximum depth of about 11m and up to about 9m of fill. Similar cut and fill earthworks will be required to form the access road which has proposed surface levels ranging between RL67m and 78m.

We have not been provided with the locations of proposed buildings or loadings and have assumed typical loadings for this type of development.

The purpose of the investigation was to obtain preliminary geotechnical information on subsurface conditions as a basis for comments and recommendations on excavation, retention, footings, on-grade floor slabs and external pavements, drainage and the geotechnical aspects of the pavement design.

We note that we prepared an email (Ref. 25371ZRemail) dated 2 December 2011 which provided preliminary geotechnical advice; this report supersedes our previous email.

A preliminary environmental site assessment has also been completed and the results are presented in a separate report (Ref. E25371KGrpt, dated December 2011) prepared by our specialist environmental division (Environmental Investigation Services (EIS)).

2 INVESTIGATION PROCEDURE

The fieldwork for the investigation was carried out on 21 and 22 November 2011 and comprised the auger drilling of twelve boreholes (BH1 to BH12) to depths ranging between 3m and 6m below the existing ground surface level using our track mounted JK300 drill rig.

The borehole locations, as indicated on the attached Figure 1, were set out using a hand held GPS device. Prior to the fieldwork commencing, the borehole locations were electro-magnetically scanned for buried services by a specialist sub-contractor.

The strength of the natural clayey soils were assessed from the Standard Penetration Test (SPT) 'N' values, which were augmented by the results of hand penetrometer readings on cohesive soil samples recovered in the SPT split tube. The strength of the bedrock was assessed from observation of drilling resistance when using a tungsten carbide ('TC') bit, examination of the recovered rock cuttings and subsequent correlation with laboratory moisture content test results.

Groundwater observations were made in the boreholes during and on completion of auger drilling. We note that water levels may not have stabilised in the short time period after drilling. No longer term groundwater monitoring was carried out.

For more details of the investigation procedures, reference should be made to the attached Report Explanation Notes.

The fieldwork was carried out under the full time direction of our geotechnical engineer (David Schwarzer), who set out the borehole locations, directed the electro-magnetic scan for buried services, logged the encountered subsurface profile and nominated in-situ testing and sampling. The borehole logs (which also include field test results and groundwater observations) are attached, together with a glossary of logging terms and symbols used.

Selected soil and rock chip samples were returned to the Soil Test Services Pty Ltd (STS) NATA registered laboratory, for moisture content, Atterberg Limit, linear shrinkage, Standard compaction and four day soaked CBR testing. The results are summarised in the attached Tables A and B.

3 RESULTS OF INVESTIGATION

3.1 Site Description

The site is situated within undulating terrain.

The subject site has southern and eastern frontages onto The Horsley Drive and Cowpasture Road, respectively. An approximately 100m length of the central portion of the eastern frontage comprised a cut slope (about 3m maximum height) which sloped down to the east at about 25°.

For descriptive purposes The Horsley Drive and Cowpasture Road which have been assumed to be orientated east-west and north-south, respectively.

At the time of the fieldwork the subject site was densely vegetated with long grass and high scrubs. Small to medium size trees were scattered throughout the site. The topography of the site was dominated by an elevated area over the central portion of the site with moderately steep slopes down to the north and south. The elevated area was approximately 20m to 22m higher than the central portion of the northern site boundary and the south-eastern corner of the site. In addition, a gully feature with gently sloping sides, orientated approximately east-west, extended over the north-western corner of the site. The surface levels over the northern side of the gully extended north beyond the site boundary. Over the southern portion of the site the site surface levels generally sloped gently down to the south and east.

Two small dams were located over the north-western corner of the site within the gully feature. The dams were densely vegetated but our restricted observations appeared to indicate that the northern side of the dam was about 2m high and sloped at about 30° down to the north.

Two private lots extended into the site from the central portion of the Cowpasture Road frontage and the eastern portion of The Horsley Drive frontage. Single storey brick and rendered houses with metal clad shed or garage structures were located close to the street frontages. Based on a cursory inspection from within the site the houses appeared to be in good external condition and the sheds in poor condition.

A concrete paved cycleway was set-back about 1m from the western site boundary. The Sydney Water Supply Channel which comprised a tunnel section under The Horsley Drive and a concrete lined section that extended north adjacent to the site and was set-back at least about 25m from the western site boundary.

We also note that a review of historical aerial photographs by EIS and our site observations has indicated the following:

- A dam was located adjacent to the western end of the northern site boundary and was backfilled between about 1994 and 2005,

- Over the central portion of the western site boundary and east into the site towards the existing dams what appeared to be abandoned cars and other scrap was evident from about 1994 and where observations through the dense vegetation were possible were evident during the fieldwork, and

- A possible fill mound covered with a thick grass cover was evident near the south west corner of the site adjacent to The Horsley Drive.

3.2 Subsurface Conditions

The 1:100,000 geological map of Penrith indicates that the site is underlain by shale, fine grained sandstone and laminite of the Bringelly Shale formation.

Generally, the boreholes revealed a subsurface profile comprising a limited thickness of topsoil fill overlying residual clays then weathered bedrock at shallow to moderate depth. Groundwater was not encountered over the depth of the investigation. For further detailed subsurface conditions at each borehole location, reference should be made to the attached borehole logs. Graphical borehole summaries are presented as Figures 2 and 3. A summary of the pertinent subsurface issues and considerations is provided below.

Topsoil

Silty clay topsoil fill was encountered from surface level in all the boreholes and ranged between 0.05m and was 0.3 or 0.5m thick.

Residual Silty Clays

Residual silty clays assessed to be of high (rarely medium) plasticity were encountered in all the boreholes below the topsoil and extended to depths ranging between 1.3m and 3.8m. The silty clays were typically assessed to be very stiff and hard; firm silty clay was encountered in BH11 below 1.2m depth and extended to 3m depth.

Weathered Bedrock

Weathered bedrock was encountered in all the boreholes beneath the residual silty clays. Shale bedrock was encountered in all boreholes except BH5, where sandstone was encountered. On first contact, the bedrock was assessed to be extremely to distinctly weathered and of extremely low to very low or low (rarely medium) strength. With depth the bedrock improved in quality, and was generally assessed to be distinctly weathered and of low strength, with medium or high strength bedrock encountered towards the base of BH4, BH5, BH6, BH7 and BH8.

Groundwater

Groundwater seepage was not encountered whilst auger drilling the boreholes. All the boreholes were also noted as ‘dry’ on borehole completion and the short time following completion that they were left open. We note that groundwater levels may not have stabilised during the relative short period between borehole completion and measurement of water levels. No longer term groundwater monitoring has been carried out.

3.3 Laboratory Test Results

The four day soaked CBR values of natural residual silty clay soils have returned values of ranging between 3.5% and 5% when compacted to 98% of Standard Maximum Dry Density (SMDD). The natural moisture contents of the samples tested were 6.5% (BH1), 5.1% (BH4), 2.8% (BH7), 1.9% (BH8), 5.6% (BH10) and 3.1% (BH12) ‘wet’ of their respective Standard Optimum Moisture Content (SOMC).

Based on the Liquid Limit and Linear Shrinkage determinations the residual clays are assessed to be of high plasticity with a high potential for shrink/swell reactivity with changes in moisture content.

The moisture content determinations on the selected rock auger cutting samples generally correlated well with our field assessment of rock strength.

4 COMMENTS AND RECOMMENDATIONS

4.1 Excavation Conditions

4.1.1 General

Excavation recommendations provided below should be complemented by reference to the Code of Practice *'Excavation Work'*, Cat No 312 (31 March 2000), by WorkCover NSW.

To achieve the proposed bulk excavation levels for the building platforms and access road (which range between RL67m and RL82m) will require excavations to a maximum depth of about 11m. The excavations will extend into the elevated area over the central portion of the site and over selected lengths of the western side of the site.

Excavations will extend through the soil profile which is expected to range between about 1.3m and 3.8m then weathered bedrock over the deeper cut areas.

All topsoil and root affected soils should be stripped and may be separately stockpiled for re-use in landscape areas, or taken off site as they are not suitable for re-use as engineered fill.

The existing dams are located over areas where fill will be placed. We recommend that the dams be de-commissioned by the removal of existing stored water and the embankments, and the stripping of any desiccated and/or softened material from the floor of the dam areas. These materials should be stockpiled separately and inspected by a geotechnical engineer to assess their suitability for re-use as engineered fill.

Excavations through the upper soil profile, and poor quality extremely low to low strength bedrock and any highly fractured bedrock may be readily completed using conventional earthmoving equipment such as tracked dozers or hydraulic excavators.

