REPORT TO

FOREST CENTRAL BUSINESS PARK PTY LTD

ON

GEOTECHNICAL INVESTIGATION

FOR

PROPOSED MEDICAL CENTRE

AT

Lot 7, DP1020015,
FOREST CENTRAL BUSINESS PARK,
49 FRENCHS FOREST ROAD EAST,
FRENCHS FOREST, NSW

Date: 4 September 2019
Ref: 32505BMrpt Rev1

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ATTACHMENTS

1 INTRODUCTION

This report presents the results of a geotechnical investigation for a proposed medical centre known as 'Project Maui Oncology' at Lot 7, DP1020015, part of Forest Central Business Park, 49 Frenchs Forest Road East, (also known as 502B Warringah Road), Frenchs Forest, NSW. A site location plan is presented as Figure 1. The investigation was commissioned by Mr Ryan Cooke of Erilyan Pty Ltd. The commission dated 22 August 2019 was on the basis of our fee proposal (Ref: P48969S) dated 18 February 2019, and emailed variations dated 26 July 2019.

We have been provided with the following:

- Architectural Floor Plan and Section drawings (Project No. 856, Drawing No. DA-100 to 103 $^{\text{Rev 2}}$, and 300 & 301 $^{\text{Rev 1}}$, all dated 28 August 2019) prepared by Team 2 Architects, and
- Brief for Geotechnical and Environmental Services (Ref. 191134, dated 2 July 2019), prepared by Taylor Thomson Whitting (TTW), indicating column loads to be in the order of 4,000kN to 7,000kN.

Based on a review of the above information, we understand that the proposed development will comprise a building with four above ground levels over three levels of basement carparking. The proposed basement carpark will have a lowest floor level, 'Basement 3', at RL149.95 requiring excavation to depths ranging from about 9m to 11m below existing surface levels. The basement extends as follows:

- to the northern boundary,
- to about 2m from the eastern boundary,
- to about 3m from the western boundary and
- to about 15m from the southern boundary.

The above ground structure does not extend to the same extent as the basement in the north-western corner, in order to accommodate a proposed turning circle at the end of the common driveway from Frenchs Forest Road East.

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

This geotechnical investigation was carried out in conjunction with a preliminary environmental site assessment by our specialist division, JK Environments (JKE). Reference should be made to the JKE report, Ref: E32505BTrpt, dated 30 August 2019 for the results of the preliminary environmental site assessment.

2 INVESTIGATION PROCEDURE

The fieldwork for the geotechnical investigation was carried out on 1 and 2 August 2019 and comprised the drilling of nine boreholes (BH1 to BH9), at the locations shown on Figure 2. BH1 to BH6 were auger drilled to depths of 3m below the existing ground surface. BH7 to BH9 were auger drilled to depths ranging from 4.75m to 5.12m and were continued thereafter to depths ranging from 12m to 12.1m by diamond coring techniques using an NMLC core barrel with water flush. The boreholes were drilled using our track mounted JK250 drill rig.

BH7 to BH9 were set out as close as practicable to locations nominated by TTW. The locations and surface reduced levels (RLs) of BH7 to BH9 were recorded by the project surveyor, Geomat, and later supplied to us on a 'Preliminary' plan dated 23/8/19 (File Ref. 1775 Lot7 DP1020015 230819). The locations of BH1 to BH6, which were also used to obtain soil samples for the environmental site assessment by JKE, were recorded by tape measurements from boundary markers. The surface levels at these locations were later estimated from spot heights and contours on the supplied survey drawing except for BH1 and BH3 which were also recorded by the surveyor. We assume that the survey datum is the Australian Height Datum (AHD).

The strength/relative density of the subsoil profile was assessed from Standard Penetration Test (SPT) 'N' values, together with hand penetrometer readings on cohesive soils recovered in the SPT split-spoon sampler and by examination of auger cuttings. Where bedrock was auger drilled the strength was assessed by monitoring the drilling resistance using a tungsten carbide (TC) bit, together with inspection of the recovered rock cuttings.

Core samples were boxed and logged on site and then returned to our laboratory where core photographs and point load strength index tests were completed. The core photographs are presented with the borehole logs. The point load test results are presented in the attached STS Table A and are also shown on the cored borehole logs.

Groundwater observations were made in all boreholes during and immediately on completion of drilling. Two Class 18 50mm PVC piezometers were installed in BH7 and BH9 to facilitate longer term groundwater monitoring and future hydrogeological analysis. Piezometer construction details are presented on the borehole logs. Note that water was used during the coring process so initial readings may have been artificially high, with more time required for levels to stabilise. A number of return visits to site were subsequently made to complete pump out tests and measure the standing water levels and recharge rates for permeability analysis. The groundwater levels and permeability results are presented in Section 3.2. A data logger has been installed in BH7 for future monitoring over a period of 2 weeks, for detailed design purposes.

Our geotechnical engineer (Warren Smith) was present full-time during the fieldwork to set out the borehole locations, log the encountered subsurface profile, nominate in-situ testing and sampling and install the piezometers. The borehole logs (which include initial groundwater observations) are attached, together with a glossary of logging terms and symbols used.

Further details of the methods and procedures employed in the investigation are presented in the attached Report Explanation Notes.

A bulk sample of the shallow soils in BH9 was returned to Soil Test Services Pty Ltd (STS), a NATA accredited laboratory, for standard compaction and four day soaked CBR testing. The results of this testing is presented in the attached STS Table B. Selected samples were also returned to Envirolab Services Pty Ltd for pH, chloride content, sulfate content, and resistivity (soil aggression) testing. The results of that testing are presented in the attached Envirolab Certificate of Analysis 223478.

3 RESULTS OF INVESTIGATION

3.1 Site Description

The site is located on top of a broad hill where the surrounding topography is relatively flat. Locally, the site slopes down to the south at about 5° towards Warringah Road which lies beyond the southern site boundary.

The site is rectangular in plan with surface levels ranging from about RL160.5m at the northern end to about RL157.5m at the southern end.

At the time of the investigation, the site was being utilised as a storage yard and was occupied by soil and gravel stockpiles along the western boundary, concrete drainage pits and supplies to the east and a storage container located to the north. The central portion of the site was mainly gravel covered. In the north-eastern corner was an electrical substation.

The site includes numerous easements and buried service pits could be seen on site, including for an existing sewer (as indicated on the Sydney Water DBYD plan) extending across the top of the site (approx. east-west) and then north to south. The sewer is noted to be 225mm PVC and partially concrete encased. Its depths are noted to be about 3m. We were also informed by site personnel of a buried concrete tank immediately to the south of BH7.

To the east of the site was an asphaltic concrete (AC) paved carpark appearing to be in poor condition with crocodile cracking, ruts, scouring and potholes observed.

Neighbouring the site to the north was a three-storey commercial building, with at least one level of basement parking, which appeared to be in good condition. The building was set back from the site boundary by about 20m. Between the building and the common boundary was a concrete surfaced carpark bordered by an (approx.) 2m wide nature strip. Also to the north of the site, but at the eastern end, is a telecommunications compound containing a tall communications tower.

To the west of the site was a four-storey commercial building, which appeared to be good condition, with at least one level of basement parking. The building, which has an irregular shape, had a variable set back of about 5m to 14m from the site boundary.

Site boundaries were generally marked by chain-link fencing. Surface levels and slopes appeared to be similar across the boundaries.

3.2 Subsurface Conditions

The 1:100,000 geological map of Sydney (Geological Survey of NSW, Geological Series Sheet 9130) indicates that the site is underlain by a sub-unit of the Hawkesbury Sandstone which often comprises shale (siltstone) and laminite, which would overlie the more common sandstone unit at depth.

The investigation revealed a generalised subsurface profile comprising silty clay fill overlaying residual silty clay grading into weathered siltstone in turn underlain by sandstone bedrock. A summary of the subsurface conditions encountered is presented below and graphical sections are presented in Figure 3 and 4, but for further specific details reference should be made to the attached borehole logs.

Fill

The fill comprised mainly silty clay of medium plasticity with inclusions of fine to medium grained igneous gravel (BH1 to BH7 and BH9) and silty clayey gravel (BH8). The fill extended to depths ranging from 0.2m (BH9) to 1m (BH1).

Residual Silty Clays

The residual silty clay beneath the fill was assessed to be of high plasticity and with strengths ranging from stiff to hard strength. BH1 to BH6 were terminated within the residual silty clay at depths of 3m. However, it should be noted that there is often an indistinct transition from residual clay to extremely weathered siltstone which is difficult to assess from disturbed samples.

Weathered Siltstone

Extremely Weathered Siltstone (locally referred to as shale) was encountered in BH7 to BH9 at depths ranging from 1.4m (BH9) to 3.2m (BH7). In BH8 and BH9, the siltstone was assessed to be highly weathered and of very low to low strength below depths of 4.8m and 2.0m, respectively.

Weathered Sandstone

In BH7, BH8 and BH9, once coring was commenced at depths of 4.9m, 5.12m and 4.75m, respectively, sandstone bedrock was encountered. The sandstone was generally assessed to be moderately to highly weathered and of very low to low strength throughout the depth of investigation, with some medium strength bands in BH8 and BH9.

Defects were relatively well spaced and generally comprised near horizontal bedding partings, extremely weathered seams and some inclined joints.

Groundwater

All the boreholes were dry upon completion of augering. The maximum depth of auger drilling was about 5.12m (BH8). In BH7 to BH9, the use of water for coring limited further measurements during drilling.

A standing water level of 3.2m was recorded on completion of coring of BH7, but is likely to have been affected by water introduced during coring. A return site visit was made on 27 August 2019 (26 days after drilling) and it was discovered that the monitoring well in BH9 had been destroyed and further groundwater measurements within that well were not possible. Within the well in BH7, groundwater was measured at a depth of 5.5m and the well was then pumped dry to allow water to recharge, with the recharge rate measured using a data logger to assess the permeability of the weathered sandstone. A site visit was again made on 28 August 2019 and groundwater was measured in BH7 at a depth of 7.05m (≈RL152.3m). A final site visit was made on 29 August 2019 and groundwater was measured in BH7 at a depth of 7.8m (≈RL151.5m).

Based on the recharge rate into BH7, the permeability of the weathered sandstone bedrock was calculated to be about 6×10^{-8} m/s, which is in the order expected for sandstone bedrock with relatively few defects. Further groundwater monitoring and analysis are being carried out for this project and will be reported separately.

3.3 Laboratory Test Results

The results of the point load strength index tests showed reasonably good correlation with our field assessment of rock strength. The estimate unconfined compressive strength (UCS), which is based on correlation with the point load strength test (Is_{50}) results, was generally within 1MPa to 2MPa but ranged from less than 1MPa to 16MPa, as shown on STS Table A.

The four day soaked CBR test on a sample of the natural silty clay from BH9 compacted to 98% of its Standard Maximum Dry Density (SMDD), returned a CBR value of 1.5%, as shown on STS Table B. As part of this test the swell was recorded to be 4.5% which together with the low CBR test result indicates high plasticity clay with a high potential for shrink/swell movements with changes in moisture content.

The pH values ranged between 5.1 and 5.6 for the samples of the residual soils and weathered sandstone, indicating acidic conditions. The chloride contents ranged from less than 10mg/kg to 10mg/kg, the sulphate contents ranged from less than 10mg/kg to 34mg/kg and the resistivity ranged from 300ohm.m to 460ohm.m. Based on these results, the samples tested would have an exposure classification of 'mild' for concrete piles and 'non-aggressive' for steel piles in accordance with Tables 6.4.2(C) and 6.5.2(C) of AS2159-2009 'Piling – Design and Installation'.

4 COMMENTS AND RECOMMENDATIONS

4.1 Geotechnical Issues

The main geotechnical issue for the proposed development will be maintaining stability to the excavation sides and nearby structures, including buried services, during excavation to depths of up to about 9m to 11m below existing surface levels.

The proposed excavation will encounter predominantly shallow clayey fill over residual silty clay grading to extremely weathered siltstone and then sandstone bedrock from a depth of about 5m. However, the sandstone is only of very low to low strength for the full depth of the boreholes, and so is not considered suitable to be cut vertically without support. The use of temporary batters to such depths is not appropriate and therefore, the proposed excavation will need to be supported by a full depth shoring system installed prior to commencement of excavation. Lateral support to the shoring piles will be required in the temporary and permanent case. In the short term, a combination of temporary anchors and/or internal props will be required with the later necessary if neighbouring basements or services prevent the use of anchors. The extent of neighbouring basements must therefore be investigated at an early stage of design to assess the appropriate temporary support. Similarly, anchors may not be permitted under the telecoms compound and this should be assessed at an early stage.

Excavation of the soils and predominantly very low strength rock will be readily achieved using conventional excavation equipment, although some assistance with rock excavation equipment will be required for the higher strength bands.

Sandstone bedrock will be uniformly exposed at bulk excavation level (BEL) so all footings for the building will be founded on sandstone bedrock. Shallow pad and strip footings are therefore feasible.

Groundwater monitoring to date indicates the water level to be at a depth of about 7m (≈RL152.3m), within the sandstone bedrock, and about 2.4m above the lowest basement level. Permeability testing at one borehole location indicates a relatively low permeability for which we expect seepage through the defects in the sandstone bedrock will be readily managed using a pump and sump system. Further hydrogeological assessment will be required to confirm these assumptions.

The above principal geotechnical issues and other considerations are discussed in further detail in the following sections.

4.2 Dilapidation Surveys

Prior to any excavation commencing, we recommend that detailed dilapidation reports be prepared for the adjoining properties and services, particularly the development to the west of the site, and the carpark and telecoms site to the north. The dilapidation surveys should comprise a detailed inspection of the adjoining properties, both externally and internally, with all defects rigorously described, e.g. defect location, defect type, crack width, crack length, etc. The respective property owners should be provided with a copy of the dilapidation reports and be asked to confirm that they present a fair representation of the existing conditions. We note that Council/RMS may also require that dilapidation reports be prepared for their adjoining assets to the south, where ancillary road works are in progress.

Such reports can be used as a baseline against which to assess possible future claims for damage arising from the works.

4.3 Excavation

Excavation for the proposed basement is expected to extend to a maximum depth of about 11m below existing levels, with locally deeper excavations required for lift overrun pits and services. Excavation to such depths will extend through the fill and residual soil profile and then into the underlying weathered bedrock.

