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Interim Report on Geotechnical Investigation and Acid Sulfate Soil Assessment

TAFE CLC Tranche 3
Bayshore Drive, Byron Bay

Prepared for
Brewster Hjorth Architects

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1. Introduction

This interim report presents the results of an additional geotechnical investigation and acid sulfate soil (ASS) assessment undertaken for a proposed TAFE connected learning centre (CLC) at Bayshore Drive, Byron Bay. The investigation was commissioned in an email dated 10 February 2021 by Nic Glass of Brewster Hjorth Architects and was undertaken in accordance with Douglas Partners Pty Ltd's (DP's) proposal 201414.P.001.Rev0 dated 9 February 2021 DP's standard "Conditions of Engagement".

DP has previously carried out an investigation within the eastern portion of the site and as detailed in the report DP (2020). It is understood that the location of the proposed TAFE CLC has moved toward the western portion of the site, as such, an additional geotechnical investigation was required. Details on structural loads and earthworks levels were not known at the time of preparation of this interim report.

The aim of the investigation was to assess the subsurface soil and groundwater conditions across the site in order to provide comment on the following:

- subsurface conditions, including groundwater (if encountered);
- site classification in accordance with AS 2870 (2011);
- suitable foundation types, allowable end bearing and shaft adhesion pressures and associated settlement;
- soil and groundwater aggressivity against buried steel and concrete;
- earthquake site sub-soil class in accordance with AS 1170.4 (2007);
- flexible pavement thickness design for on-site parking areas/internal roadways; and
- comment on site preparation measures and earthworks requirements.

It should be noted that, at the time of preparation of this report, field work had been put on hold due to extended wet weather and as such, a final report will be prepared after completion of field work and remaining laboratory testing and comments. This report has been prepared in the interim with limited preliminary comments.

At the time of preparing this report, the investigation comprised the drilling and sampling of three boreholes to depths of between 3.5 m and 6 m, laboratory testing of selected samples, engineering analysis and preparation of this report. The details of the field work are presented in this report, together with comments and recommendations on the items listed above.

This report must be read in conjunction with all of the notes in Appendix A and any other attachments and should be kept in its entirety without separation of individual pages or sections.

2. Site Description

The site is located at Bayshore Drive, Byron Bay and is bound by Bayshore Drive to the east, commercial/industrial structures to the north and south, and bushland to the west (refer attached Drawing 1).

At the time of investigation, the site was generally level and comprised a gravel access track, surrounded by well-maintained grass and scattered building materials. Photographs of the typical site conditions at the time of investigation a shown in Figures 1 and 2.

Figure 1: Typical site conditions facing southward near Bore 6.

Figure 2: Typical site conditions facing eastward Near Bore 6.

3. Regional Geology and ASS Risk Mapping

Reference to the Geological Survey of New South Wales 1:250,000 series Tweed Heads sheet indicates that the site is located within an area mapped as Quaternary aged alluvium comprising "*beach and dune sand*". Fill is also anticipated at this site.

The existing ground conditions encountered during the field work (refer to Section 5) are in general agreement with the anticipated and published geology.

Reference to NSW Department of Environment and Climate Change published Acid Sulphate Soil Risk Mapping, 1:25,000 scale, indicates that the site lies within an area mapped as "*Low probability of occurrence*".

4. Field Work Methods

The investigation comprised the drilling and sampling of three boreholes (designated Bores 1, 4 and 6) using a 4WD utility mounted drilling rig (Christie Soil Rig), using solid flight augers fitted with a tungsten carbide (TC) drill bit to depths of between 3.5 m and 6 m. Soil strata were identified by assessing the recovered auger cuttings. Standard penetration tests (SPTs) were carried out at 1.5 m depth intervals from 1.5 m depth to provide information on soil strength. A bulk sample was recovered for laboratory testing. On completion of drilling and after checking for groundwater, the boreholes were backfilled using excavated drilling spoil.

Dynamic cone penetrometer (DCP) tests were carried out adjacent to the bores intermittently to depths of between 0.2 m and 1.5 m to provide further information on soil strength.

UTM coordinates at the bore locations were determined using a hand held GPS unit, which is accurate to approximately 5 m and are recorded on the borehole logs. Ground surface levels at the borehole locations were interpolated from a client supplied survey and are presented on the borehole logs. The approximate borehole locations are indicated on Drawing 1 in Appendix B.

All field work was undertaken by experienced geotechnical personnel who logged the bores and collected samples for visual and tactile assessment.

5. Field Work Results

The subsurface conditions encountered in the bores are described in detail on the borehole logs in Appendix C and are summarised in Table 1 and shown graphically in Figure 3.

Table 1: Summary of Subsurface Conditions

Bore	Strata/Depth Range (m)(i)		Depth to groundwater (m) (i)
	Fill (Sandy/Silty Clay)	Silty Sand – medium dense (or denser)	
1	0.0 – 1.1	1.1 – 4.0(ii)	0.5
4	0.0 – 1.5	1.5 – 6.0(ii)	1.2
6	0.0 – 1.0	1.0 – 3.5(ii)	0.9

Notes (i) All depths were measured from existing site level at the time of the investigation.
(ii) Limit of investigation.

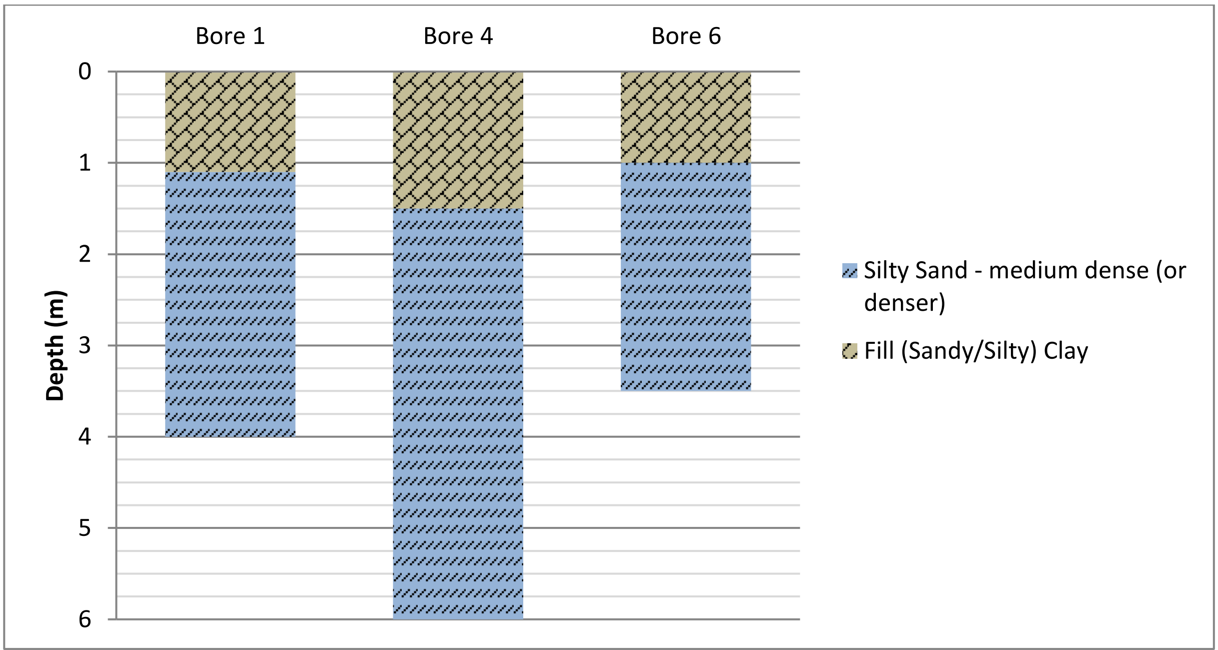

Figure 3: Graphical summary of subsurface conditions.

The fill encountered during the investigation generally appeared moderately to well compacted, however the upper 0.2 m depth at Bore 6 appeared poorly compacted probably due to water softening given the higher moisture content when compared to the underlying fill. Further, in the absence of documentation to confirm the fill was engineered and placed under 'controlled' conditions and meets the requirements of structural fill in AS 3798 (2007), it must be deemed 'uncontrolled'.

Groundwater was encountered at the depths indicated in Table 1 during the investigation. It should be noted that groundwater depths and ground moisture conditions are affected by climatic conditions and soil permeability, surface and subsurface drainage conditions and human influences, and will therefore vary with time. Seepage may also occur along the fill/natural interface during and after periods of wet weather.

6. Laboratory Testing

Geotechnical laboratory testing comprised particle size distribution (PSD), Atterberg limits and linear shrinkage (AL/LS) tests. A California bearing ratio (CBR) test was carried out on a retired bulk sample, compacted to approximately 100% standard dry density ratio at the estimated optimum moisture content (OMC) for standard compaction and soaked for four days under a surcharge loading of 4.5 kg prior to penetration.

The detailed laboratory test report sheets are attached in Appendix D, and the results are summarised in Table 2.

Table 2: Results of Atterberg Limits, Linear Shrinkage, Particle Size Distribution and Soaked CBR Testing

Bore	Depth (m)	W_F (%)	W_L (%)	PI (%)	LS (%)	Clay/Silt (%)	Sand (%)	Gravel (%)	SMDD (%)	OMC (%)	Swell (%)	CBR (%)
1	0.1 – 0.5	16.7	27	16	7	NT	NT	NT	1.82	16.0	0.0	4.0
4	0.4 – 1.0	19.0	38	16	6.0	40	22	38	1.81	16.0	3.5	3.5

Legend: W_F – field moisture content; W_L – liquid limit
W_P – plastic limit; PI – plasticity index
LS – linear shrinkage; SMDD – standard maximum dry density
OMC – optimum moisture content for Standard compaction; NT – not tested
CBR – California bearing ratio at approximately 100% SMDD after four day soak

As part of the preliminary ASS assessment, laboratory testing was performed on selected disturbed soil samples from the bores and comprised the following:

- ASS field screening after the addition of distilled water (pH_F) and peroxide (pH_{Fox}); and
- ASS analytical Chromium reducible sulfur (S_{CR}) suite.

Preliminary field screening and chemical laboratory tests for ASS were carried out with reference to the Acid Sulfate Soils Assessment Guidelines (Stone, Ahern, & Blunden, 1998). In total, eight samples recovered from the bores were submitted for field screening (pH_F and pH_{FOX}).