The lower more competent (medium or higher strength) shale (and sandstone) bedrock we expect to be excavated using ripping tynes fitted to heavy dozers (say D10 or D11) or heavy (40 tonne) excavators. Some of the high strength bedrock may contain few defects and rippability may be very marginal. In some areas it may be necessary to use hydraulic rock breaker attachments fitted to an excavator. Locally, excavation using rock breakers may be required and will be needed for detailed excavations such as for trimming excavation faces, footings, service trenches etc. A number of cored boreholes would be required to assess rock defect spacings and strengths in order to provide further information on the most suitable equipment to excavate the bedrock.

4.1.2 Potential Vibration and Ground Surface Movement Risks

Care is required with excavation as these may result in direct transmission of ground vibrations to neighbouring existing structures to the west and paved surfaces to the west, south and east. If there is any cause for concern then demolition and/or excavation should cease and further advice sought.

To reduce vibrations associated with the use of rock breakers and over-break of the bedrock, final trimming of bedrock cut faces should be completed using a grinder attachment or rock saw fitted to the excavator. Alternatively a rock saw cut may be provided close to the excavation margins prior to the use of rock breakers and provided the saw cut is maintained below the level at which the rock breaker is being used the potential for transmission of potentially damaging vibrations crossing site boundaries will be reduced. Such considerations may be of more concern if some lots are excavated following construction of buildings within other nearby lots within the site.

Where rock breakers are used, to reduce vibrations we recommend that the rock breaker be continually orientated towards the face, be operated one at a time and in short bursts only to reduce amplification of vibrations. When using the rock breakers, the resulting dust should be suppressed by spraying with water.

We recommend that periodic quantitative vibration monitoring of adjacent structures to the west be undertaken where rock breakers are being used within an off-set distance of 50m to confirm that peak particle velocities fall within acceptable limits. However, further advice from Sydney Water and Council should be sought in this regard. We note that this vibration limit will reduce the risk of vibration damage to the neighbouring structures. If potentially damaging vibrations are occurring it will be necessary to use lower energy equipment such as smaller hammers or grinders. Alternatively grid-sawing techniques can be used to dampen ground vibrations.

4.1.3 Dilapidation Surveys

Prior to excavation commencing, consideration will need to be given to completing detailed dilapidation reports on the Sydney Water Supply Channel structure and cycleway to the west and the paved road surfaces lining the southern and eastern site boundaries. Further advice from Sydney Water and Council should be sought in this regard. The owners of the neighbouring properties should be asked to sign the reports and agree that they are a fair assessment of existing conditions, as these can then be used as a benchmark in assessing potential future damage claims (due to ground surface movements and/or vibration damage).

4.1.4 Seepage

Groundwater was not encountered in the boreholes. We do not expect substantial groundwater flows or seepage to occur into the excavations, although concentrated flows may be encountered where defects or highly fractured zones of bedrock daylight in the excavation face.

The likelihood of groundwater flows along defects increases with depth. Seepage is also likely to occur along the bedrock surface, particularly in wet weather periods. We therefore recommend that the initial stages of bulk excavations are monitored, and if substantial flows are encountered, appropriate drainage measures may then be detailed. At this stage we expect the seepage that does occur will be controlled using either gravity drainage or conventional sump and pump methods.

4.2 Site Preparation

The following earthworks recommendations should be complemented by reference to AS3798-2007 "Guidelines on Earthworks for Commercial and Residential Developments" and the WorkCover Authority of NSW's "Code of Practice – Excavation Work" dated 31 March 2000 (Cat. No. 312).

4.2.1 Subgrade Preparation

Following stripping of all vegetation, topsoil and root affected soils, completion of excavations to achieve the design subgrade levels and prior to placement of fill to raise site levels, subgrade preparation over the footprint of the proposed development and access road should be completed in the following manner:

- The existing natural clay soil and extremely weathered bedrock subgrade will require proof rolling with eight passes of a minimum 5 tonne deadweight smooth drum vibratory roller.

- Proof rolling should be carried out under the direction of an experienced earthworks superintendent or geotechnical engineer to assist in the detection of soft or unstable areas not disclosed by this investigation.

- Any soft or unstable areas identified during proof rolling should be locally excavated down to a competent base and replaced with engineered fill as described below. This is of particular importance over the western end of the northern site boundary where a former dam appears to have been backfilled. Care should be taken not to over compact the clay subgrade.

- Areas of clay subgrade that contain shrinkage cracks should be watered and rolled until the shrinkage cracks disappear.

- Care will need to be exercised close to any nearby existing structures, paved surfaces and any buried services as ground borne vibrations caused by the proof rolling may cause damage. If there any causes for concern during proof rolling, then further advice should be sought and/or the non-vibration (static) mode of the roller used.

With regard to existing dams and natural slopes, it may be beneficial to cut horizontal benches within the sloping surface to facilitate more effective proof rolling and also later placement of engineered fill.

We note that if the floor slabs are suspended no particular subgrade preparation would be required.

4.2.2 Subgrade Drainage

The subgrades of a number of lots and sections of the access road will comprise clayey engineered fill and natural clays. Based on our experience on this area of Sydney the clays are likely to be highly dispersive and therefore prone to erosion. The clays may therefore be found to be unstable if proper site drainage is not implemented during construction. It is therefore important to provide good drainage in order to promote run-off, reduce ponding and erosion. Earthworks platforms should be graded to maintain cross-falls during construction. If the clays are exposed to periods of rainfall, softening may result and site trafficability will be poor. If softening occurs, the subgrade should be over-excavated to below the depth of moisture softening. The material removed should be replaced with engineered fill. Such considerations may be mitigated by the early construction of the floor slabs and pavements.

4.2.3 Engineered Fill and Earthworks Testing

Fill required to raise site levels should comprise engineered fill. The natural clays and weathered bedrock sourced from the bulk excavations may be re-used as engineered fill provided all organic material and other deleterious substances are removed. In addition, the maximum particle size should not exceed 75mm; crushing of the more competent ripped bedrock will be required to achieve the specification maximum particle size. Engineered fill should be compacted in layers no greater than 200mm loose thickness to a density strictly between 98% and 102% of SMDD and within 2% of SOMC. We note that the laboratory testing indicated that the natural clays were 'wet' of optimum and so some moisture conditioning may be required to achieve the above specification.

If preferred, well graded granular material (ripped or crushed sandstone or building rubble) free of deleterious substances and having a maximum particle size of 75mm may also be used as engineered fill and compacted to a minimum density of 98% SMDD.

To achieve satisfactory compaction over the margins of the fill platforms we recommend that the fill be placed to the site boundary. The fill may then be cut back to the appropriate temporary batter slope (see Section 4.2, above) to allow construction of the retaining wall. Alternatively, the wall could be constructed prior to placement of the fill. However, there is the possibility of damage to the retaining wall during compaction. Further, we do not expect satisfactory fill compaction will be achieved and this may lead to excessive post-construction settlements which have the potential to detrimentally affect the serviceability of both the paved surface and drainage.

Backfill to conventional retaining walls should also comprise engineered fill. Well graded granular materials such as ripped or crushed sandstone and demolition rubble would be suitable for this purpose. Such fill should be compacted in horizontal layers as above using a hand held plate compactor. Care will be required to ensure excessive compaction stresses are not transferred to the retaining walls.

We note that if single sized granular material (or 'no fines' gravel) is used as backfill to retaining walls then only nominal compaction (with no compaction testing) will be required. Further, retaining wall backfill should be provided with a clay plug at surface level to reduce the likelihood of stormwater surcharging the retaining wall.

Any proposed permanent fill batters should be no steeper than 1V in 2H and should be protected from erosion by quickly establishing a grass cover. However, for ease of maintenance a batter of 1V in 3H is more appropriate.

Care should be taken to not surcharge the batters during construction or in the longer term. In this respect, temporary stockpiles, site traffic etc should be placed well back from the crest of batters. Batters which have permanent structures founded close to their crests must be subject to geotechnical stability assessment. Fill embankment slopes higher than 3m should also be subject to geotechnical stability design checks.

The engineered fill should extend a horizontal distance of at least 1m beyond the proposed outline of new buildings. Paving around the perimeter of the warehouse will reduce potential shrinkage of both the engineered clay fill and natural clay soils.

Where fill is placed as engineered fill testing should be undertaken to check compliance with the earthwork specifications. The frequency of density tests should be at least one test per layer per 500m^2, or three tests per visit, whichever requires the greater number of tests. For backfilling of localised excavations, such as localised soft spots, testing should consist of one test per two layers per 50m^2. We recommend that at least Level 2 testing of earthworks be carried out in accordance with AS3798-2007. If the slabs are constructed 'on-grade' then we strongly recommend Level 1 testing be carried out as the long term satisfactory performance of the slabs will rely on a well constructed engineered fill platform. Preferably, the geotechnical testing authority should be engaged directly by the client rather than the earthworks subcontractor.

