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SOIL SURVEYS

GEOTECHNICAL INVESTIGATION

PROJECT NO. 1-24866

MARCH, 2022

DASH HOUSE TRUST

PROPOSED RESIDENCE

ALDERLEY STREET, RANGEVILLE

OFFICES IN BRISBANE AND GOLD COAST

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1.0 INTRODUCTION

1.1 General

This report presents the results of the geotechnical investigation carried out by Soil Surveys Engineering Pty Limited on the 3rd, 14th and 16th March, 2022 for the Proposed Residence at Alderley Street, Rangeville.

The report was carried out following authorisation from the Dash House Trust on 17th February, 2022.

The objectives of this investigation were to assess subsurface conditions at the site in accordance with the Scope of Services detailed in Section 1.3.

1.2 Proposed Development

It is understood that the proposed development is to consist of the construction of a new multi-level residence on the flatter section of the site in the south western corner of the lot.

Details about the structure are preliminary but a provided drawing suggests a structure aligned north south with an access driveway along the boundary and a pool on the eastern side of the site. The provided description indicates a double story house with basement over part of the footprint. Floors will most likely be suspended concrete.

Building loads have not been provided, however based on the provided drawing and description it has been assumed that they would be generally consistent with domestic type structural loads of this type of structure

Earthworks have not been provided and could consist of cuts and fills of up to 3m however this will need to be confirmed.

1.3 Scope of Geotechnical Services

The scope of geotechnical services provided by Soil Surveys Engineering Pty Limited was directed towards evaluating the following items as detailed in our proposal 1-24866, 2022-01-18, PR VER 2, dated 24th January, 2022:-

- Investigation of the subsurface profile at the proposed building location by drilling, sampling and in-situ testing with four boreholes.
- Laboratory testing on selected samples to assess the material parameters of the subsurface material.
- Engineering analysis of site investigation and laboratory test results to evaluate:-
    - Civil works recommendations
        - Trafficability
        - Excavatability
        - Earthworks recommendations
        - Batter recommendations
    - Foundation recommendations
        - Site classification according to AS 2870 (i.e. Reactivity)

  - Footing options and parameters
- Retaining wall design parameters
- Stability Assessment in accordance with:-
  - National Disaster Mitigation Program (NDMP), Landslide Risk Management (LRM) Guidelines, Practice Notes and Geoguidelines as published in the “Australian Geomechanics Journal” Volume 42 No. 1 March, 2007.
  - State Planning Policy July, 2017-SPP.
  - Toowoomba Regional Council (TRC) Planning Scheme.
- Construction recommendations (where applicable)
- Site management recommendations

2.0 GEOTECHNICAL INVESTIGATION

2.1 Field Investigation

Subsurface conditions at the site were investigated by the drilling and sampling of four boreholes to depths of between 0.70m and 4.00m, using a J105 drilling rig.

All boreholes terminated at ‘TC’ bit refusal. In addition, dynamic cone penetrometer tests were carried out adjacent to the boreholes. In the case of BH01, several attempts were made to penetrate through the fill layer however refusal was reached on large rock fragments at each location.

The description of the materials encountered in the field investigation is subjective, based on the experience and judgement of the rig/site supervisor and some variations in the description, from the actual material type may occur.

The soil classification descriptions, field and laboratory testing were carried out in general accordance with the following Australian Standards:-

AS 1726	Geotechnical Site Investigations
AS 1289	Methods of Testing Soils for Engineering Purposes

Notes relating to this report, borehole records and site plans (Dwg’s 1-24866-01 and 1-24866-02) showing the location of the boreholes are included in the Appendices.

The site was also mapped by a geotechnical engineer with experience in slope stability assessments.

2.2 Site Description

The site is situated at Alderley Street, Rangeville (refer Figure 1). Access onto the site was off the end of Alderley Street. At the time of the investigation, the site was clear of any structures.

The site generally slopes to the east with a fall of some 4.0m across the site (based on TRC contours). The area is moderately to well grassed with numerous small to medium sized trees. The majority of the trees on the platform suggest that the existing surface has been in place for a considerable period of time, however several trees around the edge of the structure indicate newer filling.

Refer to site photographs in Appendix D for typical site conditions.

FIGURE 1 – SITE LOCATION

2.3 Geotechnical Model

2.3.1 Regional Geology of Toowoomba

According to the Toowoomba 1:100,000 Geology Map published by the Department of Mines and Energy, 1999, the site is underlain by rocks of the mid-Tertiary (27-18 Mya) Toowoomba Volcanics which is part of the extensive Main Range Volcanics.

This unit consists primarily of basalt, inferred to be from several small local eruptive centres; mostly basalt flows, some ponded in craters and various interbeds of tuff. Rare interflow sediments and brown coals have been noted.

The unit was deposited onto an irregular terrestrial surface over immature soils covering the underlying Mesozoic sediments of the Walloon Coal Measures.

Willey (2003) describes two main periods of basalt flows between which a paleosol developed. Following the final period of flows, an upper paleosol (UP) developed on the land surface. The UP has been extensively laterised and contains no primary minerals of the parent material. It is described as very cohesive, even rocky when dry and highly permeable.

2.3.2 Site History & Observations

Published Slope Stability Assessments

A review of the site history based on published documentation and historical aerial photos indicates the following.

Two slope stability assessments were undertaken of the Toowoomba escarpment. Holmes (1981) carried out an assessment of the upper/western section of the escarpment mainly within the Toowoomba City Council area. Willmott (1984) undertook a similar assessment on the lower/eastern section of the escarpment mainly within the Gatton Shire area.

Whilst the site is presently within the Toowoomba Regional Council area the extent of the Holmes report just extended to the site with the Willmott report starting directly to the east of the existing platform.

The classification zones and their explanation used by Holmes is outlined in Table 1 below.

TABLE 1 HOLMES (1981) CLASSIFICATION ZONES

Category	Topography	Geology	Stability Constraints & Suitability
A	Plateau surface gently undulating	Lateritic soils of variable depth	No natural stability problems identified. Suitable for building for close settlement
B	Gentle to steep slopes on the top of the escarpment	Lateritic soils of variable depth	Few landslides have been recorded. Generally suitable for small acreage and close settlement but foundation investigation advisable. On-site disposal of water would be critical to land downslope.
C	Moderate to steep slopes of the escarpment	Moderately deep to deep colluvial clays and lateritic soils	Generally stable slopes, however, numerous stability problems associated with seepage zones have been identified. Land in this zone is considered suitable for small acreage settlement, but only where shown safe by geotechnical investigation. Areas of uncertain stability should be avoided. On-site disposal of stormwater and wastewater not allowable.
D	Heads of gullies and adjacent gully sides	Moderately deep to deep colluvial clays	Natural vegetated slopes mostly stable. When cleared slopes are subject to landslides both by rotational and flow sliding associated with seepage. Most unstable zone on escarpment. Not suitable for building.
E	Very steep slopes on edge of escarpment	Shallow, stoney dark brown soils	Natural vegetated slopes mostly stable. Cleared slopes are subject to shallow debris slides. Generally unsuitable for settlement. Only rare building sites available on sharp ridge crests to which access is difficult.

Holmes identified the site as being covered by two stability zones:-

- C – for the majority of the site
- D – for the southern section of the site (approximately south of BH01)

The work done by Holmes was based on air photo interpretation with site checking. In the case of this site and given the pre-fill surface the site is most likely correctly identified as D at the end of Alderley St (southern end of site) and the nearby gully to the south with the remainder more fitting to a B classification.

Air Photography

Google Earth (GE) and TRC historical photos indicate the following:-

- 2005 (Jan) - site appears to be well grassed with a track along the western boundary of the site, possible bench approximately 20m wide from western boundary. No obvious earthworks on the site (GE).
- 2009 (Jun) – the northern end of the site appears to be disturbed possible filling (GE).
- 2016 (Feb) – filling on outside edge (10m in width) of platform near BH 2 location, Fill extending from the top to the base of the batter (GE).The house to the west was constructed with what appears to be a fill embankment between that house and the subject site.
- 2017 (July) – Filling along a 25m length of the edge to the east on BH4 (TRC).
- 2017 (Dec) – further filling along the edge of the platform (25m in length) adjacent to and north of BH2. Fill is 50m horizontally from top to base (along a north south track below the site) (GE).

- 2018 (Apr) – no recent filling but what appears to be face scour adjacent to BH4 (TRC)
- 2018 (Nov) – no recent filling (TRC)
- 2019 (Jan) – fill noted in 2017 refreshed and what appears like filling (or at least clearing off the platform southeast of BH1 (GE).
- 2019 (May) – extensive fill dumped over the edge to the east of BH4 along a 30m section of crest (TRC).

2.3.3 Subsurface Profile

The subsurface profile intersected during the drilling program consisted of:-

- Fill - Identified in all the boreholes with the thickest layers in BH02 (3.5m) and was described as:-
    - Silty & Sandy CLAY (CL, CI, CH), stiff to very stiff
    - Sandy Gravelly CLAY (CL), stiff, very stiff to hard
    - Clayey & Clayey Gravelly SAND (SC), loose to medium dense
    - Silty Gravelly SAND (SM), loose
    - Clayey Sandy GRAVEL (GC), medium dense
    - GRAVEL (GP), very dense

  In the absence of documentation to confirm that the filling was placed and compacted in accordance with AS 3798-2007, then the filling must be considered as being ‘uncontrolled’.
- Clay - Encountered in boreholes BH3 and BH4 only and was described as Silty and Silty Sandy CLAY (CL & CI), stiff and very stiff.
- Sand - Identified in borehole BH4 and was described as Clayey Silty SAND (SC), dense.
- Weathered Rock - Encountered in BH2 to BH4 and was described as Basalt, extremely weathered (XW) to distinctly weathered (DW), very low to low strength.

Note AS 1726-2017 ‘Geotechnical Site Investigations’ requires that XW rock be described using soil terms.

A summary of the subsurface profile is presented in Table 2 with detailed borelogs included in the appendices.

TABLE 2 SUBSURFACE PROFILE SUMMARY

BH No.	**Fill (m)**	**Sand (m)**	**Clay (m)**	**Weathered Rock (m)**		**Total Depth (m)**	**DCP Refusal Depth[3] (m)**
				XW[6]	**DW**		
BH1[4]	0.00-TD[5]	NE	NE	NE	NE	0.90	0.83
BH1A[4]	0.00-TD[5]	NE	NE	NE	NE	0.70	0.59
BH2	0.00-3.50	NE	NE	3.50-3.80	3.80-TD[5]	4.00	0.76
BH3	0.00-0.20	NE	0.20-1.60	1.60-2.00	2.00-TD[5]	2.20	1.67
BH4	0.00-1.40	1.80-2.80	1.40-1.80	2.80-3.00	3.00-TD[5]	3.30	2.70

Notes:-
1. NE = Not Encountered; NC = Not confirmed; TD = Total Depth.
2. All depths below existing ground level (3rd, 14th and 16th March, 2022).
3. Blows >20 per 100mm.
4. Several attempts were made at the location of BH1 to penetrate the fill.
5. TC bit (rig) refusal.
6. Note AS 1726-2017 ‘Geotechnical Site Investigations’ requires that XW rock be described using soil terms.

2.3.4 Discussion

Fill

Significant amounts of fill have been identified on the site. The fill as identified in the boreholes and exposed unvegetated batters around the platform indicate an extremely variable material source. Assessment of the air photos (Section 2.3.2) indicates that the filling has been occurring over a significant period of time and that the majority appears to have been end dumped over the edge of the platform and allowed to flow down the slope. Any fill on the main platform level to the west of the centreline of the platform may have been in place for some time based on the size of some of the trees on the platform.

Residual Soils

The field description of the natural soil consists of sands and clays grading into low strength weathered rock. Whilst the descriptions are accurate from a textural aspect, the material could as a whole be described as a silty Clay or Silt. On close inspection, the material described as sand generally consists of fine, hard agglomerates of clay/silt particles that texturally appear to be sand. The surface layer also tends to have had the more mobile clay particles leached out. These leached clays collect in lower lying areas and tend to have extremely high reactivity values.

In some areas, this "sandy" layer also behaves as a silt, this can result in some problems with compaction. The layer tends to dry out quickly and is very difficult to rewet to optimum.

The soil layers also have a gravel component. This is a secondary feature which sometimes has a benefit of lowering the soils reactivity, however it is generally difficult to test and also is not necessarily consistent in extent.

2.3.5 Groundwater

The boreholes were drilled using open hole auger techniques to their total depths and groundwater was not encountered in the boreholes at the time of the investigation.

It should be noted that groundwater may be present, however due to the relatively short time that observations were made during the drilling of the boreholes and the possible effect of smear on the sides of the boreholes it was not able to be identified.

It should also be noted that the level of groundwater may fluctuate due to seasonal variations and that localised seepage may occur during and following rainfall particularly along the fill/soil and soil/rock interfaces.

2.4 Laboratory Testing

Laboratory testing was carried out on selected samples retrieved from the site investigation program and included:-

- Shrink/Swell Index - to assess the reactivity of the subsurface material
- Swell Pressure - to assess the potential uplift effects resulting from moisture variation within the upper-level clays

The results of the laboratory testing are summarised in Table 3 with full laboratory certificates given in Appendix C.

TABLE 3 SUMMARY OF LABORATORY TESTING

BH No.	Depth (m)	Material Type	FMC (%)	Shrink Swell Index (%)	Swell Pressure (kPa)
BH2	0.6	Silty CLAY (CH)	24.6	2.7	NT
BH3	0.6	Silty CLAY (CH)	20.9	3.2	220

Notes:
1. NT = Not Tested.
2. FMC = Field Moisture Content (Swell sample)

3.0 SLOPE STABILITY

3.1 Assessment Methodology

The assessment of the stability of slopes at the site has utilised a Hazard and Risk Assessment approach. In this method, the potential stability hazards on the site are assessed using certain features/properties of the site (i.e. slope angle, ground water conditions, etc.). This method provides a ranking for each of the identified hazards.

Using this hazard ranking and based on assumed consequences, an assessment of risk can then be made using a risk matrix (refer Attachment E).

The general processes in assessment and management of risks associated with landslides are given in detail in the National Disaster Mitigation Program (NDMP), "Landslide Risk Management (LRM) Guidelines", Practice Notes and Geoguidelines as published in the Australian Geomechanics Journal Volume 42 No. 1, March 2007.

The effects of earthquake on slope stability have not been included in this study.

This study has identified hazard ratings using a classification system consistent with the procedures detailed in the paper entitled "A Method of Zoning Landslide Hazards", prepared by McGregor and Taylor. This method has been adopted on a wide range of projects and has proven to be robust.

3.2 Site Factors

A recent aerial photograph of the site (refer Drawing 1-24866-01) provides an overview of the existing site conditions (along with TRC contours in Drawing 1-24866-02). Selected site photographs are included in Appendices C and D. The principal site features are:-

- The study site is located at the south western corner of 1F Alderley St, Rangeville (Lot on Plan CC555/62).
- The lot is approximately 2ha in total area with the subject section of the site is approximately 6,000m^2.
- The subject section of the covers a near level platform that runs along the western boundary (southern section).
- The platform is some 85m in length (north/south) and 40 meters wide (east/west).
- A low retaining wall runs at or near the western boundary below the adjoining lot (8 Manooka Court), this may be on adjoining site.
- The platform is well grassed with several large trees on the platform and surrounding the site.

- The edge of the platform appears to be predominately fill with slope angles of between 25° and 40° measured during the site inspection.
- Outside the platform batters the natural slopes range from 25° to 35° to the east, 15° to 20° to the north and 25° to 30° to the south east.
- The ground surface rises to the west with a section what appears to be fill directly to the west.
- Significant sections of the outer batters are obviously made up of dumped fill with large rock fragments noted within and on the surface of the batter.
- An inspection below the batters noted numerous large cobble sized fragments that have rolled down the slope.
- A detailed mapping exercise was undertaken over the platform and batters and is included in Appendix C.

3.3 Evidence of Existing and Possible Instability

Existing Land Slips

Areas of slumping were noted on the outside edge of the platform. It should be noted that there is some evidence that this has also occurred in the past.

Significant cracking was noted around the edge of most of the platform. These cracks were large and extended for several metres from the crest. Some slumping of the surface was noted. This indicates active instability and slumping of the outer material.

Refer to Appendix C for mapping of zones and instability noted.

A large slip has occurred in the bank (fill) to the west of the site which appears to have significantly affected the existing sewer line.

Surface Scour

Some evidence of surface scour was noted onsite during the site walkover particularly on the outside batter of the platform.

Soil Creep

Slight leaning and/or bowed trunks of trees are generally considered to be indicative of soil creep, with the affected trees being progressively tilted by the slow movement of soil down the slope. The degree of tilting or bowing of these trees is considered to be a sign of relatively minor movements, and is typical for trees growing on such a steep slope. Rapid or gross movements of soil downslope would be likely to uproot or kill the trees.

There was no evidence of soil creep noted onsite during the time of the site walkover.

3.4 Risk Assessment – Stability

3.4.1 General

For the purposes of this study, the assessment of Risk for the site has been based on risk to property, where Risk is the product of Likelihood and Consequence.

This method is based on the method outlined in the “Practice Note Guidelines for Landslide Risk Management 2007” (PNGLRM), produced by the Australian Geomechanics Society, 2007.

The following assessment is also based on the premise that all works are undertaken in accordance with our recommendations.

3.4.2 Identification of Potential Stability Hazards on the Site

Based on the site walkover, a review of the site information and experience on similar sites, the following potential land stability hazards have been identified for the subject site:-

I. Slumping failure of the platform and existing batters
II. Slumping failure of the embankment to the west of the subject site
III. Rocks/boulders rolling down slope
IV. Shallow Landslide within the natural slopes

3.4.3 Likelihood Estimation

Methods of Assessment

The site was assessed and zoned using the method as outlined in MacGregor and Taylor (2001). The method has been adopted as a slope stability assessment tool and used in the Brisbane, Gold Coast and Redlands Shire regional stability assessments.

Evaluation of Likelihood Rating for Existing Overall Site

Once the relative frequency of slope instability for the site is calculated, MacGregor and Taylor (2001) suggest a likelihood rating as outlined in Table 4 below.

This rating has then been compared to the Likelihood Descriptor from Appendix C of “Practice Note Guidelines for Landslides Risk Management (PNGLRM) (2007c)”.