Based on the results of the field screening tests and visual inspection of the samples, four samples were submitted for more rigorous S_{CR} analytical testing. Results of the screening tests (p_{HF} and pH_{FOX}) and S_{CR} tests are summarised in Table 3 below and are also provided in detail in the attached laboratory report sheets in Appendix D. It should be noted that S_{CR} testing was conducted on the predominant soil types encountered during the investigation. The results of all ASS testing are discussed in Section 157.8 of this report.

Table 3: Summary of ASS Screening and Chromium Suite Test Results

Depth (m)	Sample Description	Field Screening Test Results				Chromium Suite Test Results (%S)				
		pH_F	pH_{FOX}	ΔpH	Reaction (0,1,2,3) F	pH_{KCl}	Chromium Reducible Sulfur (S_{CR})	Total Actual Acidity (TAA)	Retained Acidity (S_{NAS})	Existing plus potential Acidity
Bore 1										
0.5	Fill Sandy Clay	6.9	4.6	2.3	4	–	–	–	–	–
1.0	Fill Sandy Clay	6.8	4.4	2.4	4	5.8	<0.01	0.01	–	0.01
1.5	Silty Sand	7.4	5.5	1.9	1	0 – 0	–	–	–	–
2.0	Silty Sand	5.4	3.9	1.5	1	4.5	<0.01	0.03	–	0.03
Bore 6										
0.5	Fill Silty Clay	5.3	4.3	1.0	1	4.2	<0.01	0.06	–	0.06
1.0	Silty Sand	6.1	4.8	1.3	1	–	–	–	–	–
1.5	Silty Sand	6.8	4.9	1.9	1	6.8	<0.01	–	–	<0.01
2.0	Silty Sand	6.0	4.2	1.8	1	–	–	–	–	–

Notes: (i) 1 - denotes slight effervescence; 2 - denotes moderate reaction;
3 – denotes vigorous reaction; 4 – denotes very strong effervescence accompanied by escape of gas/heat;
F – indicates a bubbly/frothy reaction (organics).
(ii) Highlighted cell denotes level of existing plus potential acidity above threshold level of 0.03%S.

7. Interim Comments

7.1 Site Classification

Site classification of foundation soil reactivity strictly only applies to residential buildings up to two-storeys and to other buildings of similar size, loading and flexibility as defined in accordance with AS 2870 (2011). Such classification provides an indication of the propensity of the ground surface to move with seasonal variation in moisture content and has been used (along with general climatic zoning and general experience) to assess the potential depth of seasonal cracking and potential for softening under soaked conditions.

In accordance with AS 2870 (2011), due to the presence of 'uncontrolled' clayey fill in excess of 0.4 m depth, the site would be designated 'Class P', requiring design by engineering principles. The following classification is therefore provided for information for design by engineering principles.

To provide an indication of the reactive surface movements of the clayey fill, the results of a plasticity test were compared with an in-house database of plasticity and shrink-swell results to estimate a presumptive shrink-swell index (I_{ss}) value of 1.5% per ΔpF for the clayey fill. Previous investigation

(DP, 2020) carried out on this site indicated an I_{ss} value of 2.3% per ΔpF for the clayey fill and was adopted for this analysis. An I_{ss} value of 1.0% was adopted for the natural sand. The I_{ss} values were input into DP's in-house program REACTIVE, to calculate the characteristic surface movement (y_s) values in general accordance with AS 2870 (2011). AS 2870 (2011) provides recommended values of change in suction (Δu) and depth of suction (H_s) for major and regional centres throughout Australia, but not Byron Bay. Based on published data by Chan and Mostyn (2009) relating climatic conditions to suction, a value of 1.2 pF was adopted for Δu and 1.5 m for H_s in the REACTIVE calculations. This is based on an 'alpine/wet coastal' climatic zone. A cracking depth of 0.75 m was used in the analysis, based on $0.5H_s$.

The results indicate that the y_s values in response to seasonal moisture variation are in the order of 20 mm to 40 mm which compare to those of a 'Class M' (Moderately reactive) classification for existing site soils. Where the existing clayey fill is removed, moisture conditioned and replaced under 'controlled' engineered conditions to approximately 1.5 m depth, the y_s values are estimated to be in the order of 40 mm to 60 mm (including long term creep settlement of newly placed 'controlled' fill), consistent with a 'Class H1' (highly reactive) classification. This is due to the need to consider uncracked conditions in the analysis for the first five years after fill placement and first three years after any bulk earthworks cut and potential 'creep' settlement due to fill self-weight.

If 'abnormal' soil moisture conditions are experienced, the site would be classified as 'Class P' which would require more extensive foundation works to avoid adverse foundation performance. Abnormal soil moisture conditions are defined in AS 2870 (Clause 1.3.3).

The above results indicate good practice in design, construction and management of the site will be required to accommodate the potential site movements. In particular, good surface and subsurface drainage will be required, along with limits on landscaping and adequate moisture preparation. QBCC (2015) provides useful advice for this.

7.2 Earthquake Factors

With reference to AS 1170.4 (2007), Byron Bay is located in an area where the Hazard Factor (Z) is considered to be 0.09. Based on the results site investigation, the site sub-soil class of the proposed Byron Bay TAFE CLC site is considered to be C_e.

7.3 Earthworks and Site Preparation

7.3.1 Risk Comparison

Where there is site fill without records to confirm if it is controlled, there may be some risk of incurring unacceptably high differential settlement of the 'uncontrolled' fill under future upper-level footing loads and due to surface water or groundwater ingress. The results of the fieldwork indicate that the existing apparently well compacted 'uncontrolled' fill is between 1.0 m and 1.5 m thick. Accordingly, the following options are suggested to manage the risks associated with 'uncontrolled' fill:

High Level of Potential Risk – The option with minimum additional work but highest risk of movement within the fill under future load is to leave the existing 'uncontrolled' fill in place and compact only the surface with a heavy roller (minimum 12-tonne static weight) until there is no apparent heave under passage of the roller. This approach is not recommended unless the building footings and slab is supported on pile or deep pad footings penetrating into competent natural material below the fill, and the risk of settlement in other parts of the development including pavements can be accepted.

Low Level of Potential Risk – The option with significant additional work but lowest risk is to remove all fill and test roll the underlying natural ground for soft or loose conditions. The fill could then be screened to remove all coarse, oversize or deleterious material prior to replacement in layers of maximum 0.3 m 'loose' thickness. Each layer should be compacted under 'Level 1' inspection and testing in accordance with the recommendations presented in Section 7.3.2 below.

Intermediate Level of Risk – It follows from the above that varying thicknesses of fill may be removed, screened and recompacted (as for 'Low Level Risk' above), leaving an existing thickness of 'uncontrolled' fill in place (after test rolling with an 12-tonne roller as above), with an inherent mid-level of risk of future fill movement.

The above procedures will require geotechnical inspection and testing services to be employed during construction. It is further noted that the intermediate level and high level of risk options will potentially require ongoing maintenance where these options are adopted beneath heavy pavement areas.

7.3.2 Subgrade Preparation and Fill Placement/Replacement

Site preparation measures associated with the 'low level of potential risk', or the 'intermediate level of potential risk', as outlined in Section 7.3.1 above, and then subsequent use of a slab-on-ground footing system founding in engineered fill (with the corresponding risk level), or the placement of controlled fill over natural soils are detailed below:

- Remove any 'uncontrolled' or deleterious, soft, wet or highly compressible material or topsoil material rich in organics or root matter, unless the structure is to be piled or a high risk or intermediate risk of damaging settlement is to be accepted (refer Section 7.3.1 above).
- Based on previous investigation (DP, 2020) at this site, the presence of approximately 0.1 m depth of clayey/silty/gravelly sand containing organics (e.g. root matter), should be noted and allowance made to strip these types of soils. 'Uncontrolled' fill was encountered at the test locations to depths of between 1.0 m and 1.5 m.
- Carefully reshape and grade the clay and/or sand subgrade beneath proposed structures and pavements to drain towards the outside from a slightly domed centre. Any internal low spots should be prevented from developing as these may act as a drainage sink and subsequently lead to localised swelling and softening.
- Assess moisture contents of the subgrade and adjust the moisture content (if required) to be within 2% of OMC, where OMC is the optimum moisture content at Standard compaction.
- Roll the exposed surface with at least six passes of a minimum 12 tonne deadweight smooth drum roller, with a final test roll pass accompanied by careful visual inspection to ensure that any deleterious materials such as soft or loose, wet or highly compressible soil and any organics are identified and removed.
- For pavement subgrades, compact the subgrade (including upper 0.5 m if in fill) to a minimum dry density ratio of 98% Standard, but 100% Standard for heavy duty pavement subgrades (i.e. where truck or other heavy vehicle movements are anticipated, if applicable). Clay fill should be limited to a maximum dry density ratio of 102% Standard to avoid over-compaction. Over-

compacted clays (i.e. minimum dry density ratio of >102%) which are dry of OMC, may swell significantly swell and lose strength if they are wetted after compaction, potentially changing the site classification and reducing subgrade strengths assumed in design, and therefore need to be avoided.

- It is noted that up to 3.5% swell was recorded during the soaking phase of the CBR testing, which is indicative of highly expansive subgrade conditions. Volume changes in expansive subgrades can be minimised by the placement of a 150 mm select fill capping layer above the expansive clay, which should extend to at least 0.5 m beyond the edge of pavement.
- For building platforms, all existing fill should be removed and stockpiled for further assessment of its suitability to be re-used as engineered fill, unless the buildings are designed to be founded on piles taken to bear within the natural soils below all fill (refer Section 7.5).
- Place fill in layers not exceeding 300 mm loose thickness, with each layer compacted to a minimum dry density ratio of 98% Standard. It is recommended that the upper 1 m of fill for any fill which is required to support building footings or upper 0.5 m under heavy duty pavements (i.e. where truck or other heavy vehicle movements are anticipated, if applicable) be compacted to a minimum dry density ratio of 100% Standard compaction. This higher dry density ratio should apply to all fill extending from a nominal horizontal distance of 2 m at the edge of each structural support footing with a nominal zone of influence of 1H:1V down and away from the proposed engineered subgrade level. Where fill is clayey, moisture content within the fill should be maintained within 2% of OMC (where OMC is the optimum moisture content at Standard compaction) during and after compaction.
- Place fill over or adjacent to existing sloping ground or cuts greater than 8H:1V in level benches with a minimum vertical height of 0.3 m adjacent to the existing ground and into natural soils to ensure compaction and interlocking, and to reduce the potential for instability between the existing soils and any newly placed ‘controlled’ fill.
- Seal or cover any compacted clayey foundation soil at or close to footing formation level as soon as practicable, to reduce the opportunity for occurrence of desiccation and cracking. It is recommended that building platforms be overlaid with a working platform of nominal 200 mm thickness of well graded clayey granular fill of minimum CBR 20% with a minimum 15% fines (<75 µm) content to reduce moisture variation (and associated shrink-swell movements) in subgrade soils, and to improve trafficability for light vehicles. Where the surface is to be trafficked by heavy vehicles/machinery, then specific pavement thickness design should be undertaken.
- ‘Level 1’ testing and inspection of fill, in accordance with AS 3798 (2007) will be required if the fill is to be used for support of structures.