4.3 Temporary and Permanent Batters and Permanent Retention

4.3.1 General

Due to the proposed deep cuts over selected areas of the site, retained heights within some lots will be between about 9m and 11m. We have no details regarding the proposed development within individual Lots. However, the locations of proposed retaining walls on lot boundaries will determine the nature of the temporary shoring of the excavation. For instance, at the lot boundaries lining the perimeter of the site there may be lack of space to accommodate temporary batters. In such instances, the preferred method of excavation support through the upper soil and poor quality bedrock profile may be to progressively install ground anchors with shotcrete or provide anchored contiguous pile walls. Permission to install ground anchors (temporary or permanent) under neighbouring properties would need to be sought.

4.3.2 Temporary and Permanent Batters

Where space permits, temporary excavation batters of 1 Vertical (V) in 1 Horizontal (H) are appropriate through the upper soil profile and the poor quality bedrock though there would still remain a possibility of localised wedge failures defined by defects within the poor quality bedrock with dips into excavations between about 30° and 45°.

The lower, better quality shale and sandstone bedrock may be temporarily cut vertically and is expected to stand unsupported, subject to frequent progressive geotechnical inspections to check for unfavourably orientated defects. Geotechnical inspections would be required as excavations progress at intervals of no more than 1.5m 'lifts'.

Any proposed permanent cut slopes through the upper soil profile and the poor quality bedrock will need to be formed at no steeper than 1V in 2H and planted with rapid growing vegetation to reduce potential erosion and spalling of the cut faces. Based on our experience on this area of Sydney the clays are likely to be highly dispersive and therefore prone to erosion.

Permanent cut slopes through the better quality bedrock may be cut at steeper angles, say up to about 60°, but regular geotechnical inspections as detailed for the lots above will be required and stabilisation of potentially unstable wedges and extremely weathered seams would require rock bolts and possibly reinforced shotcrete. The crest and toe of all permanent cut slopes will need to be provided with a concrete lined drain. Additional widths at the cut batter crest and toe (including road verges) will be required to allow for localised deterioration of the upper portion of the cut face and to collect any debris that may spall from the slope, respectively. Further geotechnical advice on such matters can be given once further design details are known and further geotechnical investigations are completed.

Where sub-vertical shale bedrock faces are not provided with permanent support selected rock bolting of potentially unstable wedges, and reinforced shotcrete protection of weak seams etc together with an accumulation zone provided at the base of the cutting can be considered. The purpose of the accumulation zone would be to collect debris that has spalled or been eroded from the cut face. We note that if such zones are provided they must not be developed and would consequently reduce the available area for development and on this basis may not be a preferred option. Access to the public would need to be restricted say by provision of a robust fence. However, access for maintenance personnel and plant to clean out debris, remove potentially unstable features from the cut slopes and/or repair the fence.

4.3.3 Permanent Retention

The selection of the most appropriate permanent retention system will depend on the nature of the material to be retained, the retained height and the available space within individual lots. General advice on the likely suitable permanent retention systems are outlined below and the selection of the most appropriate retention method will most likely be influenced by the proposed retained height.

Conventional Free Standing Retaining Walls

Where space permits we envisage that freestanding retaining walls will be constructed within the above temporary batters and subsequently be backfilled. In addition, the engineered fill placed to raise site levels may also be supported using freestanding retaining walls.

Where fill is locally required to raise site levels and needs to be retained consideration will need to be given to construction sequencing:

- Fill may be initially placed and then cut back to allow retaining wall construction.
- The retaining wall may be constructed first and then backfilled. In this case, good compaction is unlikely close to the retaining wall and a movement control joint may be required where settlement of such fill is expected.

For retaining walls founded on the crests of vertical excavation faces through bedrock, lateral restraint may be provided by starter bars drilled and grouted to a depth of at least 0.5m into the bedrock. The starter bars should be installed at a downward angle into the rock face and be provided with a vertical cogged length. If cross bedded units within sandstone bedrock are identified and slope down into the excavation, then the starter bars may have to be extended to stabilise the potentially unstable cross bedded units.

Reinforced Earth Walls and Crib Walls

If preferred, the engineered fill or cut slopes may be supported using reinforced earth (RE) walls or crib walls which would need to be designed and constructed in accordance with the suppliers specifications. We note that such walls would also require the use of select backfill materials and importing of fill may be required.

Further geotechnical input would most likely be required to assist in the design of such walls.

Anchored Retention Systems

Other long term support methods for sub-vertical shale bedrock faces include anchored reinforced shotcrete walls or anchored soldier pile walls with reinforced shotcrete infill.

We recommend that soldier piles, where selected, have the following toe embedments:

- Load bearing piles to be at least 0.5m below the bulk excavation level and below the base of any nearby footing excavations, service trenches etc. Where the socket is to provide lateral support to the pile, sockets well in excess of 0.5m will be required.

- At least 0.5m into medium strength bedrock. Where this results in the pile toes not extending below the bulk excavation level, additional support by rock bolts/anchors will have to be installed progressively to maintain stability until the structure can provide support. This will take the form of pattern bolting immediately below the piles as excavation progresses, and following geotechnical inspection.

Bored soldier piles are considered suitable although locally, difficulties may be experienced achieving clean and dry bases due to groundwater inflow and or pier wall collapse through the soil profile and any sections of fractured bedrock. All bored piers should be concreted on the day of excavation. Bored piles left open over night may require extending due to potential water softening of the base materials or fall-in from pier walls.

Medium to large capacity piling rigs with augers or buckets fitted with rock teeth will be required to socket into the medium (and higher) strength bedrock. Further advice should be sought from piling contractors to determine the cost impact of forming any such sockets.

There is also the possibility that a continuous steeply dipping defect plane may daylight towards the toe of the cut face. We therefore recommend that all anchors be extended and bonded beyond a theoretical failure plane sloping up from the toe of the excavation at 45°. Whilst the geotechnical investigations outlined in Section 4.6, below may assist in assessing whether such defects are present, it will still be imperative that close inspection of the toe of the cut faces be completed by an experienced geotechnical engineer to check for the presence of such a defect plane. If such defects are encountered, additional support, such as additional, and possibly longer, anchors may need to be provided.

4.3.4 Retention Design Parameters

The following characteristic earth pressure coefficients and subsoil parameters should be adopted for the design of the full depth retention system and conventional retaining walls:

- For progressively anchored or propped walls, we recommend the design be based on the appropriate lateral earth pressure diagram indicated on the attached Figures 4 and 5.
- Where neighbouring buildings lie within a horizontal distance of at least H from the line of the excavation, we recommend the magnitude of the lateral earth pressure distribution be based on 8H kPa, where 'H' is the full excavation depth (in metres) through the soil and the poorer quality bedrock profile (see Figure 4)

or the retained height (in metres) of the soil and the poorer quality bedrock profile (see Figure 5).

- Where minor movements may be tolerated, say where no movement sensitive structures or services are located within a horizontal distance of at least H from the line of the excavation, we recommend the magnitude of the lateral earth pressure distribution be based on 6H kPa, 'H' is the full excavation depth (in metres) through the soil and the poorer quality bedrock profile (see Figure 4) or the retained height (in metres) of the soil and the poorer quality bedrock profile (see Figure 5).
- With regard to Figure 5, we note that where a defect is revealed or may be present within the lower better quality bedrock, then the retained mass of upper material must be added as surcharge to the wedge requiring support.
- Reinforced shotcrete supporting medium or high strength bedrock may be designed using a uniform lateral earth pressure of 5kPa for the lower portion of the cut face below any anchored or propped walls (see Figure 5) and/or permanent cut slopes through such materials where an accumulation zone e to collect debris that has spalled or been eroded from the cut face cannot be accommodated.
- For conventional retaining walls that will be propped by the structure, we recommend the use of a triangular lateral earth pressure distribution with an 'at rest' earth pressure coefficient (k_o) of 0.55 for the soil profile and extremely weathered bedrock profile, assuming a horizontal backfill surface.
- Where some minor movements of retaining walls may be tolerated, using a triangular lateral earth pressure distribution, they may be designed for a coefficient of 'active' earth pressure, (k_a), of 0.35 for the soil and extremely weathered bedrock profile, assuming a horizontal backfill surface.

- A bulk unit weight value of $20kN/m^3$ adopted for the soils and poor quality bedrock and a value of $25kN/m^3$ adopted for low (or higher) strength bedrock.
- All surcharge loads affecting the walls (e.g. nearby footings, construction loads, engineered fill, sloping backfill surfaces, traffic, etc) should be allowed in the design using the appropriate earth pressure coefficient from above.
- Complete and permanent drainage of ground behind the walls should be provided. Subsurface drains must incorporate a geofabric (e.g. Bidim A34) to act as a filter against subsoil drainage. Any shotcrete walls or panels must be provided with strip drains discharging to the perimeter drainage system and/or the stormwater system.
- Lateral restraint of the retaining walls founded below bulk excavation level (BEL) should be achieved by keying the wall footing into the underlying bedrock below the base of nearby footings or service trench excavations. An allowable lateral stress of 200kPa may be adopted for key design.
- If toe restraint is provided by the passive pressure of the soil below BEL then a 'passive' earth pressure coefficient, K_p, of 3 may be adopted, provided a Factor of Safety of 2 is used in order to reduce deflections. The upper 0.3m below BEL together with any localised excavations for buried services etc should be ignored in this analysis in order to take these excavation tolerances into account.
- Anchors or rock bolts should be bonded at least 3m into bedrock of at least low strength where an allowable bond stress of 100kPa may be adopted for design. Where the anchors are bonded into medium or high strength bedrock an allowable bond stress of 300kPa may be adopted for design. The anchor bond length should commence beyond a line projected up from the base of the excavation at an angle of 45°.