TABLE 4 LIKELIHOOD RATING

MacGregor and Taylor (2001)		Landslide Risk Management (2007c) Appendix C	
Relative Frequency	**Likelihood Rating**	**Likelihood Descriptor**	**Adopted Indicative Value of Annual Probability**
< 0.2	Very Low	Barely Credible	10^{-6}
0.2 to 0.6	Low	Rare to Unlikely	10^{-5} to 10^{-4}
0.6 to 2.0	Moderate	Unlikely to Possible	10^{-4} to 10^{-3}
2.0 to 6.0	High	Possible to Likely	10^{-3} to 10^{-2}
> 6.0	Very High	Almost Certain	10^{-1}
Notes:- 1. McGregor & Taylor (2001). 2. Likelihood adopted based on Landslide Risk Management (2007c), Appendix C.			

Assessment zones

The site has been divided into five zones with the likelihood of hazards occurring within each zone discussed below (refer Dwg 1-24866-03, Appendix H).

Zone A – Main platform area away from the crest

The existing platform is relatively level and based on existing vegetation appears to have been in place for some time. The main potential hazards are:-

- Slumping of the outside batter not being addressed and the slump being allowed to migrate back into the platform. The platform zone does not extend to the crest of the platform. To allow for this, Zone B is extended back from the edge of the platform.
- Slumping of the embankment above (this has occurred). The platform zone does not extend to the rear boundary with that area being part of Zone B.

Based on the designated zone and provided any failure with Zone B is addressed and not allowed to migrate then this zone has been given a likelihood hazard rating of **Very Low,** which translates to a Likelihood Descriptor of **Barely Credible.** If the slumping in Zone B is not controlled the likelihood hazard rating will increase for this zone.

Zones B & C - Platform edge and fill batter

These zones cover the edge of the platform and the existing fill batters. This seems to cover the more recent filling which has occurred in the last five (or so) years. Following the recent rainfall event (and most likely well before that) instability has occurred in this zone.

Visual evidence is discussed in sections 2.3.2 and Appendix C and consisted of significant tension cracking parallel with the crest, slumping of the surface in these areas, slumps on the steep batters, scour of the faces and trees within the fill leaning downslope.

Therefore, this zone has been given a likelihood hazard rating of **Very High**, which translates to a Likelihood Descriptor of **Almost Certain.**

Zone D – Natural Slopes Unaffected by fill

The extent of this zone is based on air photo interpretation of the slope over time as well as the remnant vegetation and site observations. The most likely stability hazards in this area were assessed for their likelihood hazard rating based on site observations and the expected slope properties:-

- Rocks/boulders rolling down slope – likelihood hazard rating of **Moderate to High**, which translates to a Likelihood Descriptor of **Unlikely to Likely**
- Shallow Landslide within the natural slopes - likelihood hazard rating of **Moderate**, which translates to a Likelihood Descriptor of **Unlikely to Possible**

Zone E – Embankment to the west of the platform

This embankment appears to have been constructed as part of the house to the west of the site in 2016 (refer Section 2.3.2). A significant section of that embankment has slumped onto the subject site following the recent significant rainfall event.

Therefore, this zone has been given a likelihood hazard rating of **Very High**, which translates to a Likelihood Descriptor of **Almost Certain.**

3.4.4 Consequence Analysis

General

As part of the assessment of the risk of slope instability, the analysis of the consequence of any failure must also be considered.

The future development will comprise the construction of a residential dwelling as outlined in Section 1.2 of this report.

Consequence to Property

For the purposes of this study, a semi-qualitative measure of consequence to property will be adopted, as outlined in Appendix C of PNGLRM 2007c. If slope instability were to occur, the aspect of property most at risk would be the proposed dwelling.

Based on Appendix A of PNGLRM 2007c the residential structures at risk would be classified as Importance Level 2.

Considering the above information, the consequence to property for each of the hazards is as follows:-

- Zone A - **Major**
- Zone B & C - **Major**
- Zone D - **Insignificant**
- Zone E - **Medium**

3.4.5 Risk Determination and Evaluation - Property

Appendix C of "PNGLRM (2007)" outlines a method of assessing Risk to Property. The calculation of likelihood is based on the estimate outlined in Section 3.4.3 of this report. The consequence is based on the proposed development and the effects on the property of the specific hazards from Section 3.4.4.

The risk is then assessed based on the Risk Matrix shown in Attachment E. This is summarised in Table 5.

TABLE 5 SUMMARY OF RISK TO PROPERTY

Zone	Hazard Rating	Assessed Likelihood	Assessed Consequence	Assessed Risk to Property
A	Very Low	Barely Credible	Major	**Very Low**
B&C	Very High	Almost Certain	Major	**Very High**
D - Boulders	Moderate to High	Unlikely to Likely	Insignificant	**Very Low**
D - Landslides	Moderate	Unlikely to Possible	Insignificant	**Very Low to Low**
E	Very High	Almost Certain	Medium	**Very High**

Notes: Refer Appendix B of "Practice Note Guidelines for Landslides Risk Management (2007)".

3.5 Conclusions and Recommendations

3.5.1 Discussion and Conclusions

The assessed risk in different zones of the site is outlined in Table 5. The areas of major concern from a instability risk perspective are areas B&C and E. The following comments can be made with respect to the platform area:-

- The site appears to have been extensively filled over time.
- Filling of the site appears to have occurred in stages over a considerable period. Fill was identified in all the boreholes (although BH3 only encountered a thin surficial layer).

- There may be a central section of the site near BH3 and the two mature trees near BH3 where only limited filling (if any has occurred)
- The remainder of the platform has undergone significant filling.
- The most recent filling (say last five years) has been of variable quality/type and the method of filling appears to have consisted of end dumping of material either directly over the edge of the platform or subsequently pushed over the edge.
- There has been significant slumping of this material around the platform edge resulting in cracking and subsidence in areas.

The major issue with filling in the above way is the fill is generally poorly compacted and sorted. Planes of weakness develop in the fill mass which parallel the outside batter. This leads to movement of the fill mass. The peak strength is exceeded and the mass develops a lower residual strength and this results in failure of the mass. This failure unless remediated will progress into the slope resulting in a larger failure than initially observed.

The use of end dumping results in an over steepened batter particularly in the upper section of the slope with the slope having a factor of safety near unity which will eventually fail when ground water conditions result in increased pore water pressure and eventual failure.

3.5.2 Recommendations

It is strongly recommended that all uncontrolled fill be removed and where suitable re compacted in accordance with recommendations outlined in Section 4.1. At this stage given the very high risk of instability around the platform, the use of an alternative to the above (e.g., use of deep foundations and a fully suspended slab with the existing fill mass) will not be recommended.

A failure to the west of the platform was also noted. This appears to have occurred on an adjoining property in a newly constructed fill embankment. This has led to disruption of an existing sewer line and resulted in failed material slumping onto the subject property. At this stage this is likely to only effect the proposed access driveway to the west of the residence however if not repaired (including the non-failed section of the embankment), further failures may occur.

4.0 ENGINEERING ASSESSMENT

4.1 Civil Works

4.1.1 Trafficability and Site Preparation

At the time of the field investigation, trafficability was considered to be fair. Access over the site could be described as good although the presence of soft areas following rainfall and surface cracking adjacent to the edge of the platform didn’t allow machinery access.

The fieldwork for this investigation was carried out following (and during) wet weather conditions with a small 4WD truck mounted drilling rig. However, as is the case on most construction sites, some problems may be anticipated during wet weather for even light weight 4WD vehicles.

The contractor should fully inform himself of the ground conditions on site prior to commencement of earthworks. This requirement should be explicit in any earthworks specifications or contract.

The soils on site are sensitive to repetitive vehicle loading and water (i.e., they will lose strength through repetitive vehicle loading or if they become overly moist or wet). Further, seepage may also result in a subsequent loss of strength. This may limit trafficability and create difficulties for earthworks operations. This situation would be more pronounced if rainfall followed initial clearing, stripping and grubbing.

Problems may also arise from disturbance of the upper-level soil fabric with removal of vegetation and existing structures. Depressions could be formed resulting in water traps and potential softening of adjacent and underlying soils.

It is recommended that after stripping, clearing and grubbing, the exposed surface in the construction area be proof rolled (where appropriate) to assist in identifying weak areas and to improve trafficability. In areas of cut, proof rolling may be deferred until after the cut operation.

An important aspect of maintaining trafficability is seepage/drainage control.

Maintaining adequate drainage conditions is also essential. It should be ensured that runoff is diverted away from the construction area to prevent ponding of water. In addition, the construction area should be "sealed" at the completion of each day and in the event of rain.

Potential trafficability problems with this site should not be underestimated. The site will very quickly become untrafficable if appropriate seepage and drainage control measures, along with construction practices appropriate for site conditions, are not maintained.

Further, Soil Surveys Engineering offers a pre-construction meeting service. The primary objective of this service is to allow the Contractor to demonstrate to the Principal and Project Manager that they have a complete understanding of the earthwork challenges/risks associated with this site (i.e., trafficability conditions, treatment of silty/clayey sands, use of in-situ soils as structural fill, etc.) and that they have made appropriate cost and time allowances. Excavation of trial pits by the preferred contractor (just prior to commission) should form part of this exercise. Our standard rates will apply for this pre-construction meeting service. **Please note that this pre-construction meeting service is considered essential for this project.**

Working Platforms for Tracked Plant and Heavy Construction Vehicles

The scope of Soil Surveys Engineering's study **DOES NOT** include the design of a working platform for heavy construction vehicles or heavy tracked plant. If such platforms are required to be constructed on the site, general comments with respect to earthworks are included in Section 4.1.2 to allow for the preparation of costings for the project.

Detailed design of a working platform should be carried out considering the operation of actual machinery proposed to be used and the areas of work. This is particularly important when considering the use of **heavy** piling rigs and **heavy** cranes - the piling/crane contractor should be consulted regarding their requirements.

Demolition and Clearing Activities

Extreme care should be exercised during the demolition/clearing phase to ensure that excessive subgrade disturbance is not caused during removal of existing pavements, services, etc.

4.1.2 Earthworks

General

It is understood that the original proposed earthworks may have consisted of cuts of up to 3.0m in depth however based on the recommendations outlined in Section 3.5, a more extensive earthworks program will be required.

Earthwork procedures should be carried out in a responsible manner in accordance with AS 3798-2007 'Guidelines on Earthworks for Commercial and Residential Developments'. It is recommended that the earthworks contractor make themselves familiar with site conditions.

Subgrade Preparation Procedures

Subgrade preparation procedures should include the following:-

- Clearing, stripping and grubbing should be carried out in areas subject to earthworks. Also, all soils containing organic matter should be stripped from the construction area. This material is not considered suitable for use as structural fill.
- In construction areas where fill is to be placed, the existing ground surface should be proof rolled (where appropriate) under the supervision of Soil Surveys Engineering in accordance with methods and equipment as per Clause 5.5 of AS 3798-2007. In areas of cut, proof rolling may be deferred until after the cut operation. Areas demonstrating excessive movement should be treated (dried and recompacted) or removed and replaced with compacted fill, particularly loose surface clayey sands, should be compacted to the appropriate requirements. Soft, wet clays, should they be encountered, should preferably be removed. In areas of cut, proof rolling may be deferred until after the cut operation.
- Depressions formed by the removal of vegetation, existing structures, underground elements etc. should have all disturbed weakened soil cleaned out and be backfilled with compacted select material.
- Any fill material encountered should be considered uncontrolled and requiring treatment (i.e., excavate/condition/replace/compact as required). Please note that treatment standards may vary subject to slab on ground floor support requirements.
- Where fill is to be placed on sloping ground, particular care should be taken with respect to benching of the subgrade such that filling is carried out on a level, refer Section 2(i) and 2(j) of AS 3798-2007.
- Sloping ground, etc. should be benched to "key in" fill material and optimise compaction. The benches should slope back at 1V:10H and be at least 0.5m wide. Wider benches to accommodate the width of the roller may need to be adopted in some situations. Figure 2 refers.

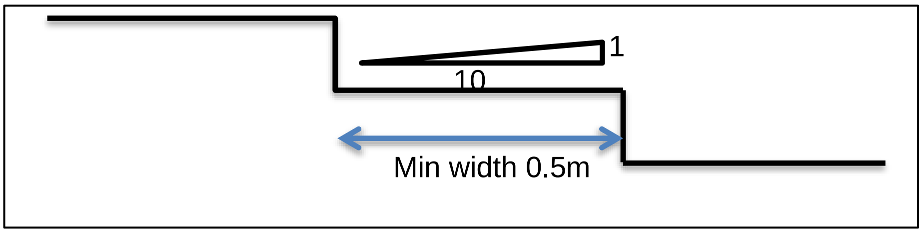


FIGURE 2

- Please note that the onsite soils are very sensitive to water and will lose strength if they become wet. Should these soils be wet at time of construction, significant works to treat these soils would be required.

Material Usage and Fill

Natural Soils

- The in-situ soils were described as highly reactive and whilst it may be able to be used for structural fill (provided they are free of organic and deleterious material) this material potentially could have a significant effect on the site reactivity. It should be noted that where reactive soils are recompacted there is the potential for over compaction of the soil to alter their reactivity. There is the potential for the reuse of this material in the lower levels of any filled area, where it affects a site-on-site reactivity would be significantly reduced.
- Use of the highly reactive clay soils may be considered with very close control on moisture content during placement and compaction. To minimise the potential for swelling and shrinkage movement a moisture content within the range of OMC -1% to OMC +2% is recommended. Foundation design must reflect the use of the potentially reactive clays if they are used as structural fill.
- Some soils may require conditioning to dry out/wet up, otherwise difficulties could be experienced in handling and achieving adequate compaction. This may require spreading, conditioning (i.e., drying/wetting up) and mixing to achieve a uniform improvement in the materials.

Existing Fill

- The existing fill has been described as uncontrolled and it is therefore suggested that this be completely excavated.
- Fill material, excluding silty/clayey sands, may be reused in the construction where Fill Specifications (see below) are met.

Imported Fill

- Imported select fill, if required for filling in structural areas/if needed to make up earthworks deficiencies, should conform to the minimum specification as set out below:-

– Soaked CBR	Minimum of 10%
– Liquid Limit	<45%
– Plasticity Index	<15%
– Maximum Aggregate Size	<75mm
– Passing 19mm Sieve	80%, minimum
– Passing 0.075mm Sieve	20%, minimum
– Shrink/Swell Index	Maximum of 1.0%

- Typical material that would conform to the above quality limits would be a good to fair quality overburden type material; such material would have a soaked CBR value of approximately 7% to 10%.
- The requirement to specify a minimum soaked CBR value is dependent on pavement design issues; this matter should be referred to the Civil Engineer.

- Pavement gravels should comply with the Local Council quality specifications for base, sub-base and blanket materials.

Compaction Procedures and Specifications

General filling

- Fill placed should be compacted in layers (approximately 250mm loose thickness) to a density not less than 98% of maximum dry density in accordance with AS 1298 Test Series 5 (Standard Compaction).
- Field density testing should be carried out to check the standard of compaction achieved and the placement moisture content. The frequency and extent of testing should be as per guidelines in AS 3798-2007, Section 8.0.
- The select fill material should be compacted in layers not exceeding 250 mm, loose thickness. However, layer thicknesses will be dependent on the compaction plant type and size, use of vibration, material type and condition. Final maximum placement layer thicknesses will need to be determined when compaction plant as well as material type and conditions are known.
- Provided the placement moisture content of the imported fill or select in-situ material approximates the optimum moisture content for compaction, suitable compaction should be achievable using typical compaction machinery, i.e., say a 5t-10t vibrating sheepsfoot roller or 25t-30t sheepsfoot compactor.
- Other comparable compaction specifications may be adopted for earthwork procedures; however, Soil Surveys Engineering Pty. Limited should be consulted prior to site works to confirm.

Service Trenches

- Backfilling for service trenches etc. should use good quality material free of organic and deleterious matter, either select fill won from site or imported fill. The backfill should be placed in uniform layers over the full width of the excavations with the layers not exceeding 200mm loose thickness using wheeled plant and 100 mm loose thickness using vibrating plates. The backfill material should be compacted to the specifications outlined above for in-situ or imported cohesive material.

Behind retaining walls

- Care should be exercised when backfilling retaining walls. Compaction against the wall could increase the long-term lateral pressure on the wall in excess of design lateral pressures.

Fill batters

- **Fill batters should be overfilled and cut-back to design batter angles (refer to AS3798-2007 Section 6.2.4).**

Trenching

Shoring of deep trench excavations is recommended. Suitable precautions to satisfy Health & Safety requirements must be adopted. Construction procedures (i.e., operation of plant, storage of materials, etc.) should also consider the nature of the site soils.

4.1.3 Earthworks Supervision and Certification

Engineering supervision of the earthworks operations by Soil Surveys Engineering Pty Limited is recommended.

Following production of AS 3798-2007, the terms "Level 1 and 2 Supervision" have been adopted in earthworks specifications to describe what could also be termed Engineering Supervision. Whilst there is no particular problem with using these terms, there does not seem to be wide agreement as to what Level 1 or 2 Supervision actually means or entails.

It should be noted that Level 1 fill may not be suitable as a founding layer and it is recommended that any filling when the intention is for the fill to be used as a founding layer be supervised and certified for a particular minimum bearing capacity by a RPEQ.

Regardless of terminology, it should be made clear in any earthworks specification as to what is actually required in terms of certification. It is recommended that the following objectives (as a minimum) be incorporated into the earthworks specification:-

- Engineering certification that all general earthworks operations (i.e., stripping, proof rolling of subgrade, subgrade treatment, etc.) have been carried out in accordance with the earthworks specification.
- Engineering certification that fills has been placed and compacted to the required minimum density in accordance with the earthworks specification.
- Engineering certification that embankment filling has been undertaken in accordance with AS3798-2007 Section 6.2.4 i.e., overfilled and cut back to the final profile.
- If required, engineering certification that the controlled fill is suitable for support of conventional high-level footings and has a recommended minimum bearing capacity (Note the removal of all uncontrolled fill would be a requirement of placement of footings in any engineered fill). Refer to Section 4.1.4.
- If required, engineering certification that the controlled fill material is suitable to support a conventional slab on ground floor.
- Engineering certification that the quality of any imported fill complies with the earthworks specification requirements.
- Engineering certification that the stability of cut/fill batters and trenches is adequate.