The above procedures will require geotechnical inspection and testing services during construction.

Due to the low to medium plasticity of the near surface clayey soils, it is expected that rubber tyred vehicles in particular will have trafficability problems during and after periods of rainfall or other increases in subgrade moisture content, and in some cases tracked plant may experience some difficulty. It will be essential to keep the site well drained during construction. As indicated previously, a granular working platform is recommended to reduce potential lost time during or following wet weather, and to reduce wetting or drying of the subgrade soils (with associated long term movements).

7.4 High Level Foundations

High level strip and pad footings to maximum widths of 1 m and 2 m respectively, can be designed using the allowable values indicated in Table 4 below. Subject to the site preparation earthworks carried out (refer 7.3 above) , stiffened raft should be adopted for slab on ground parts of the structure to suit the expected site classification movements and settlements.

Table 4: High Level Footing Design Bearing Pressures (Allowable)

Material Description	Strip Footing Allowable Bearing Pressure (kPa)	Pad Footing Allowable Bearing Pressure (kPa)
'Controlled' Fill(i)	100	
'Uncontrolled' Fill	Not recommended	
Silty Sand – medium dense (or stronger) (ii)	125	250

Notes (i) Provided site preparation is carried out in accordance with the "Low Level of Potential Risk" recommendations in Section 7.3 with acceptance of fill creep settlements of 1% of fill thickness

(ii) Minimum 1 m footing embedment depth from ground surface level.

For upper level pad or strip footings (loaded as above) it is considered that settlements under such applied loading will be less than 1% to 2% of footing width.

7.5 Piled Foundations

Should high level footings not be suitable, steel screw piles (in multiple groups with raking if needed to resist for lateral load) and multi helix for increased capacity are considered potential suitable option. The high groundwater table and sands are likely to make bored piers unsuitable unless the groundwater table is lowered (or naturally lower) at the time of construction and piers are founded in the upper level of the sand layer.

The use of steel screw piles (raked for lateral support) with a pile cap could be adopted for lightly loaded structures requiring minimal lateral resistance. Steel screw pile capacity is a function of foundation density/strength and depth.

Screw piles can be designed using the allowable parameters given in Table 5 and would need to be founded to a minimum of two pile diameters into the relevant founding layer. For the parameters below, the screw pile helix diameter has been assumed to be at least 0.6 m. It is noted that bored pile excavation would be prone to collapse and loosening in sands particularly at depth where groundwater is present and the depth of bored piles if adopted would be need to be limited to cater for these conditions.

Table 5: Allowable Screw and Bored Pile Design Pressures

Material Description and Founding Depth (m)	Allowable Shaft Adhesion (kPa)	Allowable End Bearing (kPa)(i)(ii)
Silty Sand– medium dense (or stronger) – min. 3 m depth	Not Recommended	500 (screw piles) 300 (bored piles)(iii)

Notes (i) Assumed min. pile diameter of 0.6 m.
(ii) Medium dense sand (buoyant unit weight adopted)
(iii) Bored only suitable if lower water table

For limit state design, the above allowable bearing pressures should be multiplied by the adopted factor of safety of 2.5 to equate to ultimate values. A geotechnical strength reduction factor (ϕ_g) of 0.45 is recommended for limit state design of piles in accordance with AS 2159 (2009). This is based on the data presented in this report, the method of soil strength assessment used in this investigation and after assessing the overall design average risk rating (ARR) for the site, design and installation risk factors anticipated. Higher values of ϕ_g may be applied if additional investigation is carried out at the site or if selected piles are to be subjected to confirmatory load testing.

It is essential that if bored piles are adopted, that foundation excavations be inspected by experienced Douglas Partners' personnel to ensure the design parameters adopted are suitable for ground conditions and to ensure that there is no soft or loose material remaining at the base of the excavations. Ground conditions can vary, and it is essential that adequate provision be made throughout the project to vary foundations to suit differing ground conditions.

It is important that the installation of steel screw piles be carefully controlled in the field to ensure the pile does not meet refusal prior to meeting its termination depth. In this scenario, advancement of the pile will cease, causing over rotation and disturbance of the overburden soils above the helix. This phenomenon is often encountered where steel screw piles encounter an underlying harder stratum (such as weathered rock) and the toe penetration is considerably reduced in comparison to the string rotation. Where over-rotation occurs, the bearing capacity for the helix would be substantially reduced and/or pile movements incurred.

The actual capacity of steel screw piles depends not only on the soil conditions but also on structural considerations of the piles such as the strength of the helix and the helix/shaft joint. It is considered that the structural section capacity as well as geotechnical capacity will need to be considered where the required load carrying capacity of individual steel screw piles is greater than (say) 600 kN. Measurement of installation torque should not be relied upon to indicate pile capacity, as it has been documented that significantly misleading results can be obtained. For this reason, piling contractors would be responsible for assessment of actual pile capacities for their piles.

Structural capacity of the steel screw pile should be checked, and due allowance made for inclined or eccentric loads, and possible corrosion effects.

Lateral capacity of steel screw piles could be increased by constructing concrete pile caps or by using proprietary head attachments which are dragged into the soil providing additional lateral resistance at the pile head. The lateral support is generally limited and is generally suited to non-critical structures that can accommodate some lateral movement such as light poles, signs and small towers.

The ultimate geotechnical strength ($R_{d,ug}$) of steel screw piles in uplift can be calculated using the weight of the enclosing cylinder of soil above the helix together with friction developed on the walls of this cylinder, using an the parameters given in Table 6.

Table 6: Soil Parameters for Screw Pile Uplift Calculation

Material Description	Soil density (kN/m^3)$^{(i)}$	Friction angle (°)
'Controlled' Fill	8	NA
'Uncontrolled' Fill	6	NA
Silty Sand – medium dense (or stronger)	8	32

Notes (i) Assuming a high groundwater table in the worst case.
(ii) NA – not available.

It should be noted that AS 2159 (2009) requires compressive load testing of piles to be undertaken to a test load of E_d/Φ_g. For a geotechnical strength reduction factor (Φ_g) of 0.5, this test load is twice the design action effect (E_d). The results of steel screw pile load tests, however, typically indicate that plastic deformation of the helix can occur when a screw pile is loaded to only 1.5 times E_d approximately, for piles with a helix outstand to plate thickness ratio of greater than about 10. For these piles, therefore, failure can occur prior to achievement of the required test load.

Although the test load nominated by AS 2159 (2009) is therefore unlikely to be achieved for piles with insufficient helix plate thickness, failure would not be expected to occur at normal serviceability loads; therefore, in order to achieve the nominated test load, steel screw piles should be designed with a helix outstand to plate thickness ratio of no greater than about 10.

A specialist screw piling contractor should be provided with a copy of this report, in full, to ensure they are aware of subsurface conditions.

Experience indicates that settlements of properly designed and constructed piles are unlikely to exceed 1% to 2% of the pile diameter.

Ultimately piling contractors should be contacted to confirm suitable piling types and pile capacities.

7.6 Soil and Groundwater Aggressivity

Comments on soil and groundwater aggressivity will be provided subsequent completion of remaining field work and drilling and would be provided in the revised report.

7.7 Pavements

7.7.1 Subgrade

With reference to the current conditions at the site, subgrade soils are expected to comprise a mixture of silty clayey fill overlying natural sandy soils. The results of the CBR tests on the clayey fill returned values of 3.5% and 4%. It is noted, however, that the fill is variable in constituents and included clayey fill in a number of bores. Therefore, based on the results of laboratory testing at the site and DP experience with clayey soils, a subgrade CBR value of 3% has been adopted for design.

7.7.2 Design Traffic Loading

In the absence of more detailed information, traffic loading for the site has been assumed based on the proposed development and procedures presented in Austroads (2017). The adopted design traffic for the site is summarised in Table 7 below.

Table 7: Indicative Design Traffic Loading

Street Type as defined in Austroads (2017)	Possible Application	Indicative Design Traffic (ESA)(i)
"Minor with two lane traffic"	Carpark and driveway areas subject only to light vehicle traffic (i.e. vehicles up to 3 tonnes)	8×10^3
"Local access in industrial area"	Driveways which include delivery vehicles	3×10^5

Notes: (i) 40 year design period.

The traffic loading for driveways which include delivery trucks presented above is commensurate with that required for a 'Local Street' in NRLG (2013b).

It is important that the pavement areas are carefully considered and separated into areas that are likely to and unlikely to have heavy vehicles traverse the pavement. If trucks are allowed to traffic pavement areas which have been designated for car traffic, there is a risk of reduced design life and premature pavement damage. The above loadings are not applicable for forklifts, loaders etc, which will require specific pavement design if these sorts of heavy duty vehicles are expected to traffic the site.

If the actual traffic loadings are significantly different to those assumed above, then the pavement thickness designs should be reviewed.

7.7.3 Pavement Thickness Design

The pavement thickness designs contained within this report have been prepared with reference to Council guidelines (NRLG, 2013) and Austroads (2017). The pavement thickness has been designed based on a design CBR presented in Section 7.7.1 and traffic loadings presented in Section 7.7.2, and is presented in Table 8 below.

Table 8: Pavement Thickness Design

Pavement Layer	Minimum Layer Thickness (mm)	
	Main Driveways (3 x 10^5 ESA)	Carpark (8 x 10^3 ESA)
Wearing Course[1]	two-coat spray seal or 30 mm AC10[(1)]	
Basecourse	120	100
Subbase	320	220
Total Thickness	440	320

Notes
(1) Where asphalt is to be used as a wearing course a primer seal should be placed over the basecourse.
Consideration should be given to the placement of a select layer over expansive subgrade soils, as outlined in Section 7.3.2.