- All anchors should be proof-tested to 1.3 times the working load under the direction of an experienced engineer or construction superintendent, independent of the anchor contractor. Where anchors and rock bolts extend beyond the site or lot boundaries permission from the neighbours will be required prior to installation. Where permanent lateral support of the retention structure is not provided by the new structure, permanent anchors and/or rock bolts will be required which should be designed for corrosion resistance and for long term durability.

For long term corrosion considerations we recommend that all permanent starter bars be hot dipped galvanised. Further, where permanent anchors are required, they should be designed with due regard to corrosion resistance, long term durability and performance characteristics.

4.4 Footings

Based on the laboratory test results, the natural clays are considered to be highly reactive and therefore subject to shrink-swell movements with changes in moisture content.

On the basis of the above and with regard for the need for engineered fill to raise site levels over some portions of individual Lots, we recommend that building footings and floor slab footings (if the floor slab is to be suspended) be supported entirely within similar materials to reduce the potential for long term differential settlements. However, shallow footings founded within a combination of engineered fill and natural clays may be considered and designed for a Class H2 site classification using AS2870-2011 for guidance. This would require close supervision of earthworks including Level 1 earthworks compaction testing compliance of engineered fill (see Section 4, below).

A combination of strip or pad footings may be founded within natural silty clays of at least very stiff strength or clayey engineered fill placed and tested under Level 1 and designed on the basis of a maximum allowable bearing pressure of 200kPa.

For buildings founded in bedrock, then in the majority of cut areas, shallow pad or strip footings will be feasible. However, over the areas where fill is to be placed and/or bedrock is expected at moderate depth (believed to be unlikely), piled footings will be required. Pad or strip footings and conventional bored piles founded in the weathered bedrock may be designed on the basis of an allowable bearing pressure of 600kPa. Higher bearing pressures of say 1,000kPa may be achievable in the better quality (low strength or better) bedrock but this would be subject to geotechnical inspections during pile boring. If higher bearing pressures are required then additional cored boreholes will be required to confirm the strength of bedrock, defect spacings and the presence of potentially compressible seams (clay seams etc).

With regard to conventional bored piles, the above bearing pressures assume a minimum penetration of 0.5m into the appropriate strength of bedrock. An allowable shaft adhesion in compression of 10% of the above values is also applicable and should be calculated below the initial 0.5m length of the socket. Where the piles are resisting uplift or tension the allowable shaft adhesion should be reduced to 5% of the above values.

Groundwater inflow into bored pile excavations may be expected and we expect that this will be controllable by conventional pumping methods. However, some contingency for pouring concrete by tremie methods should be allowed.

The weathered bedrock, natural clays and clayey engineered fill are susceptible to softening in the presence of water and so all footings should be bored/excavated, cleaned, inspected and poured with minimal delay (i.e. within one day). Should footing excavations be left open and water ingress occurs, then over-excavation to remove water softened material would be required. With regard to strip or pad footings a blinding layer of concrete may be provided to protect excavation bases that are to be left open.

Attention is drawn to other precautionary and foundation maintenance measures outlined in AS2870-2011 and in this regard we note the potential drying effects of trees on clay soils. In this regard we recommend that site clearance of vegetation is carried out well in advance of development within the individual Lots as shrink/swell movements in excess of those normally expected for a Class H site may occur.

4.5 On-Grade Floor Slabs, External Pavements and Drainage

Slab-on-ground construction of the floor slabs is considered feasible. This is on condition that the subgrade is prepared in accordance with the recommendations given in Section 4.4 above. The on-ground floor slabs should be independent of the footing system.

For the engineered fill and natural clay subgrades we recommend a design soaked CBR value of 3.5%, an equivalent modulus of subgrade reaction of 30kPa/mm (750mm diameter plate) or a short term Youngs Modulus of 30MPa.

For the more competent bedrock subgrades (low or higher strength) we recommend a preliminary design soaked CBR value of 5% or an equivalent Modulus of Subgrade Reaction of 35kPa/mm (750mm diameter plate) or a short term Youngs Modulus of 35MPa.

Consideration should be given to protection of the edge of ground floor slabs from excessive shrink/swell movement associated with the clayey fill and natural clay subgrades. Where surrounding pavements or slabs do not abut buildings, we recommend that there should be an edge thickening extending to at least 0.5m below external finished grade.

A sub-base layer of minimum 100mm of DGB20, compacted to at least 100% SMDD should be used beneath all concrete floor slabs and pavements where vehicle movements are expected. This will provide more uniform slab support and will reduce the “pumping” of “fines” at joints.

Where used, concrete pavements should have a sub-base layer of at least 100mm thickness of crushed rock to RTA QA specification 3051 (2010 Edition 6/Revision 1) dated October 2010 unbound base material (or equivalent good quality durable fine crushed rock) which is compacted to at least 100%SMDD. Concrete pavements should be designed with shear effective transmission of all joints by way of either dowelled or keyed joints. Additional reinforcement will be required close to the interface between soil and bedrock subgrade conditions. Concrete pavements should be used in areas where heavy vehicles manoeuvre.

We recommend that further geotechnical assessment of pavement design should be carried out once design traffic loadings are known.

Sub-soil drains should be provided along the perimeter of pavements, with inverts not less than 0.2m below subgrade level. The drainage trenches should be excavated with a longitudinal fall to appropriate discharge points so as to reduce the risk of water ponding. The pavement subgrade should be graded to promote water flow or infiltration towards sub-soil drains.

4.6 Further Geotechnical Work

This report is of a preliminary nature and we strongly recommend that further detailed geotechnical investigations (including cored boreholes) be undertaken once design details are known. In addition to the further geotechnical investigations, the following summarises the minimum scope of further geotechnical work recommended within this report. For specific details reference should be made to the relevant sections of this report.

- Detailed geotechnical design of fill embankments, RE walls, steep cuts and pavements.
- Geotechnical inspections of temporary batters and bedrock cut faces at not greater than 1.5m height intervals and progressively as the excavation faces advance.
- Monitoring of groundwater during bulk excavations.
- Visual inspection of pad footings and bored piles.
- Witnessing anchor stress testing and installation of rock face stabilisation measures.
- Witnessing proof rolling of subgrades.
- Compaction testing of engineered fill and sub-base materials.

5 GENERAL COMMENTS

The recommendations presented in this report include specific issues to be addressed during the construction phase of the project. As an example, special treatment of soft spots may be required as a result of their discovery during proof-rolling, etc. 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 and below 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.

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 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.

Reviewed By:

Paul Roberts
Senior Associate
For and on behalf of
JEFFERY AND KATAUSKAS PTY LTD.

Agi Zenon
Senior Associate

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STS
SOIL TEST SERVICES
ABN 43 002 145 173

TABLE B
SUMMARY OF FOUR DAY SOAKED C.B.R.TEST RESULTS

BOREHOLE NUMBER	1	4	7	8	10	12
DEPTH (m)	0.50 - 1.50	0.10 - 0.50	0.30 - 1.00	0.50 - 1.50	0.50 - 1.50	0.30 - 1.00
Surcharge (kg)	9.0	9.0	9.0	9.0	9.0	9.0
Maximum Dry Density (t/m^3)	1.64 STD	1.60 STD	1.67 STD	1.57 STD	1.64 STD	1.70 STD
Optimum Moisture Content (%)	18.8	20.9	16.5	24.4	18.7	18.9
Moulded Dry Density (t/m^3)	1.60	1.58	1.65	1.56	1.60	1.65
Sample Density Ratio (%)	97	99	99	99	97	97
Sample Moisture Ratio (%)	131	126	123	109	128	118
Moisture Contents						
Insitu (%)	25.3	26.0	19.3	26.3	24.3	22.0
Moulded (%)	24.7	26.3	20.4	26.6	24.0	22.3
After soaking and After Test, Top 30mm(%)	25.7	26.0	28.1	25.6	24.9	22.6
Remaining Depth (%)	21.5	23.9	20.4	23.8	22.1	22.1
Material Retained on 19mm Sieve (%)	0	0	0	0	0	0
Swell (%)	2.0	0.5	2.5	1.0	2.5	0.0
C.B.R. value: @2.5mm penetration				4.5	4.0	
@5.0mm penetration	4.5	5	3.5			4.0

