Douglas Partners
Geotechnics / Environment / Groundwater

Report on Preliminary Geotechnical Investigation and Acid Sulfate Soils Assessment

Proposed School Structures,
High School, 33 Oxford Street, Kingscliff

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The undersigned, on behalf of Douglas Partners Pty Ltd, confirm that this document and all attached drawings, logs and test results have been checked and reviewed for errors, omissions and inaccuracies.

	Signature	Date
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Report on Preliminary Geotechnical Investigation and Acid Sulfate Soils Assessment

Proposed School Structures,

High School, 33 Oxford Street, Kingscliff

1. Introduction

This report presents the results of a preliminary geotechnical investigation and acid sulfate soils (ASS) assessment undertaken for proposed school structures to be located at Kingscliff High School (KHS), 33 Oxford Street, Kingscliff. The investigation was carried out at the request of The Department of Education/School Infrastructure NSW (DoE). The works were carried out in accordance with Douglas Partners Pty Ltd (DP) fee proposal GLD190291 dated 20 August 2019 and acceptance from Rina Rodriguez representing DoE on 4 November 2019.

Details on the proposed structures, building layouts, structural loads or earthwork levels were not known at the time of the preparation of this report.

The aim of the work was to carry out a preliminary geotechnical investigation and ASS assessment and provide comments on the following:

- subsurface conditions including groundwater (if encountered);
- site classification in accordance with AS 2870 (2011);
- indicative presence or otherwise of ASS and the requirement, if any, for additional assessment;
- likely potential for contaminants based on the laboratory test results and the requirements, if any, for additional assessment; and
- suitability of high and deep level foundations, allowable bearing pressure, skin friction and estimated settlements.

The investigation comprised the drilling of three bores (designated Bores 6 to 8) at client nominated locations.

Details of the field work and laboratory testing 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 should be kept in its entirety without separation of individual pages or sections.

2. Site Description

The KHS is bound by: Oxford Street and residential structures to the north; Kingscliff TAFE to the west; Cudgen Creek to the south; and bushland to the east (refer attached Drawing 1 in Appendix B). At the time of the investigation the sites were generally level and comprised: a carpark; a sports field; several existing learning blocks; and areas of short mown grass.

Figures 1 and 2 indicate typical site conditions encountered at the time of the investigation.

Figure 1: Typical site conditions looking southward from Bore 8.

Figure 2: Typical site conditions looking westward from Bore 8.

3. Regional Geology and ASS Mapping

Based on the Geological Survey New South Wales map, Tweed Heads 1:250,000 Series, the site is located within an area of Quaternary aged alluvium deposits comprising *:gravel, sand, silt, clay”*.

The subsurface conditions encountered during the field work, (refer Section 5), are in general agreement with the above anticipated and described geology.

The Department of Environment and Climate Change published Acid Sulfate Soil Risk Map, 1:25,000 scale, indicates the site lies within an area mapped as *“low probability of occurrence”*.

4. Field Work Methods

The field work for the investigation was undertaken on 7 March 2020 and comprised the drilling of three bores (designated Bores 6 to 8) at client nominated locations.

The bores were drilled using a 4WD utility mounted Christie drilling rig employing 100 mm solid flight auger techniques fitted with a Tungsten Carbide (TC) drill bit. Representative disturbed soil samples were collected in the bores for tactile assessment and subsequent laboratory testing. At the completion of the field work, the bores were backfilled with drill spoil. Dynamic cone penetrometer (DCP) tests were carried out adjacent to each bore to refusal depths of between 0.9 m and 2.3 m.

ASS samples were retrieved within the upper 2 m depth in the bores. Three samples were also retrieved at select bores and depths for preliminary contamination assessment.

The field work for the ASS assessment aspect of the investigation was carried out in general accordance with the NSW ASS Manual (1998) which refers to the Soils Management Guidelines (2014) and Laboratory Methods Guidelines (2004).

The test locations were determined with reference to existing site features with UTM positions recorded using a hand-held GPS accurate to approximately 5 m. The coordinates are presented on the borehole logs in Appendix C with the approximate test locations indicated on Drawing 2 in Appendix B. The approximate ground surface levels were obtained from Nearmap imagery dated 6 November 2019.

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)
	Fill	Silty Sand – medium dense (or denser)	Silty Clay – soft	
6	0.0 – 0.6[ii]	0.6 – 2.7	2.7 – 3.0[iii]	0.8
7	0.0 – 0.8	0.8 – 3.0[iii]	–	1.2
8	0.0 – 0.7	0.7 – 3.0[iii]	–	1.9

Notes (i) All depths were measured from existing site level at the time of the investigation.
(ii) Existing pavement form surface to 0.6 m depth.
(iii) Limit of investigation.

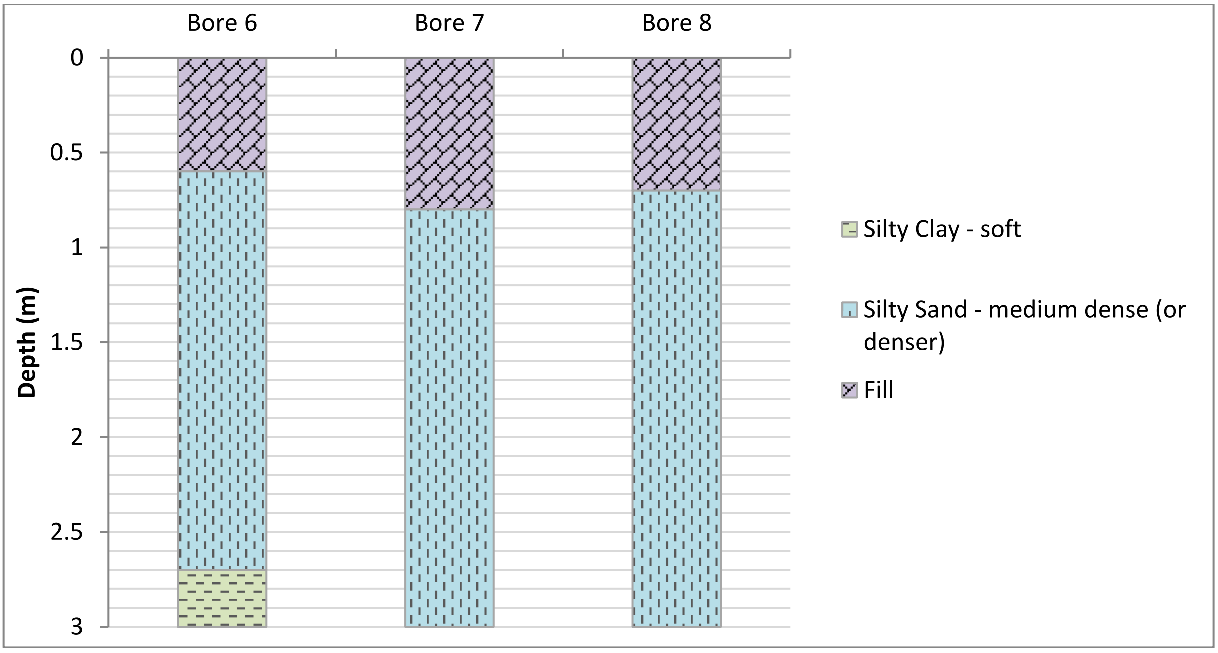

Figure 3: Graphical summary of subsurface conditions.

The fill encountered in Bore 6 comprised pavement gravel and sandy gravel to 0.6 m depth, whilst clayey/silty sand fill was encountered at Bores 7 and 8. It is probable that the fill at Bore 6 was placed under 'controlled' conditions as part of the existing pavement construction, and the fill encountered at Bores 7 and 8 appears moderately well compacted. However, in the absence of documentation to confirm the fill was engineered and placed under 'controlled' conditions and meets the requirements of structural fill as defined in AS 3798 (2007), it must be deemed 'uncontrolled'.

Groundwater was encountered within all bores during auger drilling at depths between 0.8 m and 1.8 m. It should be noted that groundwater depths and ground moisture conditions are affected by climatic conditions and tidal influences (which may be present at this site) and will therefore vary with time. Seepage may also occur along the fill/natural and the sand/clay interface during and after periods of wet weather.

6. Laboratory Testing

The following laboratory testing was carried out on select samples retrieved during the investigation:

- ASS screening;
- Chromium suite; and
- preliminary contamination testing, with selected samples analysed for a suite of contaminants of potential concern (CoPC), as detailed in Section 7.7.

Detailed laboratory test results are provided in laboratory report sheets in Appendix D and the ASS test results summarised in Table 2 in Appendix D. The results of the contamination are summarised in Tables 3 and 4 in Appendix D.

7. Comments

7.1 Site Classification

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

In strict accordance with AS 2870 (2011), due to the presence of 'uncontrolled' gravel and sand fill up to 0.8 m depth, the site (as a whole) in its present state would be given a 'Class P' classification, and will require design by engineering principles, unless all footings penetrate the 'uncontrolled' fill and are founded in natural material. The below is provided for information purposes only.