Excavation of the fill, residual soils, and the extremely weathered siltstone bedrock is expected to be readily achievable using conventional techniques, such as the buckets of medium to large sized hydraulic excavators. We also expect that excavation of the remaining very low to low strength sandstone will be achievable using large excavators with a combination of ripping tynes and toothed buckets where higher strength bands are encountered. Some assistance with rock hammers may be required for excavation of bands of higher strength material, such as the medium strength bands encountered in BH8 and BH9.

Excavation using hydraulic rock hammers must be carried out with care due to the risk of damage to adjoining structures from the vibrations generated by the hammer. In this respect, we recommend that excavation commence away from likely critical areas (i.e. commence within the central portion of the site) to allow monitoring of transmitted vibrations prior to excavation close to the adjoining structures. We recommend that the vibrations transmitted to the adjoining structures to the north and west be quantitatively monitored during rock hammer excavation works. Vibration monitors should be solidly fixed to the adjoining structures and the monitors attached to flashing warning lights, or other suitable warning systems, so that the operator is aware when acceptable limits have been reached so that excavation work can cease. If permission is not given to attach monitors to the adjoining structures then they should be set up on the site boundaries.

Vibrations, measured as Peak Particle Velocity (PPV), should be limited to no higher than 5mm/sec. However, if any particularly sensitive structures or equipment are present in adjacent properties then a lower target limit may be appropriate.

If higher vibrations are recorded than the target limits, they should be assessed against the attached Vibration Emission Design Goals as higher vibrations may be feasible depending on the associated vibration frequency. However, any on site warning devices can only be set against the PPV and not the associated vibration frequency so will need to be set for the lower PPV values. If it is confirmed that transmitted vibrations are excessive, then it would be necessary to use smaller plant or alternative lower percussion techniques, e.g. grid sawing in conjunction with ripping and rock grinders. The use of these alternative techniques will have lower productivity, but will limit vibrations. When using a rock saw or rotary grinder, the resulting dust must be suppressed by spraying with water.

We recommend use of excavation contractors with experience in such work and with a competent supervisor who is aware of vibration damage risks. The contractor should be provided with a full copy of this report and have all appropriate statutory and public liability insurances.

4.4 Groundwater

All of the boreholes were 'dry' during auger drilling. In BH7, groundwater was measured at a depth of about 7m (≈RL152.3m). Further measurements of groundwater levels are being made and will be reported separately and the comments provided below should be considered preliminary and will be confirmed once further information is available.

Whilst water is present above the lowest basement level, it was wholly within the sandstone bedrock profile. Calculations from the recharge following the initial pump out test, indicates that the sandstone is of low permeability (6 x 10^{-8}m/sec), which is what we would expect for the subsurface profile encountered. We therefore consider that design and construction of a drained basement is most appropriate for the proposed development.

During excavation, seepage may tend to occur along the soil/rock interface and through joints and bedding partings within the rock, and may increase during and following rainfall. It is also likely to reduce from probable initially higher seepage rates, as the water perched in joints and defects seeps out relatively quickly but is then usually slower to infiltrate/recharge from surface sources. The use of conventional sump and pump techniques are expected to be appropriate for control of seepage during construction. In the long term, drainage should be provided behind all retaining walls and below the lowest basement slab. The drainage system should direct seepage into sumps containing automatic and failsafe pumps to remove the collected water into the stormwater system. Observations should be made during and on completion of excavation to assess if the designed drainage system is suitable for the actual seepage flows.

Pumping of groundwater from the basement should not result in significant drawdown of groundwater in the vicinity of the site, as the groundwater is predominantly within the bedrock profile. Similarly, settlement of the near surface soils will not occur due to any drop in local groundwater levels. We therefore consider draining the basement will have a negligible impact on any nearby structures or infrastructure.

4.5 Basement Retention

Although sandstone will be encountered within the excavation the strength of the sandstone is generally only very low to low strength and it is not considered self-supporting and full depth shoring will be required. The use of temporary batters will not be feasible for the depth of the excavation proposed and the full depth shoring system will need to be installed prior to the start of excavation.

Based on the subsurface conditions encountered, the use of anchored or propped soldier pile shoring system, with reinforced shotcrete infill panels, installed prior to excavation will be suitable.

Where movements behind the wall are to be limited, such as adjacent to the telecoms compound or other structures, more rigid contiguous pile walls or closely spaced soldier piles may be required in order to limit movements. The effect of ground movements on any structures and services that lie within the influence zone of the excavation must be taken into account. The zone of influence of excavations may be defined as a horizontal distance from the wall of twice the excavation depth.

Conventional bored piles are considered suitable for use on this site and should be founded at least 1m below the base of the excavation, including excavations for footings and services and thickened edge beams etc, although deeper pile sockets may be required for stability design. The piles will need to be drilled through bands of medium strength rock and piling rigs with sufficient capacity to drill such rock should be mobilised to site. Given the expected depth of the piles, pouring using tremie methods is recommended, especially as groundwater inflow may occur into bored pile holes such that the piles would be tremie poured as 'wet' piles.

The shoring systems must be temporarily anchored or braced as the excavation progresses, by the use of external anchors or internal props. Approval from neighbouring land owners would be required prior to the installation of anchors below their property. Such permission can take some time to obtain and allowance should be made within the project program to allow time for negotiation. The location of any basements and services within the adjoining properties should also be investigated and considered so that these can be avoided during anchor installation. If permission cannot be obtained to install anchors, or where there would be insufficient space due to basements, it would be necessary to use internal props.

During excavation, reinforced shotcrete panels should be sprayed progressively during excavation to support the soil and weathered rock between the soldier piles, such that there is no more than 1.5m of vertical face of material exposed at any one time. Also, the progressive excavation and support installation sequence must be clearly stated on the design drawings to prevent over excavation and excessive deflection of the shoring piles.

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

- For anchored or propped soldier pile walls where minor movements can be tolerated, e.g. landscaped areas or similar, we recommend the use of a trapezoidal earth pressure distribution with a maximum lateral pressure of 6HkPa for the soil and weathered bedrock profile, where 'H' is the retained height in metres. These pressures should be assumed to be uniform of the central 50% of the support system, tapering to zero at the crest and toe.
- Where movements are to be limited, e.g. where neighbouring structures or movement sensitive services are located within 2H of the wall, the maximum lateral pressure should be increased to 8HkPa.
- For retention of sandstone of very low to low strength, an earth pressure of 10kPa should be adopted.
- A bulk unit weight of 20kN/m^3 should be adopted for the soil and extremely weathered bedrock.
- Any surcharge affecting the walls (e.g. traffic loading, construction loads, adjacent high level footings, etc.) should be allowed for in the design using an 'at-rest' earth pressure coefficient, K_0, of 0.5.
- The shoring walls should be designed as 'drained' and measures taken to provide permanent and effective drainage of the ground behind the walls. Strip drains should comprise a non-woven geotextile fabric (e.g. Bidim A34) to act as a filter against subsoil erosion.
- Lateral toe restraint of the fully penetrating shoring may be achieved by embedding the piles into the bedrock below the bulk excavation level. An allowable lateral resistance of 200kPa can be adopted for

the expected very low to low strength bedrock, though the upper 0.5m of socket must be ignored to allow for disturbance or possible over excavation. For piles embedded into bedrock below bulk excavation level, a minimum embedment depth (ignoring the 0.5m allowance above) of 1m should apply. Care is required not to over-excavate in front of the piles, and all excavations in front of the walls, such as for footings, tanks, buried services, etc. must be taken into account in the wall design.

- Anchors bonded into sandstone of at least very low strength bedrock may be designed on the basis of an allowable bond stress of 150kPa. All anchors should be proof loaded to at least 1.3 times their working load and then locked off at approximately 85% of their working load. Proof loading should be carried out in the presence of an engineer independent of the anchor contractor. Anchors must be bonded behind a line drawn up at 45° from the base of the excavation, with all anchors having a free length and bond length of at least 3m each. Lift off tests should be carried out on at least 10% of all anchors 24 to 48 hours following locking off to confirm that the anchors are maintaining their load.

Alternatively, the retaining walls could be designed using computer based soil structure interaction analysis methods (e.g. Plaxis), which could result in cost savings compared to a design based on the above simplified earth pressure assumptions. Analysis software treating the soil as 'equivalent springs' should not be used for this design. Analysis using soil structure interaction methods can model the actual excavation stages, including progressive anchoring/shoring, and outputs include structural actions in the piles, anchor/prop loads, and wall movements. The analysis should be completed by an engineer with a good understanding of soil-structure interaction behaviour, including an understanding of when soil wall friction should and should not be used, etc.

4.6 Landscaping Temporary Batters and Retaining Walls

For limited excavations of no more than about 3m outside of the main basement and where space permits, temporary batters within the soils and extremely weathered siltstone of 1 Vertical (V) to 1 Horizontal (H) are recommended in the short term, provided that no surcharge loads, including construction loads and existing footing loads, are placed at the top of the batters. Even if such batters are be able to be accommodated within the site boundaries, the batters may extend to, or close to, the boundaries and it may not be possible to control the placement of loads etc. in close proximity to the crests of such batters. This must be considered when assessing the feasibility of temporary batters.

Costs associated with removal of the soil and then replacement as controlled fill should also be considered along with the lost space of within the site which would often be otherwise utilised.

Permanent batters, if required, of no more than about 3m in height should be no steeper than 1V:2H, but flatter batters of the order of 1V:3H may be preferred to allow access for maintenance of vegetation. Permanent batters should be covered with topsoil and planted with a deep rooted runner grass, or other suitable coverings, to reduce erosion. All stormwater runoff should also be directed away from all temporary and permanent batters to also reduce erosion.

Long term landscaping retaining walls constructed in front of temporary batters may be designed based on a triangular earth pressure distribution using an active earth pressure coefficient, K_a, of 0.3 and a bulk unit weight of $20kN/m^3$. This assumes that some resulting ground movements are tolerable. Where movements are to be kept low the walls should be designed based on the trapezoidal distributions given above. This coefficient assumes horizontal backfill behind the wall and if inclined backfill is proposed the coefficient would need to be increased or the inclined backfill taken as a surcharge load.

All surcharges must be allowed for in design, including hydrostatic pressures unless full and effective drainage is provided for the design life of the structure.

Backfilling between temporary batters and permanent walls will need to be carried out with care to reduce the future settlement of the backfill. We recommend the use of hard and durable gravel as this is readily compactable. The use of the excavated clay and siltstone is not recommended as it will be difficult to compact within the limited space available. Only light compaction equipment should be used so that excessive lateral pressures are not placed on the walls, and therefore, the backfill will need to be placed in thin layers, say 100mm loose thickness.

The compaction specification for backfill will depend if paving will be supported on the backfill. If the fill is used to support paved areas it should be compacted to a density of at least 98% of Standard Maximum Dry Density (SMDD) for granular materials. For landscaped areas, a lower compaction specification of at least 95% of SMDD may be appropriate, provided the risk of future settlement and maintenance can be accepted.

4.7 Footings

On completion of excavation, sandstone bedrock of very low to low strength is expected to be exposed at bulk excavation level. All footings should therefore be uniformly supported within sandstone bedrock. Pad and strip footings will be suitable in the base of the excavation. If any of the above ground portions of the building extend outside the footprint of the proposed basement, they should be supported on piles founded within the rock to provide uniform support. Such piles should be founded below a line drawn up at 45° from the base of the excavation so that additional surcharge loads are not placed on the basement walls.

Pad or strip footings, or piles socketed a nominal 0.3m into the appropriate quality rock, may be designed for an allowable bearing pressure of 600kPa for extremely weathered siltstone or 1,000kPa for sandstone bedrock of at least very low strength, based on serviceability criteria. For the design of piles, allowable shaft adhesions of 10% of the above allowable bearing pressures for compressive loads, or 5% for uplift loads, may be used, provided socket roughness and cleanliness is maintained.

Higher bearing pressures would be appropriate with medium or high strength rock, but this was not encountered within the current boreholes and the use of piles below the bulk excavation level would be required to reach higher strength rock. In addition, additional deeper cored boreholes would be required to determine the design of any medium or high strength rock and assess the appropriate design parameters.

All footings must be inspected by a geotechnical engineer to confirm the material is appropriate for the design bearing pressure. All footings must be dry and clean of any loose material prior to pouring concrete.

Since the sandstone is of very low strength, blinding layers of concrete are advisable to prevent softening of the foundation material, for footings where it will takes time to form up steel reinforcement, such as at lift pits, or where other delays are anticipated.

The allowable bearing pressures given above are based on a serviceability criteria of deflections at the pile toe or footing base of less than or equal to 1% of the pile diameter or footing width. Footings on rock can also be designed using 'Limit State Design' principles. For limit state design, higher ultimate bearing capacities could be adopted provided that settlements of up to 5% of the pile diameter or footing width can be tolerated. Specific settlement analysis would be required where ultimate bearing pressures are adopted. Ultimate bearing pressures must also be used in conjunction with an appropriate geotechnical strength reduction factor (Φ_g).

4.8 Basement Floor Slab

Drainage will need to be provided below the basement slab either as a closely spaced grid of subsoil drains or a (single sized) gravel blanket. The drainage will need to be connected to a permanent fail safe pump out system, which is fitted with automatic level control pumps to avoid flooding.

If a drainage blanket is not adopted the basement slab should be designed with a subbase layer of at least 100mm thickness of crushed rock to RMS QA specification 3051 unbound base material (or other approved good quality and durable fine crushed rock), which is compacted to at least 100% of Standard Maximum Dry Density (SMDD). This subbase layer will provide a separation between the sandstone subgrade and the slab and provide a uniform base for the slab. The grid of subsoil drains would then be formed within this layer.

4.9 External Pavements

We expect that a limited area of external pavement outside of the basement excavation may be required in the north-western corner of the site linking the ramp/ground floor level turning circle to the street. Since the ramp and ground floor slabs within the basement excavation perimeter will be designed as suspended slabs supported on the structure founded within the sandstone bedrock, it would be advisable to design the external pavement the same, as a fully suspended slab supported on piles founded within the bedrock. Alternatively, a movement joint would need to be provided between the suspended slab and the external pavement to allow for differential movement. If all pavements are designed as fully suspended slabs then no particular subgrade preparation would be required.

Where pavements are to be supported on the soil subgrade, all root affected material and any loose fill and deleterious fill must be stripped and be stockpiled for use in landscaped areas only. The silty clay subgrade below should be prepared by proof rolling with a at least six passes of a minimum eight tonne smooth drum roller. The final pass should be carried out in the presence of an experienced geotechnician or geotechnical engineer engaged independently of the earthworks contractor to detect any weak or unstable subgrade areas. Any weak areas identified during proof rolling should be locally excavated to a sound base and the excavated material replaced with engineered fill, or as directed by the geotechnical engineer during the proof rolling inspection.