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
  - (c) Moisture Content : AS 1289 2.1.1

NATA
NATA Accredited Laboratory
Number:1327

Approved Signatory / Date
7/12/11
(A. Tatikonda)

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Facsimile: 02 9888 5001

Ref No: 25371ZR
Table A: Page 1 of 1

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
BOREHOLE NUMBER	DEPTH m	MOISTURE CONTENT %	LIQUID LIMIT %	PLASTIC LIMIT %	PLASTICITY INDEX %	LINEAR SHRINKAGE %
1	0.50-0.95	24.6	62	22	40	16.0
1	2.50-3.00	7.4				
2	2.50-3.00	6.5				
3	5.50-6.00	7.8				
4	0.50-0.95	22.0	52	18	34	14.5
4	2.50-3.00	4.0				
5	2.50-3.00	5.4				
6	2.50-3.00	7.4				
7	0.50-0.95	18.5	58	20	38	15.0
7	2.50-3.00	4.1				
8	5.50-6.00	6.4				
9	2.50-3.00	8.9				
10	0.50-0.95	24.6	62	20	42	16.0
10	4.00-4.50	7.1				
11	5.50-6.00	3.7				
12	5.50-6.00	10.2				

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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CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

Borehole No.
1
1/1

BOREHOLE LOG

Client: HANSEN YUNCKEN

Project: PROPOSED INDUSTRIAL ESTATE

Location: CNR. THE HORSLEY DRIVE & COWPASTURE ROAD, WETHERILL PARK, NSW

Job No. 25371ZR **Method:** SPIRAL AUGER JK300 **R.L. Surface:** ≈ 66.8m

Date: 22-11-11 **Datum:** ASSUMED

Logged/Checked by: D.S./

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			TOPSOIL: Silty clay, high plasticity, dark brown, trace of root fibres.	MC>PL			GRASS COVER
		N = 5 1,2,3			CH	SILTY CLAY: high plasticity, red brown, trace of fine to medium grained ironstone gravel and root fibres.	MC>PL	VSt	- 230 240 250	
			1			as above, but mottled light grey.				
		N > 6 8,6/80mm REFUSAL			-	SHALE: light grey and red brown.	XW	EL		VERY LOW 'TC' BIT RESISTANCE
			2			SHALE: brown.	DW	L		LOW RESISTANCE
			3			END OF BOREHOLE AT 3.0m				
			4							
			5							
			6							
			7							

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Borehole No. 2
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BOREHOLE LOG

Client: HANSEN YUNCKEN
Project: PROPOSED INDUSTRIAL ESTATE
Location: CNR. THE HORSLEY DRIVE & COWPASTURE ROAD, WETHERILL PARK, NSW

Job No. 25371ZR
Date: 22-11-11
Method: SPIRAL AUGER JK300
R.L. Surface: ≈ 73.0m
Datum: ASSUMED
Logged/Checked by: D.S./

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			TOPSOIL: Silty clay, high plasticity, red brown.				GRASS COVER
		N = 7 2,3,4			CH	SILTY CLAY: high plasticity, red brown mottled light grey, trace of fine to medium grained ironstone gravel.	MC>PL	VSt	- 270 290 310	
		N > 11 5,11/ 150mm REFUSAL	1			SILTY CLAY: high plasticity, light grey and red brown, with fine to medium grained ironstone gravel.	MC<PL	H	>600 >600 >600	
			2		-	SHALE: brown and dark grey.	DW	M	-	MODERATE 'TC' BIT RESISTANCE
			3			END OF BOREHOLE AT 3.0m				
			4							
			5							
			6							
			7							

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Borehole No.
3
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BOREHOLE LOG

Client: HANSEN YUNCKEN
Project: PROPOSED INDUSTRIAL ESTATE
Location: CNR. THE HORSLEY DRIVE & COWPASTURE ROAD, WETHERILL PARK, NSW

Job No. 25371ZR
Date: 22-11-11
Method: SPIRAL AUGER JK300
Logged/Checked by: D.S./
R.L. Surface: ≈ 68.5m
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			TOPSOIL: Silty clay, high plasticity, dark brown, trace of root fibres.	MC>PL			GRASS COVER
					CH	SILTY CLAY: high plasticity, red brown mottled light grey, trace of root fibres.	MC>PL	VSt	-	
		N = 5 2,3,2							270 270 290	
			1			SILTY CLAY: high plasticity, light grey, with fine to medium grained ironstone gravel.				
		N = 10 2,3,7	2						250 290 310	
					-	SHALE: brown and red brown.	XW	EL	-	VERY LOW 'TC' BIT RESISTANCE
		N = 22 10,19,12	3							
						SHALE: brown and dark grey.	DW	L		LOW RESISTANCE
			4							
			5							
			6			END OF BOREHOLE AT 6.0m				
			7							

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Borehole No.
4
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BOREHOLE LOG

Client: HANSEN YUNCKEN
Project: PROPOSED INDUSTRIAL ESTATE
Location: CNR. THE HORSLEY DRIVE & COWPASTURE ROAD, WETHERILL PARK, NSW

Job No. 25371ZR
Date: 21-11-11
Method: SPIRAL AUGER JK300
R.L. Surface: ≈ 82.7m
Datum: ASSUMED
Logged/Checked by: D.S./

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		CH	TOPSOIL: Silty clay, high plasticity, red brown, trace of fine to medium grained ironstone gravel and root fibres.	MC>PL	H	-	GRASS COVER
		N = 14 3,5,9				SILTY CLAY: high plasticity, red brown, trace of fine to medium grained ironstone gravel.	MC>PL		450 500 >600	
			1		-	SHALE: brown and red brown.	DW	M		MODERATE 'TC' BIT RESISTANCE
			2			SHALE: dark grey.		H		HIGH RESISTANCE
			3			END OF BOREHOLE AT 3.0m				
			4							
			5							
			6							
			7							

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BOREHOLE LOG

Borehole No. **5**
1/1

Client: HANSEN YUNCKEN
Project: PROPOSED INDUSTRIAL ESTATE
Location: CNR. THE HORSLEY DRIVE & COWPASTURE ROAD, WETHERILL PARK, NSW

Job No. 25371ZR
Date: 21-11-11
Method: SPIRAL AUGER JK300
R.L. Surface: ≈ 80.0m
Datum: ASSUMED
Logged/Checked by: D.S./

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		CH	TOPSOIL: Silty clay, high plasticity, red brown, with fine to medium grained ironstone gravel, trace of root fibres.	MC>PL	H	-	GRASS COVER
		N = 7 3,3,4				SILTY CLAY: high plasticity, red brown, with fine to medium grained ironstone gravel.	MC>PL		450 480 450	
			1							
		SPT 7/100mm REFUSAL			-	SANDSTONE: fine grained, brown and light grey.	XW	EL	-	
			2			SANDSTONE: fine grained, brown.	DW	M		MODERATE 'TC' BIT RESISTANCE
			3			END OF BOREHOLE AT 3.0m				
			4							
			5							
			6							
			7							

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Borehole No. **6**

1/1

BOREHOLE LOG

Client: HANSEN YUNCKEN
Project: PROPOSED INDUSTRIAL ESTATE
Location: CNR. THE HORSLEY DRIVE & COWPASTURE ROAD, WETHERILL PARK, NSW

Job No. 25371ZR
Date: 21-11-11
Method: SPIRAL AUGER JK300
R.L. Surface: ≈ 84.0m
Datum: ASSUMED
Logged/Checked by: D.S./

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			TOPSOIL: Silty clay, high plasticity, red brown, trace of root fibres.	MC>PL			GRASS COVER
					CH	SILTY CLAY: high plasticity, red brown.	MC>PL	VSt	-	
		N = 9 3,4,5							370 350 350	
			1							
		N > 20 10,20/ 150mm REFUSAL			-	SHALE: light grey and red brown.	XW	EL		
			2			SHALE: red brown and dark grey.	DW	L		LOW 'TC' BIT RESISTANCE
			3			END OF BOREHOLE AT 3.0m				
			4							
			5							
			6							
			7							

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Borehole No. **7**
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BOREHOLE LOG

Client: HANSEN YUNCKEN
Project: PROPOSED INDUSTRIAL ESTATE
Location: CNR. THE HORSLEY DRIVE & COWPASTURE ROAD, WETHERILL PARK, NSW

Job No. 25371ZR
Date: 22-11-11
Method: SPIRAL AUGER JK300
R.L. Surface: ≈ 81.3m
Datum: ASSUMED
Logged/Checked by: D.S./