A minimum basecourse thickness of 150 mm excluding wearing surfaces is required as per council guidelines (NRLG, 2013). It is noted, however, that for the internal pavements, these minimum requirements may not apply and hence a thinner basecourse may be applicable, as presented above. Should council guidelines (NRLG, 2013b) of 150 mm of basecourse apply, the subbase thickness could be reduced to achieve the overall 'total thicknesses' presented above.

Similarly, placement and compaction of thin layers, such as 100 mm basecourse, may be difficult to achieve. For the carpark pavement, it may be possible to place and compact a single 320 mm layer of basecourse quality material.

7.7.4 Material Quality and Compaction Requirements

The above pavement design thicknesses are minimum thicknesses and do not account for construction tolerances which should be considered during construction.

The pavement thickness designs presented above are dependent on the provision and maintenance of adequate surface drainage. Surface grades should be sufficient to prevent ponding of stormwater.

It is expected that there may be a requirement for increased maintenance in areas of tightly turning trucks due to the high shear / torsional stresses applied to the pavement surface. The use of a stiffer binder (i.e. Class 450 or Class 600 PMB bitumen) in the asphalt (if used) would be expected to reduce the damage to the asphalt surface in areas of tightly turning heavy vehicles.

It is recommended that where any new pavement abuts an existing pavement, it should be benched / keyed in a minimum width of 0.3 m. Allowance should also be made for the incorporation of intra pavement drainage. Vertical interface / joints between the new and existing sections of pavements should not be located within or along wheel paths.

Recommended pavement material quality and compactions requirements are presented in Table 9.

Table 9: Material Quality and Compaction Requirements

Pavement Layer	Material Quality	Compaction
Basecourse	CBR ≥ 80%, PI ≤ 6%, Grading in accordance with NRLG (2013) for DGB20, GMB20 or NGB20-2C	Compact to at least 100% dry density ratio Standard (AS 1289.5.2.1)
Subbase	CBR ≥ 30%, PI ≤ 12%, Grading in accordance with NRLG (2013) for DGS20, DGS40, GMS40, NGS20 or NGS40	Compact to at least 100% dry density ratio Standard (AS 1289.5.2.1)
Select Subgrade Layer	CBR ≥ 15%, PI ≤ 15%	Compact to at least 100% dry density ratio Standard (AS 1289.5.2.1)
Subgrade (clayey fill)	CBR ≥ 3%	Compact to at least 95% dry density ratio Standard (AS 1289.5.2.1)

7.7.5 Subgrade Preparation

Subgrade should be prepared as per the ground treatment recommendations and adopted approach presented in Section 7.3.

Geotechnical inspection and testing should be performed during construction in accordance with AS 3798 (2007).

7.8 Preliminary Acid Sulfate Soil (ASS) Assessment

A preliminary ASS investigation was carried out during the preliminary geotechnical investigation to assess the presence of ASS. It should be noted that addition field work and laboratory testing is required to complete the preliminary ASS assessment; the below comments are provided during the interim.

Results of the screening tests (pH_F, pH_{FOX}) were assessed based on the recommendations in the Department of Mines and Natural Resources Publications with regards to ASS to determine whether they are indicative of actual acid sulfate soils (AASS) or potential acid sulfate soils (PASS). A total of eight samples were retrieved and tested. Testing was carried out on predominant soil horizons encountered during the investigation. The results are summarised in Table 3 and are detailed in the laboratory results attached in Appendix D.

- **pH in distilled water (pH_F)** measures the existing acidity of the soil and is used to help identify whether actual ASS is present. A pH_F between 4 and 5.5 indicates acidic soils. If pH_F is less than 4, it is considered that either actual ASS is present or soils contain a high organic content.

  All samples recorded pH_F values of greater than 4.

The pH_F test method does not detect acidity bound within sulfides; therefore the pH_{FOX} test is undertaken as this gives an indication of any potential acid release.

- **A pH peroxide test (pH_{FOX})** value less than 3 combined with a pH_{FOX} reading at least one pH unit below pH_F (i.e. **ΔpH** >1) and a strong **reaction with peroxide**, strongly indicates the presence of potential ASS.

  All of the samples recorded values greater than 3. Two near surface samples from Bore 1 (0.5 m depth and 1.0 m depth) had a "very rigorous" reaction; this is likely to be due to organics. The remaining samples had a "none to slight" reaction.

On the basis of the qualitative pH screening results, the likelihood of actual ASS to occur is considered to be mostly low.

To determine more definitively if AASS or PASS are present, four samples were selected for more rigorous and quantitative chromium suite testing.

The action criterion, which triggers a requirement for ASS disturbance to be managed, is derived for ASS soils from the net acidity calculated from the Chromium Suite of tests. The net acidity is calculated from the Acid Base Account (ABA) equation in the ASSMAC (1998) as follows:

Net acidity = (S_{CR} + TAA + S_{NAS} – ANC/FF) ≥ 0.03%S (for sand and clay soils)

Where: *S_{CR} – Chromium Reducible Sulfur; TAA – Titratable Actual Acidity; S_{NAS} – Net Acid Soluble Sulfur; ANC – Acid Neutralising Capacity; and FF – Fineness factor (generally take as 1.5)*

Based on the above the existing plus potential acidity was calculated (refer to laboratory results) to be equal to or greater than 0.03%S in two of the sample tested; Bore 1 at 2 m depth with a net acidity of 0.03% and Bore 6 at 0.5 m depth with a net acidity of 0.06%. Further examination of the S_{CR} results indicates that the elevated existing plus potential acidity is due to TAA and therefore the soil is assessed to be non ASS and as such, acid sulfate soil management plan (ASSMP) will not be required if less than 1,000 t (or 500 m^3) of soil is to be disturbed. If more than 1,000 t is to be disturbed then the results indicate that an ASSMP will be required. Irrespective, due to the natural acidity in the soil, neutralisation against potential environmental harm will still be required.

Further detailed work will be required to prepare an ASSMP, however as a guide, with reference to neutralising agent (generally aglime) should be applied during site works (refer below). The TAA results can be used to guide liming rates to achieve desired pH levels. Thorough mixing, a safety factor and a fully contained treatment pad would generally not be necessary. Instead, neutralising agent may be:

- spread in key areas as part of the fill operations to intercept any acidic leachate flow;
- added to truckloads of disturbed material while being moved, thus achieving a degree of mixing during transport and placement;
- spread as a guard layer under any temporary or permanent stockpiles or treatment areas;
- incorporated as lime-enriched perimeters around temporary or permanent stockpiles or treatment areas; and

- positioned in drains and areas most likely to experience flow.

Using the highest reported level of soil acidity (i.e. existing plus potential) determined by the laboratory test results carried out during this investigation and DP's previous investigation at this site (DP, 2020), a preliminary neutralisation rate of 3.3 kg of lime per tonne of soil is required (in-situ). Assuming an overall stockpile density, after 'bulking up', of approximately 1.5 tonnes/m^3, this equates to a lime application value of 5 kg/m^3.

8. References

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Dear, S., Ahern, C., O'Brien, L., Dobos, S., McElnea, A., Moore, N., & Watling, K. (2014). *Queensland Acid Sulfate Soil Technical Manual: Soil Management Guidelines.* Brisbane: Department of Science: Department of Science, Information, Technology, Innovation and the Arts, Queensland Government.

DP. (2020). *Report on Geotechnical Investigation and Acid Sulfate Soil (ASS) Assessment, TAFE CLC Tranche 3, Bayshore Drive, Byron Bay.* Douglas Partners Pty Ltd, Project Number 98102.00,R,001,Rev0.

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9. Limitations

Douglas Partners Pty Ltd (DP) has prepared this interim report for this project at Bayshore Drive, Byron Bay in accordance with DP's proposal dated 201414.P.001.Rev0 dated 9 February 2021 with acceptance received from Nic Glass of Brewster Hjorth Architects (Brewster Hjorth) via email dated 10 February 2021. The work was carried out under DP's standard "Conditions of Engagement". This report is provided for the exclusive use of Brewster Hjorth and their consulting engineers for this project only and for the purposes as described in the report. It should not be used by or relied upon for other projects or purposes on the same or other site or by a third party. Any party so relying upon this report beyond its exclusive use and purpose as stated above, and without the express written consent of DP, does so entirely at its own risk and without recourse to DP for any loss or damage. In preparing this report DP has necessarily relied upon information provided by the client and/or their agents.

The results provided in the report are indicative of the sub-surface conditions on the site only at the specific sampling and/or testing locations, and then only to the depths investigated and at the time the work was carried out. Sub-surface conditions can change abruptly due to variable geological processes and also as a result of human influences. Such changes may occur after DP's field testing has been completed.

DP's advice is based upon the conditions encountered during this investigation and DP's previous investigation (DP, 2020) carried out at this site. The accuracy of the advice provided by DP in this report may be affected by undetected variations in ground conditions across the site between and beyond the sampling and/or testing locations. The advice may also be limited by budget constraints imposed by others or by site accessibility.

The assessment of atypical safety hazards arising from this advice is restricted to the geotechnical components set out in this report and based on known project conditions and stated design advice and assumptions. While some recommendations for safe controls may be provided, detailed 'safety in design' assessment is outside the current scope of this report and requires additional project data and assessment.

This report must be read in conjunction with all of the attached and should be kept in its entirety without separation of individual pages or sections. DP cannot be held responsible for interpretations or conclusions made by others unless they are supported by an expressed statement, interpretation, outcome or conclusion stated in this report.

This report, or sections from this report, should not be used as part of a specification for a project, without review and agreement by DP. This is because this report has been written as advice and opinion rather than instructions for construction.

The scope for work for this investigation/report did not include the assessment of surface or sub-surface materials or groundwater for contaminants, within or adjacent to the site. Should evidence of filling of unknown origin be noted in the report, and in particular the presence of building demolition materials, it should be recognised that there may be some risk that such filling may contain contaminants and hazardous building materials.

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Appendix A

About This Report
Sampling Methods
Soil Descriptions
Symbols and Abbreviations

Introduction

These notes have been provided to amplify DP's report in regard to classification methods, field procedures and the comments section. Not all are necessarily relevant to all reports.