Based on past experience it is anticipated that the silty/clayey sand and gravelly fill and sandy soils may have characteristic surface movement (y_s) values up to 20 mm, which is consistent with a 'Class S' site (slightly reactive) classification. Where the existing fill is removed, moisture conditioned and replaced under controlled engineered conditions, the y_s values are also estimated to be in the order of up to 20 mm, also consistent with a 'Class S' classification. This is due to the need to consider uncracked conditions for the first five years after fill placement and two years after cut.

If 'abnormal' soil moisture conditions are experience, 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 preparations. CSIRO (2003) on site management for homeowners provides useful advice for this site.

Final site classification will be a function of the building pad geometry and reactivity of the material comprising the building pad. The above indicative y_s values are intended as a guide and a final classification assessment will be required following bulk earthworks at the site.

7.2 Excavatability

Based on the conditions encountered within the depth of the bores, it is assessed that bulk excavation may be undertaken by small to medium sized excavation plant such as backhoes and 8 to 15 tonne hydraulic excavators.

It should be recognised that the above excavatability estimates are based on materials encountered at the test locations only and that conditions may prove more difficult (or easier) for excavatability beyond these test locations.

7.3 Re-Use of Cut Materials and Workability

Based on the results of the bores, it is expected that the material won from excavation will comprise clayey/silty sandy fill or natural silty sands.

It is expected that the excavated materials could all be re-used as structural fill. Such re-use is contingent upon the fill being free of any unsuitable and organically rich materials, particle size distribution being controlled along with moisture content, and upon minimum placement and compaction requirements being met, as indicated in Section 0.

Care should be taken not to over-wet any clayey soils which can lead to problems associated with trafficability and workability. Any clay soils, if applicable, should not be over-compacted (i.e. not more than 102% Standard) or dry of optimum, as this can lead to future swelling and softening under changes to moisture content or inundation from water. Clay subgrades (not expected) should be promptly overlaid with 150 mm of select granular fill (minimum CBR 15%) to reduce potential wetting and trafficability problems.

If clayey fill is proposed to be used in fill platforms beneath any structures, it is recommended that cohesive material be placed at depth and well graded, granular material be placed close to footing level. This will reduce the effects of seasonal moisture change and foundation soil reactivity and will improve surface trafficability.

The existing fill and silty sand materials on site are expected to be untrafficable to tyred vehicles during and after periods of rainfall or other increases in subgrade moisture content, and hence, the use of tracked plant may be preferable. Soils which become wet will need to be allowed to dry out or be replaced.

7.4 Subgrade Preparation and Fill Placement or Recompaction

Where upper level footings are to be used and on-ground slabs are adopted, it is recommended that the following site preparation be carried out for structural foundations and pavement subgrades:

- Removal of any deleterious, ‘uncontrolled’ fill, topsoil, wet or highly compressible material, or material rich in organics or root matter. Localised ‘uncontrolled’ fill was encountered up to 0.8 m depth. If the fill is left in place, suspending footings and slabs on piles founded into competent strata would be recommended.
- Test roll the subgrade beneath any proposed fill or structure, in order to detect the presence of any further soft or loose zones, which should be ripped, dried and recompacted, or excavated and replaced with compacted select fill as appropriate. Test rolling should be carried out with a smooth drum roller with a minimum static weight of 12 tonne. The tyned natural foundation soil should be compacted to a minimum density index of 80% (sands) or minimum dry density ratio of 98% Standard (clays).
- Assess moisture content of any underlying clay subgrade, which should be adjusted to within ±2% of OMC, where OMC is the optimum moisture content at standard compaction.
- Place approved fill in layers not exceeding 300 mm loose thickness, with each layer compacted to a minimum dry density ratio of 98% standard (clay) or 80% density index (sand).
- Control compaction of clays as over-compacted clays (i.e. minimum dry density ratio of >102% Standard) may swell significantly and lose strength if they are wetted after compaction, potentially changing the site classification and reducing subgrade strengths assumed in design, and therefore should be avoided.
- Seal or cover any compacted clay fill at or close to footing formation level as soon as practicable, to reduce the opportunity for occurrence of desiccation and cracking (refer Section 7.1).
- Undertake ‘Level 1’ inspection and testing as detailed in AS 3798 (2007) where new fill is required to support structures or pavement.

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

7.5 Foundations

7.5.1 General

Depending on structural loads and earthworks, both high and deep level footings may be adopted on the site. High level footings would be required to be founded below the existing ‘uncontrolled’ fill that was encountered up to 0.8 m depth. Otherwise, deeper footings (refer Section 7.5.3) would be required.

Where footings are founded in materials of differing compressibility, there is potential for differential settlement across the structure, which must be accounted for in design through careful articulation and choice of construction materials for use in the structures. If the structures are susceptible to differential movements, then it is recommended all footings be founded in the same material, whether through the use of piled footings, or a combination of high level footings and piles.

7.5.2 High Level Footings

Based on the materials encountered within the bores and subject to the earthwork recommendations being carried out as per Section 0, it is considered that high level pad footings up to 2 m wide and strip footings up to 1 m be designed using the allowable values indicated in Table 5. Localised deepening of strip footings (including the use of pedestal piers or backhoe piers could be considered) in areas of deeper fill provided the groundwater level is below the depth of excavation.

Structural footings and slab design should consider the final lot classification following bulk earthworks and include appropriate stiffening for the classifications.

Table 5: High Level Footing Design Bearing Pressures (Allowable)

Material Description	Allowable Bearing Pressure (kPa)
'Uncontrolled' Fill	Not recommended
'Controlled' Fill(i)	100
Sand – medium dense (or denser) (ii)	125 (strip footing) 250 (pad footing)

Note (i) Assuming controlled fill is placed under 'Level 1' requirements under inspection and testing by DP in accordance with the recommendations of this report.

(ii) Founded at a minimum 1 m depth.

For high level footings sized and loaded as above, it is considered that settlements under such applied loading will be less than 1% to 2% of footing width. Where footings wider than 2 m are adopted, specific assessment based on actual applied pressure and ground conditions at that location is recommended to assess specific settlement characteristics.

In addition to the above settlement estimated, the long term settlement of any bulk fill must be added if filling is proposed. Where bulk fill is placed under 'controlled' conditions, there is potential for 'creep' of the fill material as the fill settles over time under self-weight. Such settlement is expected to be in the order of approximately 0.5% to 1% of the fill thickness over a period of 10 to 20 years for well compacted clay fill and less for granular fill.

7.5.3 Deep Footings

7.5.3.1 General

As an option to high level footings, suitable piled foundation options may comprise steel screw piles for lighter loads, or otherwise driven timber piles or grout injected CFA piles for larger. Bored piles are not recommended as a foundation option, as installation is expected to be difficult due to the relatively high water table and potentially collapsible soil requiring the use of full length liners to prevent hole collapse and pumps to control water inflow. Ground vibrations and noise associated with the installation of driven piles will need to be considered and deeper investigation is recommended to confirm suitable founding depths. CFA grout injected piles are essentially vibration free but more costly to install.

It is suggested that if any of these pile types are considered that advice should be sought from an experienced piling contractor regarding the issues of potential vibration damage. It would also be prudent to undertake a building condition survey (dilapidation report) prior to undertaking any works of any sensitive nearby structures and/or inground services.

Pile capacities and suitable pile types should be confirmed by prospective piling contractors based on their particular equipment. Experience indicates that settlements of well constructed piles (loaded as per below) are unlikely to exceed the order of 1% of the pile diameter.

Groundwater inflow into excavations may occur if the work is preceded by significant wet weather, and piling contractors should include a provision for the use of temporary steel liners and cleaning buckets to remove groundwater, if required.

It should be noted that estimated soft clay was encountered at 2.7 m depth at Bore 6, therefore, if deep footings are to be used in this area, deeper investigation is recommended to confirm a suitable founding strata.

It is essential that foundation excavations (where applicable) be inspected by experienced geo-technical personnel to ensure the design parameters adopted are suitable for the ground conditions and to ensure that there is no soft or loose material remaining at the base of the excavations or smear on the side walls. Ground conditions can vary, and it is essential that adequate provision be made throughout the project to vary foundations to suit differing ground conditions.

7.5.3.2 Steel Screw Piles

The use of steel screw piles with a pile cap could be adopted, depending on structural loads, and are dependent on the structure requiring minimal lateral resistance. Similarly, screw piles could be splayed with a pile cap to provide resistance to uplift. Steel screw pile capacity is a function of foundation density/strength and depth. Steel screw piles with a helix diameter of 0.6 m founded at 1.5 m depth may be designed using an allowable end bearing pressure of 200 kPa in the medium dense silty sand encountered in Bores 7 and 8.