Engineering certification should be provided by a Registered Professional Engineer of Queensland.

Please note that Soil Surveys Engineering in our role as Earthworks Supervisor (under normal circumstances) cannot authorise variations to the contract or the use of provisional items. If the Contractor considers that any recommendation/instruction issued by Soil Surveys Engineering is a variation to the contract, the Contractor should advise the Superintendent and obtain written approval before proceeding with Soil Surveys Engineering’s recommendation/instruction.

4.1.4 Definitions

The following terms have been used in this section:-

- Engineered (certified) Fill – Fill placed in a controlled manner and certified by a RPEQ to the following:-
    - Confirmation that high level footings can found in the fill.
    - Provide a design allowable bearing capacity for the footings
    - Confirm that all the works have been undertaken in accordance with AS3798 including the requirements outlined in 4.1.3 of this report especially removal of existing fill and recommendations with respect to filling on slopes.
- Controlled fill – fill placed in a controlled manner.

Note: A "Level 1 report" may not comply with the definition of Engineered Fill.

4.1.5 Excavation Characteristics

General

It is anticipated that excavations will consist of the following:-

- Bulk Cuts - for site stripping and excavation to create building platforms and carpark/driveway areas.
- Trenching - for high level footings and underground services.
- Drilling - for bored pier foundations.

Excavatability Comments

Based on geotechnical knowledge of excavations/earthworks on projects in the local area and the findings of the investigation, the following comments can be made on excavation characteristics:-

- Bulk Works
    - Excavations in the soils and upper 0.5m to 1.0m, or so, of the very low to low strength weathered rock to 'TC' bit refusal depth (refer Table 2) should be within the capacity of a medium size backhoe (Case 580 or similar) or small excavator (e.g., 12t - 15t).
    - Below these levels (i.e., 'TC' bit refusal depths noted in Table 2), a larger excavator would be required for excavation further into the weathered rock (where required).
- Trenching
    - Trench excavations in the soils and upper 0.5m to 1.0m, or so, of the very low to low strength weathered rock to 'TC' bit refusal depth (refer Table 2) should be within the capacity of a medium size backhoe (Case 580 or similar) or small excavator (e.g., 12t - 15t).
    - Below these levels (i.e., 'TC' bit refusal depths noted in Table 2), a larger excavator would be required for excavation further into the medium to high strength weathered rock (where required).

- Bored Piers
  - It should be noted that the ability to drill piers in the weathered rock material is not only dependent on material characteristics but also the type (power and size) of the bored pier drilling rig, drilling teeth, size of pier, etc. It is recommended that the drilling contractors be consulted on this matter.
  - It is important that the drilling contractor have a 'clean-out' bucket to ensure adequate cleaning of the pier bases if hand cleaning of the bases is not possible.
  - The existing contains large fragments of rock which may make installation of bored piers difficult or not possible.

4.1.6 Batters

General

Whilst the initial development suggested that cut batters of up to 3.0m may be required, the subsequent recommendations as outlined in Section 3.5 may result in higher fill batters.

Batter Angles

Maximum batter angles for different material types are outlined in Table 6 for **un-surcharged** cut and fill batters less than 3m high on the site.

TABLE 6 DESIGN MAXIMUM BATTER ANGLES (Slopes up to 3m high)

Material	Short Term	Long Term
Existing Fill	26° (1V:2H)	15° (1V:4H)
Future Controlled Fill[1]	35° (1V:1.5H)	18° (1V:3H)
Clay Soils	45° (1V:1H)	26° (1V:2H)
Weathered Rock XW[3]	45° (1V:1H)	26° (1V:2H)

Notes:-
1. Assumes earthworks are undertaken as per section 3.1 and the following section (Fill Batters) is undertaken.
2. These values assume no seepage. If seepage is present the recommended angles would need to be significantly reduced or the use of dewatering considered.
3. Subject to inspection by an experienced geotechnical engineer/engineering geologist.

Where surcharges (e.g., footings, live loads, etc.) are located within H (height of batter) of the top of the batter, then some reduction in design angle will occur.

Steeper batters are possible with suitable surface protection such as stone pitching, etc. or by use of retaining structures (temporary and permanent).

Note values above are maximum values in the case of fill and soils. Flatter angles may be considered for ease of maintenance.

Regardless of the design, it is important that all cuts be inspected (progressively as construction proceeds) by an experienced Geotechnical Engineer/Engineering Geologist.

Fill Batters

Fill batter slopes are dependent on suitable compaction being achieved. It is recommended that all fill batters are formed by overfilling past the line of the final face and then the batter cut back (Figure 3) refer to AS3798-2007 Section 6.2.4. This allows compaction equipment to adequately compact the entire fill batter. All fill batters should consider the possible effects of scour and water runoff.

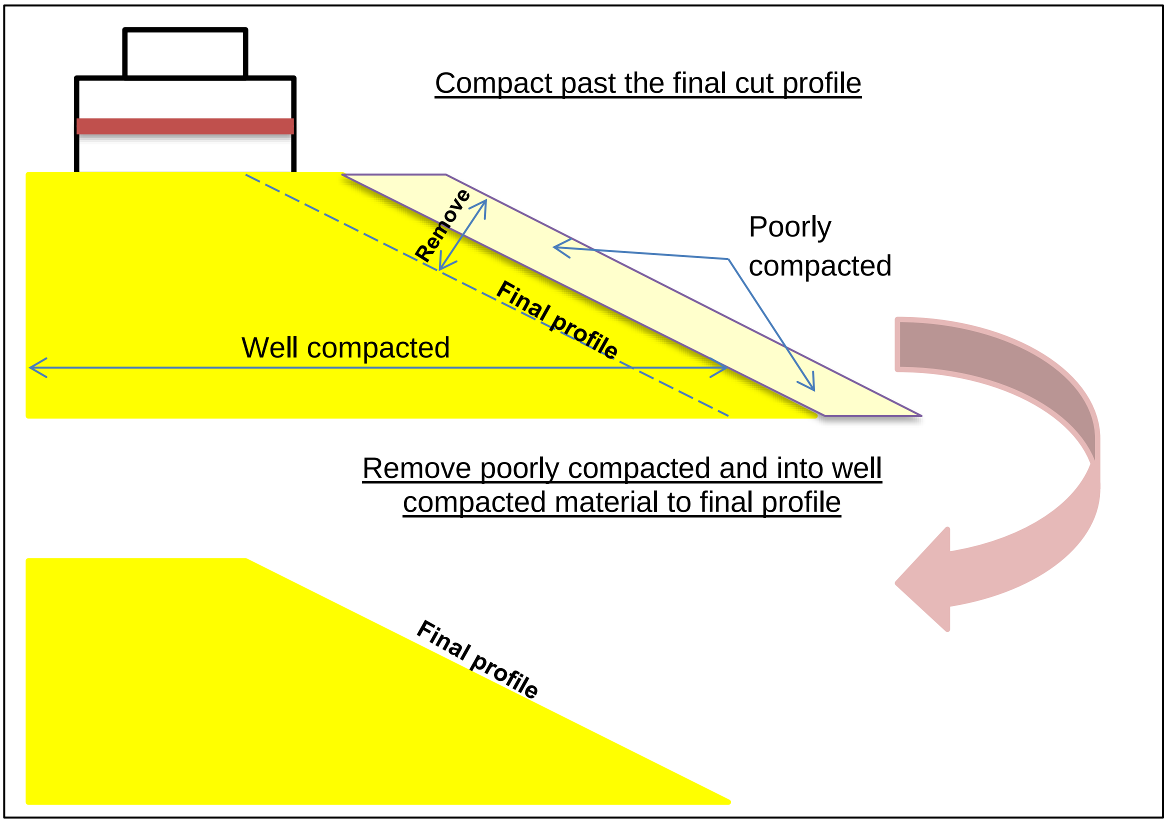


FIGURE 3

Scour Protection and Erosion Control

It is essential that permanent batters be suitably protected from erosion and scour by appropriate drainage and the establishment of ground cover and shrubs, etc. Pavement runoff should not be allowed to discharge directly across the batters without suitable scour protection.

If runoff velocities are sufficiently high, following cutting and construction of batters, erosion of the exposed soil and fractured rock could occur.

Top soiling and seeding or hydromulching on cut areas steeper than 1:5 (11 degrees) is suggested so that protection could be provided until such time as the grass cover is established.

Drain outlets would also need to consider the effects of scouring runoff water. These will need to be concrete lined near each entry and exit, with grassing further out.

It is suggested that top soiling and seeding or hydromulching also be used on all fill batters, regardless of slope, to provide protection to the slopes until permanent grass cover is established. The seed selection for the hydromulch will depend on the season when placed, but in any case, should include a rapid growing seasonal variety that could rapidly establish itself until such time as the regular grass variety is established.

It is recommended that where possible all batters have surface and subsurface drains installed above and below the slope so that water is collected and directed away from structures. This is particularly important when lots are located below the road level.

4.2 Wastewater Disposal

Appropriate drainage provisions are essential in any development. Adequate subsoil and surface drainage should also be incorporated in the batters and access driveway construction, as well as any retaining wall construction.

Wastewater from the proposed future structures should be piped to storage tanks or an approved Council discharge location. **Wastewater should not be disposed of on site without consideration of slope stability taken into account.**

Other usual treatment options associated with Good Hillside Practice, as outlined in Attachment I of this report should also be adopted. Some of these are implicit or assumed by the analysis. For example, provision of surface water drainage measures (**such as a lined drain across the top of retaining walls and batters**) and properly engineered retaining walls (where adopted). Such treatment options should be clearly stated as part of the risk management requirements.

4.3 Continued Maintenance

As with all developments, continued maintenance of the proposed site works is essential to maintain the assessed risks outlined in this report. Maintenance should include but not be limited to the following:-

- Maintaining clear drainage paths for surface water flows.
- Maintaining and promoting revegetation of exposed batter areas.
- Regular maintenance checks to ensure any damage is repaired in a timely manner.

4.4 Building Foundations

4.4.1 Site Classification as per AS 2870-2011

General

A site classification, in accordance with AS 2870 'Residential Slabs and Footings' (it is recommended that the reader satisfy themselves that the use of AS 2870 is applicable for the proposed design) has been carried out with the following information:-

- Subsurface profile
    - Up to 3.5m of fill in BH02
    - Residual soils over weathered rock
- Proposed earthworks
    - Cuts of up to 3.0m
    - Fill depths to be confirmed
- Soil Moisture Model
    - A zone of moisture variation of 2.3m
    - A crack zone of 1.15m
    - ΔpF = 1.2
- Laboratory test results
    - Refer to Section 2.4 of this report and attached certificates

Site Classification

Based on the site investigation the site has been assessed as being a reactive site according to AS 2870-2011 Section 1.8.44.

According to AS 2870-2011 Clause 2.1.3 the site has been designated Class 'P' due to:-

- The site being potentially subject to moisture changes due to site conditions more severe than normal site conditions or other factors resulting in foundation movement beyond the reactive soil movements resulting from moisture changes due to the normal site conditions refer AS 2870-2011 Clause 1.3.2 (i.e., Clause 1.3.3 abnormal moisture conditions). Abnormal moisture conditions can occur on reactive site when there is:-
    - removal of trees prior to construction,
    - presence of trees on the building site or adjacent site,
    - unusual moisture conditions caused by drains, channels, ponds, dams, swimming pools, effluent disposal areas of tanks, which are to be maintained or removed from the site.
- The bearing capacity is less than that specified in AS 2870-2011 Clause 2.4.5;
- Excessive foundation settlement may occur due to the design loading on the foundations;
- The presence of uncontrolled or controlled fill as defined in AS 2870-2011 Clause 2.5.3 i.e., in excess of 0.4m in depth;
- The site may be subject to mine subsidence, landslip, collapse activity or coastal erosion

The above site classification has been assessed based on the provided/assumed earthworks. Should the extent of earthworks change, the site classification may need to be reassessed by Soil Surveys Engineering to confirm it has not changed.

It should also be noted that the site classification provided above applies to the design of the structural foundation system. Further comments are provided in Section 4.4.2.

Should a site classification with respect to plumbing and sanitary drainage be required please refer to the Section 4.4.3.

4.4.2 Further Comments Regarding Foundations

Foundation Design

As noted above this site has been classified as ‘P’, however, if the site was classified by the soils reactivity alone, based on the site investigation results, laboratory testing and adopting the **EXISTING SURFACE**, a ground surface movement (ys) of 55mm (maximum) has been calculated, i.e., 'H1' Class site.

The recommendation above may change if any:-

- Filling in excess of 0.40m occurs on the site
- Cuts in excess of 0.50m occurs on the site

The above is based on the existing site. Given the potential for additional earthworks to be undertaken on the site, it is strongly recommended that the above be reassessed once these earthworks have been completed.

The above calculation has also been based on the recommendations outlined in Sections 4.1 'Civil Works' and 4.4.12 'Site Management' being complied with. Where these recommendations cannot be met then the ys value and classification will need to be reviewed.

It should also be noted that the methodology adopted for determination of the site classification assumes the structure has performance requirements similar to domestic type construction.

Effect of Reactivity on Services

It should be noted that a site classification of ‘H1’ was assessed for this site as such the detailing of services at the edge of the buildings should be carefully considered to take into effect the potential for movement differentials at the edges of the structure and the potential effect on service lines. Reference to Section 5.5.4 of AS 2870-2011 is recommended.

The effect of this possible movement should also be considered in points of egress and ingress to the structure to ensure that differential movement does not result in a safety hazard.

Trees

As noted in AS 2870-2011 Section 1.3.3, if existing trees are left in place (or subsequently planted) ‘close’ to future building locations, significantly greater movements than those nominated above may occur due to an increased soil suction magnitude and depth. Also, if trees proposed to be removed are not removed well in advance of construction, significantly greater shrink-swell movements could also occur on ‘wetting up’ following tree removal.

Further, AS 2870-2011 Appendix H, includes an informative section on the design of footings for trees and in particular the calculations of the maximum potential surface movement due to the tree-induced suction change (that is in addition to the normal design suction profile). The Designer should refer to AS 2870-2011 for detailed discussion on this matter.

Development of Abnormal Soil Conditions

It is recommended that design works for the proposed development take into account the long term effect of any design elements on the subsurface moisture conditions under and around the structure.

These design works should consider aspects of the design that may contribute to the development of abnormal moisture conditions i.e.:-

- Site drainage
- Gardens and landscaping
- Future planting of trees and shrubs
- Services and repair of leaks

The designer is directed to AS 2870-2011 Appendix B and Section 4.4.12 ‘Site Management’ of this report which outlines measures that should be considered.

Settlement of Bulk Filling

Where bulk filling is placed under 'controlled' conditions, there is potential for 'creep' settlement of the filling material as the filling settles over a period of years due to the self-weight of the fill. Estimates of bulk filling creep settlement under self-weight will vary in accordance with the depth and quality of the filling. Given the variation of fill encountered on site, this may result in differential settlements.

Potential movements of 'controlled' bulk filling are estimated to be in the order of 0.1% to 0.5% of the total fill thickness i.e. for maximum encountered fill depths = 5mm to 25mm with an additional settlement of another 25% due to slab loads. This range should be considered for sensitivity assessment and is in addition to any movement due to reactivity of the fill (or in the case of the south western corner the existing subsurface profile).

The creep settlement estimates of both filling under self-weight must be added to the settlement estimates of any footings founded in the controlled filling.

4.4.3 Site Classification for the Purpose of Sanitary Drainage

It should be noted that the soil classification to AS 2870 for the purpose of design of plumbing and sanitary drainage systems (Form 1 Section 5) is 'H1'. Reference to Section 5.6.4 of AS 2870-2011 is recommended.

As the site has been classified as 'H1' an articulation report will be required. We would recommend that a suitably qualified person be engaged to complete this for submission.

4.4.4 Hazard factor and Site Sub-soil Class as per AS 1170.4-2007

It is recommended that the following coefficients be adopted for the design of the structure on this site according to AS 1170.4-2007 Structural design actions Part 4: Earthquake actions in Australia :-

- Hazard Design Factor (Z) = 0.08 - Figure 3.2 (F)
- Site sub-soil class, depending on founding level for the main structure shall be (Refer Section 4 of code): Class C_e – Shallow Soils Site

4.4.5 Settlement

A preliminary assessment of likely settlements of footings is required. The assessment of settlement depends upon the following factors:-

- The properties of the subsurface profile at and below the footings
- The type of footings adopted on the site
- The size of the footing
- The applied loads

Given the supplied information with respect to the proposed structure (proposed earthworks and design loads) and the results of the geotechnical investigation; provided the recommendations contained in this report are followed it is expected that the total settlement of the footings to be <10mm. It should be noted that the effects of closely spaced footings have not been taken into account and this may need to be reviewed once the footing layout and applied loads have been finalised.

4.4.6 Foundation Options

The results of the drilling and site observations to date suggest that high level foundations may only be possible in a small portion of the development. The remainder is underlain by a significant thickness of uncontrolled fill which has the potential to undergo slope failure. As such the recommendations made in this report is to completely remove and replace with more suitable fill placed in the correct manner.

The existing fill also appears to contain (in part at least) significant amounts of over sized material. This would make the installation of deep foundations very difficult if not impossible. The additional benefit of replacing the fill would be to remove these oversize fragments.

On the basis of these recommendations the use of a combination high-level/deep foundation system dimensioned for the loads and the bearing capacity of the founding material would generally be acceptable for the proposed development once the existing fill had been suitably replaced.

It should be noted that the use of the same type of footing system founding in similar material is recommended to minimise the potential for differential settlement over the structure.

4.4.7 High Level Footings

It should be noted that the use of high-level footings in highly reactive clay soils may result in shrink or swell movements of the footings adopted for the structure.

The amount of movement will depend upon:-

- the depth and size of footings,
- the material parameters of the founding layer,
- the potential for change in the moisture regime around and under the footings (i.e. in general the shallower the footing the greater the potential).

Where high level footings are adopted, our recommendation is that all footings:-

- Should be founded at least 200mm into natural stiff or better CLAYS or WEATHERED ROCK.
- Found at least (assuming masonry veneer construction):-
    - 500mm below platform level for strip footings
    - 1,000mm below platform level for pad footings
- Not be founded in Topsoil, FILL, soft or firm CLAYS.
- All footings be founded in similar stratum material.