DP's reports are based on information gained from limited subsurface excavations and sampling, supplemented by knowledge of local geology and experience. For this reason, they must be regarded as interpretive rather than factual documents, limited to some extent by the scope of information on which they rely.

Copyright

This report is the property of Douglas Partners Pty Ltd. The report may only be used for the purpose for which it was commissioned and in accordance with the Conditions of Engagement for the commission supplied at the time of proposal. Unauthorised use of this report in any form whatsoever is prohibited.

Borehole and Test Pit Logs

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

Interpretation of the information and its application to design and construction should therefore take into account the spacing of boreholes or pits, the frequency of sampling, and the possibility of other than 'straight line' variations between the test locations.

Groundwater

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

- In low permeability soils groundwater may enter the hole very slowly or perhaps not at all during the time the hole 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. They may not be the same at the time of construction as are indicated in the report; and
- 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 first be washed out of the hole if water measurements are to be made.

More reliable measurements can be made by installing standpipes which are read at intervals over several days, or 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 a perched water table.

Reports

The report has been prepared by qualified personnel, is based on the information obtained from field and laboratory testing, and has been undertaken to current engineering standards of interpretation and analysis. Where the report has been prepared for a specific design proposal, the information and interpretation may not be relevant if the design proposal is changed. If this happens, DP will be pleased to review the report and the sufficiency of the investigation work.

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

- Unexpected variations in ground conditions. The potential for this will depend partly on borehole or pit spacing and sampling frequency;
- Changes in policy or interpretations of policy by statutory authorities; or
- The actions of contractors responding to commercial pressures.

If these occur, DP will be pleased to assist with investigations or advice to resolve the matter.

About this Report

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, DP requests that it be immediately notified. Most problems are much more readily resolved when conditions are exposed rather than at some later stage, well after the event.

Information for Contractual Purposes

Where information obtained from this report 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. DP would be pleased to assist in this regard and/or to make additional report copies available for contract purposes at a nominal charge.

Site Inspection

The company will always be pleased to provide engineering inspection services for geotechnical and environmental aspects of work to which this report is related. This could range from a site visit to confirm that conditions exposed are as expected, to full time engineering presence on site.

Sampling

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

Disturbed samples taken during drilling provide information on colour, type, inclusions and, depending upon the degree of disturbance, some information on strength and structure.

Undisturbed samples are taken by pushing a thin-walled sample tube into the soil and withdrawing it to obtain a sample of the soil in a relatively undisturbed state. Such samples yield information on structure and strength, and are necessary for laboratory determination of shear strength and compressibility. Undisturbed sampling is generally effective only in cohesive soils.

Test Pits

Test pits are usually excavated with a backhoe or an excavator, allowing close examination of the in-situ soil if it is safe to enter into the pit. The depth of excavation is limited to about 3 m for a backhoe and up to 6 m for a large excavator. A potential disadvantage of this investigation method is the larger area of disturbance to the site.

Large Diameter Augers

Boreholes can be drilled using a rotating plate or short spiral auger, generally 300 mm or larger in diameter commonly mounted on a standard piling rig. The cuttings are returned to the surface at intervals (generally not more than 0.5 m) and are disturbed but usually unchanged in moisture content. Identification of soil strata is generally much more reliable than with continuous spiral flight augers, and is usually supplemented by occasional undisturbed tube samples.

Continuous Spiral Flight Augers

The borehole is advanced using 90-115 mm diameter continuous spiral flight augers which are withdrawn at intervals to allow sampling or in-situ testing. This is a relatively economical means of drilling in clays and sands above the water table. Samples are returned to the surface, or may be collected after withdrawal of the auger flights, but they are disturbed and may be mixed with soils from the sides of the hole. Information from the drilling (as distinct from specific sampling by SPTs or undisturbed samples) is of relatively low reliability, due to the remoulding, possible mixing or softening of samples by groundwater.

Non-core Rotary Drilling

The borehole is advanced using a rotary bit, with water or drilling mud being pumped down the drill rods and returned up the annulus, carrying the drill cuttings. Only major changes in stratification can be determined from the cuttings, together with some information from the rate of penetration. Where drilling mud is used this can mask the cuttings and reliable identification is only possible from separate sampling such as SPTs.

Continuous Core Drilling

A continuous core sample can be obtained using a diamond tipped core barrel, usually with a 50 mm internal diameter. Provided full core recovery is achieved (which is not always possible in weak rocks and granular soils), this technique provides a very reliable method of investigation.

Standard Penetration Tests

Standard penetration tests (SPT) are used as a means of estimating the density or strength of soils and also of obtaining a relatively undisturbed sample. The test procedure is described in Australian Standard 1289, Methods of Testing Soils for Engineering Purposes - Test 6.3.1.

The test is carried out in a borehole by driving a 50 mm diameter split sample tube under the impact of a 63 kg hammer with a free fall of 760 mm. It is normal for the tube to be driven in three successive 150 mm increments and the 'N' value is taken as the number of blows for the last 300 mm. In dense sands, very hard clays or weak rock, the full 450 mm 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 150 mm of, say, 4, 6 and 7 as:

  4,6,7
  N=13

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

  15, 30/40 mm

The results of the SPT tests can be related empirically to the engineering properties of the soils.

Dynamic Cone Penetrometer Tests / Perth Sand Penetrometer Tests

Dynamic penetrometer tests (DCP or PSP) are carried out by driving a steel rod into the ground using a standard weight of hammer falling a specified distance. As the rod penetrates the soil the number of blows required to penetrate each successive 150 mm depth are recorded. Normally there is a depth limitation of 1.2 m, but this may be extended in certain conditions by the use of extension rods. Two types of penetrometer are commonly used.

- Perth sand penetrometer - a 16 mm diameter flat ended rod is driven using a 9 kg hammer dropping 600 mm (AS 1289, Test 6.3.3). This test was developed for testing the density of sands and is mainly used in granular soils and filling.
- Cone penetrometer - a 16 mm diameter rod with a 20 mm diameter cone end is driven using a 9 kg hammer dropping 510 mm (AS 1289, Test 6.3.2). This test was developed initially for pavement subgrade investigations, and correlations of the test results with California Bearing Ratio have been published by various road authorities.

Description and Classification Methods

The methods of description and classification of soils and rocks used in this report are generally based on Australian Standard AS1726:2017, Geotechnical Site Investigations. In general, the descriptions include strength or density, colour, structure, soil or rock type and inclusions.

Soil Types

Soil types are described according to the predominant particle size, qualified by the grading of other particles present:

Type	Particle size (mm)
Boulder	>200
Cobble	63 - 200
Gravel	2.36 - 63
Sand	0.075 - 2.36
Silt	0.002 - 0.075
Clay	<0.002

The sand and gravel sizes can be further subdivided as follows:

Type	Particle size (mm)
Coarse gravel	19 - 63
Medium gravel	6.7 - 19
Fine gravel	2.36 – 6.7
Coarse sand	0.6 - 2.36
Medium sand	0.21 - 0.6
Fine sand	0.075 - 0.21

Definitions of grading terms used are:

- Well graded - a good representation of all particle sizes
- Poorly graded - an excess or deficiency of particular sizes within the specified range
- Uniformly graded - an excess of a particular particle size
- Gap graded - a deficiency of a particular particle size with the range

The proportions of secondary constituents of soils are described as follows:

In fine grained soils (>35% fines)

Term	Proportion of sand or gravel	Example
And	Specify	Clay (60%) and Sand (40%)
Adjective	>30%	Sandy Clay
With	15 – 30%	Clay with sand
Trace	0 - 15%	Clay with trace sand

In coarse grained soils (>65% coarse)
- with clays or silts

Term	Proportion of fines	Example
And	Specify	Sand (70%) and Clay (30%)
Adjective	>12%	Clayey Sand
With	5 - 12%	Sand with clay
Trace	0 - 5%	Sand with trace clay

In coarse grained soils (>65% coarse)
- with coarser fraction

Term	Proportion of coarser fraction	Example
And	Specify	Sand (60%) and Gravel (40%)
Adjective	>30%	Gravelly Sand
With	15 - 30%	Sand with gravel
Trace	0 - 15%	Sand with trace gravel

The presence of cobbles and boulders shall be specifically noted by beginning the description with ‘Mix of Soil and Cobbles/Boulders’ with the word order indicating the dominant first and the proportion of cobbles and boulders described together.

Soil Descriptions

Cohesive Soils

Cohesive soils, such as clays, are classified on the basis of undrained shear strength. The strength may be measured by laboratory testing, or estimated by field tests or engineering examination. The strength terms are defined as follows:

Description	Abbreviation	Undrained shear strength (kPa)
Very soft	VS	<12
Soft	S	12 - 25
Firm	F	25 - 50
Stiff	St	50 - 100
Very stiff	VSt	100 - 200
Hard	H	>200
Friable	Fr	-

Cohesionless Soils

Cohesionless soils, such as clean sands, are classified on the basis of relative density, generally from the results of standard penetration tests (SPT), cone penetration tests (CPT) or dynamic penetrometers (PSP). The relative density terms are given below:

Relative Density	Abbreviation	Density Index (%)
Very loose	VL	<15
Loose	L	15-35
Medium dense	MD	35-65
Dense	D	65-85
Very dense	VD	>85

Soil Origin

It is often difficult to accurately determine the origin of a soil. Soils can generally be classified as:

- Residual soil - derived from in-situ weathering of the underlying rock;
- Extremely weathered material – formed from in-situ weathering of geological formations. Has soil strength but retains the structure or fabric of the parent rock;
- Alluvial soil – deposited by streams and rivers;
- Estuarine soil – deposited in coastal estuaries;
- Marine soil – deposited in a marine environment;
- Lacustrine soil – deposited in freshwater lakes;
- Aeolian soil – carried and deposited by wind;
- Colluvial soil – soil and rock debris transported down slopes by gravity;
- Topsoil – mantle of surface soil, often with high levels of organic material.
- Fill – any material which has been moved by man.

Moisture Condition – Coarse Grained Soils

For coarse grained soils the moisture condition should be described by appearance and feel using the following terms:

- Dry (D) Non-cohesive and free-running.
- Moist (M) Soil feels cool, darkened in colour.
  Soil tends to stick together.
  Sand forms weak ball but breaks easily.
- Wet (W) Soil feels cool, darkened in colour.
  Soil tends to stick together, free water forms when handling.