Engineered fill should comprise well graded granular material such as crushed sandstone compacted to at least 98% of SMDD. The use of excavated clay fill, residual silty clay or excavated siltstone is not recommended within the limited space available.

Following preparation of the soil subgrade as detailed above, pavement design may be based on the measured soaked CBR value of 1.5%. This CBR is low and we recommend that consideration be given to the use of a select layer of good quality granular material to replace the upper subgrade and reduce the thickness of the overlying pavement materials.

Surface and subsoil drainage should be provided on the high side of the pavements to prevent moisture ingress into the subgrade and pavement. The subsoil drains should have an invert level of at least 300mm below the adjacent subgrade level and be excavated with a uniform longitudinal fall to appropriate discharge points so as to reduce the risk of ponding in the base of the drain. In addition, the surface of the adjacent pavement subgrade should be provided with a uniform cross fall towards the subsoil drain to assist with drainage.

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

4.10 Earthquake Subsoil Classification

The site sub-soil class is a ‘Class C_e - Shallow Soil Site’ in accordance with AS1170-2007 with Amdt 1 and 2. However, following excavation of the soil and extremely weathered siltstone, the main building will be founded directly in Class Be rock, being rock with an (estimated unconfined) compressive strength between 1MPa and 50MPa.

4.11 Further Geotechnical Input

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

- Investigation of neighbouring basement levels and extents (to north and west)
- Mapping of buried services (for diversions)
- Dilapidation surveys
- Further groundwater monitoring and seepage analysis (detailed hydrogeological assessment)
- Shoring design and deflection analysis

- Inspection of initial shoring pile drilling
- Inspection of temporary anchor drilling installation and proof loading
- Vibration Monitoring
- Inspection of seepage
- Inspection of footings
- Proof rolling of subgrade for external pavements

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 JK Geotechnics accept no responsibility whatsoever for the performance of the structure where recommendations are not implemented in full and properly tested, inspected and documented.

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

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

This report provides 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.

This report has been prepared for the particular project described and no responsibility is accepted for the use of any part of this report in any other context or for any other purpose. If there is any change in the proposed development described in this report then all recommendations should be reviewed. Copyright in this report is the property of JK Geotechnics. We have used a degree of care, skill and diligence normally exercised by consulting engineers in similar circumstances and locality. No other warranty expressed or implied is made or intended. Subject to payment of all fees due for the investigation, the client alone shall have a licence to use this report. The report shall not be reproduced except in full.

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SOIL TEST SERVICES

TABLE A
POINT LOAD STRENGTH INDEX TEST REPORT

Client:	JK Geotechnics	**Ref No:**	32505S
Project:	Proposed Medical Centre	**Report:**	A
Location:	Forest Central Business Park, Frenchs Forest, NSW	**Report Date:**	8/08/2019
		Page 1 of 2	

BOREHOLE NUMBER	DEPTH m	$I_{S\ (50)}$ MPa	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)
7	5.14 - 5.17	0.1	2
	5.74 - 5.77	0.2	4
	6.22 - 6.24	0.03	1
	6.71 - 6.74	0.09	2
	7.39 - 7.42	0.1	2
	7.84 - 7.87	0.2	4
	8.26 - 8.29	0.05	1
	8.63 - 8.66	0.1	2
	9.02 - 9.05	0.07	1
	9.67 - 9.70	0.05	1
	10.07 - 10.10	0.2	4
	10.79 - 10.82	0.2	4
	11.19 - 11.22	0.08	2
	11.62 - 11.65	0.01	<1
8	5.30 - 5.33	0.4	8
	5.78 - 5.81	0.06	1
	6.23 - 6.26	0.5	10
	6.68 - 6.71	0.1	2
	7.19 - 7.23	0.8	16
	7.66 - 7.69	0.3	6
	8.17 - 8.20	0.08	2
	8.78 - 8.81	0.07	1
	9.20 - 9.22	0.04	1
	9.74 - 9.77	0.06	1
	10.19 - 10.22	0.06	1

NOTES: See Page 2 of 2

115 Wicks Road
Macquarie Park NSW 2113
Telephone: 02 9888 5000
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SOIL TEST SERVICES

TABLE A
POINT LOAD STRENGTH INDEX TEST REPORT

Client: JK Geotechnics
Project: Proposed Medical Centre
Location: Forest Central Business Park, Frenchs Forest, NSW
Ref No: 32505S
Report: A
Report Date: 8/08/2019
Page 2 of 2

BOREHOLE NUMBER	DEPTH m	$I_{S\ (50)}$ MPa	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)
8	10.58 - 10.61	0.1	2
	11.18 - 11.21	0.02	<1
	11.58 - 11.62	0.1	2
9	5.08 - 5.11	0.3	6
	5.55 - 5.58	0.04	1
	5.79 - 5.82	0.1	2
	6.08 - 6.11	0.03	1
	6.50 - 6.53	0.09	2
	6.82 - 6.85	0.04	1
	7.68 - 7.71	0.05	1
	8.10 - 8.15	0.04	1
	8.79 - 8.82	0.09	2
	9.47 - 9.51	0.09	2
	10.04 - 10.08	0.1	2
	10.48 - 10.42	0.02	<1
	10.85 - 10.88	0.06	1
	11.08 - 11.11	0.06	1
	11.85 - 11.88	0.1	2

NOTES:

1. In the above table testing was completed in the Axial direction.
2. The above strength tests were completed at the 'as received' moisture content.
3. Test Method: RMS T223.
4. For reporting purposes, the $I_{S(50)}$ has been rounded to the nearest 0.1MPa, or to one significant figure if less than 0.1MPa
5. The Estimated Unconfined Compressive Strength was calculated from the Point Load Strength Index by the following approximate relationship and rounded off to the nearest whole number :
   U.C.S. = 20 $I_{S\ (50)}$

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SOIL TEST SERVICES
ABN 43 002 145 173

TABLE B
FOUR DAY SOAKED CALIFORNIA BEARING RATIO TEST REPORT

Client: JK Geotechnics
Project: Proposed Medical Centre
Location: Forest Central Business Park, Frenchs Forest, NSW
Ref No: 32505S
Report: B
Report Date: 15/08/2019
Page 1 of 1

BOREHOLE NUMBER	BH 9
DEPTH (m)	0.20 - 1.00
Surcharge (kg)	4.5
Maximum Dry Density (t/m^3)	1.78 STD
Optimum Moisture Content (%)	15.1
Moulded Dry Density (t/m^3)	1.75
Sample Density Ratio (%)	98
Sample Moisture Ratio (%)	101
Moisture Contents	
Insitu (%)	15.8
Moulded (%)	15.2
After soaking and	
After Test, Top 30mm(%)	26.3
Remaining Depth (%)	20.5
Material Retained on 19mm Sieve (%)	0
Swell (%)	4.5
C.B.R. value: @2.5mm penetration	1.5

NOTES: Sampled and supplied by client. Sample tested as received.
- Refer to appropriate Borehole logs for soil descriptions
- Test Methods: AS 1289 6.1.1, 5.1.1 & 2.1.1.
- Date of receipt of sample: 07/08/2019.

15/08/2019

Authorised Signature / Date
(D. Treweek)

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CERTIFICATE OF ANALYSIS 223478

Client Details

Client	JK Geotechnics
Attention	Warren Smith
Address	PO Box 976, North Ryde BC, NSW, 1670

Sample Details

Your Reference	**32505S, Frenchs Forest**
Number of Samples	3 Soil
Date samples received	08/08/2019
Date completed instructions received	08/08/2019

Analysis Details

Please refer to the following pages for results, methodology summary and quality control data.

Samples were analysed as received from the client. Results relate specifically to the samples as received.

Results are reported on a dry weight basis for solids and on an as received basis for other matrices.

Report Details

Date results requested by	15/08/2019
Date of Issue	14/08/2019

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Accredited for compliance with ISO/IEC 17025 - Testing. **Tests not covered by NATA are denoted with ***

Results Approved By
Nancy Zhang, Laboratory Manager, Sydney

Authorised By

Nancy Zhang, Laboratory Manager

ACCREDITED FOR TECHNICAL COMPETENCE

Misc Inorg - Soil				
Our Reference		223478-1	223478-2	223478-3
Your Reference	UNITS	BH2	BH9	BH7
Depth		1.5.195	5.8-6	7.8-8
Date Sampled		01/08/2019	02/08/2019	01/08/2019
Type of sample		Soil	Soil	Soil
Date prepared	-	12/08/2019	12/08/2019	12/08/2019
Date analysed	-	12/08/2019	12/08/2019	12/08/2019
pH 1:5 soil:water	pH Units	5.3	5.6	5.1
Electrical Conductivity 1:5 soil:water	µS/cm	33	30	22
Chloride, Cl 1:5 soil:water	mg/kg	<10	10	10
Sulphate, SO4 1:5 soil:water	mg/kg	34	10	<10
Resistivity in soil*	ohm m	300	330	460

Method ID	Methodology Summary
Inorg-001	pH - Measured using pH meter and electrode in accordance with APHA latest edition, 4500-H+. Please note that the results for water analyses are indicative only, as analysis outside of the APHA storage times.
Inorg-002	Conductivity and Salinity - measured using a conductivity cell at 25°C in accordance with APHA latest edition 2510 and Rayment & Lyons.
Inorg-002	Conductivity and Salinity - measured using a conductivity cell at 25oC in accordance with APHA 22nd ED 2510 and Rayment & Lyons. Resistivity is calculated from Conductivity (non NATA). Resistivity (calculated) may not correlate with results otherwise obtained using Resistivity-Current method, depending on the nature of the soil being analysed.
Inorg-081	Anions - a range of Anions are determined by Ion Chromatography, in accordance with APHA latest edition, 4110-B. Waters samples are filtered on receipt prior to analysis. Alternatively determined by colourimetry/turbidity using Discrete Analyser.

QUALITY CONTROL: Misc Inorg - Soil					Duplicate				Spike Recovery %	
Test Description	Units	PQL	Method	Blank	#	Base	Dup.	RPD	LCS-1	[NT]
Date prepared	-			12/08/2019	1	12/08/2019	12/08/2019		12/08/2019	[NT]
Date analysed	-			12/08/2019	1	12/08/2019	12/08/2019		12/08/2019	[NT]
pH 1:5 soil:water	pH Units		Inorg-001	[NT]	1	5.3	5.3	0	102	[NT]
Electrical Conductivity 1:5 soil:water	µS/cm	1	Inorg-002	<1	1	33	36	9	106	[NT]
Chloride, Cl 1:5 soil:water	mg/kg	10	Inorg-081	<10	1	<10	<10	0	88	[NT]
Sulphate, SO4 1:5 soil:water	mg/kg	10	Inorg-081	<10	1	34	35	3	94	[NT]
Resistivity in soil*	ohm m	1	Inorg-002	<1	1	300	280	7	[NT]	[NT]

Result Definitions

NT	Not tested
NA	Test not required
INS	Insufficient sample for this test
PQL	Practical Quantitation Limit
<	Less than
>	Greater than
RPD	Relative Percent Difference
LCS	Laboratory Control Sample
NS	Not specified
NEPM	National Environmental Protection Measure
NR	Not Reported

Quality Control Definitions

Blank	This is the component of the analytical signal which is not derived from the sample but from reagents, glassware etc, can be determined by processing solvents and reagents in exactly the same manner as for samples.
Duplicate	This is the complete duplicate analysis of a sample from the process batch. If possible, the sample selected should be one where the analyte concentration is easily measurable.
Matrix Spike	A portion of the sample is spiked with a known concentration of target analyte. The purpose of the matrix spike is to monitor the performance of the analytical method used and to determine whether matrix interferences exist.
LCS (Laboratory Control Sample)	This comprises either a standard reference material or a control matrix (such as a blank sand or water) fortified with analytes representative of the analyte class. It is simply a check sample.
Surrogate Spike	Surrogates are known additions to each sample, blank, matrix spike and LCS in a batch, of compounds which are similar to the analyte of interest, however are not expected to be found in real samples.

Australian Drinking Water Guidelines recommend that Thermotolerant Coliform, Faecal Enterococci, & E.Coli levels are less than 1cfu/100mL. The recommended maximums are taken from "Australian Drinking Water Guidelines", published by NHMRC & ARMC 2011.

Laboratory Acceptance Criteria

Duplicate sample and matrix spike recoveries may not be reported on smaller jobs, however, were analysed at a frequency to meet or exceed NEPM requirements. All samples are tested in batches of 20. The duplicate sample RPD and matrix spike recoveries for the batch were within the laboratory acceptance criteria.

Filters, swabs, wipes, tubes and badges will not have duplicate data as the whole sample is generally extracted during sample extraction.

Spikes for Physical and Aggregate Tests are not applicable.

For VOCs in water samples, three vials are required for duplicate or spike analysis.

Duplicates: >10xPQL - RPD acceptance criteria will vary depending on the analytes and the analytical techniques but is typically in the range 20%-50% – see ELN-P05 QA/QC tables for details; <10xPQL - RPD are higher as the results approach PQL and the estimated measurement uncertainty will statistically increase.

Matrix Spikes, LCS and Surrogate recoveries: Generally 70-130% for inorganics/metals; 60-140% for organics (+/-50% surrogates) and 10-140% for labile SVOCs (including labile surrogates), ultra trace organics and speciated phenols is acceptable.

In circumstances where no duplicate and/or sample spike has been reported at 1 in 10 and/or 1 in 20 samples respectively, the sample volume submitted was insufficient in order to satisfy laboratory QA/QC protocols.

When samples are received where certain analytes are outside of recommended technical holding times (THTs), the analysis has proceeded. Where analytes are on the verge of breaching THTs, every effort will be made to analyse within the THT or as soon as practicable.

Where sampling dates are not provided, Envirolab are not in a position to comment on the validity of the analysis where recommended technical holding times may have been breached.

Measurement Uncertainty estimates are available for most tests upon request.

Analysis of aqueous samples typically involves the extraction/digestion and/or analysis of the liquid phase only (i.e. NOT any settled sediment phase but inclusive of suspended particles if present), unless stipulated on the Envirolab COC and/or by correspondence. Notable exceptions include certain Physical Tests (pH/EC/BOD/COD/Apparent Colour etc.), Solids testing, total recoverable metals and PFAS where solids are included by default.