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			TOPSOIL: Silty clay, high plasticity, dark brown and grey.	MC>PL			GRASS COVER
					CH	SILTY CLAY: high plasticity, light grey mottled red brown, with fine to medium grained ironstone gravel.	MC>PL	(St)	-	
		N = 16 6,6,10					MC<PL	H	>600 >600 >600	
			1							
					-	SHALE: red brown and light grey.	XW	EL	-	VERY LOW 'TC' BIT RESISTANCE
		SPT 10/150mm REFUSAL				SHALE: red brown and dark grey.	DW	L		LOW RESISTANCE
			2							
						SHALE: brown and red brown.		H		MODERATE RESISTANCE
			3			END OF BOREHOLE AT 3.0m				
			4							
			5							
			6							
			7							

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Borehole No. **8**

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BOREHOLE LOG

Client: HANSEN YUNCKEN

Project: PROPOSED INDUSTRIAL ESTATE

Location: CNR. THE HORSLEY DRIVE & COWPASTURE ROAD, WETHERILL PARK, NSW

Job No. 25371ZR

Date: 21-11-11

Method: SPIRAL AUGER JK300

R.L. Surface: ≈ 72.4m

Datum: ASSUMED

Logged/Checked by: D.S./

Groundwater Record	Field Tests	Depth (m)	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DRY ON COMPLETION		0	CH	TOPSOIL: Silty clay, high plasticity, brown.	MC>PL		-	GRASS COVER
	N = 4 1,2,2			SILTY CLAY: high plasticity, red brown mottled light grey, trace of fine to medium grained ironstone gravel and root fibres.	MC>PL	VSt	250 280 300	
		1						
	N = 9 3,4,5						240 250 280	
		2						
		3						
	N = 17 6,7,10						320 350 350	
		4	-	SHALE: brown.	DW	L		LOW 'TC' BIT RESISTANCE
		5				M		MODERATE RESISTANCE
		6		END OF BOREHOLE AT 6.0m				
		7						

Samples: ES, U50, DB, DS

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BOREHOLE LOG

Borehole No. **9**

Client: HANSEN YUNCKEN

Project: PROPOSED INDUSTRIAL ESTATE

Location: CNR. THE HORSLEY DRIVE & COWPASTURE ROAD, WETHERILL PARK, NSW

Job No. 25371ZR

Date: 21-11-11

Method: SPIRAL AUGER JK300

R.L. Surface: ≈ 75.2m

Datum: ASSUMED

Logged/Checked by: D.S./

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		CH	TOPSOIL: Silty clay, high plasticity, brown, trace of root fibres.	MC>PL	VSt	-	GRASS COVER
		N = 9 4,5,4				SILTY CLAY: high plasticity, light grey mottled red brown.	MC<PL		320 350 370	
			1							
					-	SHALE: red brown and brown.	DW	VL-L		VERY LOW TO LOW 'TC' BIT RESISTANCE
			2							
			3			END OF BOREHOLE AT 3.0m				
			4							
			5							
			6							
			7							

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Borehole No.
10
1/1

BOREHOLE LOG

Client: HANSEN YUNCKEN
Project: PROPOSED INDUSTRIAL ESTATE
Location: CNR. THE HORSLEY DRIVE & COWPASTURE ROAD, WETHERILL PARK, NSW

Job No. 25371ZR **Method:** SPIRAL AUGER JK300 **R.L. Surface:** ≈ 72.0m
Date: 21-11-11 **Datum:** ASSUMED
Logged/Checked by: D.S./

Groundwater Record	Field Tests	Depth (m)	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DRY ON COMPLETION		0	CH	TOPSOIL: Silty clay, medium plasticity, brown, trace of root fibres.	MC>PL	VSt	-	GRASS COVER
	N = 4 1,2,2			SILTY CLAY: high plasticity, light grey mottled brown, trace of root fibres.	MC>PL		260 230 260	
	N = 19 4,7,12	1		SILTY CLAY: high plasticity, light grey, with fine to medium grained ironstone gravel.	MC<PL	H	>600 520 500	
		2	-	SHALE: light grey and red brown.	XW	EL	-	VERY LOW 'TC' BIT RESISTANCE
		3		SHALE: brown and red brown.	DW	VL-L		LOW RESISTANCE
		4	-	SHALE: brown, red brown and dark grey.		L-M		LOW TO MODERATE RESISTANCE
				END OF BOREHOLE AT 4.5m				
		5						
		6						
		7						

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CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

Borehole No. **11**

1/1

BOREHOLE LOG

Client: HANSEN YUNCKEN

Project: PROPOSED INDUSTRIAL ESTATE

Location: CNR. THE HORSLEY DRIVE & COWPASTURE ROAD, WETHERILL PARK, NSW

Job No. 25371ZR

Date: 22-11-11

Method: SPIRAL AUGER JK300

R.L. Surface: ≈ 68.0m

Datum: ASSUMED

Logged/Checked by: D.S./

Groundwater Record	Field Tests	Depth (m)	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DRY ON COMPLETION		0	CL	TOPSOIL: Silty clay, medium plasticity, brown, with fine to medium grained ironstone gravel, trace of root fibres.	MC>PL	VSt-H	-	GRASS COVER
	N = 6 2,3,3			SILTY CLAY: medium plasticity, brown, with fine to medium grained ironstone gravel.	MC>PL		370 420 420	
	N = 7 3,3,4	1	CH	SILTY CLAY: high plasticity, red brown, with fine to coarse grained sand.		F	90 110 150	
	N > 18 10,7, 11/100mm REFUSAL	3		SILTY CLAY: high plasticity, with fine to medium grained ironstone gravel.		VSt	260 320 290	
			-	SHALE: red brown and light grey.	XW	EL	-	VERY LOW 'TC' BIT RESISTANCE
		4		SHALE: brown and dark grey.	DW	L		LOW RESISTANCE
		5				H		HIGH RESISTANCE
		6		END OF BOREHOLE AT 6.0m				
		7						

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BOREHOLE LOG

Borehole No. 12
1/1

Client: HANSEN YUNCKEN
Project: PROPOSED INDUSTRIAL ESTATE
Location: CNR. THE HORSLEY DRIVE & COWPASTURE ROAD, WETHERILL PARK, NSW

Job No. 25371ZR
Date: 22-11-11
Method: SPIRAL AUGER JK300
R.L. Surface: ≈ 64.2m
Datum: ASSUMED
Logged/Checked by: D.S./

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			TOPSOIL: Silty clay, medium plasticity, brown, trace of root fibres.				GRASS COVER
		N = 8 3,3,5			CH	SILTY CLAY: high plasticity, red brown, trace of fine to medium grained ironstone gravel.	MC>PL	F	250 220 260 270	
		N = 7 3,3,4	1			SILTY CLAY: high plasticity, red brown mottled grey, fine to coarse grained ironstone gravel.	MC<PL	VSt	300 350	
			2		-	SHALE: light grey and red brown.	XW	EL	-	BANDS OF VERY LOW 'TC' BIT RESISTANCE
		SPT 11/150mm	3			as above, but with clay seams.	XW	EL		
						SHALE: brown, red brown and dark grey.	DW	VL		VERY LOW RESISTANCE
			6			END OF BOREHOLE LOG AT 6.0m				

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SCALE (m)