Moisture Condition – Fine Grained Soils

For fine grained soils the assessment of moisture content is relative to their plastic limit or liquid limit, as follows:

- 'Moist, dry of plastic limit' or 'w <PL' (i.e. hard and friable or powdery).
- 'Moist, near plastic limit' or 'w ≈ PL (i.e. soil can be moulded at moisture content approximately equal to the plastic limit).
- 'Moist, wet of plastic limit' or 'w >PL' (i.e. soils usually weakened and free water forms on the hands when handling).
- 'Wet' or 'w ≈LL' (i.e. near the liquid limit).
- 'Wet' or 'w >LL' (i.e. wet of the liquid limit).

Symbols & Abbreviations

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Introduction

These notes summarise abbreviations commonly used on borehole logs and test pit reports.

Drilling or Excavation Methods

C	Core drilling
R	Rotary drilling
SFA	Spiral flight augers
NMLC	Diamond core - 52 mm dia
NQ	Diamond core - 47 mm dia
HQ	Diamond core - 63 mm dia
PQ	Diamond core - 81 mm dia

Water

▷	Water seep
▽	Water level

Sampling and Testing

A	Auger sample
B	Bulk sample
D	Disturbed sample
E	Environmental sample
U_{50}	Undisturbed tube sample (50mm)
W	Water sample
pp	Pocket penetrometer (kPa)
PID	Photo ionisation detector
PL	Point load strength Is(50) MPa
S	Standard Penetration Test
V	Shear vane (kPa)

Description of Defects in Rock

The abbreviated descriptions of the defects should be in the following order: Depth, Type, Orientation, Coating, Shape, Roughness and Other. Drilling and handling breaks are not usually included on the logs.

Defect Type

B	Bedding plane
Cs	Clay seam
Cv	Cleavage
Cz	Crushed zone
Ds	Decomposed seam
F	Fault
J	Joint
Lam	Lamination
Pt	Parting
Sz	Sheared Zone
V	Vein

Orientation

The inclination of defects is always measured from the perpendicular to the core axis.

h	horizontal
v	vertical
sh	sub-horizontal
sv	sub-vertical

Coating or Infilling Term

cln	clean
co	coating
he	healed
inf	infilled
stn	stained
ti	tight
vn	veneer

Coating Descriptor

ca	calcite
cbs	carbonaceous
cly	clay
fe	iron oxide
mn	manganese
slt	silty

Shape

cu	curved
ir	irregular
pl	planar
st	stepped
un	undulating

Roughness

po	polished
ro	rough
sl	slickensided
sm	smooth
vr	very rough

Other

fg	fragmented
bnd	band
qtz	quartz

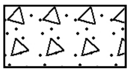
Symbols & Abbreviations

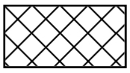
Graphic Symbols for Soil and Rock

General

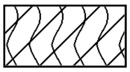
 Asphalt

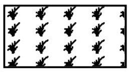
 Road base

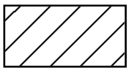
 Concrete

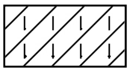
 Filling

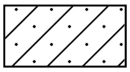
Soils

 Topsoil

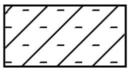
 Peat

 Clay

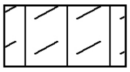
 Silty clay

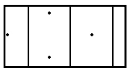
 Sandy clay

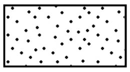
 Gravelly clay

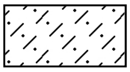
 Shaly clay

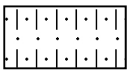
 Silt

 Clayey silt

 Sandy silt

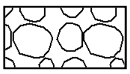
 Sand

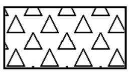
 Clayey sand

 Silty sand

 Gravel

 Sandy gravel

 Cobbles, boulders

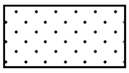
 Talus

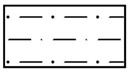
Sedimentary Rocks

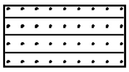
 Boulder conglomerate

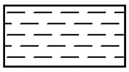
 Conglomerate

 Conglomeratic sandstone

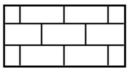
 Sandstone

 Siltstone

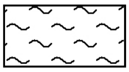
 Laminite

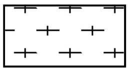
 Mudstone, claystone, shale

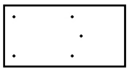
 Coal

 Limestone

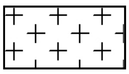
Metamorphic Rocks

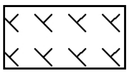
 Slate, phyllite, schist

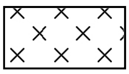
 Gneiss

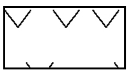
 Quartzite

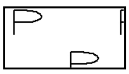
Igneous Rocks

 Granite

 Dolerite, basalt, andesite

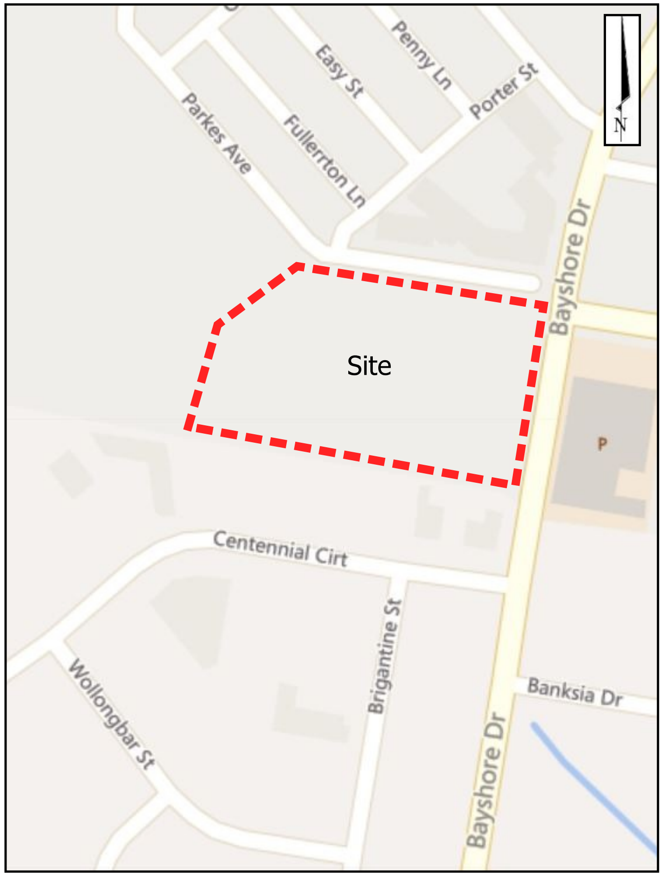
 Dacite, epidote

 Tuff, breccia

 Porphyry

Appendix B

Drawing 1 – Site and Test Location Plan



Site Locality

Legend:

Approximate Bore Location and Number

Image obtained from Metromap dated 24 August 2020.
Site locatlity not to scale.

Douglas Partners
Geotechnics | Environment | Groundwater

CLIENT: Hjorth Architects	**Site and Test Location Plan**	PROJECT No:	201414.00
OFFICE: Ballina	**TAFE CLC Tranche 3**	DRAWING No:	1
DATE: April 2021	**Bayshore Drive, Byron Bay**	REVISION:	0

Appendix C

Borehole Log Sheets (Bores 1, 4 and 6)

BOREHOLE LOG

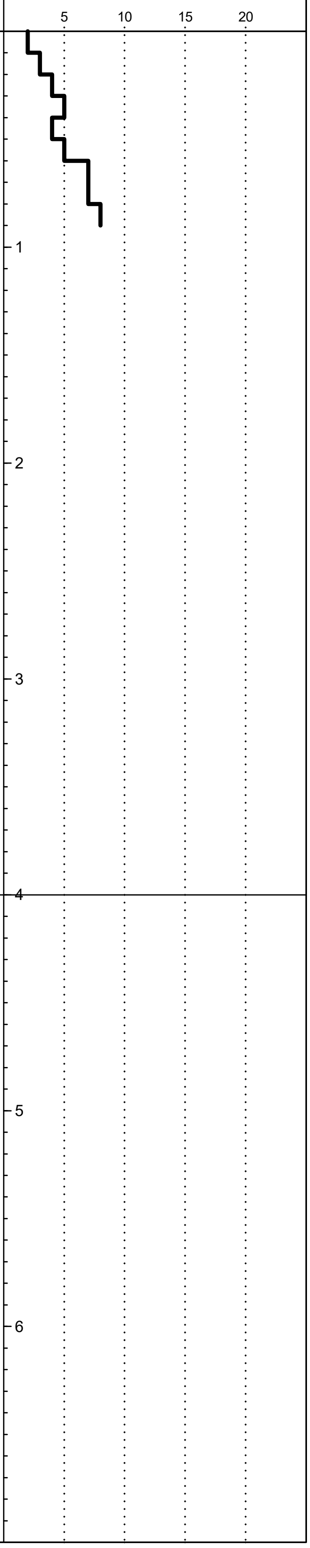
CLIENT: Brewster Hjorth Architects
PROJECT: TAFE CLC Tranche 3
LOCATION: Byshore Drive, Byron Bay

SURFACE LEVEL: 4.5 AHD
EASTING: 556828
NORTHING: 6832570
DIP/AZIMUTH: 90°/--

BORE No: 1
PROJECT No: 201414.00
DATE: 18-2-2021
SHEET 1 OF 1

Depth (m)	Description of Strata	Sampling Type	Sampling Depth	Sample	Results & Comments	Water
	FILL Sandy CLAY CL-CI: low to medium plasticity, dark brown with orange, fine to coarse sand with fine to medium gravel, moist, w>PL, appeared well compacted	D B	0.1 – 0.2		Samples collected at 0.5 m depth intervals to 2.5 m depth for ASS Assessment	
0.5	- wet		0.5 – 1.0			▼ (0.5 m)
0.6	- with clay bands					
1.1	Silty SAND SM: fine to medium grained, grey, wet, medium dense, alluvial	S	1.0 – 1.45		5,7,10 N = 17	
1.6	- dark brown	D	1.7			
2.6	- brown	S	2.5 – 2.95		4,8,10 N = 18	
4.0	Bore discontinued at 4.0m . Limit of investigation.					

Dynamic Penetrometer Test (blows per 100mm)



RIG: Christie Soil Rig **DRILLER:** Geoserve **LOGGED:** MM/BMc **CASING:** NA
TYPE OF BORING: Auger
WATER OBSERVATIONS: Groundwater encountered at 0.5 m depth.
REMARKS: Surface level interpolated from client supplied survery.