JKGeotechnics

BOREHOLE LOG

Borehole No.
1
1 / 1

Client: ERILYAN
Project: PROPOSED MEDICAL CENTRE
Location: FOREST CENTRAL BUSINESS PARK, FRENCHS FOREST, NSW

Job No.: 32505S **Method:** SPIRAL AUGER **R.L. Surface:** 158.72 m
Date: 1/8/19 **Datum:** AHD
Plant Type: JK250 **Logged/Checked By:** W.S./P.S.

Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
DRY ON COMPLETION	■ ■			■	N = 7 3,4,3	158		▩		FILL: Silty gravelly clay, medium plasticity, dark grey and red brown, fine to medium grained igneous gravel.	w>PL			GRASS COVER APPEARS MODERATELY COMPACTED
	■ ■			■			1	▨	CH	Silty CLAY: high plasticity, orange brown mottled red brown.	w>PL	St		RESIDUAL
				■ ■	N = 18 11,9,9	157 156	2	▨		as above, but light grey.	w~PL	VSt - Hd	350 450 480	
						155 154 153 152	3 4 5 6			END OF BOREHOLE AT 3.00 m				

JKGeotechnics

BOREHOLE LOG

Borehole No.
2
1 / 1

Client: ERILYAN
Project: PROPOSED MEDICAL CENTRE
Location: FOREST CENTRAL BUSINESS PARK, FRENCHS FOREST, NSW

Job No.: 32505S **Method:** SPIRAL AUGER **R.L. Surface:** 159.3 m
Date: 1/8/19 **Datum:** AHD
Plant Type: JK250 **Logged/Checked By:** W.S./P.S.

Groundwater Record	SAMPLES ES	SAMPLES U50	SAMPLES DB	SAMPLES DS	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
DRY ON COMPLETION	■			■		159		▩		FILL: Silty clay, medium plasticity, dark brown, trace of fine to medium grained igneous gravel and ash.	w>PL			GRAVEL COVER
	■			■	N = 9 7,4,5		1	▨	CH	SILTY CLAY: high plasticity, red brown mottled orange brown, trace of fine to medium grained ironstone gravel.	w>PL	VSt - Hd	450 550 540	RESIDUAL
				■		158		▨		SILTY CLAY: high plasticity, yellow brown.				
				■	N = 12 5,6,6	157	2	▨		as above, but light grey.	w~PL		520 550 600	
				■				▨		as above, but light grey and red brown.				
						156 155 154 153	3 4 5 6			END OF BOREHOLE AT 3.00 m				

JKGeotechnics

BOREHOLE LOG

Borehole No.
3
1 / 1

Client: ERILYAN
Project: PROPOSED MEDICAL CENTRE
Location: FOREST CENTRAL BUSINESS PARK, FRENCHS FOREST, NSW

Job No.: 32505S **Method:** SPIRAL AUGER **R.L. Surface:** 160.01 m
Date: 1/8/19 **Datum:** AHD
Plant Type: JK250 **Logged/Checked By:** W.S./P.S.

Groundwater Record	SAMPLES ES	SAMPLES U50	SAMPLES DB	SAMPLES DS	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
DRY ON COMPLETION	■			■				▦		FILL: Silty clay, medium plasticity, dark grey and brown, with fine to coarse grained igneous gravel.	w>PL			GRAVEL COVER
	■			■	N = 7 4,4,3	159	1	▨	CH	Silty CLAY: high plasticity, light grey and red brown, trace of fine to medium grained ironstone gravel.	w>PL	St	300 350 510	RESIDUAL
				■ ■	N = 3 1,2,1	158	2	▨		Silty CLAY: high plasticity, pale grey.	w~PL	F - St	100 150 160	
						157	3			END OF BOREHOLE AT 3.00 m				
						156	4							
						155	5							
						154	6							

JKGeotechnics

BOREHOLE LOG

Borehole No.
4
1 / 1

Client: ERILYAN
Project: PROPOSED MEDICAL CENTRE
Location: FOREST CENTRAL BUSINESS PARK, FRENCHS FOREST, NSW

Job No.: 32505S
Method: SPIRAL AUGER
R.L. Surface: 160.0 m
Date: 1/8/19
Datum: AHD
Plant Type: JK250
Logged/Checked By: W.S./P.S.

Groundwater Record	SAMPLES ES	SAMPLES U50	SAMPLES DB	SAMPLES DS	Field Tests	RL (m AHD)	Depth (m)	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
DRY ON COMPLETION	■								FILL: Silty clay, dark brown, medium plasticity, trace of fine to medium grained igneous gravel.	w>PL			
								CH	Silty CLAY: high plasticity, light grey mottled orange brown and red brown.	w>PL	VSt - Hd		REFUSAL
	■			■	N = 8 4,4,4							450 500 550	
						159	1						
									as above, but mottled light grey and orange brown.	w~PL	Hd		
				■	N = 14 5,6,8							500 560 >600	
						158	2						
				■									
						157	3		END OF BOREHOLE AT 3.00 m				
						156	4						
						155	5						
						154	6						

JKGeotechnics

BOREHOLE LOG

Borehole No.
5
1 / 1

Client: ERILYAN
Project: PROPOSED MEDICAL CENTRE
Location: FOREST CENTRAL BUSINESS PARK, FRENCHS FOREST, NSW

Job No.: 32505S **Method:** SPIRAL AUGER **R.L. Surface:** 159.9 m
Date: 1/8/19 **Datum:** AHD
Plant Type: JK250 **Logged/Checked By:** W.S./P.S.

Groundwater Record	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
DRY ON COMPLETION				▩		FILL: Silty clay, medium plasticity, dark brown, trace of fine to medium grained igneous gravel and ash.				
	N = 16 19,10,6	159	1	▨	CH	SILTY CLAY: high plasticity, light grey mottled orange brown.	w~PL	Hd	>600 >600 >600	RESIDUAL
	N = 11 5,5,6	158	2	▨		as above, but light grey and orange brown.		VSt	480 520 530	
		157	3	▨		as above, but light grey.				
		156 155 154 153	4 5 6			END OF BOREHOLE AT 3.00 m				

Samples: ES, U50, DB, DS

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BOREHOLE LOG

Borehole No.
6
1 / 1

Client: ERILYAN
Project: PROPOSED MEDICAL CENTRE
Location: FOREST CENTRAL BUSINESS PARK, FRENCHS FOREST, NSW

Job No.: 32505S **Method:** SPIRAL AUGER **R.L. Surface:** 159.3 m
Date: 1/8/19 **Datum:** AHD
Plant Type: JK250 **Logged/Checked By:** W.S./P.S.

Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
DRY ON COMPLETION	■			■		159		▩		FILL: Silty clay, medium plasticity, dark brown, with fine to medium grained igneous gravel.	w>PL			
	■			■	N = 11 11,6,5			▨	CH	Silty CLAY: high plasticity, light grey mottled orange brown and red brown, trace of root fibres.	w~PL	VSt	400 350	RESIDUAL
						158	1	▨						
				■	N = 10 4,4,6	157	2	▨				Hd	>600 >600 >600	
				■				▨						
						156	3			END OF BOREHOLE AT 3.00 m				
						155	4							
						154	5							
						153	6							

BOREHOLE LOG

Borehole No.
7
1 / 3

Client: ERILYAN
Project: PROPOSED MEDICAL CENTRE
Location: FOREST CENTRAL BUSINESS PARK, FRENCHS FOREST, NSW

Job No.: 32505S **Method:** SPIRAL AUGER **R.L. Surface:** 159.31 m
Date: 1/8/19 **Datum:** AHD
Plant Type: JK250 **Logged/Checked By:** W.S./P.S.

Groundwater Record	SAMPLES ES	SAMPLES U50	SAMPLES DB	SAMPLES DS	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
			■			159				FILL: Silty clay, medium plasticity, dark brown, trace of fine to medium grained igneous and ironstone gravel, and ash.	w>PL			
				■	N = 4 9,2,2	158	1		CH	Silty CLAY: high plasticity, light grey mottled orange brown and red brown.	w>PL	VSt	300 350	RESIDUAL
				■	N = 32 11,16,16	157	2 3			as above, but light grey.				
ON COMPLETION OF CORING						156 155	4		-	Extremely Weathered siltstone: silty CLAY, high plasticity, light grey.	XW	Hd	300 550 >600	HAWKESBURY SANDSTONE VERY LOW 'TC' BIT RESISTANCE MODERATE RESISTANCE
						154 153	5 6			REFER TO CORED BOREHOLE LOG				GROUNDWATER MONITORING WELL INSTALLED TO 12.0m. CLASS 18 MACHINE SLOTTED 50mm DIA. PVC STANDPIPE 12.0m TO 9.0m. CASING 9.0m TO 0m. 2mm SAND FILTER PACK 12.0m TO 8.0m. BENTONITE SEAL 8.0m TO 0.1m. BACKFILLED WITH SAND TO THE SURFACE. COMPLETED WITH A CONCRETED GATIC COVER.

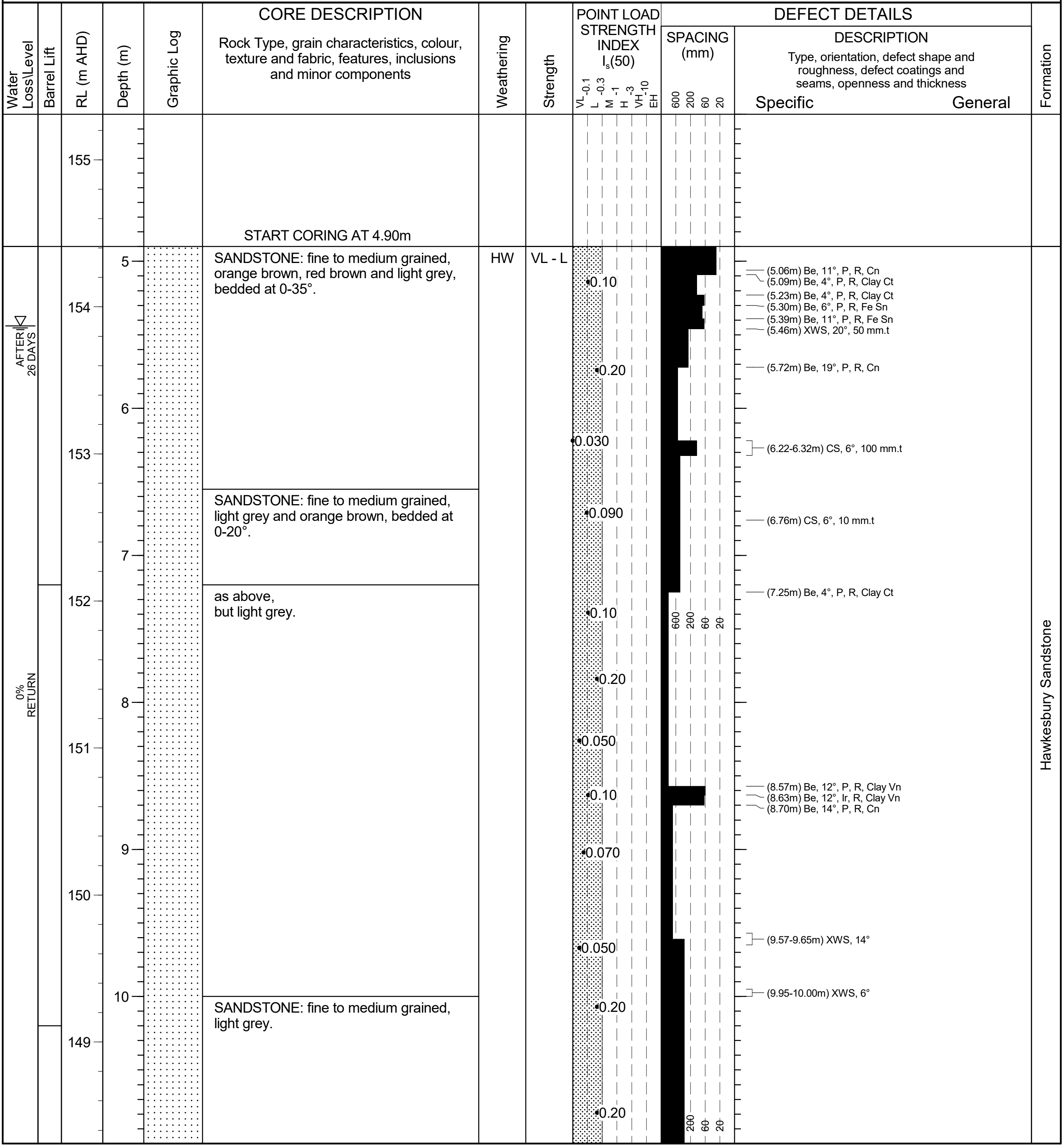
JK 9.02.4 LIB.GLB Log JK AUGERHOLE - MASTER 32505S FRENCHSFOREST.GPJ <<DrawingFile>> 03/09/2019 14:29 10.01.00.01 Datgel Lab and In Situ Tool - DGD | Lib: JK 9.02.4 2019-05-31 Prj: JK 9.01.0 2018-03-20

CORED BOREHOLE LOG

Borehole No.
7
2 / 3

Client: ERILYAN
Project: PROPOSED MEDICAL CENTRE
Location: FOREST CENTRAL BUSINESS PARK, FRENCHS FOREST, NSW

Job No.: 32505S
Core Size: NMLC
R.L. Surface: 159.31 m
Date: 1/8/19
Inclination: VERTICAL
Datum: AHD
Plant Type: JK250
Bearing: N/A
Logged/Checked By: W.S./P.S.



Depth (m)	CORE DESCRIPTION Rock Type, grain characteristics, colour, texture and fabric, features, inclusions and minor components	Weathering	Strength
4.90	START CORING AT 4.90m		
4.90	SANDSTONE: fine to medium grained, orange brown, red brown and light grey, bedded at 0-35°.	HW	VL - L
6.55	SANDSTONE: fine to medium grained, light grey and orange brown, bedded at 0-20°.		
7.20	as above, but light grey.		
10.00	SANDSTONE: fine to medium grained, light grey.		