0 150

BOREHOLE LOCATION PLAN

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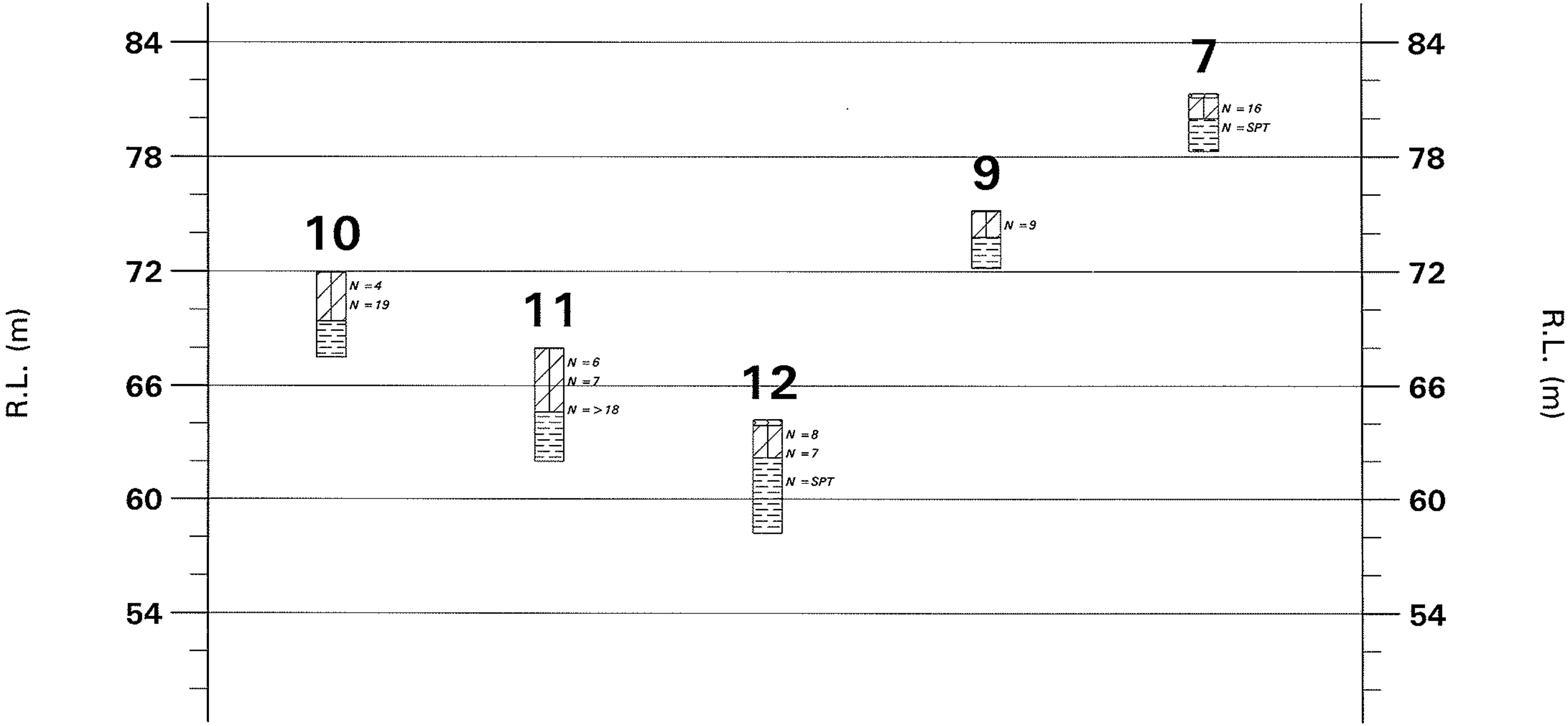
Report No. 25371ZR **Figure No. 1**

GRAPHICAL BOREHOLE SUMMARY



Topsoil	N	SPT "N" VALUE
Silty Clay	Nc	SOLID CONE BLOW COUNTS PER 150mm
Shale		

NOTE: REFER TO BOREHOLE LOGS

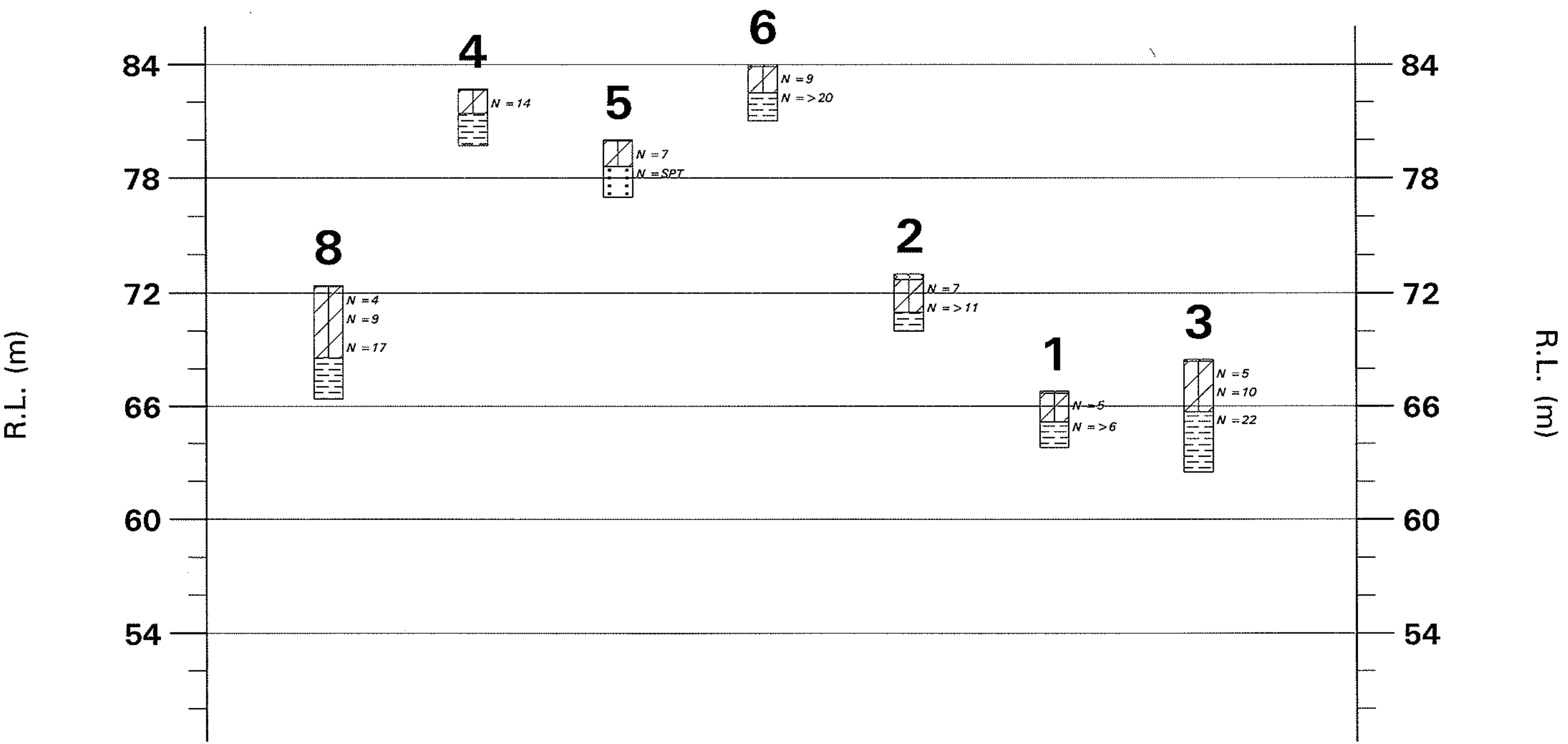
Scale: 1 : 300 (vert) ; NTS (horiz)

Jeffery and Katauskas Pty Ltd

Job No.: 25371ZR

Figure No.: 2

GRAPHICAL BOREHOLE SUMMARY



	Topsoil		Sandstone/ Greywacke
	Silty Clay	N	SPT "N" VALUE
	Shale	Nc	SOLID CONE BLOW COUNTS PER 150mm