☐ Sand Penetrometer AS1289.6.3.3
☒ Cone Penetrometer AS1289.6.3.2

SAMPLING & IN SITU TESTING LEGEND

A	Auger sample	G	Gas sample	PID	Photo ionisation detector (ppm)
B	Bulk sample	P	Piston sample	PL(A)	Point load axial test Is(50) (MPa)
BLK	Block sample	U_x	Tube sample (x mm dia.)	PL(D)	Point load diametral test Is(50) (MPa)
C	Core drilling	W	Water sample	pp	Pocket penetrometer (kPa)
D	Disturbed sample	▷	Water seep	S	Standard penetration test
E	Environmental sample	▼	Water level	V	Shear vane (kPa)

Douglas Partners
Geotechnics / Environment / Groundwater

BOREHOLE LOG

CLIENT: Brewster Hjorth Architects
PROJECT: TAFE CLC Tranche 3
LOCATION: Byshore Drive, Byron Bay

SURFACE LEVEL: 4.5 AHD
EASTING: 556795
NORTHING: 6832575
DIP/AZIMUTH: 90°/--

BORE No: 4
PROJECT No: 201414.00
DATE: 18-2-2021
SHEET 1 OF 1

Depth (m)	Description of Strata	Type	Depth	Sample	Results & Comments	Water
0.0–0.4	FILL Sandy CLAY CL-CI: low to medium plasticity, dark brown with orange, fine to coarse sand, with fine to medium gravel, moist, w>PL, appeared moderately compacted	D	0.1–0.3		Samples collected at 0.5 m depth intervals to 2.5 m depth for ASS Assessment	
0.4–1.5	FILL Silty CLAY CL-CI: low to medium plasticity, orange with grey and red, with fine to medium grained sand, w<PL, appeared well compacted	B	0.4–1.0			
1.2	- wet	S	1.0–1.45		4,4,5 N = 9	▼ 1.2
1.5–6.0	Silty SAND SM: fine to medium grained, grey, wet, medium dense, alluvial					
1.6	- dark brown, indurated in part	S	2.5–2.95		16,21,25 N = 46	
6.0	Bore discontinued at 6.0m . Limit of investigation.					

Dynamic Penetrometer Test (blows per 100mm)

RIG: Christie Soil Rig **DRILLER:** Geoserve **LOGGED:** MM/BMc **CASING:** NA
TYPE OF BORING: Auger
WATER OBSERVATIONS: Groundwater encountered at 1.2 m depth.
REMARKS: Surface level interpolated from client supplied survery.

☐ Sand Penetrometer AS1289.6.3.3
☒ Cone Penetrometer AS1289.6.3.2

SAMPLING & IN SITU TESTING LEGEND

A	Auger sample	G	Gas sample	PID	Photo ionisation detector (ppm)
B	Bulk sample	P	Piston sample	PL(A)	Point load axial test Is(50) (MPa)
BLK	Block sample	U_x	Tube sample (x mm dia.)	PL(D)	Point load diametral test Is(50) (MPa)
C	Core drilling	W	Water sample	pp	Pocket penetrometer (kPa)
D	Disturbed sample	▷	Water seep	S	Standard penetration test
E	Environmental sample	▼	Water level	V	Shear vane (kPa)

Douglas Partners
Geotechnics / Environment / Groundwater

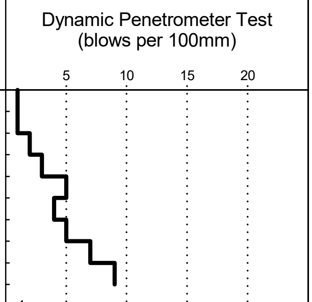
BOREHOLE LOG

CLIENT: Brewster Hjorth Architects
PROJECT: TAFE CLC Tranche 3
LOCATION: Byshore Drive, Byron Bay

SURFACE LEVEL: 4.5 AHD
EASTING: 556763
NORTHING: 6832584
DIP/AZIMUTH: 90°/--

BORE No: 6
PROJECT No: 201414.00
DATE: 18-2-2021
SHEET 1 OF 1

Depth (m)	Description of Strata	Type	Depth	Sample	Results & Comments	Water
0.2	FILL Sandy CLAY CL-CI: low to medium plasticity, dark brown with orange, fine to coarse grained sand, with fine to medium gravel, moist, w>PL, appeared poorly compacted		0.1		Samples collected at 0.5 m depth intervals to 2.5 m depth for ASS Assessment	
0.9	FILL Silty CLAY CL-CI: low to medium plasticity, orange with grey and red, with fine to medium grained sand, moist, w<PL, appeared well compacted					Water level at 0.9 m
1.0	- wet					
	Silty SAND SM: fine to medium grained, grey, wet, medium dense, alluvial	S	1.0		3,5,9 N = 14	
1.6	- dark brown					
		S	2.5		3,7,8 N = 15	
3.5	Bore discontinued at 3.5m . Limit of investigation.					



RIG: Christie Soil Rig **DRILLER:** Geoserve **LOGGED:** MM/BMc **CASING:** NA
TYPE OF BORING: Auger
WATER OBSERVATIONS: Groundwater encountered at 0.9 m depth.
REMARKS: Surface level interpolated from client supplied survery.

☐ Sand Penetrometer AS1289.6.3.3
☒ Cone Penetrometer AS1289.6.3.2

SAMPLING & IN SITU TESTING LEGEND

A	Auger sample	G	Gas sample	PID	Photo ionisation detector (ppm)
B	Bulk sample	P	Piston sample	PL(A)	Point load axial test Is(50) (MPa)
BLK	Block sample	U_x	Tube sample (x mm dia.)	PL(D)	Point load diametral test Is(50) (MPa)
C	Core drilling	W	Water sample	pp	Pocket penetrometer (kPa)
D	Disturbed sample	▷	Water seep	S	Standard penetration test
E	Environmental sample	▼	Water level	V	Shear vane (kPa)

Douglas Partners
Geotechnics / Environment / Groundwater

Appendix D

Laboratory Test Results

Douglas Partners
Geotechnics / Environment / Groundwater

Douglas Partners Pty Ltd
ABN 75 053 980 117
www.douglaspartners.com.au
Unit 4, 482 Scottsdale Drive
Varsity Lakes QLD 4227
Phone (07) 5568 8900
Fax (07) 5568 8999

FIELD pH SCREENING - ACID SULFATE SOILS

Client : Brewster Hjorth Architects
Project : Proposed TAFE CLC Tranche 3
Location : Bayshore Drive, Byron Bay

Project No. : 201414.00
Report No. : 201414.00-1
Report Date: 9.3.21

Tested By: NP
Date Tested: 4.3.21
Reported By: CW
Date Reported: 9.3.21

Sample No.	1	2	3	4	5	6	7	8
Chainage (m)								
Location	Bore 1	Bore 1	Bore 1	Bore 1	Bore 6	Bore 6	Bore 6	Bore 6
Offest (m)								
Level of test (m)(RL)	0.5	1.0	1.5	2.0	0.5	1.0	1.5	2.0
Soil Description	Sandy Clay	Sandy Clay	Sand	Silty Sand	Sandy Gravelly Clay	Silty Sand	Silty Sand	Silty Sand
Field pH (pH_F)	6.9	6.8	7.4	5.4	5.3	6.1	6.8	6.0
Oxidised pH (pH_{FOX})	4.6	4.4	5.5	3.9	4.3	4.8	4.9	4.2
Reactivity Code (see below)	4	4	1	1	1	1	1	1
AASS / PASS (see below)								

Reactivity codes: 1 = None to slight 2 = Moderate, 3 = Vigorous, 4 = Very vigorous (gas & heat generated)

F= Bubbling/ Frothy Reaction (organics) , N/A = Not Assessed

ASS / PASS intereptation : if pHf < 4.0 = ASS (actual acid sulphate soil) if pHfox < 3.0 = PASS (potential acid sulphate soil)

CHECKED BY:..................CW.......................................

MAZLAB Pty Ltd
Environmental Testing, Sampling and Drilling (Vibracore)
ABN 90 151 684 436 ACN 151 684 436

U1/ 33 MACHINERY DR., TWEED HEADS SOUTH, 2486
PO BOX 6879 TWEED HEADS SOUTH MC., 2486
PHONE: (07) 55239922 FAX: (07) 55239822
EMAIL: mazlab@bigpond.com

Client: Douglas Partners **Project:** Prop. TAFE, Bayshore Dr., Byron Bay (201414.00)

Mazlab Job No: DPB3147 **Date:** 12/03/2021

LABORATORY TEST RESULTS

Certificate of Test Results – Chromium Reducible Sulphur

Sample No.	Client I.D	Soil Description (truncated)	pH KCL	SCr mol (H+/t) %S	TAA mol (H+/t)	SNAS %S	ANC mol (H+/t) NA= Scr< action limit	Net Acidity mol (H+/t)	Liming Rate (Kg/ dry/ t)
46207	BH1-1.00	SAND(SP) brown/grey brown	5.8	<2 <0.01%	5 0.01%	-	-	5 0.01%	Nil
46208	BH1-2.00	SAND(SP) dark grey/black	4.5	<2 <0.01%	20 0.03%	-	-	20 0.03%	1.6
46209	BH6-0.50	Silty CLAY(CL) light brown (Decomposed Siltstone)	4.2	<2 <0.01%	35 0.06%	<0.02%	-	35 0.06%	2.7
46210	BH6-1.50	SAND(SP) grey	6.8	<2 <0.01%	-	-	NA	<2 <0.01%	Nil

: :

Laboratory Test Methods follow procedures described in : QASSIT – Acid Sulphate Soils Laboratory Methods Guidelines – Version 2.1 June 2004