POINT LOAD STRENGTH INDEX $I_s(50)$: 0.10, 0.20, 0.030, 0.090, 0.10, 0.20, 0.050, 0.10, 0.070, 0.050, 0.20, 0.20

DEFECT DETAILS — DESCRIPTION (Type, orientation, defect shape and roughness, defect coatings and seams, openness and thickness) — Specific:

(5.06m) Be, 11°, P, R, Cn
(5.09m) Be, 4°, P, R, Clay Ct
(5.23m) Be, 4°, P, R, Clay Ct
(5.30m) Be, 6°, P, R, Fe Sn
(5.39m) Be, 11°, P, R, Fe Sn
(5.46m) XWS, 20°, 50 mm.t
(5.72m) Be, 19°, P, R, Cn
(6.22-6.32m) CS, 6°, 100 mm.t
(6.76m) CS, 6°, 10 mm.t
(7.25m) Be, 4°, P, R, Clay Ct
(8.57m) Be, 12°, P, R, Clay Vn
(8.63m) Be, 12°, Ir, R, Clay Vn
(8.70m) Be, 14°, P, R, Cn
(9.57-9.65m) XWS, 14°
(9.95-10.00m) XWS, 6°

General

FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS

CORED BOREHOLE LOG

Borehole No.
7

Client: ERILYAN
Project: PROPOSED MEDICAL CENTRE
Location: FOREST CENTRAL BUSINESS PARK, FRENCHS FOREST, NSW

Job No.: 32505S
Core Size: NMLC
R.L. Surface: 159.31 m
Date: 1/8/19
Inclination: VERTICAL
Datum: AHD
Plant Type: JK250
Bearing: N/A
Logged/Checked By: W.S./P.S.

Water Loss\Level	Barrel Lift	RL (m AHD)	Depth (m)	Graphic Log	CORE DESCRIPTION Rock Type, grain characteristics, colour, texture and fabric, features, inclusions and minor components	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$ VL 0.1, L 0.3, M 1, H 3, VH 10, EH	DEFECT DETAILS SPACING (mm) 600 200 60 20	DEFECT DETAILS DESCRIPTION Specific Type, orientation, defect shape and roughness, defect coatings and seams, openness and thickness	General	Formation
0% RETURN		148			SANDSTONE: fine to medium grained, light grey. *(continued)*	DW	VL	0.080 0.010				
		147 146 145 144 143 142	12 13 14 15 16 17		END OF BOREHOLE AT 12.00 m							

JK 9.02.4 LIB.GLB Log JK CORED BOREHOLE - MASTER 32505S FRENCHSFOREST.GPJ <<DrawingFile>> 03/09/2019 14:29 10.01.00.01 Datgel Lab and In Situ Tool - DGD | Lib: JK 9.02.4 2019-05-31 Prj: JK 9.01.0 2018-03-20


JK Geotechnics
JOB No 32505S BH7 START DEPTH AT 4·90m
4
5
6
7
8
9
10
11
END OF BOREHOLE AT 12.0m

JKGeotechnics

BOREHOLE LOG

Borehole No.
8
1 / 2

Client: ERILYAN
Project: PROPOSED MEDICAL CENTRE
Location: FOREST CENTRAL BUSINESS PARK, FRENCHS FOREST, NSW

Job No.: 32505S **Method:** SPIRAL AUGER **R.L. Surface:** 160.25 m
Date: 2/8/19 **Datum:** AHD
Plant Type: JK250 **Logged/Checked By:** W.S./P.S.

Groundwater Record	ES	U50	DB	DS	Field Tests	RL (m AHD)	Depth (m)	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
DRY ON COMPLETION OF AUGERING			●	●		160			FILL: Silty clayey gravel, fine to medium grained sub-angular igneous gravel, trace of fine to medium grained sand.	M			GRAVEL COVER
	●			●	N = 4 3,2,2	159	1	CL	Silty CLAY: high plasticity, red brown and orange brown, trace of fine to medium grained ironstone gravel and root fibres.	w>PL	St - VSt	150 350 450	RESIDUAL
				●	N = 19 6,8,11	158	2		Silty CLAY: high plasticity, light grey.	w~PL	VSt - Hd	420 550 >600	
				●		157 156	3 4	-	Extremely Weathered siltstone: silty CLAY, high plasticity, light grey.	XW	Hd		HAWKESBURY SANDSTONE
				●			5		SILTSTONE: dark grey.	HW	L		LOW RESISTANCE MODERATE RESISTANCE
						155 154	6		REFER TO CORED BOREHOLE LOG				

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CORED BOREHOLE LOG

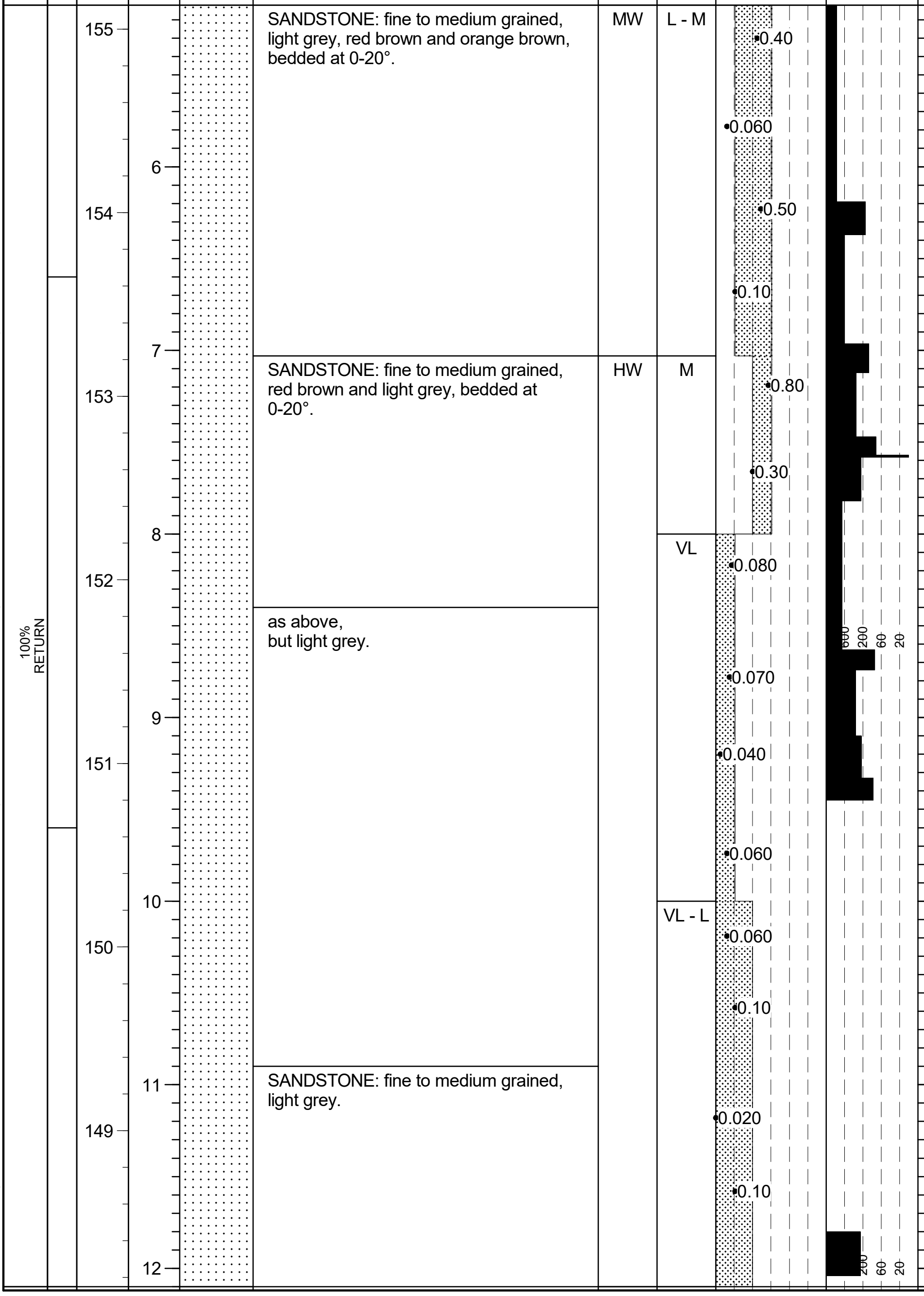
Borehole No.
8
2 / 2

Client: ERILYAN
Project: PROPOSED MEDICAL CENTRE
Location: FOREST CENTRAL BUSINESS PARK, FRENCHS FOREST, NSW

Job No.: 32505S
Core Size: NMLC
R.L. Surface: 160.25 m
Date: 2/8/19
Inclination: VERTICAL
Datum: AHD
Plant Type: JK250
Bearing: N/A
Logged/Checked By: W.S./P.S.

Water Loss\Level | Barrel Lift | RL (m AHD) | Depth (m) | Graphic Log | CORE DESCRIPTION — Rock Type, grain characteristics, colour, texture and fabric, features, inclusions and minor components | Weathering | Strength | POINT LOAD STRENGTH INDEX $I_s(50)$ | DEFECT DETAILS: SPACING (mm); DESCRIPTION — Type, orientation, defect shape and roughness, defect coatings and seams, openness and thickness | Formation

START CORING AT 5.12m



CORE DESCRIPTION	Weathering	Strength
SANDSTONE: fine to medium grained, light grey, red brown and orange brown, bedded at 0-20°.	MW	L - M
SANDSTONE: fine to medium grained, red brown and light grey, bedded at 0-20°.	HW	M
as above, but light grey.		VL
		VL - L
SANDSTONE: fine to medium grained, light grey.		

Specific	General
(6.19m) Jh, 18°, Cn	
(6.26-6.48m) XWS, 10°	
(6.90-7.03m) XWS, 6°	
(7.12m) Be, 8°, P, R, Fe Sn	
(7.47m) Be, 6°, P, R, Clay Ct	
(7.57m) Be, 12°, P, R, Clay Vn	
(7.58m) Be, 12°, P, R, Clay Vn	
(7.82m) Be, 20°, P, R, Clay Vn	
(8.63m) Jh, 41°, Ir, R, Cn	
(8.74m) J, 44°, Ir, R, Cn	
(9.00-9.20m) FRACTURED ZONE	
(9.33m) J, 55°, Ir, R, Cn	
(9.45m) J, 57°, Ir, R, Cn	
(11.80m) Be, 4°, P, S, Clay Ct	
(12.04m) J, 78°, Ir, R, Cn	

Formation: Hawkesbury Sandstone

END OF BOREHOLE AT 12.10 m

FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS


JK Geotechnics
JOB No. 32505S BH8 START DEPTH AT 5·12m
5
6
7
8
9
10
11
12
END OF BOREHOLE AT 12·10m

JKGeotechnics

BOREHOLE LOG

Borehole No.
9
1 / 3

Client: ERILYAN
Project: PROPOSED MEDICAL CENTRE
Location: FOREST CENTRAL BUSINESS PARK, FRENCHS FOREST, NSW

Job No.: 32505S **Method:** SPIRAL AUGER **R.L. Surface:** 159.98 m
Date: 2/8/19 **Datum:** AHD
Plant Type: JK250 **Logged/Checked By:** W.S./P.S.

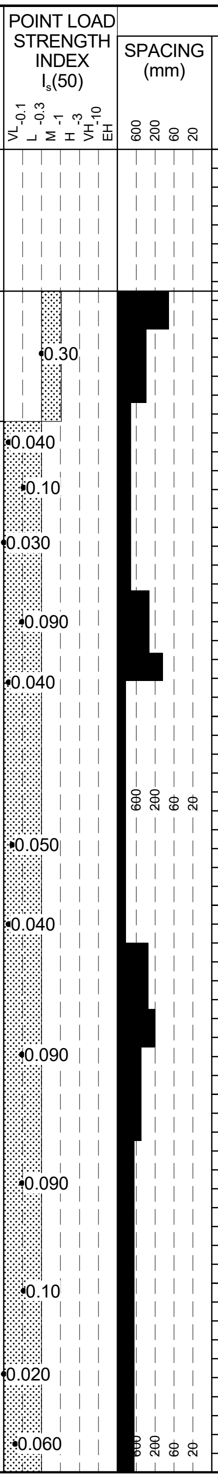
Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
DRY ON COMPLETION OF AUGERING				■						FILL: Silty clay, medium plasticity, dark grey, with fine to medium grained igneous gravel, trace of sand.	w>PL			
			■	■	N = 11 6,6,5	159	1		CH	Silty CLAY: high plasticity, light grey, dark grey and orange brown, trace of fine to medium grained ironstone gravel, ash and root fibres.	w>PL		350 400 500	RESIDUAL
										Silty CLAY: high plasticity, light grey.				
				■	N > 7 16,7/ 10mm REFUSAL				-	Extremely Weathered siltstone: silty CLAY, high plasticity, light grey.	XW	Hd	>600 >600 >600	HAWKESBURY SANDSTONE
				■		158 157 156	2 3 4			SILTSTONE: light grey.	HW	VL - L		VERY LOW 'TC' BIT RESISTANCE
						155 154	5 6			REFER TO CORED BOREHOLE LOG				GROUNDWATER MONITORING WELL INSTALLED TO 12.1m. CLASS 18 MACHINE SLOTTED 50mm DIA. PVC STANDPIPE 12.1m TO 9.1m. CASING 9.1m TO 0m. 2mm SAND FILTER PACK 12.1m TO 8.0m. BENTONITE SEAL 8.0m TO 0.1m. BACKFILLED WITH SAND TO THE SURFACE. COMPLETED WITH A CONCRETED GATIC COVER.

CORED BOREHOLE LOG

Borehole No.
9
2 / 3

Client: ERILYAN
Project: PROPOSED MEDICAL CENTRE
Location: FOREST CENTRAL BUSINESS PARK, FRENCHS FOREST, NSW

Job No.: 32505S | **Core Size:** NMLC | **R.L. Surface:** 159.98 m
Date: 2/8/19 | **Inclination:** VERTICAL | **Datum:** AHD
Plant Type: JK250 | **Bearing:** N/A | **Logged/Checked By:** W.S./P.S.