Footings may be dimensioned for an allowable bearing pressure as outlined in Table 7.

TABLE 7 ALLOWABLE BEARING CAPACITIES FOR HIGH LEVEL FOOTINGS

Material		Allowable Bearing Capacity (kPa)[2]	
		Strip Footing	Pad Footing
Fill	- Uncontrolled - Controlled[3] - Existing	NR 100 NR	NR 125 NR
Clay	- Stiff - Very Stiff - Hard	100 200 350	125 250 425
Weathered Rock	- Above 'TC' Bit Refusal - Below 'TC' Bit Refusal	500 650	550 750

Notes:
1. NR = Not Recommended.
2. To be confirmed by inspection.
3. To be confirmed by the earthworks certifying engineer (refer Section 4.1.3)

Where necessary, footings deeper than the values indicated above may be made up with mass concrete poured to the underside of the footings, or alternatively, footings may be constructed over mass concrete filled, backhoe excavated pedestals.

4.4.8 Deep Foundations

General

A deep foundation system utilizing bored piers/screw piles founding:-

- into the top of the weathered rock or
- to a suitable founding depth based on loads and soil capacity or
- below the base of services

could be considered are recommended.

Ultimate Strength Design

The design of a deep foundation system should consider the following:-

- Compressional capacity i.e. base bearing and skin friction design values
- Tensional capacity and effects i.e. potential uplift due to expansive clays
- Lateral capacity of the piles
- Design considerations
- Construction considerations

Compressional Capacity

It is recommended that the deep foundation system on this project be designed in accordance with AS 2159-2009 'Piling - Design and Installation'. This code uses the limit state design method.

In the limit state design method the following must be taken into account:-

- Considering limit state analysis (AS 2159-2009), the design geotechnical strength $R_{d,g}$ is calculated by multiplying the ultimate geotechnical strength $R_{d,ug}$ by the geotechnical strength reduction factor ϕ_g, i.e.

$$R_{dg} = R_{d,ug}\ \phi_g \qquad \textbf{AS 2159-2009 1.3.7}$$

- Ultimate strength - the design of a single pile or a pile group must be such that both the design geotechnical strength ($R_{d,g}$) and the structural strength ($R_{d,s}$) are greater than or equal to the Design action effect (E_d) i.e.

$$R_{d,g} \geq E_d \text{ and } R_{d,s} \geq E_d$$ **AS 2159-2009 3.2.2 (d)**

- Serviceability - Single piles and pile groups shall be designed for serviceability by controlling or limiting pile movements.
- Durability - This is outlined in Section 6 of AS 2159-2009 and will not be discussed any further here.
- Any other factors that need to be considered i.e. stability, scour, fatigue, cyclic loading or seismic actions - As we are not aware that any of these factors will affect the pile design they will not be considered any further.

The design geotechnical strength ($R_{d,g}$) can be calculated as the design ultimate geotechnical strength ($R_{d,ug}$) multiplied by the geotechnical strength reduction factor (ϕ_g). The ultimate geotechnical strength parameters for the materials encountered on the site are outlined in Table 8.

TABLE 8 ULTIMATE GEOTECHNICAL STRENGTH PARAMETERS ($R_{d,ug}$)

Material		**Base Bearing (kPa)**		**Skin Friction (kPa)**
		L<4D	**L>4D**	
Fill		NR	NR	NC
Clay	- Stiff - Very Stiff - Hard	300 600 900	450 900 1350	15 30 45
Rock	- Above 'TC' Bit Refusal	1500	2200	60
	- Below 'TC' Bit Refusal	3000		100

Notes:
1. NR - Not Recommended; NC - Not considered in skin friction calculations.
2. Recommended geotechnical strength reduction factor (ϕ_g) - Refer AS 2159-2009 - for a moderate risk category and a low redundancy system a value of 0.48 is recommended. This should be confirmed by designer.
3. Considering limit state analysis (AS 2159-2009), the design geotechnical strength $R_{d,g}$ is calculated by multiplying the ultimate geotechnical strength $R_{d,ug}$ by the geotechnical strength reduction factor ϕ_g, i.e. $R_{d,g} = R_{d,ug} \times \phi_g$.
4. Should a "working stress" approach be adopted, a minimum factor of safety of 3.0 on base and 2.0 on skin friction is recommended.
5. Ignore top 2.3m of clay profile in skin friction calculations; shrinkage of the upper-level clays may occur.
6. The above parameters are for single piers. If piers are spaced at closer than three diameters, a reduction factor (Group Efficiency Ratio) may apply.
7. All values to be confirmed by inspection.

Tensional Effects - Potential Uplift Due to Swelling Soils

General

It is recommended that individual piers be assessed for uplift capacity as well as the overall pier capacity. The following sections of the Piling Code AS 2159-2009 should be considered:-

Section 3 Design Requirements and Procedures

A review of Section 3 of the piling code indicates that in Section 3.3 ACTIONS AND COMBINATIONS FOR STRENGTH AND SERVICEABILITY DESIGN that:-

"Where a pile is situated in swelling soils, such as reactive clays or those subjected to frost action, allowance shall be made for the compressive and tensile actions (F_{ex}) that may be developed in the pile." AS 2159-2009 3.3.1.2(b).

Section 3.3.2 Load combinations for strength design and where the actions are induced by ground movement, they shall be computed as below:-

- Structural Design – AS 2159-2009 3.3.2(b)(i)(B) - compressive and tensile actions – refer Section 5.
- Geotechnical Design – AS 2159-2009 3.3.2(b)(ii) - "Loads induced by soil movement shall not be taken into account"

Section 4 Geotechnical Design

In the case of geotechnical design, the effects of swelling soils are mentioned in several sections of AS 2159-2009 Section 4 Geotechnical Design. The designer is directed to review the following sections with respect to their design:-

- 4.4.6 Soil Swelling
- 4.6.4 Pile heave due to soil swelling

Section 5 Structural Design

In the case of structural design, the effect of swelling soils should be considered as part of the structural design of the foundation and as noted above:-

$$S_u = 1.5F_{es}$$ **AS 2159-2009 3.3.2 (b)(i)(B)**

Where Su is the "Ultimate value of various actions appropriate for particular combinations" AS 2159-2009 Table 1.1 (refer also AS 1170) and Fes is the tensile (in this case) actions in the pile induced by vertical ground movements.

F_{es} can be calculated using the "Rational Pier Formula" as

$$F_{es} = 0.15 * SP * A_{mv}$$

- SP = Assessed Swell Pressure of soil (kPa)
- A_{mv} = Surface area of pile in intimate contact with soil in the zone affected by swelling (i.e. above 2.3m)

Based on testing carried out on samples from the site, for design purposes SP = 220kPa.

Uplift may be reduced by constructing a separation e.g. sleeving between the pile shaft and the soil in the unstable zone.

4.4.9 Slab on Ground

General

The expected subgrade following the required earthworks is controlled or engineered fill.

Field and laboratory test results indicate that the existing natural soils, along with any controlled fill, are suitable for slab on ground construction provided that earthworks are carried out in accordance with recommendations in Section 4.1.

However, the existing fill is not considered to be suitable in its current state (certification status) to support slab on ground floors. Should earthworks operations not result in a subgrade suitable for slab support, the slab on ground floor should be supported on pedestals/piers founding in competent natural soils.

Options

Slab on ground floor options include:-

- Stiffened raft - A stiffened raft comprises perimeter and internal beams and is designed to accommodate potential ground surface movements.
- Fully suspended slab - A fully suspended slab system involves the support of the slab on piers with provision of a void beneath the slab to accommodate potential reactive clay movement, span over service trenches or areas above retaining walls to limit additional surcharges on the walls. The design of piers should be in accordance with Section 4.4.8.

4.4.10 Design Considerations

Articulation

Given the potentially reactive nature of the subsurface material, the importance of architectural and structural detailing cannot be over-emphasised. It is essential that the building be suitably articulated/detailed to allow for ground surface movement. In addition, the overall design of the building and surrounds should consider the likely ground surface movement and site classification, with the objective being to maintain a stable moisture environment.

It is recommended that any masonry walls be articulated. This articulation may be achieved by the use of full height (footings to eaves) openings or vertical construction joints at regular intervals. Guidelines on articulation are contained in the Cement and Concrete Association's Technical Note 61, 'Articulated Walling'.

Underground Services/Retaining Walls

Where footings are located adjacent to underground services or retaining walls, the footings should extend to base a minimum of 200mm below the trench/wall base level for a distance of 1.0m out from the trench/wall. Beyond 1.0m the footings should be taken a minimum of 200mm below an imaginary line drawn up at 45° from the trench/wall base level. Figure 4 refers.

These requirements do not override minimum footing levels.

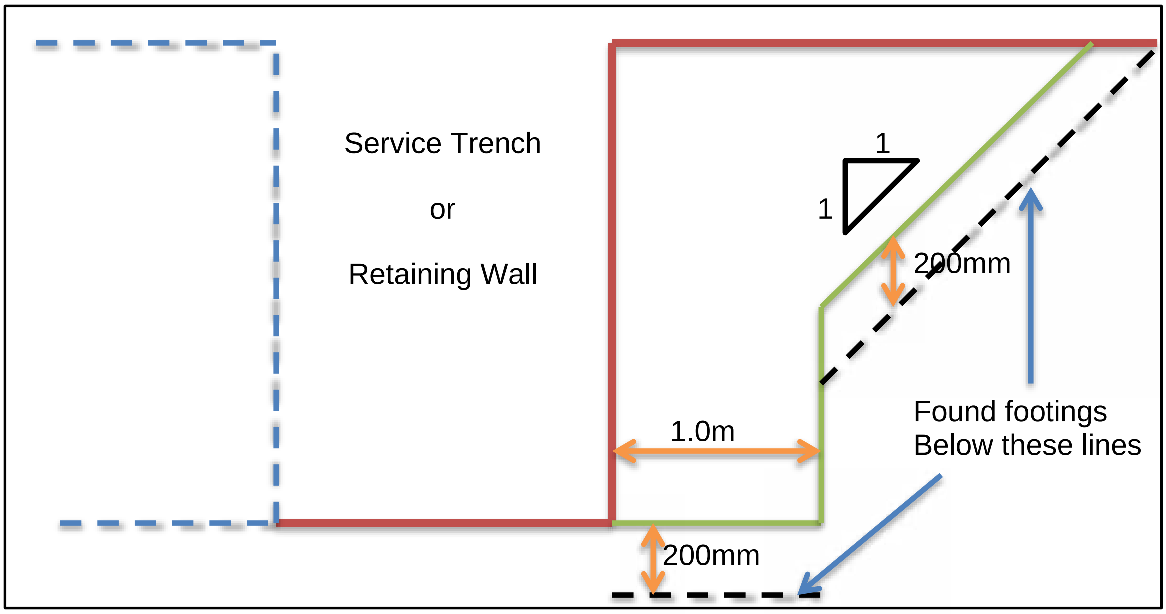


FIGURE 4

4.4.11 Construction Considerations

Inspections

It is recommended that inspections be undertaken by an experienced and qualified geotechnical engineer or engineering geologist following footing excavations and during pier excavations to confirm the adequacy of the founding material. **Inspections should be carried out prior to placement of reinforcing steel and ordering of concrete.**

High Level Footings

Given the properties of the founding materials it is recommended that the footings be poured as soon as possible following excavation, to minimise the potential for desiccation/wetting up of the founding material. Where the footings cannot be poured the same day as excavation, it is recommended that a blinding layer of concrete, at least 50mm thick, be placed immediately following excavation, cleaning and inspection of the footing bases.

Bored Piers

Some difficulty with fall-in may occur with bored piers, particularly when drilling through fill material and clayey sand. It should be ensured that all loose material is removed from the base of piers prior to pouring of concrete. The use of a 'clean-out' bucket should be explicit in instructions to the drilling contractor. The practice of 'using water and spinning the augers' to remove loose material from the pier base is generally unacceptable.

Although groundwater was not encountered in the boreholes (refer Section 2.3.4) at the time of the investigation:-

- It is recommended that, should a bored pier foundation system be adopted, some allowance for dewatering and the use of liners should be made. In addition, it may be prudent to drill a "trial pier" to fully assess construction difficulties.
- The potential for seepage still exists particularly if the piers are not poured straight after drilling. If the piers exhibit seepage or if left for a significant period (e.g. over night), at the

very least it is recommended that the pier holes should be covered and/or soil placed around the piers to minimise the possibility of water entering the pier from the surface. It is also likely that the piers will need to be cleaned and deepened to remove any softened material.

Considering the depth of fill encountered in some of the boreholes, liners may need to be considered in those holes to minimise the issue of wall collapse during the installation of the piers.

Given the nature and strength of the subsurface material encountered, it is recommended that inspections are undertaken by a qualified geotechnical engineer/engineering geologist from Soil Surveys Engineering Pty Limited during pier excavations to confirm the adequacy of the founding material. Inspections should be carried out prior to placement of reinforcing steel and following cleaning of pier bases.

Given the properties of the founding materials, it is recommended that the pier holes be poured as soon as possible following boring, to minimise the potential for desiccation/wetting up of the founding material.

4.4.12 Site Management

It is important that proper site management methods be observed for the existing soil conditions by both the builder at the time of the construction and the owner throughout the life of the proposed development.

Particular reference to this is set out in AS 2870-2011. It should be noted that where proper site management particularly with respect to change in moisture conditions is not followed the foundation recommendations contained in this report could be considered void.

The following are some specific comments with respect to site management and the reader is also directed to CSIRO pamphlet, 10-91, 'to Home Owners on Foundation Maintenance and Footing Performance' for further information.

- It is important that the site be well drained. The ground around the structures should slope away at 1 in 20 for 2 metres and then fall to the stormwater system to prevent ponding of water adjacent to the building.
- Founding soils should not be allowed to become saturated.
- It is recommended that service trenches under the structures be minimised as disruption or breakage of these service lines could lead to saturation of the under-slab/footing material resulting in an increase to the site classification. If this were to occur all footing recommendations contained within this report would be void. Where service trenches are adopted the possible effects of leakage should be considered in the design
- Where service lines are placed under a structure it is recommended that the base of these trenches fall towards the outside of the structure and that these trenches are backfilled in an appropriate manner to minimise the potential for seepage into the trench and therefore under the structure.
- Footings should be placed with minimal delay after excavation to avoid desiccation or wetting of the founding soils. If footings cannot be poured on the same day as excavation, a blinding layer of 50mm thickness is recommended. Piers should be poured immediately following excavation.

- Do not let the slab subgrade "dry out" prior to casting.
- Future shrubs and trees should be planted at a distance at least equivalent to one times their mature height away from the building to avoid shrinkage movement in the potentially expansive founding soils. Existing trees that may encroach within this restriction should be removed. It is recommended that trees to be removed as early as possible prior to building construction to enable soil moisture to reach equilibrium. Should this not be done the effect of these trees must be taken into account in the design of the structure.

4.5 Retaining Walls

4.5.1 General

It is understood that retaining walls may be required for sections of the proposed development.

The lateral earth pressure distribution that affects the retaining walls on the site will depend upon the following parameters:-

- In-situ and backfill material properties
- Design water regime at the rear of the wall
- Wall and cut geometry
- Surcharges affecting the wall
- Wall type
- The structural bracing of the wall and the timing of backfilling behind the wall

4.5.2 Design Requirements

It is recommended that any retaining structure is designed in accordance with AS 4678-2002 ‘Earth Retaining Structures’. Section 3 of this code outlines the design requirements for these retaining structures which specifically includes both Ultimate and Serviceability Limit Modes, it is recommended that the retaining structures be assessed for each of these modes.

Note the following sections provide general recommendations with respect to some of these limit modes however it doesn't provide a detailed assessment in full accordance with the above-mentioned modes for AS 4678 as this will require additional information. A detailed assessment can be undertaken of the geotechnical aspects of the Limit Modes for the proposed retaining structures once details of the wall have been provided. **Of particular importance is the assessment for Limit Mode U5 Global Failure for the proposed walls.**

4.5.3 Pressure Distribution

The following situations should be considered:-

- For cantilever walls, which allow some movement at the top, (i.e., at least 0.005H in clays) the active case (K_a) applies with a triangular distribution in both the short- and long-term situations.
- For cantilever walls which cannot tolerate this movement, the at rest case (K_o) applies with a triangular distribution in both the short- and long-term situations.

- For structurally braced walls the wall design should be checked for both a trapezoidal (clay soils - short term conditions) and triangular distribution with K_o values (long term conditions).

The pressure distributions as referred to above are shown in Figures 5 and 6. The parameters selected for use in the figures are dependent on the preconstruction geometry of the face being retained. Where the material has been cut using the recommendations as outlined in Section 4.1.6, the backfill parameters will control. Where a steeper angle is used the earth parameters will control.