Care is required with structures to avoid bearing capacity failure, if they are to be founded on screw piles near Bore 6, as the ‘soft’ clay at depth will fail in shear if loaded to greater than about 25 kPa. Hence, the bearing capacity of the soil subgrade overall in this area will depend upon the following:

- the width/diameter of the applied load (since the greater the width, the greater the risk of excessive bearing pressures being transferred to the underlying ‘soft’ clay, etc);
- the punching resistance of any slab (does not apply to a pad/screw pile footing);
- the thickness of material between the applied load and ‘soft’ ground and hence its capability to reduce applied pressure on the underlying ‘soft’ materials; and
- the proximity of the applied load to the edge of any slab.

In the latter case, where excessive load is applied over sufficient width close to the edge of the slab, then the contribution of the slab towards bridging is minimised and localised shear failure will occur.

As preliminary guidance for the design of screw piles near Bore 6, shear failure will occur in the underlying ‘soft’ clay soil if the pile diameter and thickness of bridging material are outside the suggested guidelines in Table 6 below. Maximum pile diameters are suggested in relation to the thickness of the bridging material between the underside of the screw pile and the top of the ‘soft’ clay. Alternatively, a minimum thickness of bridging material is suggested for any given diameter of screw pile. These suggested constraints are presented for a maximum applied bearing pressure of 200 kPa (working stress). It should be recognised, however, that settlements may become intolerably high before shear failure can occur.

Table 6: Suggested Geometry and Bridging Cover for Screw Piles Over 'soft' clay Near Bore 6

Design Bearing Pressure (kPa)	Maximum Pile Diameter (D)	Minimum Bridging Cover Above ‘soft’ Clay (H)
200	0.60 H	1.63 D

An advantage of this form of pile installation is that high ground water levels have no effect on pile construction. Disadvantages of the method are the slender shaft dimensions and the ability to penetrate dense materials.

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 very dense sand) 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 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.

The structural capacity of steel screw piles should be checked and due allowance made for inclined or eccentric loads, and possible corrosion effects.

Lateral capacity of vertical 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 average buoyant (assuming a high groundwater table in the worst case) soil density of 9 kN/m^3 and friction of 30°. Reference is made to the comments above regarding appropriate factors for working stress or limit state design analysis.

It should be noted that AS 2159–2009 requires compressive load testing of piles to be undertaken to a test load of $Ed/\Phi g$. For a geotechnical strength reduction factor (Φg) of 0.5, this test load is twice the design action effect (Ed). 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 Ed 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.

7.5.3.3 CFA Piles

Grout injected continuous flight auger, (CFA) piles are essentially vibration free, because an auger is drilled into the ground to the required founding level. When design founding level is reached auger rotation is stopped, then the auger is withdrawn, bringing the disturbed soil with it, and the void below is replaced with grout which is pumped under pressure through the hollow stem of the auger. Because CFA piles are a non-displacement method, there is no spoil generated that needs to be removed from the site and the need for bentonite or liners is mitigated. CFA piles also have the advantage as they are not affected by high water tables. Care is required in installing CFA piles into hard strata to ensure the pile advance is sufficient compared to the rotation to avoid ‘ground loss’, whereby significant material is brought to the surface by the auger flights, resulting in significant disturbance of the ground.

CFA piles founded a minimum 1.5 m depth, could be designed using an allowable end bearing pressure of 200 kPa in the medium dense (or denser) silty sand near Bores 7 and 8. Similarly, when founding near Bore 6 with CFA piles, the before mentioned comments in Section 7.5.3.1 regarding ‘soft’ clay and suggested geometry given in Table 6 will apply. Where limit state methods are used to design the piles, the ultimate geotechnical strength ($R_{d,ug}$) can be calculated by multiplying the allowable bearing pressure by the adopted safety factor of 2.5, and then by a suitable geotechnical strength reduction factor (ϕ_g) to obtain the design geotechnical strength ($R_{d,g}$). After assessing the overall design average risk rating in accordance with the guidelines presented in AS2159 (2009), a minimum Φ_g value of 0.5 is suggested for the site.

7.6 Assessment of Preliminary ASS Results

7.6.1 General

A preliminary ASS assessment was carried out to assess the presence of ASS across the proposed development area. ASS sampling was carried out in the upper 2 m depth of all bores.

7.6.2 Soil pH Screening Tests

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 twelve samples were retrieved and tested. Testing was carried out on predominant soil horizons encountered during the investigation. The results are summarised in Table 2 in Appendix D.

- **pH in distilled water (pH_F)** features 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 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.

- **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.

  Two of the twelve samples tested were were below the trigger value of 3, however, the samples had a “none to slight” reaction with peroxide.

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

7.6.3 Chromium Reducible Sulfur Suite Analysis of Soils

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

The action criterion on which the presence of ASS is made from the Chromium suite test results with the results that may trigger a requirement for an ASSMP based on the Soil Management Guidelines and the Laboratory Methods Guidelines. The Soil Management Guidelines, 2014 indicate that the assessment of whether or not a soil is AASS or PASS should be determined using only the ‘existing plus potential’ acidity calculation, and not the former ‘net acidity’ calculation, unless the ANC term (as used in the ‘net acidity’ calculation) meets certain requirements. The guideline indicates that the ANC term is only used if the soil contains fine (i.e. <0.5 mm in size) shell fragments or other similar fine calcium carbonate material, such as skeletal fragments, coral and foraminifera. Furthermore, the guidelines specifically cautions against relying on the ANC term when the material comprises clay without any discernible carbonate material.

In the absence of the site soils meeting the above, the existing plus potential acidity equation (i.e. S_{CR} + TAA + S_{NAS}) of greater than or equal to 0.03%S (sulfur trail) or 18 mol H^+/tonne (acid trail) has been adopted for this site.

Where: S_{CR} = Chromium Reducible Sulfur
TAA = Titratable Actual Acidity
S_{NAS} = Net Acid Soluble Sulfur (retained acidity)

Based on the above, the existing plus potential acidity was calculated (refer Table 2 in Appendix D) to be less than 0.03%S in all of the samples tested, as such, it is considered that an ASS management plan (ASSMP) is not necessary if bulk excavation levels do not exceed 1 m depth. If bulk excavations of greater than 1 m depth are proposed, additional assessment would be required.

7.6.4 Groundwater

The presence of ASS soils could lead to leaching of acid and metals during and after rainfall events. Accordingly, groundwater should not be discharged off site without prior testing in order to confirm the suitability of the water quality in accordance with Council guidelines/criteria.

7.7 Preliminary Contamination

Testing of soil contamination was undertaken by analysing three selected soil samples for a suite of commonly encountered contaminants of potential concern (CoPC), which included the following:

- Metals/metalloids (As, Cd, Cr, Cu, Pb, Hg, Ni and Zn);
- Total recoverable hydrocarbons (TRH);
- Benzene, toluene, ethylbenzene and xylenes (BTEX);
- Polycyclic aromatic hydrocarbons (PAH);
- Organochloride pesticides (OCP);
- Organophosphate pesticides (OPP);
- Polychlorinated biphenyls (PCB);
- Phenols; and
- Asbestos.

The samples were analysed by a NATA-accredited laboratory and the analytical results were compared to the following relevant Tier 1 screening criteria in accordance with the National Environment Protection Council (NEPC) (2013) *National Environmental Protection (Assessment of Site Contamination) Measure 1999 (as amended 2013)*:

1. Human Health

- Health Investigation Levels (HILs) for a sensitive land use exposure scenario (HIL-A), which includes all schools including preschools and primary schools;
- Health Screening Levels (HSLs) for a ‘secondary school building’ exposure scenario (HSL-A); and
- Asbestos was assessed on the basis of presence/absence. Asbestos HSLs were not adopted as detailed asbestos quantification was not undertaken.

2. Ecological – terrestrial ecosystems

- Ecological Investigation Levels (EILs) for urban residential/ public open space land uses;
- Ecological Screening Levels (ESLs) for TRH fractions, naphthalene, and BTEX in coarse soils for urban residential, parkland and public open space land uses; and
- These thresholds would provide an indication of ‘worst-case-scenario’ potential ecological risks. In the absence of ambient background concentrations (ABC), added concentration limits (ACL) for a soil pH of 4.0, a cation exchange capacity of 5 cmolc/kg and clay content of 10% have been adopted.

The analytical results are summarised in the attached Tables 6 and 7 in Appendix D. The results indicated the following:

- Metals/metalloids: all analytical results were below the screening criteria;
- TRH: all analytical results were below the screening criteria;
- BTEX: all analytical results were below the screening criteria;
- PAH: all analytical results were below the screening criteria;
- OCP: all analytical results were below the screening criteria;
- OPP: all analytical results were below the screening criteria;
- PCB: all analytical results were below the screening criteria;
- Phenols: all analytical results were below the screening criteria;
- Asbestos: asbestos was not detected in the samples analysed.