Water Loss\Level	Barrel Lift	RL (m AHD)	Depth (m)	Graphic Log	CORE DESCRIPTION Rock Type, grain characteristics, colour, texture and fabric, features, inclusions and minor components	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$	DEFECT DETAILS – DESCRIPTION Type, orientation, defect shape and roughness, defect coatings and seams, openness and thickness – Specific / General	Formation
					START CORING AT 4.75m					
100% RETURN		155	5		SANDSTONE: fine to medium grained, light grey and orange brown, bedded at 0-25°.	MW	M	0.30	(4.85m) Be, 24°, P, R, Clay Vn (4.95m) Be, 8°, P, S, Clay Vn (5.24-5.44m) XWS, 6°	Hawkesbury Sandstone
		154	6			HW	VL - L	0.040 0.10 0.030 0.090 0.040	(6.29-6.38m) XWS, 7° (6.60-6.73m) XWS, 0° (6.81m) Be, 16°, Ir, R, Fe Sn	
		153	7					0.050		
		152	8		as above, but light grey and red brown.			0.040	(8.20m) Be, 14°, P, R, Clay Vn	
					SANDSTONE: fine to medium grained, light grey, bedded at 0-20°.			0.090	(8.55m) Be, 9°, C, R, Cn (8.75m) Be, 14°, P, R, Clay Vn	
		151	9					0.090 0.10	(9.18-9.31m) XWS, 18° (9.97-10.05m) XWS, 16°	
		150	10		SANDSTONE: fine to medium grained, light grey.			0.020 0.060		

JK 9.02.4 LIB.GLB Log JK CORED BOREHOLE - MASTER 32505S FRENCHSFOREST.GPJ <<DrawingFile>> 03/09/2019 14:29 10.01.00.01 Datgel Lab and In Situ Tool - DGD | Lib: JK 9.02.4 2019-05-31 Prj: JK 9.01.0 2018-03-20

FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS

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CORED BOREHOLE LOG

Borehole No.
9
3 / 3

Client: ERILYAN
Project: PROPOSED MEDICAL CENTRE
Location: FOREST CENTRAL BUSINESS PARK, FRENCHS FOREST, NSW

Job No.: 32505S **Core Size:** NMLC **R.L. Surface:** 159.98 m
Date: 2/8/19 **Inclination:** VERTICAL **Datum:** AHD
Plant Type: JK250 **Bearing:** N/A **Logged/Checked By:** W.S./P.S.

Water Loss\Level	Barrel Lift	RL (m AHD)	Depth (m)	Graphic Log	CORE DESCRIPTION Rock Type, grain characteristics, colour, texture and fabric, features, inclusions and minor components	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$ VL $_{-0.1}$ L $_{-0.3}$ M $_{-1}$ H $_{-3}$ VH $_{-10}$ EH	DEFECT DETAILS SPACING (mm) 600 200 60 20	DEFECT DETAILS DESCRIPTION Type, orientation, defect shape and roughness, defect coatings and seams, openness and thickness Specific General	Formation
100% RETURN		148	12		SANDSTONE: fine to medium grained, light grey. as above, but light grey and red brown.	HW	VL - L	0.060 0.10			Hawkesbury Sandstone
		147 146 145 144 143	13 14 15 16 17		END OF BOREHOLE AT 12.10 m				600 200 60 20 600 200 60 20		

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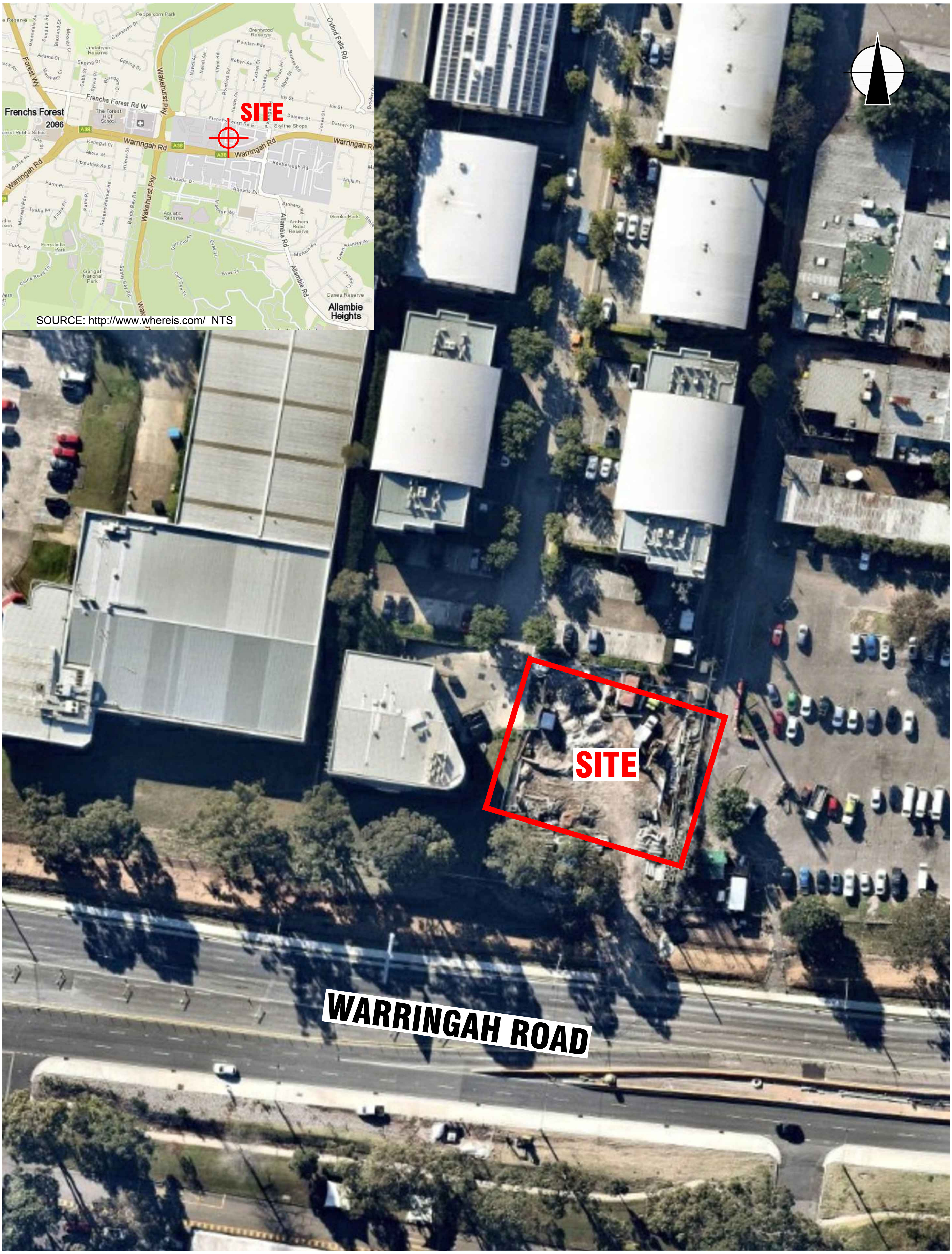
FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS


JK Geotechnics
JOB No. 32505S BH9 · START DEPTH AT 4.75m
4
5
6
7
8
9
10
11
12
END OF BOREHOLE AT 12.10m



SOURCE: http://www.whereis.com/ NTS

AERIAL IMAGE SOURCE: MAPS.AU.NEARMAP.COM

0 10 20 30 40 50

SCALE 1:1000 @A4 METRES

This plan should be read in conjunction with the JK Geotechnics report.

Title:

SITE LOCATION PLAN

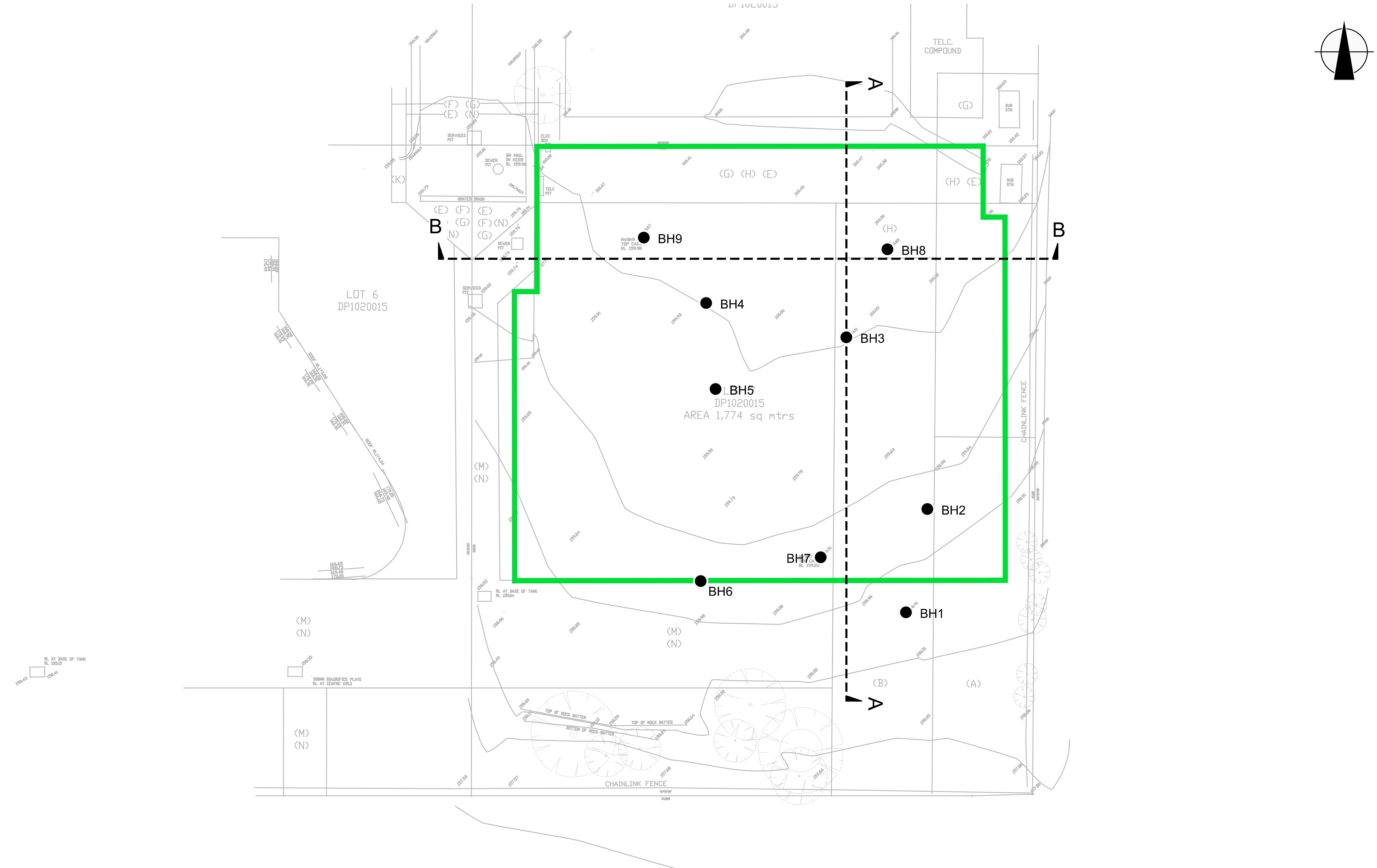
Location: FRENCHS FOREST BUSINESS PARK
FRENCHS FOREST, NSW

Report No: 32505S

Figure: 1

JKGeotechnics

PLOT DATE: 20/08/2019 3:18:10 PM DWG FILE: S:\6 GEOTECHNICAL\6F GEOTECHNICAL JOBS\32000'S\32505S FRENCHS FOREST\CAD\32505S.DWG



LEGEND:

🟩 OUTLINE OF PROPOSED BASEMENT 3

0 2.5 5 7.5 10 12.5

SCALE 1:250 @A3 METRES

This plan should be read in conjunction with the JK Geotechnics report.

Title:
BOREHOLE LOCATION PLAN

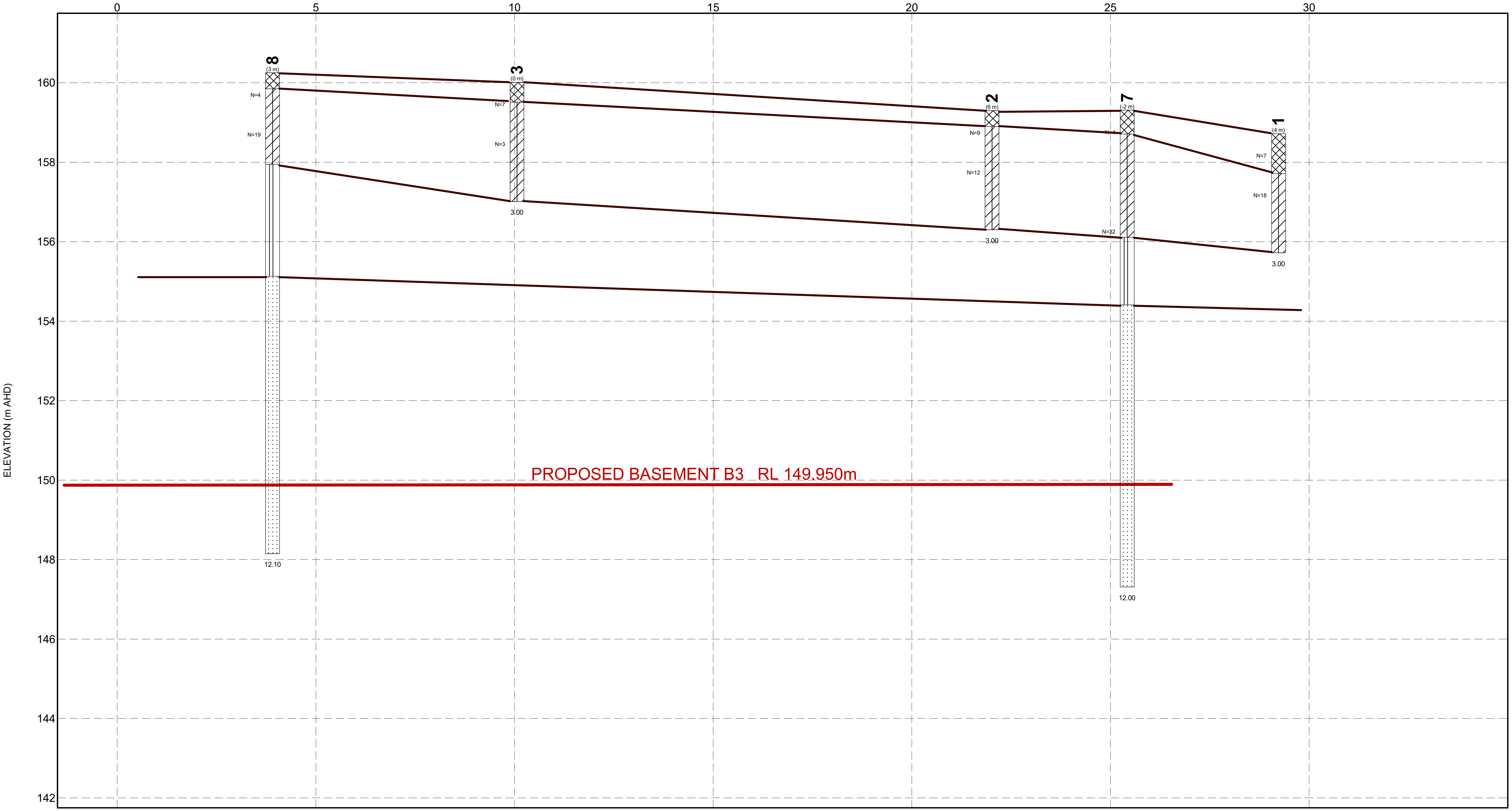
Location: FRENCHS FOREST BUSINESS PARK
FRENCHS FOREST, NSW