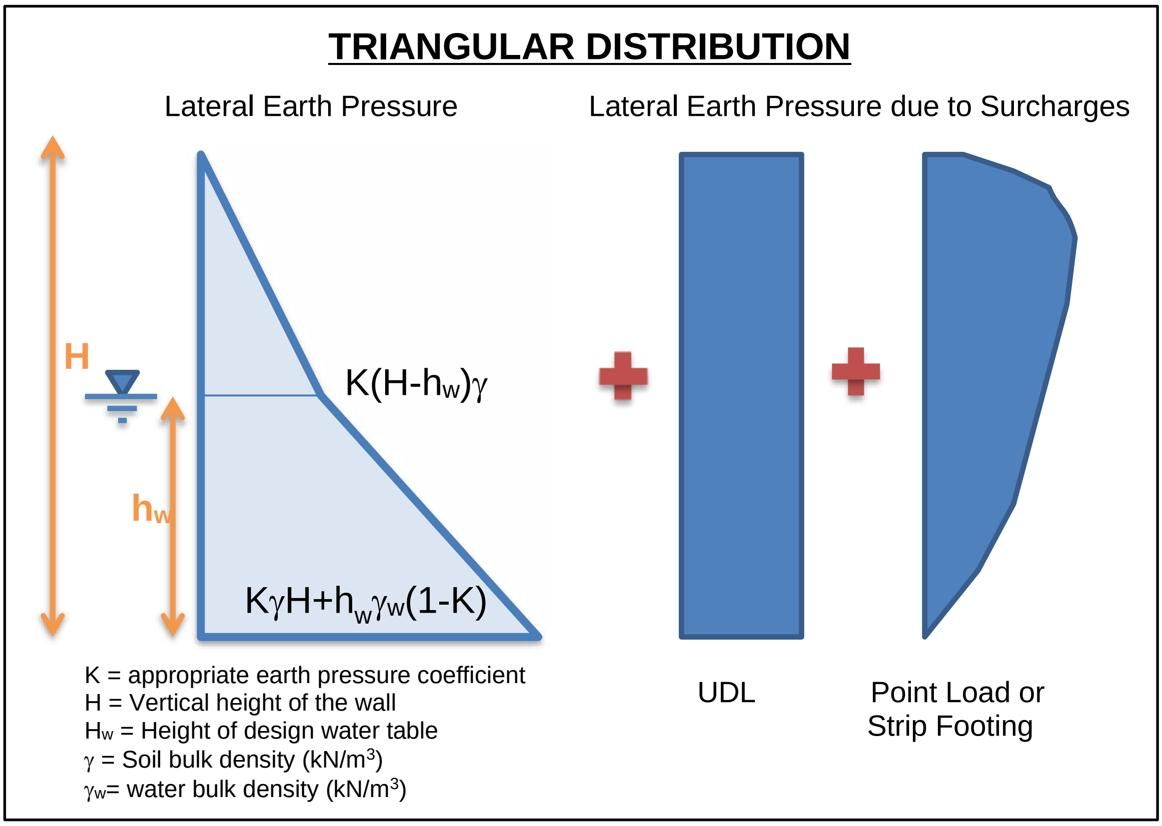


FIGURE 5

The lateral pressure distributions shown in Figures 5 and 6 include hydrostatic pressure and show typical pressure distributions due to surcharge loadings. It is recommended that where the retaining walls are expected to be surcharged (e.g., by footings, traffic loads, sloping ground surface, etc.) Soil Surveys Engineering should be contacted to provide a recommended lateral earth pressure distribution.

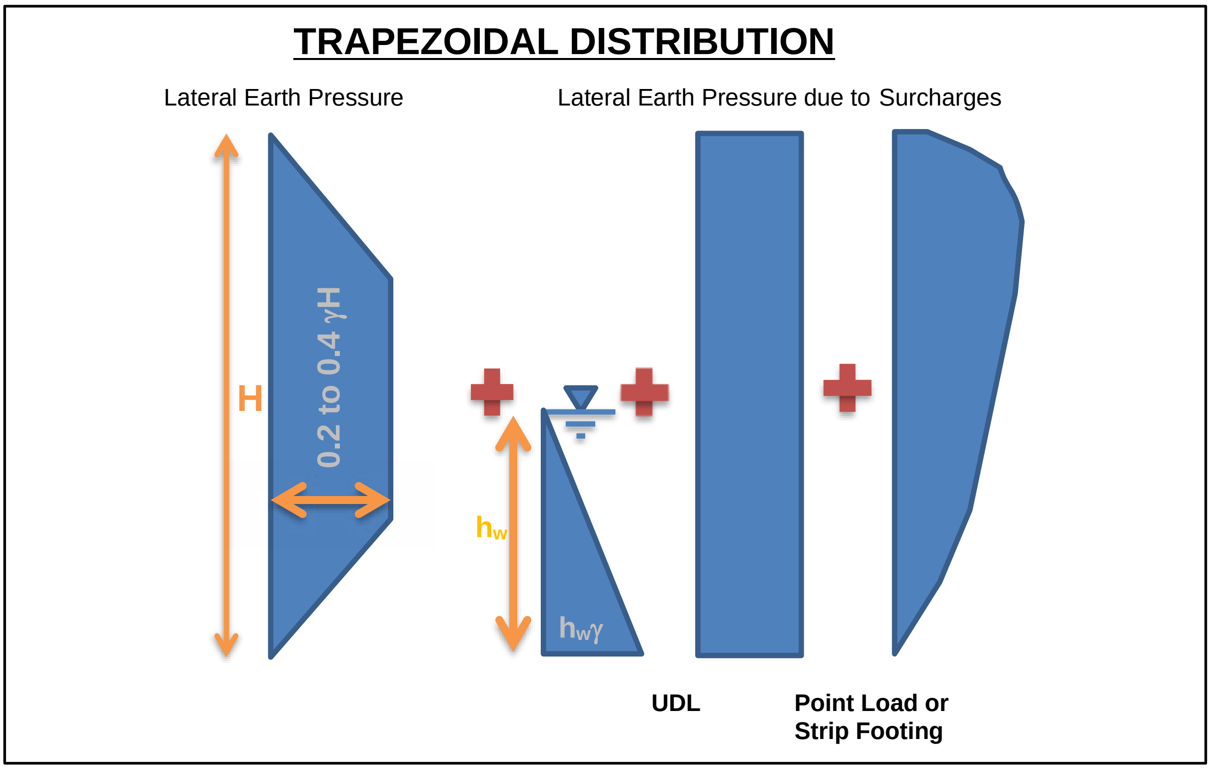


FIGURE 6

Parameters (unfactored) for assessment of lateral earth pressures are outlined in Table 9.

TABLE 9 RETAINING WALL DESIGN PARAMETERS (UNFACTORED)

Material	Density (kN/m^3)	Earth Pressure Coefficient Vertical Wall			Long Term Drained ϕ (º)
		K_a	K_o	K_p	
Existing Fill	18	0.41	0.58	NR	25
Compacted Fill	19	0.36	0.53	2.8	28
Gravel Back Fill^2	20	0.27	0.43	NR	35
Residual Clay	19	0.36	0.53	2.8	28
Weathered Rock	21	0.27	0.43	3.7	35

Notes:-
1. NC = Not Considered, NR = Not recommended.
2. Drainage gravel.

The above recommend values are based on a drained model but has assumed a cohesion value of 0kPa for all materials. Whilst testing will indicate a value in excess of 0kPa there is some concern that in the long term a very low cohesion would be more appropriate for design. Should an undrained model be more appropriate values can be provided.

4.5.4 Resistance To Lateral Forces

Retaining walls can be designed to resist lateral pressure distribution in several ways i.e.:-

- Passive resistance at the toe of the wall
- Passive resistance at the key of the wall
- Frictional resistance due to the weight of the wall
- Dowelling into rock

- Positive resistance at the base for Bored pier walls (Figure 7)

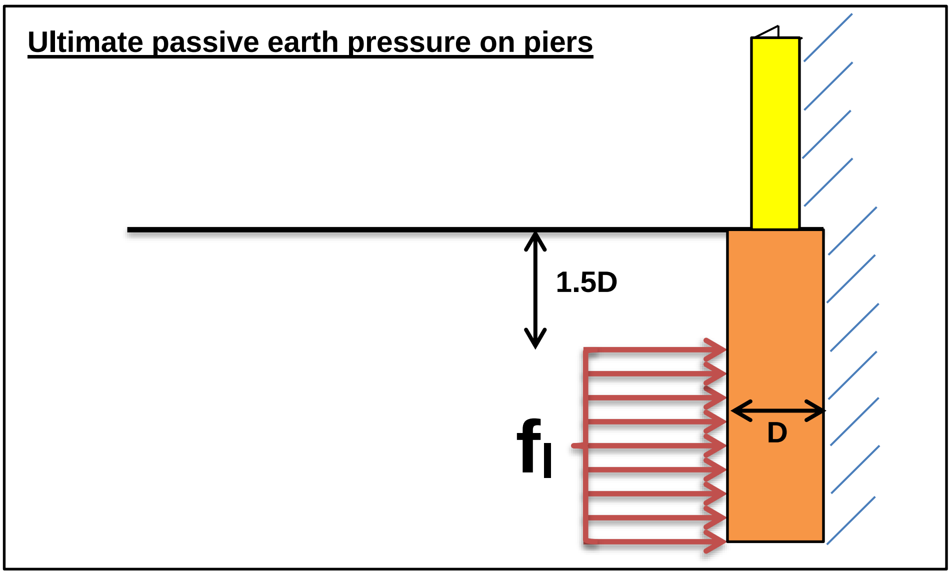


FIGURE 7

Because the resistance can be affected by various parameters, Soil Surveys Engineering should be contacted once the general wall geometry has been identified to provide further comment. For design purposes adopt f_l (design ultimate lateral capacity) values equal to the Ultimate base bearing pressure for the various material types as outlined in Table 8 in Section 4.4.8.

4.5.5 Surcharge Loads

AS 4678 requires that all structures are designed to resist live loads resulting from the intended use of the structure but not less than the values outlined in Table 4.1 of the code.

AS 4678 (Appendix J) also provides some guidance on the effect of earth pressures resulting from:-

- The geometric shape of the retaining structure.
- The effect of compaction adjacent to the retaining structure.
- Various types of surcharges.

As a general guide (using an elastic model) line loads located 2.5H away from the wall (where H is the wall height) results in lateral forces of less than 10% of the line load.

4.5.6 Backfill and Drainage

Any backfill placed behind the wall should be loose granular material. The backfill should not be heavily compacted since research has shown that compaction can raise the earth pressure to above the 'at rest' pressure (refer AS 4678 Appendix J).

Adequate surface and subsoil drainage should be provided for all retaining walls on the site. Cut-off/interceptor drains should also be provided around the high side of the wall to ensure stormwater runoff from the area above the wall is suitably diverted.

The placement of a filter fabric between the retained soil and the drainage material (e.g., granular backfill) for protection against silting of the drainage material is recommended. The outlets to subsoil pipe drains must be located beyond the ends of the walls and connected to a proper drainage system. It is suggested the pipes be wrapped in filter fabric to minimise silting.

In weather exposed locations, to reduce infiltration by surface runoff, the surface of the backfill should be sealed. This can be achieved by either compacting a material of low permeability i.e., on site clay or concrete, etc. with a slope towards an open drain.

Due to possible long-term problems with blocking of gravel filters and drains and short-term storm conditions that could flood the fill behind retaining walls, it is recommended that all retaining walls be designed for some water pressure distribution. For ultimate limit state the suggested water pressure distribution for retaining walls on this site would be half height water pressure.

During installation of any retaining walls, the in-situ soils should be battered back to minimise fall-in and subsequent disruption of works. Temporary batter angles are given in Section 4.1.6. Suitable precautions to satisfy Health & Safety requirements must also be adhered to.

5.0 CONSTRUCTION INSPECTIONS

It is recommended that placement of all structural fill, footing excavations, and cut batter slopes in soil should be inspected by Soil Surveys Engineering Pty. Limited or a duly qualified Geotechnical Engineer. Should subsurface conditions other than those described in this report be encountered, Soil Surveys Engineering Pty. Limited should be consulted immediately and appropriate modifications developed and implemented if necessary.

6.0 FURTHER WORK

It is recommended that following the preparation of design drawings that they be reviewed by Soil Surveys to confirm that they comply with the recommendations outlined in this report. Particularly with respect to the required earthworks control and certification.

7.0 LIMITATIONS

We have prepared this report for the use of **Dash House Trust**, for design purposes in accordance with generally accepted geotechnical engineering practices. No other warranty, expressed or implied, is made as to the professional advice included in this report. This report has not been prepared for use by parties other than **Dash House Trust**; it may not contain sufficient information for purposes of other parties or for other uses. Please note that any third party relying on the information contained in this report for any purpose whatsoever does so entirely at its own risk, and any duty of care to that third party is excluded.

Any interpretation or recommendation given by Soil Surveys Engineering shall be understood to be based on judgement and experience and not on greater knowledge of the facts than the reported investigations would imply. The interpretation and recommendations are therefore opinions provided for our Client’s sole use in accordance with the specific brief. As such they do not necessarily address all aspects of ground behaviour on the subject site. Information provided by others has been taken in good faith, but no liability can be accepted for information provided by others.

Your attention is drawn to 'Appendix A', 'Notes Relating to this Report'. Interpretation of factual data given in this report is based on judgement, not a greater knowledge of facts other than those reported.

Interpretation of the information shown on the logs, and its application to design and construction, should therefore take into account the spacing and depth of boreholes, the method of drilling, the frequency of sampling and testing and the possibility of other than "straight line" variations between the boreholes. Subsurface conditions between and below boreholes may vary significantly from conditions encountered at the borehole locations.

Please note that should, following detailed design, footings or piers/piles be required to extend within 3B (B = footing width/pier diameter) above the termination level of the boreholes or any excavations extend below borehole termination levels, Soil Surveys Engineering should be contacted immediately. In this case, geotechnical data in this report should be considered preliminary only; additional investigation is likely to be required.

In the event that conditions encountered on site during construction appear to vary from those expected from the information contained in the report, the Company strongly recommends that it immediately be notified. Most problems are more readily resolved when conditions are exposed than at some later stage, after the event. Should Soil Surveys Engineering not be notified or if this notification is delayed, then Soil Surveys cannot be held responsible for the affect that any variation has on any aspect of the development.

Soil Surveys Engineering consider that a documentation review service (during the design phase and prior to construction) to verify that the intent of geotechnical recommendations is properly reflected in the design, along with construction inspections, forms a very important component of the geotechnical engineering design service/process.

The geotechnical review ensures geotechnical risks to our Client and their project are minimised at the design and tender stage of the project. Further, with Soil Surveys Engineering being commissioned to carry out geotechnical construction inspections, an opportunity at the time of construction to confirm any assumptions made in the preparation of the report and allow the effect of any normally occurring variation in ground conditions to be assessed with respect to construction becomes available.

The above statements are not intended to reduce the level of responsibility accepted by Soil Surveys Engineering in accordance with our commission, but rather to ensure that all parties who may rely on this report are aware of the responsibilities each assumes in doing so and the risks they accept should they decline to have Soil Surveys Engineering carry out a geotechnical documentation review and geotechnical construction inspections.

It is highly recommended that the Client avail themselves of these review and inspection services; our standard rates will apply.

N.T. PERKINS (RPEQ 7527)
PRINCIPAL GEOTECHNICAL ENGINEER

For and on behalf of
SOIL SURVEYS ENGINEERING PTY LIMITED

APPENDICES

APPENDIX A

NOTES RELATING TO THIS REPORT

INTRODUCTION

These notes are provided by Soil Surveys Engineering Pty Limited (the Company) to complement the geotechnical report in regard to classification methods and field procedures. Not all notes are necessarily relevant to all reports.

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

DESCRIPTION AND CLASSIFICATION METHODS

Soils - The methods of description and classification of soils and rocks used in this report are based on Australian Standard 1726-2017 (Geotechnical Site Investigations), where appropriate. In general, descriptions cover the following properties - soil or rock type, colour, structure, strength or density, and inclusions. Identification and classification of soil and rock involves judgement and the Company infers accuracy only to the extent that is common in current geotechnical practice.

Soil types are described according to the dominant particle size and behaviour as set out in AS 1726-2017.

Cohesive soils are classified on the basis of strength (consistency) either by use of hand penetrometer, shear vane, laboratory testing or engineering examination. The strength terms are defined in AS 1726-2017 Table 11.

Non-cohesive soils are classified on the basis of relative density usually based on insitu testing or engineering examination (see AS 1726-2017 Table 12).

Rocks - Rock types are classified by their geological names (AS 1726-2017 Tables 15 to 18), together with descriptive terms regarding weathering (AS 1726-2017 Table 20), strength (AS 1726-2017 Table 19), defects (AS 1726-2017 Table 22), etc.

SAMPLING

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

Disturbed samples taken during drilling provide information on plasticity, grain size, colour, moisture content, minor constituents and, depending upon sample disturbance, (information on strength and structure).

Undisturbed samples are taken by pushing a thin walled sample tube, usually 50mm diameter (U50), into the soil and withdrawing it with a sample of the soil contained in a relatively undisturbed state. Such samples yield information on structure and strength, and are necessary for laboratory determination of shear strength, volume change potential and compressibility. Undisturbed sampling is generally effective only in cohesive soils.

Details of the type and method of sampling used are given on the attached logs.

SAMPLE STORAGE – SOIL, ROCK AND WATER SAMPLES

Soil samples (not subject to testing) are not stored beyond a period of 90 days of taking or receiving said soil sample. Rock core (not subject to testing) is not stored beyond a period of six months of taking or receiving said rock core.

Should any party require that soil samples (not subject to testing) be stored beyond 90 days, or rock core (not subject to testing) be stored beyond six months, please contact Soil Surveys Engineering.

Water samples (not subject to testing) are not stored beyond a period of seven days of taking or receiving water samples.

TEST LOCATIONS

Test locations (e.g. boreholes, CPT's, test pits etc.) were based on available access at the time of testing. Test locations may have been shifted if access was not suitable.

Unless noted otherwise, accuracy of test locations are to the accuracy of hand held GPS equipment.

INVESTIGATION METHODS

The following is a brief summary of investigation methods currently adopted by the Company and some comments on their use and application.

Test Pits - These are normally excavated with a backhoe or a tracked excavator, allowing close examination of the insitu soils if it is safe to descend into the pit. The depth of penetration is limited to approximately 3.0m for a backhoe and up to 6.0m for an excavator. Limitations of test pits are the problems associated with disturbance and difficulty of reinstatement and the consequent effects on close-by structures. Care must be taken if construction is to be carried out near test pit locations to either properly recompact the backfill during construction or to design and construct the structure so as not to be adversely affected by poorly compacted backfill at the test pit location.

Hand Auger Drilling - A borehole of 50mm to 100mm diameter is advanced by manually operated equipment. Refusal of the augers can occur on a variety of materials such as hard clay, gravel or rock fragments and does not necessarily indicate rock level.

Continuous Spiral Flight Augers - The borehole is advanced using 75mm to 300 mm diameter continuous spiral flight augers, which are withdrawn at intervals to allow sampling or insitu testing. This is a relatively economical means of drilling in clays and in sands above the water table. Samples are returned to the surface by the flights or may be collected after withdrawal of the augers. Information from the drilling (as distinct from specific sampling) is of relatively lower reliability due to remoulding, inclusion of cuttings from above or softening of samples by groundwater, or uncertainties as to the original depth of the samples. Augering below the groundwater table has a lower reliability than augering above the water table. Various drill bits are attached to the base of the augers during the drilling. The depth of refusal of the different bit types can provide information as to the strength of the material encountered. Generally the 'TC' bit (a tungsten carbide tipped screw type bit) is used.