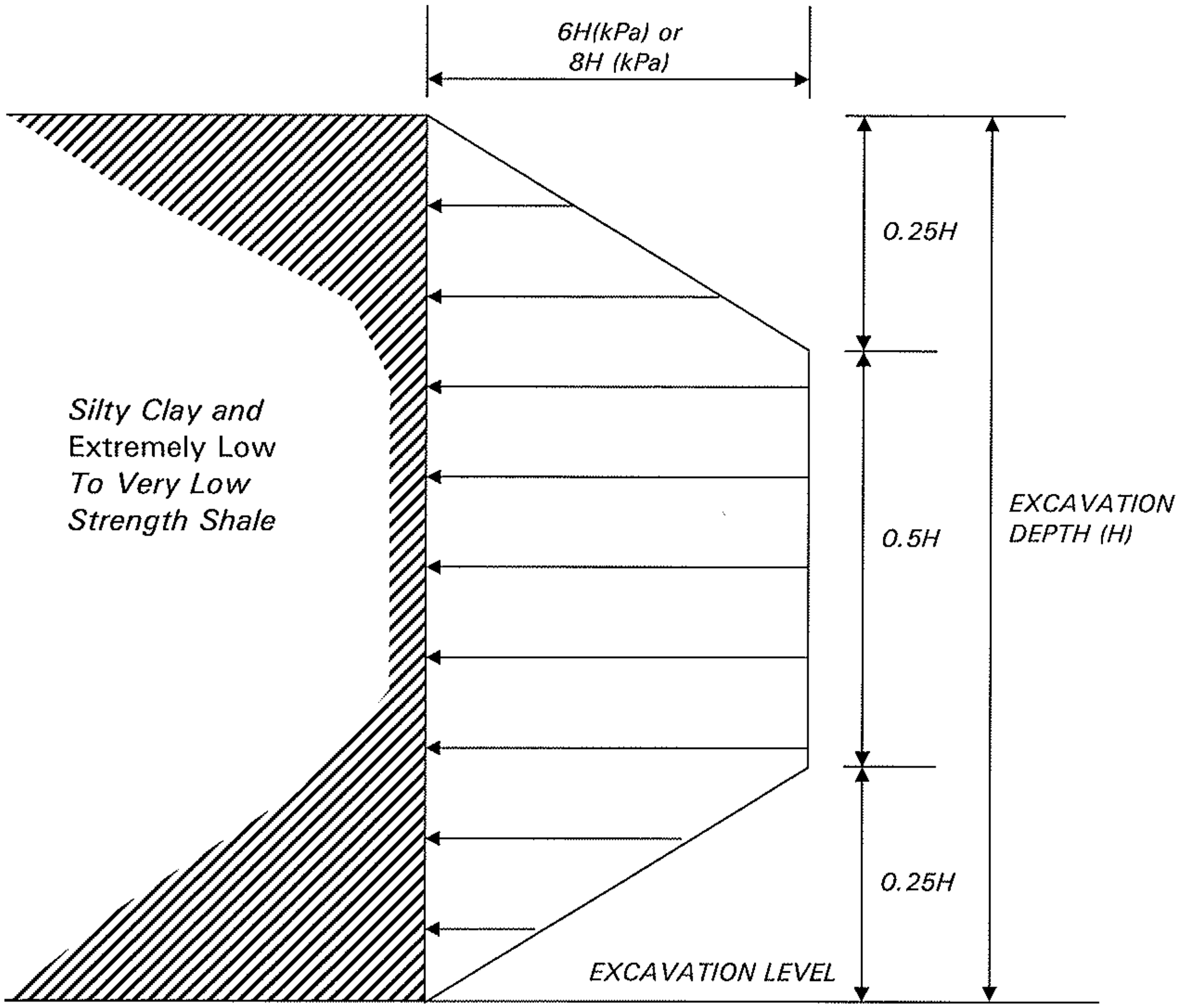
NOTE: REFER TO BOREHOLE LOGS

Scale: 1 : 300 (vert) ; NTS (horiz)

Jeffery and Katauskas Pty Ltd

Job No.: 25371ZR

Figure No.: 3



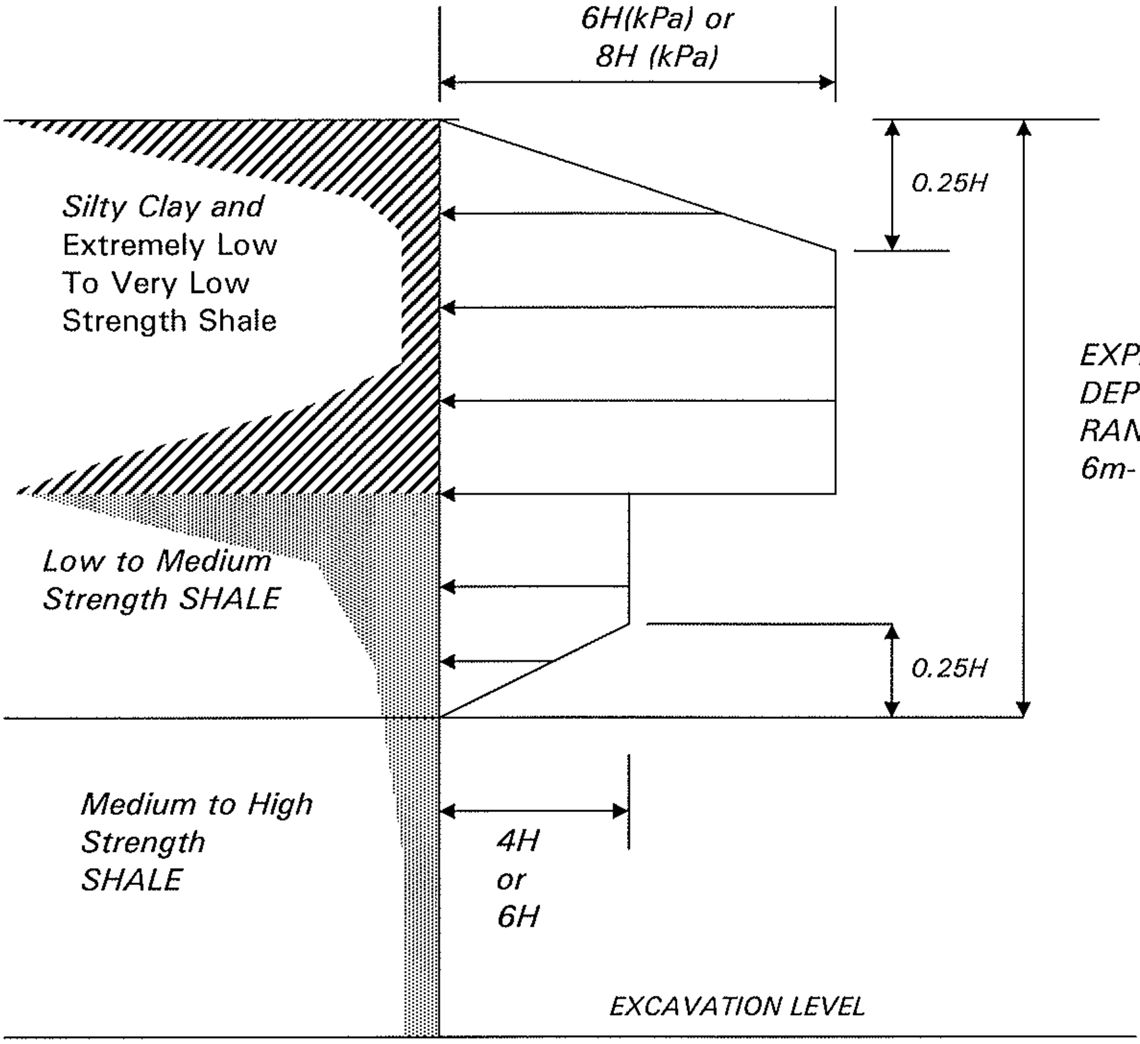
NOTES:

1. USE 6H FOR DESIGN WHERE NO MOVEMENT SENSITIVE STRUCTURES OR SERVICES ARE LOCATED WITHIN H FROM LINE OF EXCAVATION.
2. USE 8H FOR DESIGN WHERE MOVEMENT SENSITIVE STRUCTURES OR SERVICES ARE LOCATED WITHIN H FROM LINE OF EXCAVATION.
3. SURCHARGE AND GROUNDWATER PRESSURES MUST BE ADDED TO THE ABOVE IF APPLICABLE.
4. REFER TO TEXT OF REPORT.

RECOMMENDED DESIGN PRESSURES FOR ANCHORED OR PROPPED RETAINING WALLS

Jeffery & Katauskas Pty Ltd

Report No. 25371ZR Figure No. 4



NOTES:

1. USE 6H AND 4H FOR DESIGN WHERE NO MOVEMENT SENSITIVE STRUCTURES OR SERVICES ARE LOCATED WITHIN H FROM LINE OF EXCAVATION.
2. USE 8H AND 6H FOR DESIGN WHERE MOVEMENT SENSITIVE STRUCTURES OR SERVICES ARE LOCATED WITHIN H FROM LINE OF EXCAVATION.
3. SURCHARGE AND GROUNDWATER PRESSURES MUST BE ADDED TO THE ABOVE IF APPLICABLE.
4. VERTICAL SHALE BATTERS ARE LIKELY SUBJECT TO PROGRESSIVE GEOTECHNICAL INSPECTIONS; ANCHORING OF UNSTABLE ROCK WEDGES MAY BE REQUIRED TOGETHER WITH PATTERN BOLTING AND REINFORCED SHOTCRETE.
5. REFER TO TEXT OF REPORT

RECOMMENDED DESIGN PRESSURES FOR ANCHORED OR PROPPED RETAINING WALLS

Jeffery & Katauskas Pty Ltd

Report No. 25371ZR Figure No. 5

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

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UNIFIED SOIL CLASSIFICATION TABLE

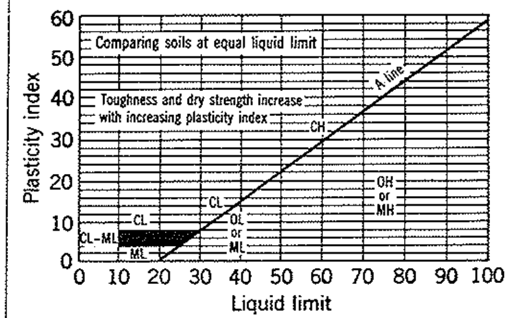
Major divisions			Field Identification Procedures (Excluding particles larger than 75 μm and basing fractions on estimated weights)	Group Symbols[a]	Typical Names	Information Required for Describing Soils	Laboratory Classification Criteria
Coarse-grained soils More than half of material is *larger* than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

(The 75 μm sieve size is about the smallest particle visible to naked eye)

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.

Identification Procedures on Fraction Smaller than 380 μm Sieve Size

Major divisions		Dry Strength (crushing character-istics)	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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LOG SYMBOLS

LOG COLUMN	SYMBOL	DEFINITION
Groundwater Record		Standing water level. Time delay following completion of drilling may be shown.
		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.
	ASB	Soil sample taken over depth indicated, for asbestos screening.
	ASS	Soil sample taken over depth indicated, for acid sulfate soil analysis.
	SAL	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	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.
	T 60	Penetration of auger string in mm under static load of rig applied by drill head hydraulics without rotation of augers.

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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	