Report No: 32505S

Figure: 2

JKGeotechnics

PLOT DATE: 30/08/2019 7:48:27 AM DWG FILE: S:\6 GEOTECHNICAL\6F GEOTECHNICAL JOBS\32000'S\32505S FRENCHS FOREST\CAD\32505S.DWG



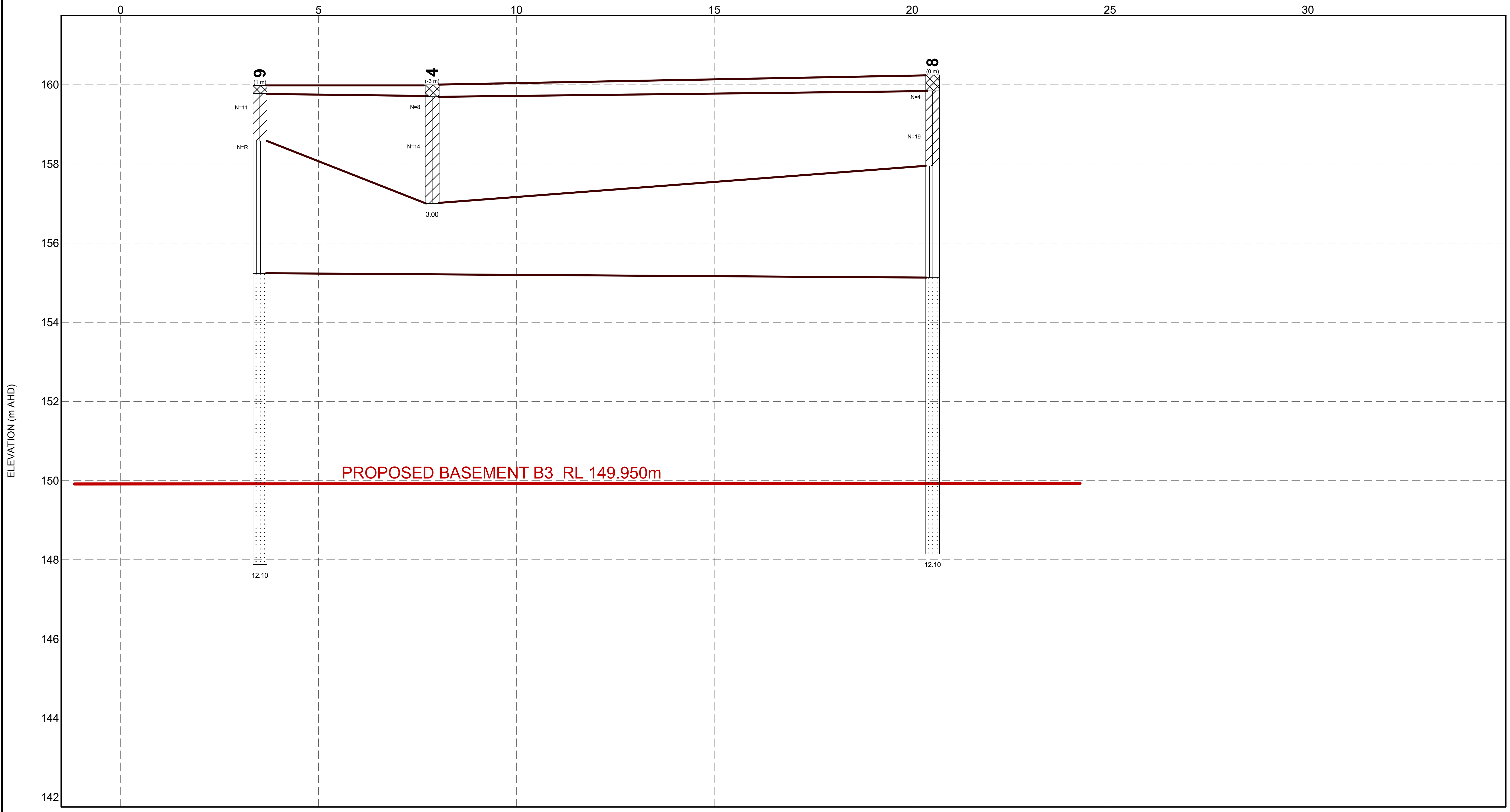
MATERIAL GRAPHIC

- ▨ SILTY CLAY (CL, CI, CH)
- ▥ SILTSTONE
- ☒ FILL
- ⛆ SANDSTONE

GRAPHICAL BOREHOLE SUMMARY
ERILYAN
FOREST CENTRAL BUSINESS PARK,
FRENCHS FOREST, NSW
PROPOSED MEDICAL CENTRE

DRAWN D.M.		DATE 28/08/2019	
CHECKED M.P.		DATE 28/08/2019	
SCALE H 1:100 V 1:100			A3
PROJECT No 32505S		FIGURE No 3	

JK 9.02.4 LIB.GLB Fence FENCE A3L 32505S FRENCHSFOREST.GPJ 32505S FIG 3.GDW 28/08/2019 11:46 10.01.00.01 Datgel Lab and In Situ Tool - DGD | Lib: JK 9.02.4 2019-05-31 Prj: JK 9.01.0 2018-03-20



MATERIAL GRAPHIC

▨ SILTY CLAY (CL, CI, CH)
▥ SILTSTONE
▩ FILL
⬚ SANDSTONE

GRAPHICAL BOREHOLE SUMMARY
ERILYAN
FOREST CENTRAL BUSINESS PARK,
FRENCHS FOREST, NSW
PROPOSED MEDICAL CENTRE

DRAWN D.M.	DATE 28/08/2019		
CHECKED M.P.	DATE 28/08/2019		
SCALE H 1:100 V 1:100		A3	
PROJECT No 32505S	FIGURE No 4		

JK 9.02.4 LIB.GLB Fence FENCE A3L 32505S FRENCHSFOREST.GPJ 32505S FIG 4.GDW 28/08/2019 11:47 10.01.00.01 Datgel Lab and In Situ Tool - DGD | Lib: JK 9.02.4 2019-05-31 Prj: JK 9.01.0 2018-03-20

VIBRATION EMISSION DESIGN GOALS

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

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

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

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

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

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

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

REPORT EXPLANATION NOTES

INTRODUCTION

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

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

DESCRIPTION AND CLASSIFICATION METHODS

The methods of description and classification of soils and rocks used in this report are based on Australian Standard 1726:2017 *'Geotechnical Site Investigations'*. 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 soil classification table qualified by the grading of other particles present (eg. sandy clay) as set out below:

Soil Classification	Particle Size
Clay	< 0.002mm
Silt	0.002 to 0.075mm
Sand	0.075 to 2.36mm
Gravel	2.36 to 63mm
Cobbles	63 to 200mm
Boulders	> 200mm

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 (VL)	< 4
Loose (L)	4 to 10
Medium dense (MD)	10 to 30
Dense (D)	30 to 50
Very Dense (VD)	> 50

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

Classification	Unconfined Compressive Strength (kPa)	Indicative Undrained Shear Strength (kPa)
Very Soft (VS)	≤ 25	≤ 12
Soft (S)	> 25 and ≤ 50	> 12 and ≤ 25
Firm (F)	> 50 and ≤ 100	> 25 and ≤ 50
Stiff (St)	> 100 and ≤ 200	> 50 and ≤ 100
Very Stiff (VSt)	> 200 and ≤ 400	> 100 and ≤ 200
Hard (Hd)	> 400	> 200
Friable (Fr)	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 fissile mudstone, with a weakness parallel to bedding. Rocks with alternating inter-laminations of different grain size (eg. siltstone/claystone and siltstone/fine grained sandstone) is referred to as 'laminite'.

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 shrink-swell behaviour, 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 methods except test pits, hand auger drilling and portable Dynamic Cone Penetrometers require the use of a mechanical rig which is commonly mounted on a truck chassis or track base.

Test Pits: These are normally excavated with a backhoe or a tracked excavator, allowing close examination of the insitu soils and 'weaker' bedrock 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 a large 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. Refusal of the hand auger can occur on a variety of materials such as obstructions within any fill, tree roots, hard clay, gravel or ironstone, cobbles and boulders, 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 limited 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 cuttings. 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 assessed 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. 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, NMLC or HQ triple tube core barrels, which give a core of about 50mm and 61mm diameter, respectively, 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 NO CORE. The location of NO CORE recovery is determined on site by the supervising engineer; where the location is uncertain, the loss is placed at the bottom 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 1289.6.3.1–2004 (R2016) *'Methods of Testing Soils for Engineering Purposes, Soil Strength and Consolidation Tests – Determination of the Penetration Resistance of a Soil – Standard Penetration Test (SPT)'*.

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 63.5kg 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.

A modification to the SPT 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.

Cone Penetrometer Testing (CPT) and Interpretation: The cone penetrometer is sometimes referred to as a Dutch Cone. The test is described in Australian Standard 1289.6.5.1–1999 (R2013) *'Methods of Testing Soils for Engineering Purposes, Soil Strength and Consolidation Tests – Determination of the Static Cone Penetration Resistance of a Soil – Field Test using a Mechanical and Electrical Cone or Friction-Cone Penetrometer'*.

In the tests, a 35mm or 44mm 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 a hydraulic ram system. Measurements are made of the end bearing resistance on the cone and the frictional resistance on a separate 134mm or 165mm 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. The CPT does not provide soil sample recovery.

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. There are two scales presented for the cone resistance. The lower scale has a range of 0 to 5MPa and the main scale has a range of 0 to 50MPa. For cone resistance values less than 5MPa, the plot will appear on both scales.
- 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 CPT and SPT values can be developed for both sands and clays but may be site specific.

Interpretation of CPT 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.

There are limitations when using the CPT in that it may not penetrate obstructions within any fill, thick layers of hard clay and very dense sand, gravel and weathered bedrock. Normally a 'dummy' cone is pushed through fill to protect the equipment. No information is recorded by the 'dummy' probe.

Flat Dilatometer Test: The flat dilatometer (DMT), also known as the Marchetti Dilometer comprises a stainless steel blade having a flat, circular steel membrane mounted flush on one side.

The blade is connected to a control unit at ground surface by a pneumatic-electrical tube running through the insertion rods. A gas tank, connected to the control unit by a pneumatic cable, supplies the gas pressure required to expand the membrane. The control unit is equipped with a pressure regulator, pressure gauges, an audio-visual signal and vent valves.

The blade is advanced into the ground using our CPT rig or one of our drilling rigs, and can be driven into the ground using an SPT hammer. As soon as the blade is in place, the membrane is inflated, and the pressure required to lift the membrane (approximately 0.1mm) is recorded. The pressure then required to lift the centre of the membrane by an additional 1mm is recorded. The membrane is then deflated before pushing to the next depth increment, usually 200mm down. The pressure readings are corrected for membrane stiffness.

The DMT is used to measure material index (I_D), horizontal stress index (K_D), and dilatometer modulus (E_D). Using established correlations, the DMT results can also be used to assess the 'at rest' earth pressure coefficient (K_o), over-consolidation ratio (OCR), undrained shear strength (C_u), friction angle (ϕ), coefficient of consolidation (C_h), coefficient of permeability (K_h), unit weight (γ), and vertical drained constrained modulus (M).

The seismic dilatometer (SDMT) is the combination of the DMT with an add-on seismic module for the measurement of shear wave velocity (V_s). Using established correlations, the SDMT results can also be used to assess the small strain modulus (G_o).

Portable Dynamic Cone Penetrometers: Portable Dynamic Cone Penetrometer (DCP) tests are carried out by driving a 16mm diameter rod with a 20mm diameter cone end with a 9kg hammer dropping 510mm. The test is described in Australian Standard 1289.6.3.2–1997 (R2013) *'Methods of Testing Soils for Engineering Purposes, Soil Strength and Consolidation Tests – Determination of the Penetration Resistance of a Soil – 9kg Dynamic Cone Penetrometer Test'*.

The results are used to assess the relative compaction of fill, the relative density of granular soils, and the strength of cohesive soils. Using established correlations, the DCP test results can also be used to assess California Bearing Ratio (CBR).

Refusal of the DCP can occur on a variety of materials such as obstructions within any fill, tree roots, hard clay, gravel or ironstone, cobbles and boulders, and does not necessarily indicate rock level.

Vane Shear Test: The vane shear test is used to measure the undrained shear strength (C_u) of typically very soft to firm fine grained cohesive soils. The vane shear is normally performed in the bottom of a borehole, but can be completed from surface level, the bottom and sides of test pits, and on recovered undisturbed tube samples (when using a hand vane).

The vane comprises four rectangular blades arranged in the form of a cross on the end of a thin rod, which is coupled to the bottom of a drill rod string when used in a borehole. The size of the vane is dependent on the strength of the fine grained cohesive soils; that is, larger vanes are normally used for very low strength soils. For borehole testing, the size of the vane can be limited by the size of the casing that is used.

For testing inside a borehole, a device is used at the top of the casing, which suspends the vane and rods so that they do not sink under self-weight into the 'soft' soils beyond the depth at which the test is to be carried out. A calibrated torque head is used to rotate the rods and vane and to measure the resistance of the vane to rotation.

With the vane in position, torque is applied to cause rotation of the vane at a constant rate. A rate of 6° per minute is the common rotation rate. Rotation is continued until the soil is sheared and the maximum torque has been recorded. This value is then used to calculate the undrained shear strength. The vane is then rotated rapidly a number of times and the operation repeated until a constant torque reading is obtained. This torque value is used to calculate the remoulded shear strength. Where appropriate, friction on the vane rods is measured and taken into account in the shear strength calculation.

LOGS

The borehole or test pit logs presented herein are an engineering and/or geological interpretation of the subsurface 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 terms and symbols used in preparation of the logs are defined in the following pages.

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 reliable water observations are to be made.