Based on the results of the preliminary assessment for soil contamination, DP considers that there is a low risk of contamination at the site. However, it is noted that the contamination assessment was preliminary in nature and has been undertaken without knowledge of the specific details of the proposed development nor the sites history. Due to the presence of fill material of unknown origin at the site, there is some potential for undetected sources of contamination. Further, the number of samples taken and tested may be insufficient depending on the volume of material that may be disturbed, if applicable. Also, additional assessment may be required if any material is to be removed from the site, and a waste classification assessment of the material will be required. Douglas Partners recommends that when specific details of the proposed development are known, location of new buildings, earthwork levels and volumes of material to be disturbed, a Preliminary Site Investigation (PSI) for contamination, also known as a Stage 1 assessment, should be undertaken. The analytical results of the current assessment would be incorporated into the PSI.

8. Limitations

Douglas Partners (DP) has prepared this preliminary report for this project at KHS, 33 Oxford Street, Kingscliff in accordance with DP's proposal GLD190291 dated 20 August 2019 in accordance with The Department of Education/School Infrastructure NSW (DoE) Standard Form Agreement. This report is provided for the exclusive use of DoE or their consulting engineers, as authorised, 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 scope for work for this report did not include the assessment of surface or subsurface 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.

The results provided in the report are indicative of the sub-surface conditions on the site only at the specific 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. 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 testing locations. The advice may also be limited by budget constraints imposed by others or by site accessibility.

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 contents of this report do not constitute formal design components such as are required, by the Health and Safety Legislation and Regulations, to be included in a Safety Report specifying the hazards likely to be encountered during constructions and the controls required to mitigate risk. This design process requires risk assessment to be undertaken, with such assessment being dependent upon factors relating to likelihood of occurrence and consequences of damage to property and to life. This, in turn, requires project data and analysis presently beyond the knowledge and project role respectively of DP. DP may be able, however, to assist the client in carrying out a risk assessment of potential hazards contained in the Comments section of this report, as an extension to the current scope of works, if so requested, and provided that suitable additional information is made available to DP. Any such risk assessment would, however, be necessarily restricted to the geotechnical and environmental components set out in this report and to their application by the project designers to project design, construction, maintenance and demolition.

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Appendix A

About This Report
Sampling Methods
Soil Descriptions
Symbols and Abbreviations
CSIRO Sheet BTF 18 ‘Foundation Maintenance and Footing Performance’

Foundation Maintenance and Footing Performance:
A Homeowner's Guide

BTF 18 replaces Information Sheet 10/91

Buildings can and often do move. This movement can be up, down, lateral or rotational. The fundamental cause of movement in buildings can usually be related to one or more problems in the foundation soil. It is important for the homeowner to identify the soil type in order to ascertain the measures that should be put in place in order to ensure that problems in the foundation soil can be prevented, thus protecting against building movement.

This Building Technology File is designed to identify causes of soil-related building movement, and to suggest methods of prevention of resultant cracking in buildings.

Soil Types

The types of soils usually present under the topsoil in land zoned for residential buildings can be split into two approximate groups – granular and clay. Quite often, foundation soil is a mixture of both types. The general problems associated with soils having granular content are usually caused by erosion. Clay soils are subject to saturation and swell/shrink problems.

Classifications for a given area can generally be obtained by application to the local authority, but these are sometimes unreliable and if there is doubt, a geotechnical report should be commissioned. As most buildings suffering movement problems are founded on clay soils, there is an emphasis on classification of soils according to the amount of swell and shrinkage they experience with variations of water content. The table below is Table 2.1 from AS 2870, the Residential Slab and Footing Code.

Causes of Movement

Settlement due to construction

There are two types of settlement that occur as a result of construction:

- Immediate settlement occurs when a building is first placed on its foundation soil, as a result of compaction of the soil under the weight of the structure. The cohesive quality of clay soil mitigates against this, but granular (particularly sandy) soil is susceptible.
- Consolidation settlement is a feature of clay soil and may take place because of the expulsion of moisture from the soil or because of the soil's lack of resistance to local compressive or shear stresses. This will usually take place during the first few months after construction, but has been known to take many years in exceptional cases.

These problems are the province of the builder and should be taken into consideration as part of the preparation of the site for construction. Building Technology File 19 (BTF 19) deals with these problems.

Erosion

All soils are prone to erosion, but sandy soil is particularly susceptible to being washed away. Even clay with a sand component of say 10% or more can suffer from erosion.

Saturation

This is particularly a problem in clay soils. Saturation creates a bog-like suspension of the soil that causes it to lose virtually all of its bearing capacity. To a lesser degree, sand is affected by saturation because saturated sand may undergo a reduction in volume – particularly imported sand fill for bedding and blinding layers. However, this usually occurs as immediate settlement and should normally be the province of the builder.

Seasonal swelling and shrinkage of soil

All clays react to the presence of water by slowly absorbing it, making the soil increase in volume (see table below). The degree of increase varies considerably between different clays, as does the degree of decrease during the subsequent drying out caused by fair weather periods. Because of the low absorption and expulsion rate, this phenomenon will not usually be noticeable unless there are prolonged rainy or dry periods, usually of weeks or months, depending on the land and soil characteristics.

The swelling of soil creates an upward force on the footings of the building, and shrinkage creates subsidence that takes away the support needed by the footing to retain equilibrium.

Shear failure

This phenomenon occurs when the foundation soil does not have sufficient strength to support the weight of the footing. There are two major post-construction causes:

- Significant load increase.
- Reduction of lateral support of the soil under the footing due to erosion or excavation.
- In clay soil, shear failure can be caused by saturation of the soil adjacent to or under the footing.

GENERAL DEFINITIONS OF SITE CLASSES

Class	Foundation
A	Most sand and rock sites with little or no ground movement from moisture changes
S	Slightly reactive clay sites with only slight ground movement from moisture changes
M	Moderately reactive clay or silt sites, which can experience moderate ground movement from moisture changes
H	Highly reactive clay sites, which can experience high ground movement from moisture changes
E	Extremely reactive sites, which can experience extreme ground movement from moisture changes
A to P	Filled sites
P	Sites which include soft soils, such as soft clay or silt or loose sands; landslip; mine subsidence; collapsing soils; soils subject to erosion; reactive sites subject to abnormal moisture conditions or sites which cannot be classified otherwise

Tree root growth
Trees and shrubs that are allowed to grow in the vicinity of footings can cause foundation soil movement in two ways:

- Roots that grow under footings may increase in cross-sectional size, exerting upward pressure on footings.
- Roots in the vicinity of footings will absorb much of the moisture in the foundation soil, causing shrinkage or subsidence.

Unevenness of Movement

The types of ground movement described above usually occur unevenly throughout the building's foundation soil. Settlement due to construction tends to be uneven because of:

- Differing compaction of foundation soil prior to construction.
- Differing moisture content of foundation soil prior to construction.

Movement due to non-construction causes is usually more uneven still. Erosion can undermine a footing that traverses the flow or can create the conditions for shear failure by eroding soil adjacent to a footing that runs in the same direction as the flow.

Saturation of clay foundation soil may occur where subfloor walls create a dam that makes water pond. It can also occur wherever there is a source of water near footings in clay soil. This leads to a severe reduction in the strength of the soil which may create local shear failure.

Seasonal swelling and shrinkage of clay soil affects the perimeter of the building first, then gradually spreads to the interior. The swelling process will usually begin at the uphill extreme of the building, or on the weather side where the land is flat. Swelling gradually reaches the interior soil as absorption continues. Shrinkage usually begins where the sun's heat is greatest.

Effects of Uneven Soil Movement on Structures

Erosion and saturation
Erosion removes the support from under footings, tending to create subsidence of the part of the structure under which it occurs. Brickwork walls will resist the stress created by this removal of support by bridging the gap or cantilevering until the bricks or the mortar bedding fail. Older masonry has little resistance. Evidence of failure varies according to circumstances and symptoms may include:

- Step cracking in the mortar beds in the body of the wall or above/below openings such as doors or windows.
- Vertical cracking in the bricks (usually but not necessarily in line with the vertical beds or perpends).

Isolated piers affected by erosion or saturation of foundations will eventually lose contact with the bearers they support and may tilt or fall over. The floors that have lost this support will become bouncy, sometimes rattling ornaments etc.

Seasonal swelling/shrinkage in clay
Swelling foundation soil due to rainy periods first lifts the most exposed extremities of the footing system, then the remainder of the perimeter footings while gradually permeating inside the building footprint to lift internal footings. This swelling first tends to create a dish effect, because the external footings are pushed higher than the internal ones.

The first noticeable symptom may be that the floor appears slightly dished. This is often accompanied by some doors binding on the floor or the door head, together with some cracking of cornice mitres. In buildings with timber flooring supported by bearers and joists, the floor can be bouncy. Externally there may be visible dishing of the hip or ridge lines.

As the moisture absorption process completes its journey to the innermost areas of the building, the internal footings will rise. If the spread of moisture is roughly even, it may be that the symptoms will temporarily disappear, but it is more likely that swelling will be uneven, creating a difference rather than a disappearance in symptoms. In buildings with timber flooring supported by bearers and joists, the isolated piers will rise more easily than the strip footings or piers under walls, creating noticeable doming of flooring.