Wash Boring - The borehole is usually advanced by a rotary bit with water or fluid pumped down the hollow drill rods and returned up in the space between the rods and the soil or casing, carrying the drill cuttings. Only major changes in stratification can be determined from the cuttings, together with some information from "feel" and rate of penetration. More accurate information on soil strata is gained by regular testing and sampling using the Standard Penetration Test (SPT) and undisturbed thin walled tube samples (U50).

Mud Stabilized Drilling - Either Wash Boring or Continuous Core Drilling can use drilling mud as a circulating fluid to stabilize the borehole. The term "mud" encompasses a range of products ranging from bentonite to polymers such as Revert or Biogel. The mud tends to mask the cuttings and reliable identification is only possible from regular intact sampling (e.g. from SPT and U50 samples) or from rock coring, etc.

Continuous Core Drilling - A continuous core sample is obtained using a diamond or tungsten carbide tipped core barrel. Provided full core recovery is achieved (which is not always possible in very weak rocks and granular soils), this technique provides a very reliable method of investigation. In rocks, NMLC coring (nominal 52 mm diameter) is usually used with water flush. The length of core recovered is compared to the length drilled and any length not recovered is shown as CORE LOSS. The location of losses is determined on site by the supervisor. If the location of the loss is uncertain, it is placed at the top end of the run, when the core is placed in a storage tray and recorded on the log.

Standard Penetration Tests - Standard Penetration Tests (SPT) are used mainly in non-cohesive soils, but can also be used in cohesive soils, as a means of indicating density or strength. 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 50mm diameter split sample tube with a tapered shoe, 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, the upper 150 mm being neglected due to possible disturbance from the drilling method. In dense sands, very hard clays or weak rock, the full 450 mm penetration may not be practicable and the test is discontinued at a reduced penetration.

In the case where full penetration is obtained with successive blow counts for each 150 mm of, say 4, 6 and 7 blows, the record shows,

4, 6, 7   N = 13

In a case where the test is discontinued short of full penetration, say after 15 blows for the first 150 mm and 30 blows for the next 40 mm, the record shows:

15, 30/40mm

The results of the test can be related empirically to the engineering properties of the soil.

Occasionally, the drop hammer is used to drive 50mm diameter thin walled sample tubes (U50) in clays. In such circumstances, it is noted on the borehole logs.

A modification to the SPT test is where the same driving system is used with a solid 600 tipped steel cone of the same diameter as the SPT hollow sampler. The solid cone can be continuously driven for some distance in soft clays or loose sands, or may be used where damage would otherwise occur to the SPT. The results of this Solid SPT are shown as "N_c" on the borehole logs, together with the number of blows per 150 mm penetration.

Cone Penetration Tests - Test Method - Cone Penetration Tests (CPT) are carried out in accordance with AS 1289 Test 6.5.1-1999, using an electrical friction-cone penetrometer.

The test essentially comprises the measurement of resistance to penetration of a cone of 35.7 mm diameter pushed into the soil at a rate of 10-20 mm per second by hydraulic force. The resistance to penetration is recorded in terms of pressure on the end area of the cone (cone resistance, q_c, in MPa) and friction on the side of the 135 mm long sleeve immediately above the top of the cone (friction resistance, f_s, in kPa). These forces are measured by electrical transducers (strain gauges) within the cone device. The ratio between friction resistance and cone resistance is also calculated as a percentage, i.e.-

$$\textbf{Friction Ratio (FR)} = \frac{Friction\ Resistance,\ \ f_s\ (kPa)\ x\ 100}{cone\ resistance,\ \ q_c\ (kPa)}$$

The friction ratio, FR, is generally low in sands (less than 1% or 2%) and generally higher in clays (say 3% or more). The interpretation of sandy clays, clayey sands and material with a high silt content is more difficult, but intermediate values (between 1% and 3%) would be expected. Highly organic clays and peats generally have a friction ratio in excess of 5%.

Static cone data is recorded in the field on disc for later presentation using computer aided drafting.

The equipment can be operated from any conventional drill rig. A total applied load in the range of 4 to 10 tonnes is required for practical purposes, although lighter loads may be used. The cone penetrometers are available with various capacities of cone resistance ranging up to 100 MPa for general purpose investigations, while a range of 0 to 10 MPa can be used where more sensitive investigations of soft clay are required.

The cone resistance value provides a continuous measure of soil strength or density, and together with the friction ratio, provide very useful indications of the presence of narrow bands of geotechnically significant layers such as thin, soft clay layers or lenses of sand which might otherwise be missed using conventional drilling methods.

The lithology of the encountered soils is interpreted from static cone data and is generally presented on the static cone log sheets.

It is important to note that the lithology is interpreted information and is based on research by Schmertmann (1970), Sanglerat (1972), Robinson and Campinalli (1986), modified to suit local conditions as indicated by borehole information and laboratory testing.

As soils generally change gradually it is sometimes difficult to accurately describe depths of strata changes, although greater accuracy is obtained with the static cone compared with conventional drilling. In addition, friction ratios decrease in accuracy with low cone resistance values, and in desiccated soils. As a result, some overlap and minor discrepancies may exist between static cone and nearby borehole information.

Portable Dynamic Cone Penetrometers - Portable Dynamic Cone Penetrometer (DCP) tests are carried out by driving a rod into the ground with a falling weight hammer and measuring the blows for successive 100mm increments of penetration.

The DCP comprises a Cone of 20 mm diameter with 30 degree taper attached to steel rods of smaller section.

The cone end is driven with a 9 kg hammer falling 510 mm (AS 1289 Test 6.3.2). The test was developed initially for pavement subgrade investigations, and empirical correlations of the test results with California Bearing Ratio have been published by various Road Authorities. The Company has developed their own correlations with Standard Penetration tests and Density Index tests in sands.

LOGS

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

The attached explanatory notes define the terms and symbols used in preparation of the logs.

Interpretation of the information shown on the logs, and its application to design and construction, should therefore take into account the spacing of boreholes or test pits, the method of drilling or excavation, the frequency of sampling and testing and the possibility of other than "straight line" variations between the boreholes or test pits. Subsurface conditions between boreholes or test pits may vary significantly from conditions encountered at the borehole or test pit locations.

GROUNDWATER

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

- Although groundwater may be present in lower permeability soils, it may enter the hole slowly or perhaps not at all during the time the hole is open.
- A localized perched water table may lead to an erroneous indication of the true water table.
- Water table levels will vary from time to time with seasons or recent weather changes and may not be the same at the time of construction.
- The use of water or mud as a drilling fluid will mask any groundwater inflow. Water has to be bailed out of the bore and mud must be washed out of the hole or "reverted" if water observations are to be made.

More reliable measurements can be made by use of standpipes which are read after stabilizing at periods ranging from several days to perhaps weeks for low permeability soils. Piezometers, sealed in a particular stratum, may be advisable in low permeability soils or where there may be interference from perched water tables or surface water.

FILL

The presence of fill materials can often be determined only by the inclusion of foreign objects (e.g. bricks, steel, etc.) or by distinctly unusual colour, texture or fabric. Identification of the extent of fill materials will also depend on investigation methods and frequency. Where natural soils similar to those at the site are used for fill, it may be difficult with limited testing and sampling to reliably determine the extent of the fill.

The presence of fill materials is usually regarded with caution as the possible variation in density, strength and material type is much greater than with natural soil deposits. Consequently, there is an increased risk of adverse engineering characteristics or behaviour. If the volume and quality of fill is important to a project, then frequent test pit excavations are preferable to boreholes.

LABORATORY TESTING

Laboratory testing is normally carried out in accordance with Australian Standard 1289 "Methods of Testing Soil for Engineering Purposes". Details of the test procedure used are given on the individual report forms and the attached explanatory notes summarize important aspects of the Laboratory Test Procedures adopted.

ENGINEERING REPORTS

Engineering reports are prepared by qualified personnel and are based on the information obtained and on current engineering standards of interpretation and analysis. The information provided in Soil Surveys Engineering reports is opinion and interpretation and not factual. The client/contractor increases their risk by not retaining the person who authored the geotechnical report, to carry out site inspection and review (overseeing role) during construction, to confirm opinion and interpretation expressed in the report is accurate. 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, the Company will be pleased to review the report and the sufficiency of the investigation work.

Every care is taken with the report as it relates to interpretation of subsurface conditions, discussion of geotechnical aspects and recommendations or suggestions for design and construction. Since the test sites in any exploration represent a very small proportion of the total site and since the exploration only identifies actual ground conditions at the test sites, even under the best circumstances actual conditions may vary from those inferred to exist. No responsibility is taken for:-

- Unexpected variations in ground and/or groundwater conditions.
- Changes in policy or interpretation of policy by statutory authorities.
- The actions of other persons.
- Any work where the company is not given the opportunity to supervise the construction using the Companies designs/recommendations.

If differences occur, the Company will be pleased to assist with investigation or advice to resolve any problems occurring.

SITE ANOMALIES

In the event that conditions encountered on site during construction appear to vary from those expected from the information contained in the report, the Company requests that it immediately be notified. Most problems are more readily resolved when conditions are exposed than at some later stage, well after the event.

Extreme events including but not limited to the results of climate change, e.g. flood levels above previously identified levels, beach scour or erosion beyond normal expectations (as identified by local authorities) extreme rainfall events, war, espionage, sabotage may result in different conditions between time of investigation and time of construction.

REPRODUCTION OF INFORMATION FOR CONTRACTUAL PURPOSES

Attention is drawn to the document “Guidelines for the Provision of Geotechnical Information in Construction Contracts (1987)”, published by the Institution of Engineers, Australia. Where information obtained from this investigation is provided for tendering purposes, it is recommended that all information, including the written report and discussion, be made available. In circumstances, where the discussion or comments section is not relevant to the contractual situation, it may be appropriate to prepare a specially edited document. The Company would be pleased to assist in this regard and/or to make additional report copies available for contract purposes at a nominal charge.

REVIEW OF DESIGN

Where major civil or structural developments are propose or where only a limited investigation has been completed or where the geotechnical conditions/ constraints are quite complex, it is prudent to have a joint design review which involves a senior geotechnical engineer. We would be happy to assist in this regard as an extension of our investigation commission. Construction drawings should be reviewed by Soil Surveys Engineering, with sufficient time to allow changes if required, prior to inspections. Otherwise Soil Surveys Engineering reserves the right to refuse to carry out inspections.

SITE INSPECTION

The Company will always be pleased to provide engineering inspection services for geotechnical aspects of work to which this report is related.

i. Site visits during construction to confirm reported ground conditions
ii. Site visits to assist the contractor or other site personnel in identifying various soil/rock types such as appropriate footing or pier founding depths, the stability of a filled or excavated slope; or
iii. Full-time engineering presence on site.

In the vast majority of cases it is advantageous to the principal for the geotechnical engineer who wrote the investigation report to be involved in the construction stage of the project.

The geotechnical engineer cannot take responsibility for variations in encountered conditions, where he is not given the opportunity to review plans for the proposed development with sufficient time to allow review and make changes to the proposed development if required, and where he is not given the opportunity to inspect the site and oversee construction methods with regard to site conditions with sufficient time to observe all relevant site conditions and operations.

RESPONSIBLE USE OF GEOTECHNICAL INFORMATION

Recommendations in our report are for design purposes only and provided on the basis that inspections are carried out to allow finalisation of opinions and recommendations contained in our report.

The geotechnical investigation consisting of field and laboratory testing has been carried out to indicate typical conditions by indicating conditions and parameters at the specific locations of boreholes/test pits. Subsurface conditions are indicated at these locations only and the inference of conditions between or away from these locations (interpolation and extrapolation) involves a certain degree of risk. Persons inferring such conditions or carrying out such inferences should do so with a degree of caution and conservatism which is commensurate with the consequences of the risk of error.

Estimates of volumes based on our findings require interpolation and extrapolation between test locations and as such may be significantly different from actual volumes.

APPENDIX B

ASSESSMENT GUIDELINES

B.1 STATE PLANNING POLICY (JULY, 2017)

B.1.1 General

In 2003 the Queensland State Government adopted under the Integrated Planning Act 1997 (IPA) the State Planning Policy 1/03 "Mitigating the Adverse Impacts of Flood, Bushfire and Landslide" and an associated guideline.

In 2014, the Queensland State Government reissued all the State Planning Policies under a single document (State Planning Policy July 2014 - SPP).

In July 2017, the Queensland State Government reviewed the State Planning Policies, releasing the updated State Planning Policy July 2017 – SPP.

Landslide is addressed under two sections of the document, these being:-

Part E - State interest policies and assessment benchmarks

The SPP contains state interest policies and where relevant, the assessment benchmarks for each state interest. These are relevant for landslide hazards in the following sections:-

- Planning for the environment and heritage
- Planning for safety and resilience to hazards
- Natural hazards, risk and resilience

As noted in the policy, the risks associated with natural hazards, including the projected impacts of climate change, are avoided or mitigated to protect people and property and enhance the community's resilience to natural hazards.

All of the following state interest policies must be appropriately integrated in planning and development outcomes, where relevant:

1. Natural Hazard areas are identified, including:-
    a. Bushfire prone areas
    b. Flood hazard areas
    c. Landslide hazard areas
    d. Storm tide inundation areas
    e. Erosion prone areas
2. A fit-for-purpose risk assessment is undertaken to identify and achieve an acceptable or tolerable level of risk for personal safety and property in natural hazard areas.

Bushfire, flood, landslide, storm tide inundation, and erosion prone areas:

1. Land in an erosion prone area is not to be used for urban purposes, unless the land is located in:
    a. an urban area in a planning scheme; or
    b. an urban footprint identified in a regional plan.
2. Development in bushfire, flood, landslide, storm tide inundation or erosion prone natural hazard areas:
    a. avoids the natural hazard area; or
    b. where it is not possible to avoid the natural hazard area, development mitigates the risks to people and property to an acceptable or tolerable level.
3. Development in natural hazard areas:
    a. supports, and does not hinder disaster management capacity and capabilities
    b. directly, indirectly and cumulatively avoids an increase in the exposure or severity of the natural hazard and the potential for damage on the site or to other properties

    c. avoids risks to public safety and the environment from the location of the storage of hazardous materials and the release of these materials as a result of a natural hazard
    d. maintains or enhances the protective function of landforms and vegetation that can mitigate risks associated with the natural hazard.
4. Community infrastructure is located and designed to maintain the required level of functionality during and immediately after a natural hazard event.
5. Coastal protection work in an erosion prone area is undertaken only as a last resort where coastal erosion or inundation presents an imminent threat to public safety or existing buildings and structures[5], and all of the following apply:
    a. The building or structure cannot reasonably be relocated or abandoned.
    b. Any erosion control structure is located as far landward as practicable and on the lot containing the property to the maximum extent reasonable.
    c. Any increase in coastal hazard risk for adjacent areas from the coastal protection work is mitigated.

Assessment benchmarks – natural hazards, risk and resilience

These performance outcomes apply to the following development applications, to the extent the SPP has not been identified in a local planning instrument as being appropriately integrated.

A development application for a material change of use, reconfiguration of a lot or operational works on premises in any of the following:

1. bushfire prone areas
2. flood hazard areas
3. landslide hazard areas
4. storm tide inundation areas
5. erosion prone area

All of the following requirements are assessment benchmarks for the development in relation to all natural hazard areas:

1. Development supports and does not hinder disaster management response or recovery capacity and capabilities.
2. Development directly, indirectly and cumulatively avoids an increase in the severity of the natural hazard and the potential for damage on the site or to other properties.
3. Risks to public safety and the environment from the location of hazardous materials and the release of these materials as a result of a natural hazard are avoided.
4. The natural processes and the protective function of landforms and the vegetation that can mitigate risks associated with the natural hazard are maintained or enhanced.

In accordance with the SPP, risks to the community for proposed developments in identified Natural Hazard Management Areas should be adequately considered during design and assessment of a proposed development. Where risks are unacceptable they should be suitably minimised and/or controlled.

Part F – Glossary

The SPP defines the following:-

- **Landslide hazard area** means an area that is:

a) identified by a local government in a local planning instrument as a landslide hazard area, based on a localised landslide study prepared by a suitably qualified person; or
b) if the local government has not identified landslide hazard areas in a local planning instrument in accordance with (a) above; land with a slope greater than or equal to 15 per cent.

- **Natural hazard** means a naturally occurring situation or condition, such as a flood, bushfire, landslide, coastal erosion or storm tide inundation, with the potential for loss or harm to the community, property or environment.
- **Natural hazard area** means a flood hazard area, a bushfire hazard area, a landslide hazard area, an erosion prone area or a storm tide inundation area.