More reliable measurements can be made by installing standpipes which are read after the groundwater level has stabilised 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 assess 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 Soils for Engineering Purposes'* or appropriate NSW Government Roads & Maritime Services (RMS) test methods. 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.

Reasonable 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.
- Details of the development that the Company could not reasonably be expected to anticipate.

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 rather than at some later stage, well after the event.

REPRODUCTION OF INFORMATION FOR CONTRACTUAL PURPOSES

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. Licence to use the documents may be revoked without notice if the Client is in breach of any obligation 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 an experienced geotechnical engineer/engineering geologist.

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 and appropriate footing or pile founding depths, or

iii) full time engineering presence on site.

SYMBOL LEGENDS

SOIL

▩	FILL
≋	TOPSOIL
▨	CLAY (CL, CI, CH)
▥	SILT (ML, MH)
░	SAND (SP, SW)
◌	GRAVEL (GP, GW)
▨	SANDY CLAY (CL, CI, CH)
▨	SILTY CLAY (CL, CI, CH)
▨	CLAYEY SAND (SC)
░	SILTY SAND (SM)
▨	GRAVELLY CLAY (CL, CI, CH)
▨	CLAYEY GRAVEL (GC)
▥	SANDY SILT (ML, MH)
⁂	PEAT AND HIGHLY ORGANIC SOILS (Pt)

ROCK

◌	CONGLOMERATE
░	SANDSTONE
▤	SHALE/MUDSTONE
▥	SILTSTONE
▤	CLAYSTONE
■	COAL
▥	LAMINITE
▦	LIMESTONE
≋	PHYLLITE, SCHIST
▤	TUFF
⁘	GRANITE, GABBRO
+	DOLERITE, DIORITE
∨	BASALT, ANDESITE
≈	QUARTZITE

OTHER MATERIALS

▦	BRICKS OR PAVERS
░	CONCRETE
■	ASPHALTIC CONCRETE

CLASSIFICATION OF COARSE AND FINE GRAINED SOILS

Major Divisions		Group Symbol	Typical Names	Field Classification of Sand and Gravel	Laboratory Classification	
Coarse grained soil (more than 65% of soil excluding oversize fraction is greater than 0.075mm)	GRAVEL (more than half of coarse fraction is larger than 2.36mm	GW	Gravel and gravel-sand mixtures, little or no fines	Wide range in grain size and substantial amounts of all intermediate sizes, not enough fines to bind coarse grains, no dry strength	≤ 5% fines	$C_u > 4$ $1 < C_c < 3$
		GP	Gravel and gravel-sand mixtures, little or no fines, uniform gravels	Predominantly one size or range of sizes with some intermediate sizes missing, not enough fines to bind coarse grains, no dry strength	≤ 5% fines	Fails to comply with above
		GM	Gravel-silt mixtures and gravel-sand-silt mixtures	'Dirty' materials with excess of non-plastic fines, zero to medium dry strength	≥ 12% fines, fines are silty	Fines behave as silt
		GC	Gravel-clay mixtures and gravel-sand-clay mixtures	'Dirty' materials with excess of plastic fines, medium to high dry strength	≥ 12% fines, fines are clayey	Fines behave as clay
	SAND (more than half of coarse fraction is smaller than 2.36mm)	SW	Sand and gravel-sand mixtures, little or no fines	Wide range in grain size and substantial amounts of all intermediate sizes, not enough fines to bind coarse grains, no dry strength	≤ 5% fines	$C_u > 6$ $1 < C_c < 3$
		SP	Sand and gravel-sand mixtures, little or no fines	Predominantly one size or range of sizes with some intermediate sizes missing, not enough fines to bind coarse grains, no dry strength	≤ 5% fines	Fails to comply with above
		SM	Sand-silt mixtures	'Dirty' materials with excess of non-plastic fines, zero to medium dry strength	≥ 12% fines, fines are silty	N/A
		SC	Sand-clay mixtures	'Dirty' materials with excess of plastic fines, medium to high dry strength	≥ 12% fines, fines are clayey	

Major Divisions		Group Symbol	Typical Names	Field Classification of Silt and Clay			Laboratory Classification
				Dry Strength	Dilatancy	Toughness	% < 0.075mm
ine grained soils (more than 35% of soil excluding oversize fraction is less than 0.075mm)	SILT and CLAY (low to medium plasticity)	ML	Inorganic silt and very fine sand, rock flour, silty or clayey fine sand or silt with low plasticity	None to low	Slow to rapid	Low	Below A line
		CL, CI	Inorganic clay of low to medium plasticity, gravelly clay, sandy clay	Medium to high	None to slow	Medium	Above A line
		OL	Organic silt	Low to medium	Slow	Low	Below A line
	SILT and CLAY (high plasticity)	MH	Inorganic silt	Low to medium	None to slow	Low to medium	Below A line
		CH	Inorganic clay of high plasticity	High to very high	None	High	Above A line
		OH	Organic clay of medium to high plasticity, organic silt	Medium to high	None to very slow	Low to medium	Below A line
	Highly organic soil	Pt	Peat, highly organic soil	–	–	–	–

Laboratory Classification Criteria

A well graded coarse grained soil is one for which the coefficient of uniformity $Cu > 4$ and the coefficient of curvature $1 < C_c < 3$. Otherwise, the soil is poorly graded. These coefficients are given by:

$$C_U = \frac{D_{60}}{D_{10}} \qquad and \qquad C_C = \frac{(D_{30})^2}{D_{10}\,D_{60}}$$

Where D_{10}, D_{30} and D_{60} are those grain sizes for which 10%, 30% and 60% of the soil grains, respectively, are smaller.

NOTES:

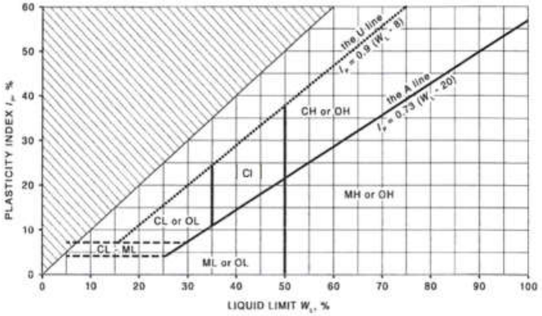
1 For a coarse grained soil with a fines content between 5% and 12%, the soil is given a dual classification comprising the two group symbols separated by a dash; for example, for a poorly graded gravel with between 5% and 12% silt fines, the classification is GP-GM.

2 Where the grading is determined from laboratory tests, it is defined by coefficients of curvature (C_c) and uniformity (C_u) derived from the particle size distribution curve.

3 Clay soils with liquid limits > 35% and ≤ 50% may be classified as being of medium plasticity.

4 The U line on the Modified Casagrande Chart is an approximate upper bound for most natural soils.

Modified Casagrande Chart for Classifying Silts and Clays according to their Behaviour



LOG SYMBOLS

Log Column	**Symbol**	**Definition**		
Groundwater Record	▼	Standing water level. Time delay following completion of drilling/excavation may be shown.		
	—C—	Extent of borehole/test pit collapse shortly after drilling/excavation.		
	▶—	Groundwater seepage into borehole or test pit noted during drilling or excavation.		
Samples	ES	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 analysis.		
	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. 'Refusal' refers to apparent hammer refusal within the corresponding 150mm depth increment.		
	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° 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 (Fine Grained Soils)	*w* > PL	Moisture content estimated to be greater than plastic limit.		
	w ≈ PL	Moisture content estimated to be approximately equal to plastic limit.		
	w < PL	Moisture content estimated to be less than plastic limit.		
	w ≈ LL	Moisture content estimated to be near liquid limit.		
	w > LL	Moisture content estimated to be wet of liquid limit.		
(Coarse Grained 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 ≤ 25kPa.		
	S	SOFT – unconfined compressive strength > 25kPa and ≤ 50kPa.		
	F	FIRM – unconfined compressive strength > 50kPa and ≤ 100kPa.		
	St	STIFF – unconfined compressive strength > 100kPa and ≤ 200kPa.		
	VSt	VERY STIFF – unconfined compressive strength > 200kPa and ≤ 400kPa.		
	Hd	HARD – unconfined compressive strength > 400kPa.		
	Fr	FRIABLE – strength not attainable, soil crumbles.		
	( )	Bracketed symbol indicates estimated consistency based on tactile examination or other assessment.		
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 and ≤ 35	4 – 10
	MD	MEDIUM DENSE	> 35 and ≤ 65	10 – 30
	D	DENSE	> 65 and ≤ 85	30 – 50
	VD	VERY DENSE	> 85	> 50
	( )	Bracketed symbol indicates estimated density based on ease of drilling or other assessment.		
Hand Penetrometer Readings	300 250	Measures reading in kPa of unconfined compressive strength. Numbers indicate individual test results on representative undisturbed material unless noted otherwise.		

Log Column	Symbol	Definition
Remarks	'V' bit	Hardened steel 'V' shaped bit.
	'TC' bit	Twin pronged tungsten carbide bit.
	T_{60}	Penetration of auger string in mm under static load of rig applied by drill head hydraulics without rotation of augers.
	Soil Origin	The geological origin of the soil can generally be described as:
		RESIDUAL – soil formed directly from insitu weathering of the underlying rock. No visible structure or fabric of the parent rock.
		EXTREMELY WEATHERED – soil formed directly from insitu weathering of the underlying rock. Material is of soil strength but retains the structure and/or fabric of the parent rock.
		ALLUVIAL – soil deposited by creeks and rivers.
		ESTUARINE – soil deposited in coastal estuaries, including sediments caused by inflowing creeks and rivers, and tidal currents.
		MARINE – soil deposited in a marine environment.
		AEOLIAN – soil carried and deposited by wind.
		COLLUVIAL – soil and rock debris transported downslope by gravity, with or without the assistance of flowing water. Colluvium is usually a thick deposit formed from a landslide. The description 'slopewash' is used for thinner surficial deposits.
		LITTORAL – beach deposited soil.

Classification of Material Weathering

<table>
<tr><th colspan="2">Term</th><th colspan="2">Abbreviation</th><th>Definition</th></tr>
<tr><td colspan="2">Residual Soil</td><td colspan="2">RS</td><td>Material is weathered to such an extent that it has soil properties. Mass structure and material texture and fabric of original rock are no longer visible, but the soil has not been significantly transported.</td></tr>
<tr><td colspan="2">Extremely Weathered</td><td colspan="2">XW</td><td>Material is weathered to such an extent that it has soil properties. Mass structure and material texture and fabric of original rock are still visible.</td></tr>
<tr><td>Highly Weathered</td><td rowspan="2">Distinctly Weathered (Note 1)</td><td>HW</td><td rowspan="2">DW</td><td>The whole of the rock material is discoloured, usually by iron staining or bleaching to the extent that the colour of the original rock is not recognisable. Rock strength is significantly changed by weathering. Some primary minerals have weathered to clay minerals. Porosity may be increased by leaching, or may be decreased due to deposition of weathering products in pores.</td></tr>
<tr><td>Moderately Weathered</td><td>MW</td><td>The whole of the rock material is discoloured, usually by iron staining or bleaching to the extent that the colour of the original rock is not recognisable, but shows little or no change of strength from fresh rock.</td></tr>
<tr><td colspan="2">Slightly Weathered</td><td colspan="2">SW</td><td>Rock is partially discoloured with staining or bleaching along joints but shows little or no change of strength from fresh rock.</td></tr>
<tr><td colspan="2">Fresh</td><td colspan="2">FR</td><td>Rock shows no sign of decomposition of individual minerals or colour changes.</td></tr>
</table>

NOTE 1: The term 'Distinctly Weathered' is used where it is not practicable to distinguish between 'Highly Weathered' and 'Moderately Weathered' rock. 'Distinctly Weathered' is defined as follows: *'Rock strength usually changed by weathering. The rock may be highly discoloured, usually by iron staining. Porosity may be increased by leaching, or may be decreased due to deposition of weathering products in pores'*. There is some change in rock strength.

Rock Material Strength Classification

			Guide to Strength	
Term	**Abbreviation**	**Uniaxial Compressive Strength (MPa)**	**Point Load Strength Index $Is_{(50)}$ (MPa)**	**Field Assessment**
Very Low Strength	VL	0.6 to 2	0.03 to 0.1	Material crumbles under firm blows with sharp end of pick; can be peeled with knife; too hard to cut a triaxial sample by hand. Pieces up to 30mm thick can be broken by finger pressure.
Low Strength	L	2 to 6	0.1 to 0.3	Easily scored with a knife; indentations 1mm to 3mm show in the specimen with firm blows of the pick point; has dull sound under hammer. A piece of core 150mm long by 50mm diameter may be broken by hand. Sharp edges of core may be friable and break during handling.
Medium Strength	M	6 to 20	0.3 to 1	Scored with a knife; a piece of core 150mm long by 50mm diameter can be broken by hand with difficulty.
High Strength	H	20 to 60	1 to 3	A piece of core 150mm long by 50mm diameter cannot be broken by hand but can be broken by a pick with a single firm blow; rock rings under hammer.
Very High Strength	VH	60 to 200	3 to 10	Hand specimen breaks with pick after more than one blow; rock rings under hammer.
Extremely High Strength	EH	> 200	> 10	Specimen requires many blows with geological pick to break through intact material; rock rings under hammer.

Abbreviations Used in Defect Description

Cored Borehole Log Column		Symbol Abbreviation	Description
Point Load Strength Index		• 0.6	Axial point load strength index test result (MPa)
		x 0.6	Diametral point load strength index test result (MPa)
Defect Details	– Type	Be	Parting – bedding or cleavage
		CS	Clay seam
		Cr	Crushed/sheared seam or zone
		J	Joint
		Jh	Healed joint
		Ji	Incipient joint
		XWS	Extremely weathered seam
	– Orientation	Degrees	Defect orientation is measured relative to normal to the core axis (ie. relative to the horizontal for a vertical borehole)
	– Shape	P	Planar
		C	Curved
		Un	Undulating
		St	Stepped
		Ir	Irregular
	– Roughness	Vr	Very rough
		R	Rough
		S	Smooth
		Po	Polished
		Sl	Slickensided
	– Infill Material	Ca	Calcite
		Cb	Carbonaceous
		Clay	Clay
		Fe	Iron
		Qz	Quartz
		Py	Pyrite
	– Coatings	Cn	Clean
		Sn	Stained – no visible coating, surface is discoloured
		Vn	Veneer – visible, too thin to measure, may be patchy
		Ct	Coating ≤ 1mm thick
		Filled	Coating > 1mm thick
	– Thickness	mm.t	Defect thickness measured in millimetres