Trees can cause shrinkage and damage

As the weather pattern changes and the soil begins to dry out, the external footings will be first affected, beginning with the locations where the sun's effect is strongest. This has the effect of lowering the external footings. The doming is accentuated and cracking reduces or disappears where it occurred because of dishing, but other cracks open up. The roof lines may become convex.

Doming and dishing are also affected by weather in other ways. In areas where warm, wet summers and cooler dry winters prevail, water migration tends to be toward the interior and doming will be accentuated, whereas where summers are dry and winters are cold and wet, migration tends to be toward the exterior and the underlying propensity is toward dishing.

Movement caused by tree roots
In general, growing roots will exert an upward pressure on footings, whereas soil subject to drying because of tree or shrub roots will tend to remove support from under footings by inducing shrinkage.

Complications caused by the structure itself
Most forces that the soil causes to be exerted on structures are vertical – i.e. either up or down. However, because these forces are seldom spread evenly around the footings, and because the building resists uneven movement because of its rigidity, forces are exerted from one part of the building to another. The net result of all these forces is usually rotational. This resultant force often complicates the diagnosis because the visible symptoms do not simply reflect the original cause. A common symptom is binding of doors on the vertical member of the frame.

Effects on full masonry structures
Brickwork will resist cracking where it can. It will attempt to span areas that lose support because of subsided foundations or raised points. It is therefore usual to see cracking at weak points, such as openings for windows or doors.

In the event of construction settlement, cracking will usually remain unchanged after the process of settlement has ceased.

With local shear or erosion, cracking will usually continue to develop until the original cause has been remedied, or until the subsidence has completely neutralised the affected portion of footing and the structure has stabilised on other footings that remain effective.

In the case of swell/shrink effects, the brickwork will in some cases return to its original position after completion of a cycle, however it is more likely that the rotational effect will not be exactly reversed, and it is also usual that brickwork will settle in its new position and will resist the forces trying to return it to its original position. This means that in a case where swelling takes place after construction and cracking occurs, the cracking is likely to at least partly remain after the shrink segment of the cycle is complete. Thus, each time the cycle is repeated, the likelihood is that the cracking will become wider until the sections of brickwork become virtually independent.

With repeated cycles, once the cracking is established, if there is no other complication, it is normal for the incidence of cracking to stabilise, as the building has the articulation it needs to cope with the problem. This is by no means always the case, however, and monitoring of cracks in walls and floors should always be treated seriously.

Upheaval caused by growth of tree roots under footings is not a simple vertical shear stress. There is a tendency for the root to also exert lateral forces that attempt to separate sections of brickwork after initial cracking has occurred.

The normal structural arrangement is that the inner leaf of brickwork in the external walls and at least some of the internal walls (depending on the roof type) comprise the load-bearing structure on which any upper floors, ceilings and the roof are supported. In these cases, it is internally visible cracking that should be the main focus of attention, however there are a few examples of dwellings whose external leaf of masonry plays some supporting role, so this should be checked if there is any doubt. In any case, externally visible cracking is important as a guide to stresses on the structure generally, and it should also be remembered that the external walls must be capable of supporting themselves.

Effects on framed structures
Timber or steel framed buildings are less likely to exhibit cracking due to swell/shrink than masonry buildings because of their flexibility. Also, the doming/dishing effects tend to be lower because of the lighter weight of walls. The main risks to framed buildings are encountered because of the isolated pier footings used under walls. Where erosion or saturation cause a footing to fall away, this can double the span which a wall must bridge. This additional stress can create cracking in wall linings, particularly where there is a weak point in the structure caused by a door or window opening. It is, however, unlikely that framed structures will be so stressed as to suffer serious damage without first exhibiting some or all of the above symptoms for a considerable period. The same warning period should apply in the case of upheaval. It should be noted, however, that where framed buildings are supported by strip footings there is only one leaf of brickwork and therefore the externally visible walls are the supporting structure for the building. In this case, the subfloor masonry walls can be expected to behave as full brickwork walls.

Effects on brick veneer structures
Because the load-bearing structure of a brick veneer building is the frame that makes up the interior leaf of the external walls plus perhaps the internal walls, depending on the type of roof, the building can be expected to behave as a framed structure, except that the external masonry will behave in a similar way to the external leaf of a full masonry structure.

Water Service and Drainage

Where a water service pipe, a sewer or stormwater drainage pipe is in the vicinity of a building, a water leak can cause erosion, swelling or saturation of susceptible soil. Even a minuscule leak can be enough to saturate a clay foundation. A leaking tap near a building can have the same effect. In addition, trenches containing pipes can become watercourses even though backfilled, particularly where broken rubble is used as fill. Water that runs along these trenches can be responsible for serious erosion, interstrata seepage into subfloor areas and saturation.

Pipe leakage and trench water flows also encourage tree and shrub roots to the source of water, complicating and exacerbating the problem.
Poor roof plumbing can result in large volumes of rainwater being concentrated in a small area of soil:

- Incorrect falls in roof guttering may result in overflows, as may gutters blocked with leaves etc.
- Corroded guttering or downpipes can spill water to ground.
- Downpipes not positively connected to a proper stormwater collection system will direct a concentration of water to soil that is directly adjacent to footings, sometimes causing large-scale problems such as erosion, saturation and migration of water under the building.

Seriousness of Cracking

In general, most cracking found in masonry walls is a cosmetic nuisance only and can be kept in repair or even ignored. The table below is a reproduction of Table C1 of AS 2870.

AS 2870 also publishes figures relating to cracking in concrete floors, however because wall cracking will usually reach the critical point significantly earlier than cracking in slabs, this table is not reproduced here.

Prevention/Cure

Plumbing
Where building movement is caused by water service, roof plumbing, sewer or stormwater failure, the remedy is to repair the problem. It is prudent, however, to consider also rerouting pipes away from the building where possible, and relocating taps to positions where any leakage will not direct water to the building vicinity. Even where gully traps are present, there is sometimes sufficient spill to create erosion or saturation, particularly in modern installations using smaller diameter PVC fixtures. Indeed, some gully traps are not situated directly under the taps that are installed to charge them, with the result that water from the tap may enter the backfilled trench that houses the sewer piping. If the trench has been poorly backfilled, the water will either pond or flow along the bottom of the trench. As these trenches usually run alongside the footings and can be at a similar depth, it is not hard to see how any water that is thus directed into a trench can easily affect the foundation's ability to support footings or even gain entry to the subfloor area.

Ground drainage
In all soils there is the capacity for water to travel on the surface and below it. Surface water flows can be established by inspection during and after heavy or prolonged rain. If necessary, a grated drain system connected to the stormwater collection system is usually an easy solution.

It is, however, sometimes necessary when attempting to prevent water migration that testing be carried out to establish watertable height and subsoil water flows. This subject is referred to in BTF 19 and may properly be regarded as an area for an expert consultant.

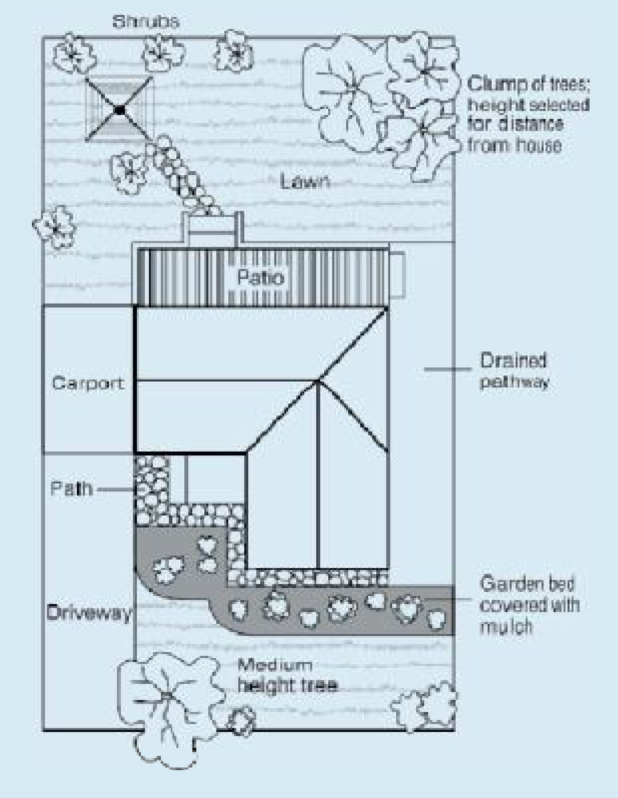
Protection of the building perimeter
It is essential to remember that the soil that affects footings extends well beyond the actual building line. Watering of garden plants, shrubs and trees causes some of the most serious water problems.