B.2 TOOWOOMBA *REGIONAL PLANNING SCHEME, VERSION 21, AUGUST, 2019*

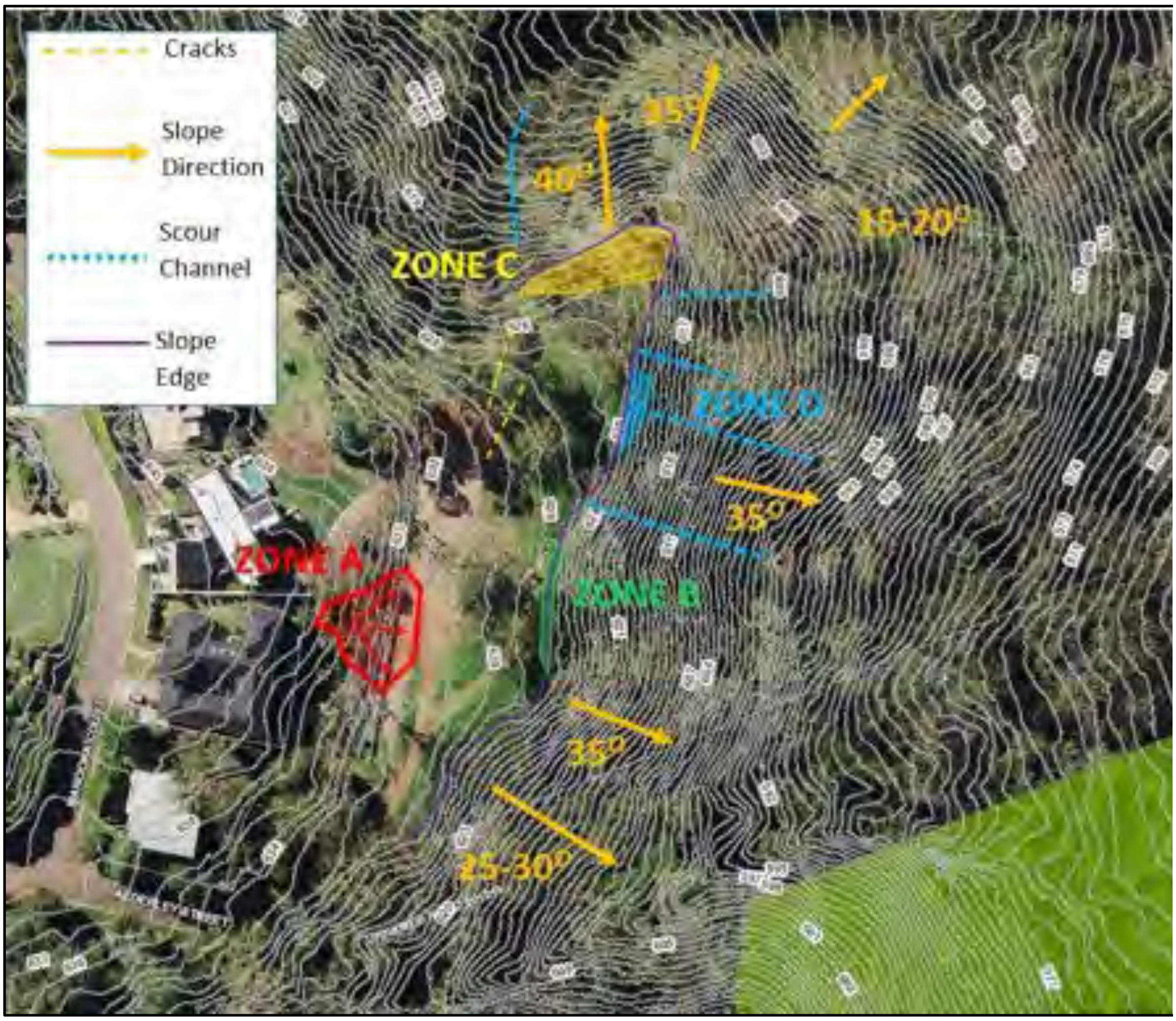
The Toowoomba Regional Council Planning Scheme Version 21, 2019 includes two items in relation to Landslide hazards:-

- Part 8.2.4 Landslide Hazard Overlay Code – Contains application information, purpose and overall outcomes and criteria for assessment.
- Schedule 6.1.6 Additional Information required by Development and Overlay Code - Contains additional information requirements triggered by the Landslide Hazard Overlay Code.

Figure 1 shows the relevant section of the Landslide Hazard Overlay and based on this, essentially the entire site is situated within the Landslide Hazard High Risk Zone**.** Therefore, the TRC Planning Scheme Version 21 2019 will apply to the development.

APPENDIX C

SITE REPORT



ZONE A – Slip on neighbouring land to the west.

- Predominantly Silty Clay with sand, gravels and organics.
- 1m high blockwork retaining wall destroyed, slump spilling into subject site. Overturned sewer manhole, vegetation and retaining wall debris noted in slump material. Soil at toe of slump downslope from manhole is wet.




ZONE B – Sunken section along edge of slope.

- Dished section running along slope edge in this area, approximately 0.3m deep in the centre, grass covered.
- Expected fill has consolidated in this area, no visible scouring on slope.


ZONE C – Cracked and slumped ground at top of slope.

- Cracks running the entire length of the slumped area from west to east. Crack widths ranging from 15-40mm.
- Ground has slumped approximately 0.95m from edge of slump to lowest point of slump.
- Some 10-20mm cracks also noted to the south west of slumped area.

ZONE D – Eroded section at top of hill leading to scour channel.

- Fill has slumped and scoured away in this area, revealing boulders and large cobbles in the fill.
- Many rills branching off the main scour channel and throughout this area of the slope face. Little to no vegetation in this area.

APPENDIX D

PHOTOGRAPHS

PLATE 1 – SITE LOOKING NORTH FROM NEAR BH1

PLATE 2 – SITE LOOKING WEST. NOTE: SLUMP ON ADJACENT SITE

PLATE 3 – RECENT SLUMP ON OUTER EDGE

PLATE 4 – NORTHERN END OF PLATFORM

PLATE 5 – CLOSE UP OF FILL BATTER

PLATE 6 – CRACKING OF FILL BATTER

APPENDIX E

LRM2007 PRACTICE NOTE GUIDELINES – RISK MATRIX

APPENDIX C: – QUALITATIVE TERMINOLOGY FOR USE IN ASSESSING RISK TO PROPERTY (CONTINUED)

QUALITATIVE RISK ANALYSIS MATRIX – LEVEL OF RISK TO PROPERTY

LIKELIHOOD		CONSEQUENCES TO PROPERTY (With Indicative Approximate Cost of Damage)				
	Indicative Value of Approximate Annual Probability	**1: CATASTROPHIC 200%**	**2: MAJOR 60%**	**3: MEDIUM 20%**	**4: MINOR 5%**	**5: INSIGNIFICANT 0.5%**
A – ALMOST CERTAIN	10^{-1}	VH	VH	VH	H	M or **L** (5)
B - LIKELY	10^{-2}	VH	VH	H	M	**L**
C - POSSIBLE	10^{-3}	VH	H	M	M	**VL**
D - UNLIKELY	10^{-4}	H	M	**L**	**L**	**VL**
E - RARE	10^{-5}	M	**L**	**L**	**VL**	**VL**
F - BARELY CREDIBLE	10^{-6}	**L**	**VL**	**VL**	**VL**	**VL**

Notes: (5) For Cell A5, may be subdivided such that a consequence of less than 0.1% is Low Risk.

(6) When considering a risk assessment it must be clearly stated whether it is for existing conditions or with risk control measures which may not be implemented at the current time.

RISK LEVEL IMPLICATIONS

Risk Level		**Example Implications** (7)
VH	VERY HIGH RISK	Unacceptable without treatment. Extensive detailed investigation and research, planning and implementation of treatment options essential to reduce risk to Low; may be too expensive and not practical. Work likely to cost more than value of the property.
H	HIGH RISK	Unacceptable without treatment. Detailed investigation, planning and implementation of treatment options required to reduce risk to Low. Work would cost a substantial sum in relation to the value of the property.
M	MODERATE RISK	May be tolerated in certain circumstances (subject to regulator's approval) but requires investigation, planning and implementation of treatment options to reduce the risk to Low. Treatment options to reduce to Low risk should be implemented as soon as practicable.
L	LOW RISK	Usually acceptable to regulators. Where treatment has been required to reduce the risk to this level, ongoing maintenance is required.
VL	VERY LOW RISK	Acceptable. Manage by normal slope maintenance procedures.

Note: (7) The implications for a particular situation are to be determined by all parties to the risk assessment and may depend on the nature of the property at risk; these are only given as a general guide.

APPENDIX F

BOREHOLE RECORD SHEETS

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SOIL SURVEYS

BOREHOLE RECORD SHEET

Location Number: BH 01
Project Number: 1-24866
Project Name: Proposed Residence
Location: Alderley Street, Rangeville
Client: Dash House Trust
Date: 03/03/2022

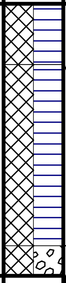
Page: 1 OF 1

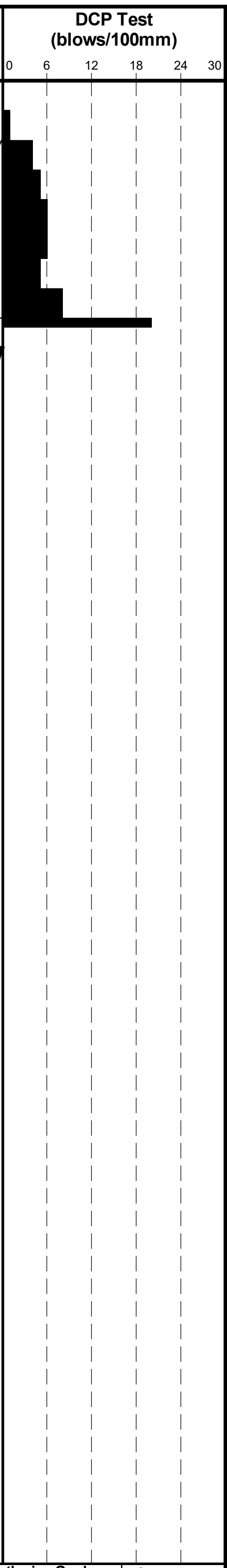
Easting: 400247 Northing: 6947797 RL:
Logger: JCG Operator: JCG Machine: J 105

Drilling Method (TC / WB / RR / NMLC / Casing)	Depth	Description	Samples and Remarks
TC	0.20	FILL Sandy Gravelly CLAY (CL): Stiff, low plasticity, orange red brown dark grey, fine sized gravel, fine to coarse grained sand, w=pl	D
TC	0.80	FILL Sandy CLAY (CL): Stiff to very stiff, low plasticity, red brown mottled dark grey, fine to coarse grained sand, trace fine sized gravel, w=pl	U50 PP=>600 Rec = 100%
TC	0.90	FILL GRAVEL (GP): Very dense, coarse sized, light brown mottled light grey, with fine grained sand, with boulders, moist	

BOREHOLE BH 01 TERMINATED AT 0.90 m

Depth scale: 0.5, 1.0, 1.5, 2.0, 2.5, 3.0, 3.5, 4.0, 4.5, 5.0





Comments:
1. Groundwater not encountered during augering.
2. DCP refusal at 0.83m.
3. TC bit refusal at 0.90m.

Water First Noted Water Steady Level Water Loss

Weathering Grades
RS - Residual Soil
XW - Extremely weathered
HW - Highly weathered
DW - Distinctly weathered
MW - Moderately weathered
SW - Slightly weathered
FR - Fresh

Rock Strength
RS - Residual Soil
VL - Very low
L - Low
M - Medium
H - High
VH - Very high
EH - Extremely high

Samples
U50 █
SPT]
Disturbed Sample [
Bulk Sample [

Approved: NTP
Date: 9/05/2023

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SOIL SURVEYS

BOREHOLE RECORD SHEET

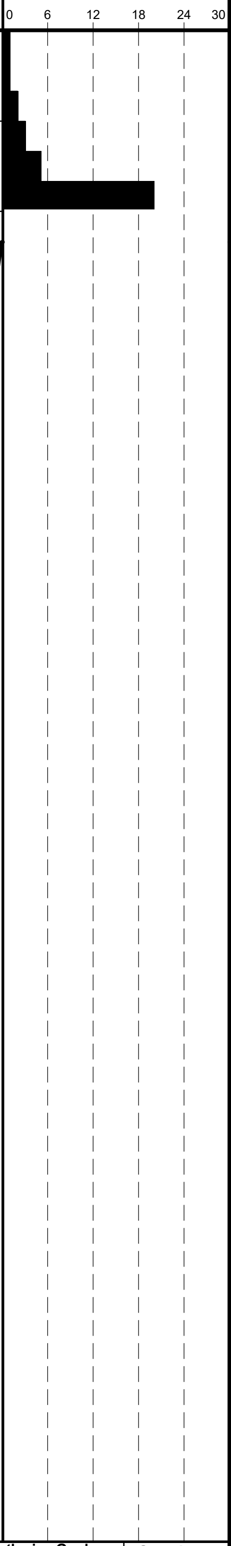
Location Number: BH 01 A
Project Number: 1-24866
Project Name: Proposed Residence
Location: Alderley Street, Rangeville
Client: Dash House Trust
Date: 03/03/2022
Page: 1 OF 1

Easting: 400248 Northing: 6947798 RL:
Logger: JCG Operator: JCG Machine: J 105

Drilling Method	Depth	Description	Samples and Remarks
TC	0.00 – 0.30	FILL Sandy Gravelly CLAY (CL): Firm, low plasticity, red brown mottled dark grey, fine to medium sized gravel, fine to coarse grained sand, w>pl	
TC	0.30 – 0.60	FILL Sandy Gravelly CLAY (CL): Stiff, low plasticity, red brown mottled dark grey, fine to medium sized gravel, fine to coarse grained sand, w=pl	
TC	0.60 – 0.70	FILL GRAVEL (GP): Dense, coarse sized, light grey mottled light brown, with fine to medium grained sand, moist (cobble/boulder)	
		BOREHOLE BH 01 A TERMINATED AT 0.70 m	

Depth scale: 0.5, 1.0, 1.5, 2.0, 2.5, 3.0, 3.5, 4.0, 4.5, 5.0

DCP Test (blows/100mm)



Comments:
1. Groundwater not encountered during augering.
2. DCP refusal at 0.59m.
3. TC bit refusal at 0.70m.

Water First Noted, Water Steady Level, Water Loss

Weathering Grades
RS - Residual Soil
XW - Extremely weathered
HW - Highly weathered
DW - Distinctly weathered
MW - Moderately weathered
SW - Slightly weathered
FR - Fresh

Rock Strength
RS - Residual Soil
VL - Very low
L - Low
M - Medium
H - High
VH - Very high
EH - Extremely high

Samples
U50
SPT
Disturbed Sample
Bulk Sample

Approved: NTP
Date: 9/05/2023

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SOIL SURVEYS

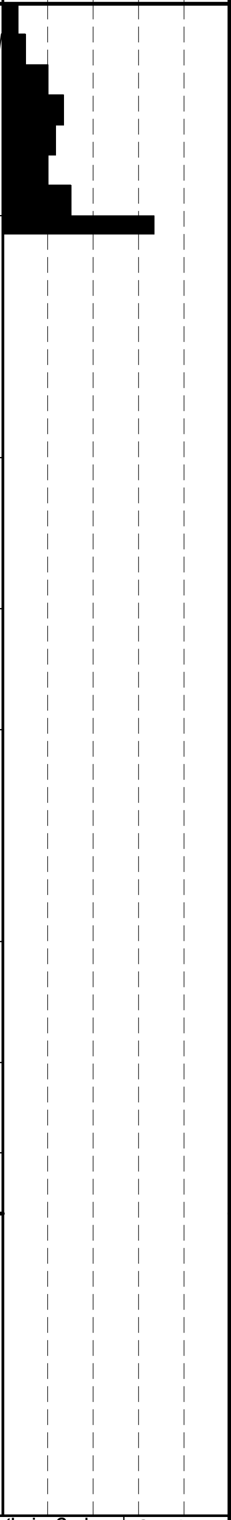
BOREHOLE RECORD SHEET

Location Number: BH 02
Project Number: 1-24866
Project Name: Proposed Residence
Location: Alderley Street, Rangeville
Client: Dash House Trust
Date: 03/03/2022
Page: 1 OF 1

Easting: 400266 Northing: 6947813 RL:
Logger: JCG Operator: JCG Machine: J 105

Drilling Method	Depth	Description	Samples and Remarks
TC	0.10	FILL Clayey Gravelly SAND (SC): Loose, fine to coarse grained, dark grey light brown, low plasticity clay, fine to medium sized gravel, moist	D
TC	0.70	FILL Sandy CLAY (CL): Very stiff, low plasticity, orange red brown mottled dark grey, fine to medium grained sand, trace fine to medium sized gravel, w<pl	D U50 PP=>600 Rec = 100%
TC	1.50	FILL Clayey Sandy GRAVEL (GC): Medium dense, coarse sized, red brown dark grey, low plasticity clay, fine to medium grained sand, with cobble, moist	
TC	2.00	FILL Silty CLAY (CH): Very stiff, high plasticity, light brown, with fine to coarse grained sand, w>pl	D
TC	2.40	FILL Sandy CLAY (CI): Stiff to very stiff, medium plasticity, dark brown mottled red brown dark grey, fine to medium grained sand, w>pl	U50 PP=260 Rec = 60%
TC	3.10	FILL Sandy CLAY (CH): Stiff to very stiff, high plasticity, dark brown mottled red brown dark grey, fine to coarse grained sand, with fine sized gravel, w>pl	U50 PP=>600 Rec = 89%
TC	3.50	FILL Clayey Gravelly SAND (SC): Medium dense, fine to medium grained, dark grey yellow, low plasticity clay, fine to medium sized gravel, moist (possibly colluvium)	
TC	3.80	NATURAL Extremely weathered Basalt (XW) recovered as Sandy SILT (ML): Hard, low plasticity, yellow brown, fine grained sand, inorganic, moist	D
TC	4.00	BASALT (DW): Distinctly weathered, low strength, fine grained, brown yellow, moist	
		BOREHOLE BH 02 TERMINATED AT 4.00 m	

DCP Test (blows/100mm)



Comments:
1. Groundwater not encountered during augering.
2. DCP refusal at 0.76m.
3. TC bit refusal at 4.00m.