Form Number MAZREP08

Material Test Report

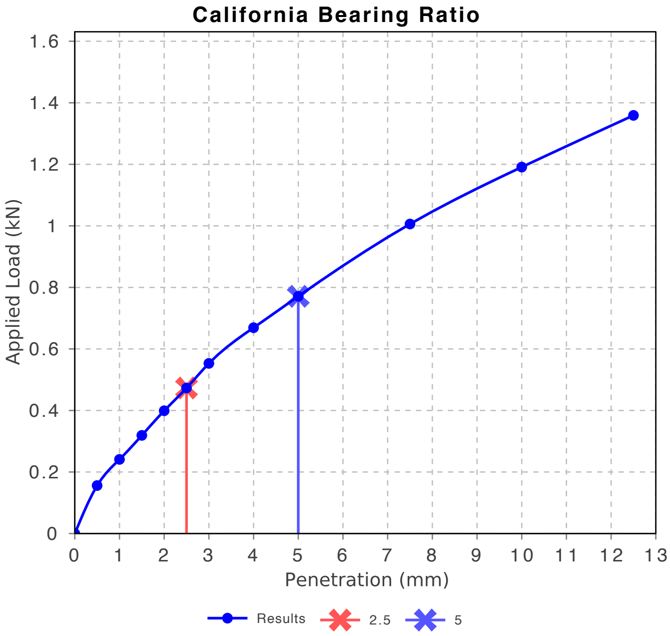
Report Number:	**201414.00-1**
Issue Number:	*2 - This version supersedes all previous issues*
Reissue Reason:	*amended*
Date Issued:	08/04/2021
Client:	Brewster Hjorth Architects Level 1, Surry Hills NSW 2010
Contact:	Nic Glass
Project Number:	201414.00
Project Name:	TAFE CLC Tranche 3
Project Location:	Bayshore Drive, Byron Bay
Work Request:	8043
Sample Number:	GL-8043A
Date Sampled:	01/03/2021
Dates Tested:	02/03/2021 - 12/03/2021
Sample Location:	**Bore 1, Depth: 0.1m - 0.5m**

Douglas Partners
Geotechnics / Environment / Groundwater
Douglas Partners Pty Ltd
Gold Coast Laboratory
Unit 2/3 Distribution Avenue Molendinar QLD 4214
Phone: (07) 5568 8900
Email: chad.whatley@douglaspartners.com.au

Accredited for compliance with ISO/IEC 17025 - Testing

Approved Signatory: Chad Whatley
Lab Manager
Laboratory Accreditation Number: 828

California Bearing Ratio (AS 1289 6.1.1 & 2.1.1)		Min	Max
CBR taken at	5 mm		
CBR %	**4.0**		
Method of Compactive Effort	Standard		
Method used to Determine MDD	AS 1289 5.1.1 & 2.1.1		
Method used to Determine Plasticity	Visual Assessment		
Maximum Dry Density (t/m^3)	1.82		
Optimum Moisture Content (%)	16.0		
Laboratory Density Ratio (%)	100.0		
Laboratory Moisture Ratio (%)	99.5		
Dry Density after Soaking (t/m^3)	1.83		
Field Moisture Content (%)	16.7		
Moisture Content at Placement (%)	15.9		
Moisture Content Top 30mm (%)	18.2		
Moisture Content Rest of Sample (%)	17.3		
Mass Surcharge (kg)	4.5		
Soaking Period (days)	4		
Curing Hours	70.2		
Swell (%)	0.0		
Oversize Material (mm)	19		
Oversize Material Included	Excluded		
Oversize Material (%)	4.9		



Atterberg Limit (AS1289 3.1.2 & 3.2.1 & 3.3.1)		Min	Max
Sample History	Oven Dried		
Preparation Method	Dry Sieve		
Liquid Limit (%)	27		
Plastic Limit (%)	11		
Plasticity Index (%)	**16**		

Linear Shrinkage (AS1289 3.4.1)		Min	Max
Moisture Condition Determined By	AS 1289.3.1.2		
Linear Shrinkage (%)	7.0		
Cracking Crumbling Curling	None		

Material Test Report

Douglas Partners
Geotechnics / Environment / Groundwater
Douglas Partners Pty Ltd
Gold Coast Laboratory
Unit 2/3 Distribution Avenue Molendinar QLD 4214
Phone: (07) 5568 8900
Email: chad.whatley@douglaspartners.com.au

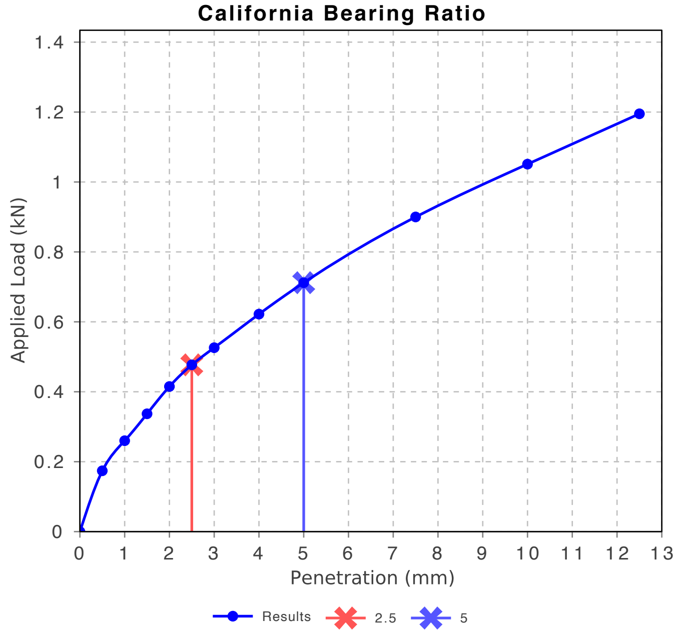
Report Number:	**201414.00-1**
Issue Number:	*2 - This version supersedes all previous issues*
Reissue Reason:	*amended*
Date Issued:	08/04/2021
Client:	Brewster Hjorth Architects Level 1, Surry Hills NSW 2010
Contact:	Nic Glass
Project Number:	201414.00
Project Name:	TAFE CLC Tranche 3
Project Location:	Bayshore Drive, Byron Bay
Work Request:	8043
Sample Number:	GL-8043B
Date Sampled:	01/03/2021
Dates Tested:	02/03/2021 - 12/03/2021
Sample Location:	**Bore 4, Depth: 0.4m - 1.0m**

ilac-MRA NATA

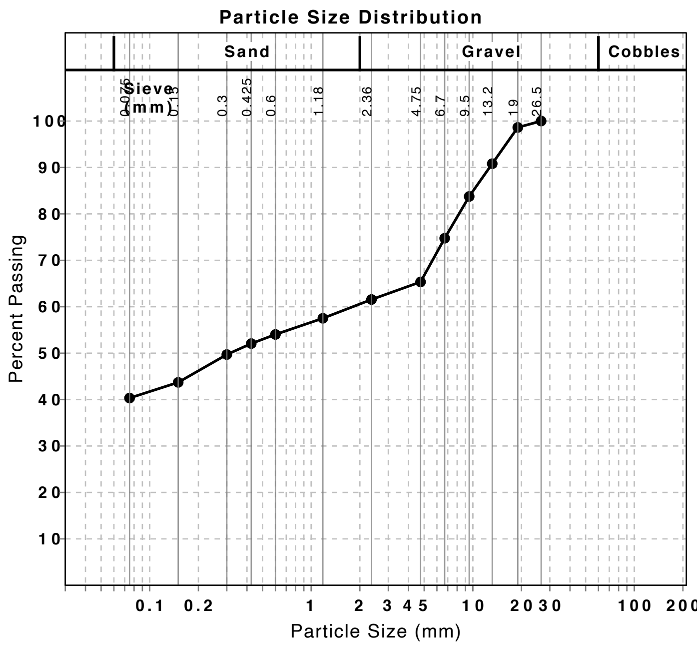
Accredited for compliance with ISO/IEC 17025 - Testing

Approved Signatory: Chad Whatley
Lab Manager
Laboratory Accreditation Number: 828

California Bearing Ratio (AS 1289 6.1.1 & 2.1.1)		Min	Max
CBR taken at	2.5 mm		
CBR %	**3.5**		
Method of Compactive Effort	Standard		
Method used to Determine MDD	AS 1289 5.1.1 & 2.1.1		
Method used to Determine Plasticity	Visual Assessment		
Maximum Dry Density (t/m^3)	1.81		
Optimum Moisture Content (%)	16.0		
Laboratory Density Ratio (%)	96.5		
Laboratory Moisture Ratio (%)	100.5		
Dry Density after Soaking (t/m^3)	1.69		
Field Moisture Content (%)	19.0		
Moisture Content at Placement (%)	16.3		
Moisture Content Top 30mm (%)	24.0		
Moisture Content Rest of Sample (%)	21.2		
Mass Surcharge (kg)	4.5		
Soaking Period (days)	4		
Curing Hours	113.3		
Swell (%)	3.5		
Oversize Material (mm)	19		
Oversize Material Included	Excluded		
Oversize Material (%)	0		



Particle Size Distribution (AS1289 3.6.1)						
Sieve	Passed %	Passing Limits		Retained %	Retained Limits	
26.5 mm	**100**			0		
19 mm	**99**			1		
13.2 mm	**91**			8		
9.5 mm	**84**			7		
6.7 mm	**75**			9		
4.75 mm	**65**			9		
2.36 mm	**62**			4		
1.18 mm	**58**			4		
0.6 mm	**54**			3		
0.425 mm	**52**			2		
0.3 mm	**50**			2		
0.15 mm	**44**			6		
0.075 mm	**40**			3		



Material Test Report

Report Number:	**201414.00-1**
Issue Number:	*2 - This version supersedes all previous issues*
Reissue Reason:	*amended*
Date Issued:	08/04/2021
Client:	Brewster Hjorth Architects Level 1, Surry Hills NSW 2010
Contact:	Nic Glass
Project Number:	201414.00
Project Name:	TAFE CLC Tranche 3
Project Location:	Bayshore Drive, Byron Bay
Work Request:	8043
Sample Number:	GL-8043B
Date Sampled:	01/03/2021
Dates Tested:	02/03/2021 - 12/03/2021
Sample Location:	**Bore 4, Depth: 0.4m - 1.0m**

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Accredited for compliance with ISO/IEC 17025 - Testing

Approved Signatory: Chad Whatley
Lab Manager
Laboratory Accreditation Number: 828

Atterberg Limit (AS1289 3.1.2 & 3.2.1 & 3.3.1)		Min	Max
Sample History	Oven Dried		
Preparation Method	Dry Sieve		
Liquid Limit (%)	38		
Plastic Limit (%)	22		
Plasticity Index (%)	**16**		

Linear Shrinkage (AS1289 3.4.1)		Min	Max
Moisture Condition Determined By	AS 1289.3.1.2		
Linear Shrinkage (%)	6.0		
Cracking Crumbling Curling	None		