For this reason, particularly where problems exist or are likely to occur, it is recommended that an apron of paving be installed around as much of the building perimeter as necessary. This paving

CLASSIFICATION OF DAMAGE WITH REFERENCE TO WALLS

Description of typical damage and required repair	Approximate crack width limit (see Note 3)	Damage category
Hairline cracks	<0.1 mm	0
Fine cracks which do not need repair	<1 mm	1
Cracks noticeable but easily filled. Doors and windows stick slightly	<5 mm	2
Cracks can be repaired and possibly a small amount of wall will need to be replaced. Doors and windows stick. Service pipes can fracture. Weathertightness often impaired	5–15 mm (or a number of cracks 3 mm or more in one group)	3
Extensive repair work involving breaking-out and replacing sections of walls, especially over doors and windows. Window and door frames distort. Walls lean or bulge noticeably, some loss of bearing in beams. Service pipes disrupted	15–25 mm but also depend on number of cracks	4

Gardens for a reactive site



should extend outwards a minimum of 900 mm (more in highly reactive soil) and should have a minimum fall away from the building of 1:60. The finished paving should be no less than 100 mm below brick vent bases.

It is prudent to relocate drainage pipes away from this paving, if possible, to avoid complications from future leakage. If this is not practical, earthenware pipes should be replaced by PVC and backfilling should be of the same soil type as the surrounding soil and compacted to the same density.

Except in areas where freezing of water is an issue, it is wise to remove taps in the building area and relocate them well away from the building – preferably not uphill from it (see BTF 19).

It may be desirable to install a grated drain at the outside edge of the paving on the uphill side of the building. If subsoil drainage is needed this can be installed under the surface drain.

Condensation

In buildings with a subfloor void such as where bearers and joists support flooring, insufficient ventilation creates ideal conditions for condensation, particularly where there is little clearance between the floor and the ground. Condensation adds to the moisture already present in the subfloor and significantly slows the process of drying out. Installation of an adequate subfloor ventilation system, either natural or mechanical, is desirable.

Warning: Although this Building Technology File deals with cracking in buildings, it should be said that subfloor moisture can result in the development of other problems, notably:

- Water that is transmitted into masonry, metal or timber building elements causes damage and/or decay to those elements.
- High subfloor humidity and moisture content create an ideal environment for various pests, including termites and spiders.
- Where high moisture levels are transmitted to the flooring and walls, an increase in the dust mite count can ensue within the living areas. Dust mites, as well as dampness in general, can be a health hazard to inhabitants, particularly those who are abnormally susceptible to respiratory ailments.

The garden

The ideal vegetation layout is to have lawn or plants that require only light watering immediately adjacent to the drainage or paving edge, then more demanding plants, shrubs and trees spread out in that order.

Overwatering due to misuse of automatic watering systems is a common cause of saturation and water migration under footings. If it is necessary to use these systems, it is important to remove garden beds to a completely safe distance from buildings.

Existing trees

Where a tree is causing a problem of soil drying or there is the existence or threat of upheaval of footings, if the offending roots are subsidiary and their removal will not significantly damage the tree, they should be severed and a concrete or metal barrier placed vertically in the soil to prevent future root growth in the direction of the building. If it is not possible to remove the relevant roots without damage to the tree, an application to remove the tree should be made to the local authority. A prudent plan is to transplant likely offenders before they become a problem.

Information on trees, plants and shrubs

State departments overseeing agriculture can give information regarding root patterns, volume of water needed and safe distance from buildings of most species. Botanic gardens are also sources of information. For information on plant roots and drains, see Building Technology File 17.

Excavation

Excavation around footings must be properly engineered. Soil supporting footings can only be safely excavated at an angle that allows the soil under the footing to remain stable. This angle is called the angle of repose (or friction) and varies significantly between soil types and conditions. Removal of soil within the angle of repose will cause subsidence.

Remediation

Where erosion has occurred that has washed away soil adjacent to footings, soil of the same classification should be introduced and compacted to the same density. Where footings have been undermined, augmentation or other specialist work may be required. Remediation of footings and foundations is generally the realm of a specialist consultant.

Where isolated footings rise and fall because of swell/shrink effect, the homeowner may be tempted to alleviate floor bounce by filling the gap that has appeared between the bearer and the pier with blocking. The danger here is that when the next swell segment of the cycle occurs, the extra blocking will push the floor up into an accentuated dome and may also cause local shear failure in the soil. If it is necessary to use blocking, it should be by a pair of fine wedges and monitoring should be carried out fortnightly.

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About this Report

Douglas Partners

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.

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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.

Soil Descriptions

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

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

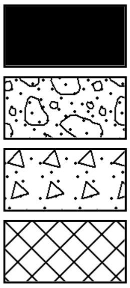
Other

fg	fragmented
bnd	band
qtz	quartz

Symbols & Abbreviations

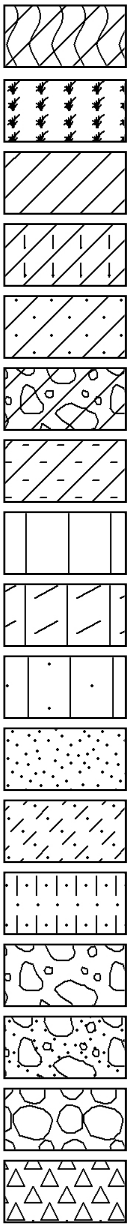
Graphic Symbols for Soil and Rock

General



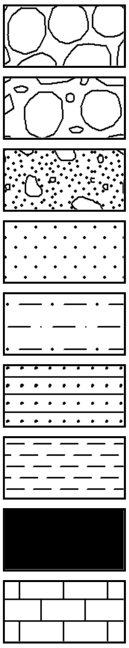
- Asphalt
- Road base
- Concrete
- Filling

Soils



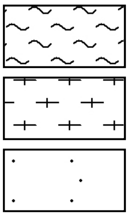
- Topsoil
- Peat
- Clay
- Silty clay
- Sandy clay
- Gravelly clay
- Shaly clay
- Silt
- Clayey silt
- Sandy silt
- Sand
- Clayey sand
- Silty sand
- Gravel
- Sandy gravel
- Cobbles, boulders
- Talus

Sedimentary Rocks



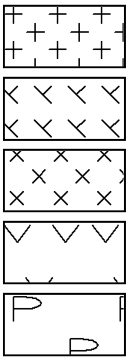
- Boulder conglomerate
- Conglomerate
- Conglomeratic sandstone
- Sandstone
- Siltstone
- Laminite
- Mudstone, claystone, shale
- Coal
- Limestone

Metamorphic Rocks



- Slate, phyllite, schist
- Gneiss
- Quartzite

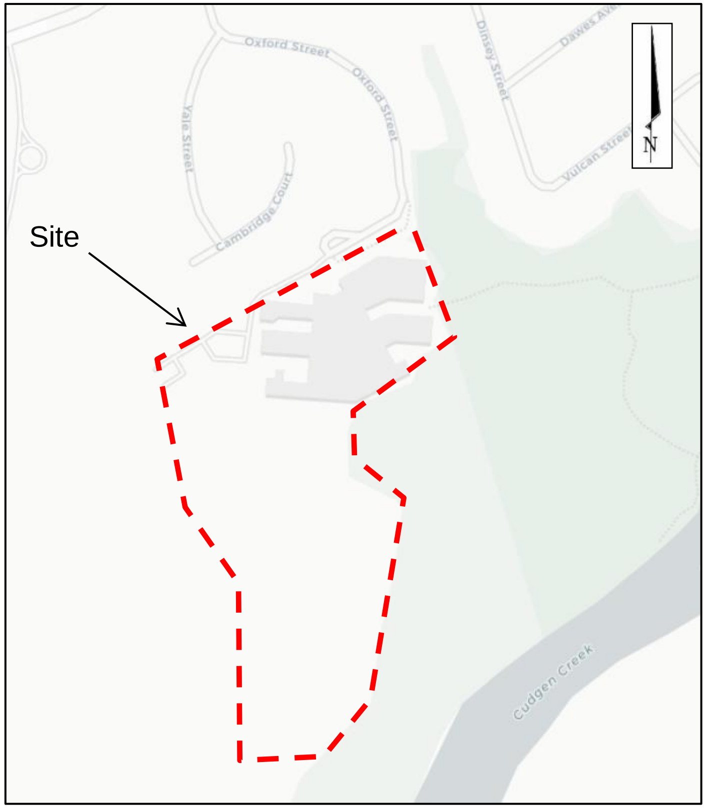
Igneous Rocks



- Granite
- Dolerite, basalt, andesite
- Dacite, epidote
- Tuff, breccia
- Porphyry

Appendix B

Drawing 2 – Site and Test Location Plan



Site Locality

Notes:

1. Test locations are approximate only and are shown with reference to existing site features.
2. Drawing Not To Scale.
3. Drawing adapted from Neamap imagery dated 6 November 2019.

Legend:

⊕ Bore number and location

Douglas Partners
Geotechnics / Environment / Groundwater

CLIENT:	School Infrastructure NSW/ Department of Education
OFFICE:	Gold Coast
DATE:	April 2020

Site and Test Location Plan

Proposed School Structures

Oxford Street, Kingscliff

PROJECT No:	98084.00
DRAWING No:	2
REVISION:	0

Appendix C

Bore Log Sheets (Bores 6 to 8)

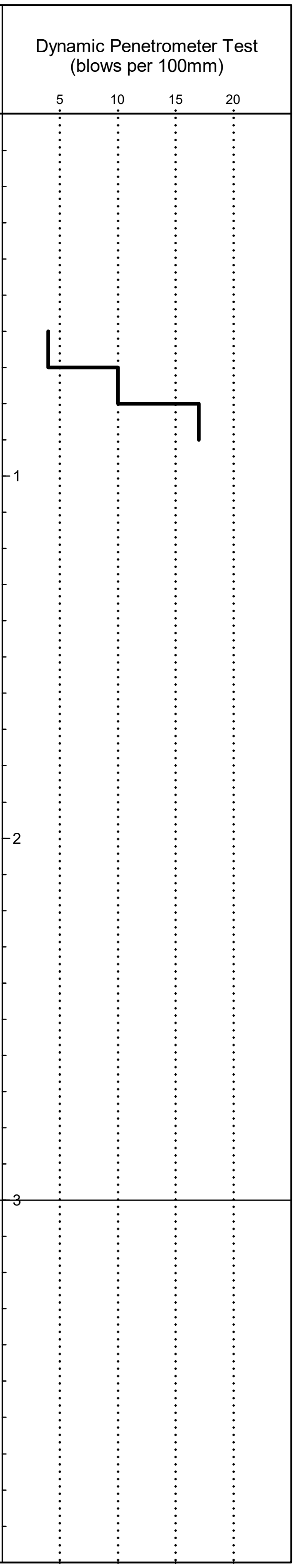
BOREHOLE LOG

CLIENT: School Infrastructure NSW/Dept. of Education
PROJECT: Proposed School Structures
LOCATION: 33 Oxford Street, Kingscliff

SURFACE LEVEL: 8.5 Approx
EASTING: 555985
NORTHING: 6873158
DIP/AZIMUTH: 90°/--

BORE No: 6
PROJECT No: 98084.00
DATE: 7-3-2020
SHEET 1 OF 1

RL	Depth (m)	Description of Strata	Graphic Log	Sampling & In Situ Testing: Type	Depth	Sample	Results & Comments	Water
		ASPHALTIC CONCRETE						
	0.1	FILL Sandy Gravel (GP): fine to medium gravel, pale brown, fine to coarse grained sand, moist, apparently well compacted		E	0.1 0.2			
8				D	0.5			
	0.6	Silty SAND (SM): fine to medium grained sand, grey, moist, medium dense to dense, alluvial		E	0.6 0.7			
	0.8	- wet						▼
				E	0.9 1.0			
	1			D	1.0			
7				D	1.5			
	2			D	2.0			
	2.3	- dark grey, estimated loose to medium dense						
6								
	2.7	Silty CLAY (CI): medium plasticity, dark brown, wet, estimated soft		D	2.7 2.8			
	3 3.0	Bore discontinued at 3.0m . Limit of Investigation.						
5								



RIG: Christie Soil Rig **DRILLER:** GeoServe **LOGGED:** BM/BMc **CASING:** N/A

TYPE OF BORING: Auger

WATER OBSERVATIONS: Groundwater encountered at 0.8 m depth.

REMARKS: Approximate surface level obtained from Nearmap imagery dated 6 November 2019.

☐ 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)

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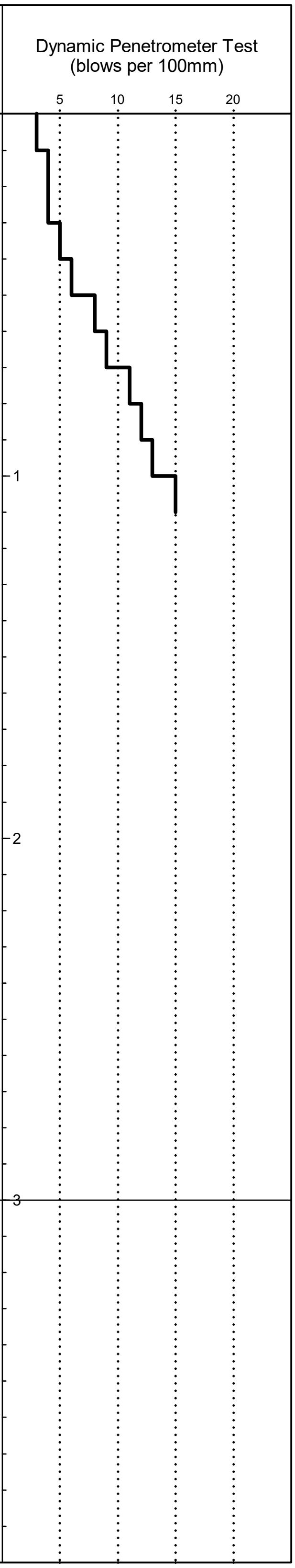
BOREHOLE LOG

CLIENT: School Infrastructure NSW/Dept. of Education
PROJECT: Proposed School Structures
LOCATION: 33 Oxford Street, Kingscliff

SURFACE LEVEL: 8.0 Approx
EASTING: 556136
NORTHING: 6873111
DIP/AZIMUTH: 90°/--

BORE No: 7
PROJECT No: 98084.00
DATE: 7-3-2020
SHEET 1 OF 1

RL	Depth (m)	Description of Strata	Sampling & In Situ Testing: Type	Depth	Sample	Results & Comments	Water
8	0.0	FILL Clayey SAND (SC): fine to medium grained sand, dark brown, moist, apparently well compacted	E	0.0 0.1			
	0.2	FILL Silty SAND (SM): fine to medium grained sand, grey-brown, moist, apparently well compacted	E D	0.4 0.5			
	0.8	Silty SAND (SM): fine to medium grained sand, pale brown, moist, medium dense to dense, alluvial	E	0.9			
7	1 1.0	- grey	D	1.0			
	1.2	- wet	D	1.5			▼
6	1.9	- dark brown	D	2.0			
	2.4	- estimated dense					
5	3 3.0	Bore discontinued at 3.0m . Limit of Investigation.					



RIG: Christie Soil Rig **DRILLER:** GeoServe **LOGGED:** BM/BMc **CASING:** N/A

TYPE OF BORING: Auger

WATER OBSERVATIONS: Groundwater encountered at 1.2 m depth.

REMARKS: Approximate surface level obtained from Nearmap imagery dated 6 November 2019.

☐ 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)

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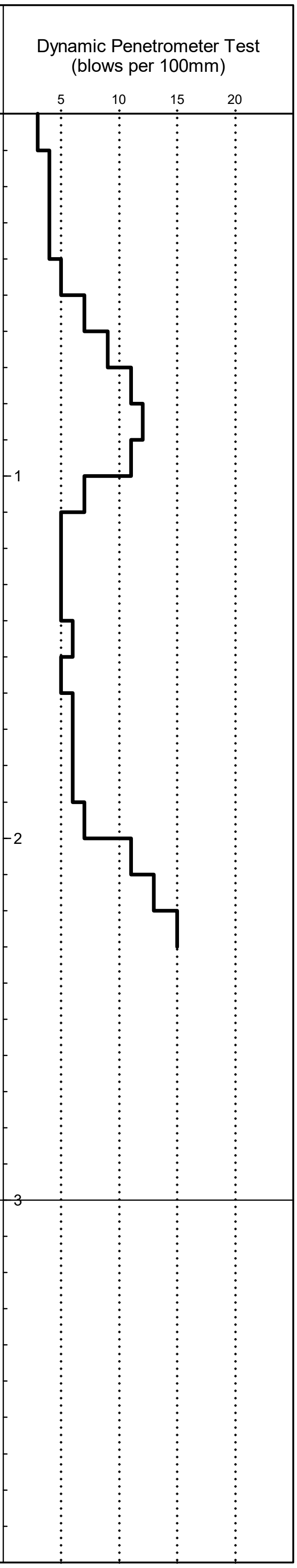
BOREHOLE LOG

CLIENT: School Infrastructure NSW/Dept. of Education
PROJECT: Proposed School Structures
LOCATION: 33 Oxford Street, Kingscliff

SURFACE LEVEL: 8.5 Approx
EASTING: 556052
NORTHING: 6873022
DIP/AZIMUTH: 90°/--

BORE No: 8
PROJECT No: 98084.00
DATE: 7-3-2020
SHEET 1 OF 1

RL	Depth (m)	Description of Strata	Sampling & In Situ Testing: Type	Depth	Sample	Results & Comments	Water
		FILL Clayey SAND (SC): fine to medium grained sand, dark brown, moist, apparently well compacted	E	0.0 0.1			
	0.3	FILL Silty SAND (SM): fine to medium grained sand, grey-brown, moist, apparently well compacted					
8			D	0.5			
			E	0.5 0.6			
	0.7	Silty SAND (SM): fine to medium grained sand, dark grey, moist, medium dense, alluvial					
			E	0.9 1.0			
	1		D	1.0			
7			D	1.5			
	1.9	- wet					▼
	2		D	2.0			
	2.1	- dense					
	2.4	- grey					
6							
	3 3.0	Bore discontinued at 3.0m . Limit of Investigation.					
5							



RIG: Christie Soil Rig **DRILLER:** GeoServe **LOGGED:** BM/BMc **CASING:** N/A
TYPE OF BORING: Auger
WATER OBSERVATIONS: Groundwater encountered at 1.9 m depth.
REMARKS: Approximate surface level obtained from Nearmap imagery dated 6 November 2019.

☐ 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)

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Appendix D

Laboratory Test Results

Table 2: 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 6										
0.5	Fill Sandy Gravel	6.5	4.4	2.1	1	–	–	–	–	–
1.0	Silty Sand	6.5	2.5	4.0	1	6.8	0.01	–	–	0.01
1.5	Silty Sand	6.1	3.1	3	1	–	–	–	–	–
2.0	Silty Sand	5.9	3.3	2.6	1	6.9	<0.01	–	–	<0.01
Bore 7										
0.5	Fill Silty Sand	5.5	3.5	2	1	–	–	–	–	–
1.0	Silty Sand	6.2	4.3	1.9	1	6.8	<0.01	–	–	<0.01
1.5	Silty Sand	6.0	3.7	2.3	1	–	–	–	–	–
2.0	Silty Sand	5.6	2.5	3.1	1	6.7	0.01	–	–	0.01
Bore 8										
0.5	Fill Silty Sand	5.5	3.7	1.8	1	6.5	<0.01	–	–	<0.01
1.0	Silty Sand	6.3	4.3	2.0	1	–	–	–	–	–
1.5	Silty Sand	6.8	4.7	2.1	1	7.0	<0.01	–	–	<0.01
2.0	Silty Sand	6.9	5.4	1.7	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.

Douglas Partners
Geotechnics / Environment / Groundwater

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Phone (07) 5568 8900
Fax (07) 5568 8999

FIELD pH SCREENING - ACID SULFATE SOILS

Client : Kingscliff PS and HS
Project : Proposed School Works
Location : Kingscliff, NSW

Project No. : 98084.00
Report No. : 98084.00-1
Report Date: 16.3.20

Tested By: MT
Date Tested: 16.3.20

Reported By: CW
Date Reported: 16.3.20

Sample No.	1	2	3	4	5	6	7	8
Chainage (m)								
Location	Bore 6	Bore 6	Bore 6	Bore 6	Bore 7	Bore 7	Bore 7	Bore 7
Offest (m)								
Level of test (m)(RL)	0.5m	1.0m	1.5m	2.0m	0.5m	1.0m	1.5m	2.0m
Soil Description	Sand	Sand	Sand	Sand	Sand	Sand	Sand	Sand
Field pH (pH_F)	6.5	6.5	6.1	5.9	5.5	6.2	6.0	5.6
Oxidised pH (pH_{FOX})	4.4	2.5	3.1	3.3	3.5	4.3	3.7	2.5
Reactivity Code (see below)	1	1	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.......................................

Douglas Partners
Geotechnics / Environment / Groundwater

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FIELD pH SCREENING - ACID SULFATE SOILS

Client : Kingscliff PS and HS
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Report Date: 16.3.20

Tested By: MT
Date Tested: 16.3.20

Reported By: CW
Date Reported: 16.3.20

Sample No.	1	2	3	4	5	6	7	8
Chainage (m)								
Location	Bore 8	Bore 8	Bore 8	Bore 8				
Offest (m)								
Level of test (m)(RL)	0.5m	1.0m	1.5m	2.0m				
Soil Description	Sand	Sand	Sand	Sand				
Field pH (pH_F)	5.5	6.3	6.8	6.9				
Oxidised pH (pH_{FOX})	3.7	4.3	4.7	5.4				
Reactivity Code (see below)	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)

LDC_GC_009_22.04.2014_Rev 00

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:** Kingscliff High School (98084.00)

Mazlab Job No: DPB3083 **Date:** 19/03/2020

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)
45499	BH6-1.00	SAND(SP) grey/grey brown	6.8	7 0.01%	-	-	NA	7	Nil
45500	BH6-2.00	SAND(SP) grey/grey brown	6.9	<2 <0.01%	-	-	NA	<2	Nil
45501	BH7-1.00	SAND(SP) light brown, some grey brown	6.8	<2 <0.01%	-	-	NA	<2	Nil
45502	BH7-2.00	SAND(SP) grey brown, some light brown	6.7	4 0.01%	-	-	NA	4	Nil
45503	BH8-0.50	SAND(SP) grey brown	6.5	<2 <0.01%	-	-	-	<2	Nil
45504	BH8-1.50	SAND(SP) light brown	7.0	<2 <0.01%	-	-	NA	<2	Nil

: :

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

Table 3: Summary of Metal/Metalloid, PCBs, Phenols, OCP/OPP, PAH and Asbestos Results (mg/kg)

Sample ID	Lithology	Moisture Content (%)	Analyte															
																PAH		
			Asbestos	Arsenic	Cadmium	Chromium	Copper	Lead	Nickel	Zinc	Mercury	PCBs	Phenol	Pentachlorophenol	OCP/OPP	Benzo(a)pyrene	Total PAH[4]	Carcinogenic PAHs (as BaP TEQ) (zero)
Bore 6 - 0-0.1m	Fill - Sandy Gravel	2.9	ND	**8**	<1	**4**	**8**	<5	<2	<5	<0.1	<0.1	<LOR	<LOR	<LOR	<0.5	<0.5	<0.5
Bore 7 - 0.4-0.5m	Fill - Silty Sand	6.5	ND	<5	<1	<2	<5	<5	<2	<5	<0.1	<0.1	<LOR	<LOR	<LOR	<0.5	<0.5	<0.5
Bore 8 - 0-0.1m	Fill - Clayey Sand	15.0	ND	<5	<1	**3**	<5	<5	**2**	**14**	<0.1	<0.1	<LOR	<LOR	<LOR	<0.5	<0.5	<0.5
Site Assessment Criteria																		
EIL[1]			~	100	~	190[a]	60[a]	1,100[a]	30[a]	100[a]	~	~	~	~	Various^	~	~	~
ESL[2] (Coarse Soil)			~	~	~	~	~	~	~	~	~	~	~	~	~	0.7	~	~
HIL-A[3]			~	100	20	100*	6,000	300	400	7,400	40**	1	3,000	100	Various^	~	300	3

Notes

1	NEPM Schedule B1 Table 1B(5) Ecological investigation levels - urban residential/ public open space
2	NEPM Schedule B1 Table 1B(6) Ecological screening levels for urban residential and public open space
3	NEPM Schedule B1 Table 1A(1) Health-based investigation levels for residential premises with garden/accessible soil
4	Based on sum of standard 16 PAHs most commonly reported for contaminated sites
a	EIL determined using Ecological Investigation Level Calculation Spreadsheet (NEPM) assuming 5cmolc/kg cation exchange capacity, 4.5 pH, 1% organic carbon, 1% clay, aged contamination for QLD with a high/low traffic volume and no ambient background concentrations (ABCs)
<LOR	Less than laboratory's limit of reporting
ND	Not detected
~	No guideline available at time of investigation
*	Chromium threshold for Chromium VI
**	Inorganic mercury
Various^	Various thresholds exist for individual compounds

Table 4: Summary of Total Recoverable Hydrocarbons and Monocyclic Aromatic Hydrocarbon Results (mg/kg)

Sample ID	Lithology	Analyte (mg/kg)										
		Naphthalene	F1	C_6-C_{10}	F2	>C_{10}-C_{16}	>C_{16}-C_{34}	>C_{34}-C_{40}	Benzene	Toluene	Ethyl-benzene	Xylenes
Bore 6 - 0-0.1m	Fill - Sandy Gravel	<1	<10	<10	<50	<50	<100	<100	<0.2	<0.5	<0.5	<0.5
Bore 7 - 0.4-0.5m	Fill - Silty Sand	<1	<10	<10	<50	<50	<100	<100	<0.2	<0.5	<0.5	<0.5
Bore 8 - 0-0.1m	Fill - Clayey Sand	<1	<10	<10	<50	<50	<100	<100	<0.2	<0.5	<0.5	<0.5
Site Assessment Criteria												
ESL[1] (coarse soil)		~	180	~	~	120	300	2,800	50	85	70	105
EIL[2]		170	~	~	~	~	~	~	~	~	~	~
HSL-A & HSL-B[3] (0 m to <1 m) Sand		3	45	~	110	~	~	~	0.5	160	55	40

Notes

1	NEPM Schedule B1 Table 1B(6) Ecological screening levels for urban residential and public open space
2	NEPM Schedule B1 Table 1B(5) Ecological investigation levels - urban residential/ public open space
3	NEPM Schedule B1 Table 1A(5) Health-based screening levels (vapour intrusion) for low - high density residential premises with garden/accessible soil
F1	C_6-C_{10} less BTEX
F2	>C_{10}-C_{16} less Naphthalene
<LOR	Less than laboratory's limit of reporting
mbGL	metres below ground level
-	Not tested
~	No guideline available at time of investigation
NL	Not limiting