Water First Noted Water Steady Level Water Loss

Weathering Grades
RS - Residual Soil
XW - Extremely weathered
HW - Highly weathered
DW - Distinctly weathered
MW - Moderately weathered
SW - Slightly weathered
FR - Fresh

Rock Strength
RS - Residual Soil
VL - Very low
L - Low
M - Medium
H - High
VH - Very high
EH - Extremely high

Samples
U50
SPT
Disturbed Sample
Bulk Sample

Approved: NTP
Date: 9/05/2023

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BOREHOLE RECORD SHEET

Location Number: BH 03
Project Number: 1-24866
Project Name: Proposed Residence
Location: Alderley Street, Rangeville
Client: Dash House Trust
Date: 16/03/2022

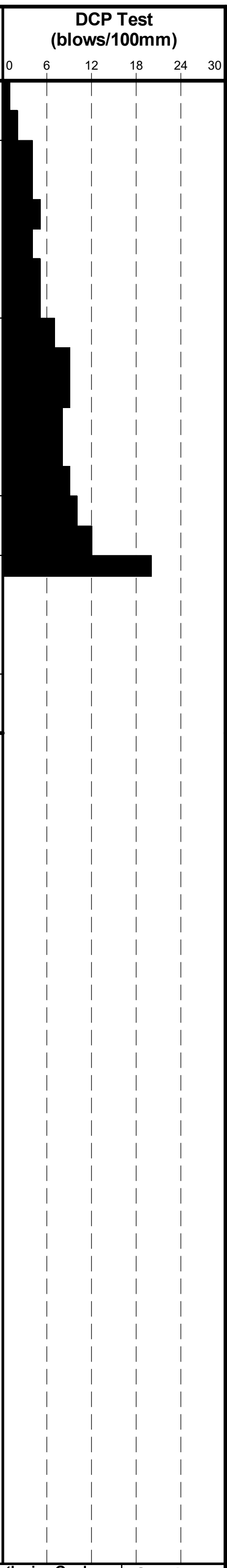
Page: 1 OF 1

Easting: 400252 Northing: 6947840 RL:
Logger: JCG Operator: JCG Machine: J 105

Drilling Method: TC | WB | RR | NMLC | Casing — TC to 2.20

Depth	Description	Samples and Remarks
0.00–0.20	FILL Silty Gravelly SAND (SM): Loose, fine to coarse grained, light brown dark grey, fine to coarse sized gravel, moist	
0.20–0.80	NATURAL Silty CLAY (CI): Stiff to very stiff, medium plasticity, red brown, w=pl	D
0.80–1.40	Silty CLAY (CL): Very stiff, low plasticity, red brown mottled dark grey, with fine grained sand, w<pl	U50 PP=>600 Rec = 96%
1.40–1.60	Hard	
1.60–2.00	Extremely weathered Basalt (XW) recovered as Clayey SAND (SC): Dense, fine to medium grained, yellow grey brown, low plasticity clay, moist	D
2.00–2.20	BASALT (DW): Distinctly weathered, low strength, fine to medium grained, light grey brown, moist	

BOREHOLE BH 03 TERMINATED AT 2.20 m



Depth: 0.5, 1.0, 1.5, 2.0, 2.5, 3.0, 3.5, 4.0, 4.5, 5.0

Comments:
1. Groundwater not encountered during augering.
2. DCP refusal at 1.67m.
3. TC bit refusal at 2.20m.

▽ Water First Noted ▼ Water Steady Level ◀ Water Loss

Weathering Grades
RS - Residual Soil
XW - Extremely weathered
HW - Highly weathered
DW - Distinctly weathered
MW - Moderately weathered
SW - Slightly weathered
FR - Fresh

Rock Strength
RS - Residual Soil
VL - Very low
L - Low
M - Medium
H - High
VH - Very high
EH - Extremely high

Samples
U50 █
SPT]█
Disturbed Sample [
Bulk Sample [

Approved: NTP
Date: 9/05/2023

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SOIL SURVEYS

BOREHOLE RECORD SHEET

Location Number: BH 04

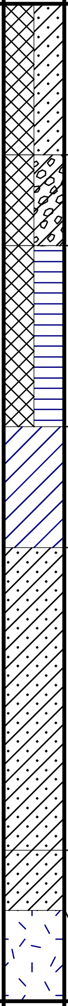
Project Number: 1-24866
Project Name: Proposed Residence
Location: Alderley Street, Rangeville
Client: Dash House Trust
Date: 16/03/2022
Page: 1 OF 1

Easting: 400269   Northing: 6947834   RL:
Logger: JCG   Operator: JCG   Machine: J 105

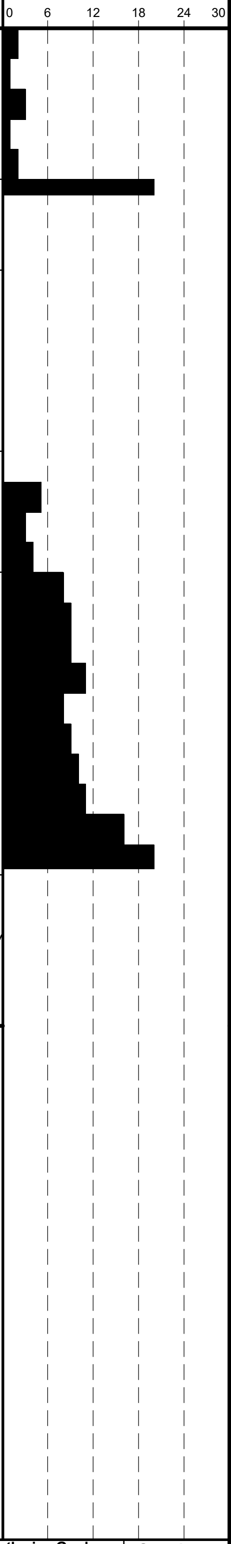
Drilling Method (TC / WB / RR / NMLC / Casing)	Depth	Description	Samples and Remarks
TC	0.00 – 0.50	FILL Clayey SAND (SC): Loose, fine to medium grained, red brown dark brown, low plasticity clay, moist	D
TC	0.50 – 0.80	FILL Clayey Sandy GRAVEL (GC): Medium dense, fine to coarse sized, red brown dark grey, low plasticity clay, fine to medium grained sand, with cobbles, moist	
TC	0.80 – 1.40	FILL Sandy Gravelly CLAY (CL): Very stiff to hard, low plasticity, dark brown dark grey red brown, fine to coarse sized gravel, fine to medium grained sand, w<pl	D
TC	1.40 – 1.80	NATURAL Silty Sandy CLAY (CI): Stiff, medium plasticity, dark brown mottled red brown, fine to medium grained sand, w=pl	D
TC	1.80 – 2.80	Clayey Silty SAND (SC): Dense, fine to medium grained, yellow grey brown, low plasticity clay, moist	D
TC	2.80 – 3.00	Extremely weathered Basalt (XW) recovered as Clayey SAND (SC): Dense, fine to medium grained, yellow grey brown, low plasticity clay, moist	
TC	3.00 – 3.30	BASALT (DW): Distinctly weathered, low strength, fine to medium grained, yellow grey brown, moist	

BOREHOLE BH 04 TERMINATED AT 3.30 m

Graphic



DCP Test (blows/100mm)



Comments:
1. Groundwater not encountered during augering.
2. DCP refusal at 0.55m and 2.78m.
3. TC bit refusal at 3.30m.

⊻ Water First Noted ⊻ Water Steady Level ◀ Water Loss

Weathering Grades
RS - Residual Soil
XW - Extremely weathered
HW - Highly weathered
DW - Distinctly weathered
MW - Moderately weathered
SW - Slightly weathered
FR - Fresh

Rock Strength
RS - Residual Soil
VL - Very low
L - Low
M - Medium
H - High
VH - Very high
EH - Extremely high

Samples
U50 ■
SPT]
Disturbed Sample [
Bulk Sample [

Approved: NTP
Date: 9/05/2023

APPENDIX G

LABORATORY TEST CERTIFICATES

SOIL SURVEYS ENGINEERING PTY LIMITED
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SHRINK SWELL INDEX REPORT

Client :	**Dash House Trust**	Report No:	WHL22-0202-S1-S2
Client Address :	**C/O 2/19 Finchley Street, Milton, QLD, 4064**	Report Date:	24/03/2022
Project Number :	**1-24866**	Issue Number:	1
Project :	**Proposed Residence**	Page 1 of 1	
Location :	**Rangeville**		

SAMPLE DETAILS:					
Sample ID:	WHL22-0202-S1	WHL22-0202-S2			
Date Sampled:	3/03/2022	3/03/2022			
Date Tested:	21/03/2022	21/03/2022			
Soil Description:	Silty Clay (CH) Trace of Sand & Gravel, Brown, Black & Orange Mottled	Silty Clay (CH) Trace of Sand & Gravel, Brown/ Red Brown			
Sampling Method:	As Supplied	As Supplied			
Sampled by:	SSE	SSE			
Sample Location:	**BH02**	**BH03**			
Depth (m):	**0.6**	**0.6**			
Source:	Borehole	Borehole			
Material:	Unknown	Unknown			
TEST RESULTS:					
Swell Test (AS1289.7.1.1)					
Swell on Saturation(%)	**1.4**	**5.8**			
Moisture Content Before(%)	24.6	20.9			
Moisture Content After(%)	27.5	27.0			
Shrink Test (AS1289.7.1.1)					
Shrink on Drying(%)	**4.1**	**2.8**			
Shrinkage Moisture Content(%)	25.0	22.1			
Est. Inert Material(%)	5	2			
Crumbling	None	None			
Cracking	Slight	Slight			
Results:					
Shrink Swell Index - Iss (%):	**2.7**	**3.2**			
Swell Pressure (kPa)		220			
Unit Weight (t/m^3):	1.98	2.15			
Remarks :					

This document is issued in accordance with Soil Surveys Engineering quality requirements.Accredited for compliance with ISO/IEC 17025 - Testing.This document shall not be reproduced, except in full without the approval of Soil Surveys Engineering Pty Limited.Results Only apply to test items in this report. Swell Pressure readings are not Covered By this Laboratories Current Scope of Accreditation

APPROVED SIGNATORY
C.Ferguson-Hannah

Nata Accreditation number: 15301

END OF DOCUMENT

FORM NUMBER **REP-ISS-05** V3 19/9/21

APPENDIX H

SITE PLANS

LEGEND

🔴 Borehole Locations

╱ Approx. Lot Boundary

Image from Nearmap.com (15/11/2021)

SCALE 1:750 @ A4

REFERENCE V:\Milton\Milton 1-24001 - 1-25000\1-24801 - 1-25000\1-24866 Rangeview\Graphics

A4

DRW NO 1-24866-01

DATE 18/02/2022

CHECKED NTP

CLIENT Dash House Trust

LOCATION 1F Alderley Street
Rangeview

DRAWING TITLE

Bore Hole Locations

Soil Surveys
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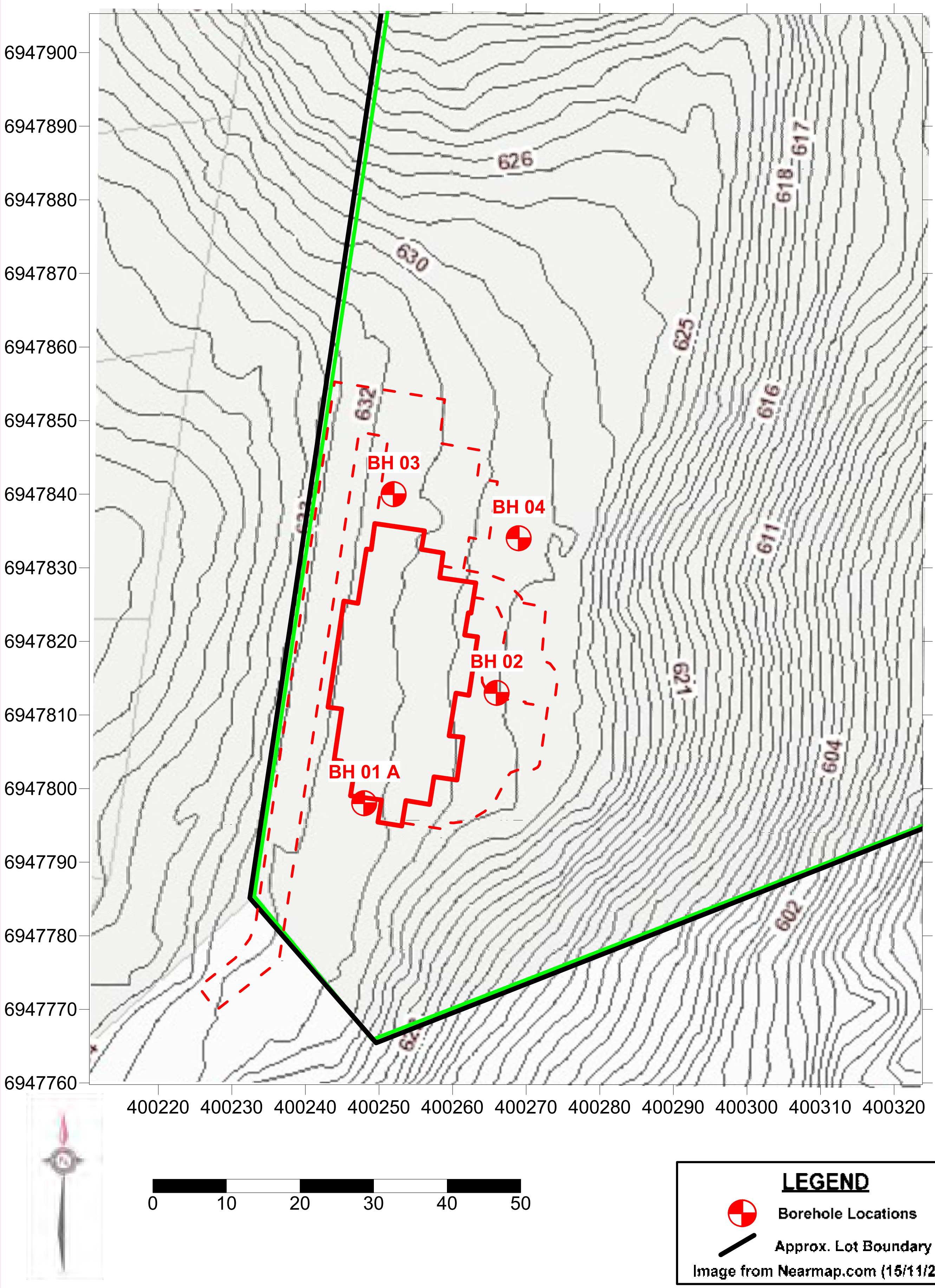
Specialists in Applied Geotechnics



LEGEND

🔴 Borehole Locations

╱ Approx. Lot Boundary

Image from Nearmap.com (15/11/2021)

SCALE 1:750 @ A4

REFERENCE V:\Milton\Milton 1-24001 - 1-25000\1-24801 - 1-25000\1-24866 Rangeview\Graphics

A4

DRW NO 1-24866-02

DATE 18/02/2022

CHECKED NTP

CLIENT Dash House Trust

LOCATION 1F Alderley Street
Rangeview

DRAWING TITLE

TRC Contours

Soil Surveys
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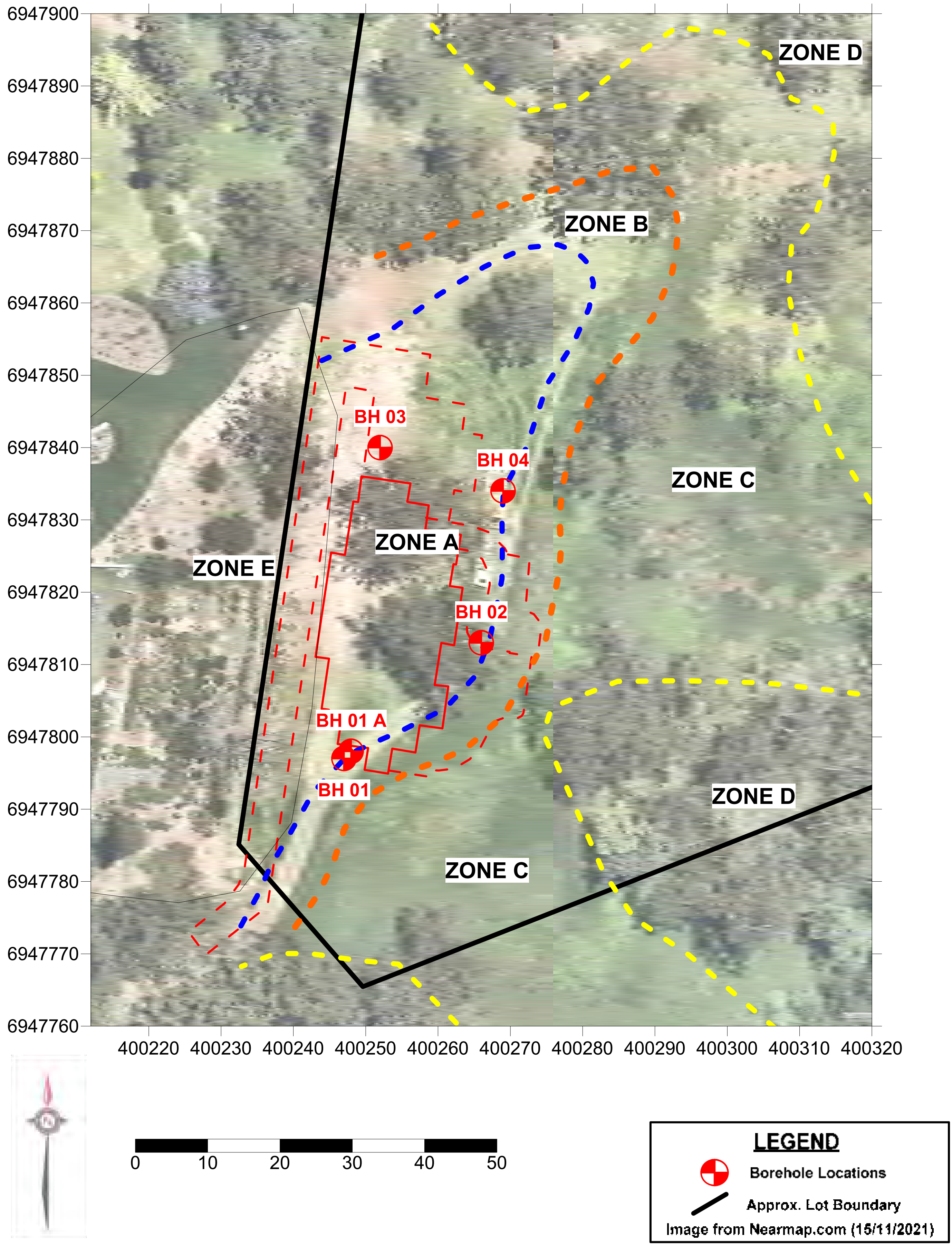

SCALE 1:750 @ A4

REFERENCE V:\Milton\Milton 1-24001 - 1-25000\1-24801 - 1-25000\1-24866 Rangeview\Graphics

A4

DRW NO 1-24866-03

DATE 18/02/2022

CHECKED NTP

CLIENT Dash House Trust

LOCATION 1F Alderley Street
Rangeview

DRAWING TITLE

Stability Zones

Soil Surveys
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Specialists in Applied Geotechnics

APPENDIX I

HILLSIDE CONSTRUCTION GUIDELINES

AUSTRALIAN GEOGUIDE LR8 (CONSTRUCTION PRACTICE)

HILLSIDE CONSTRUCTION PRACTICE

Sensible development practices are required when building on hillsides, particularly if the hillside has more than a low risk of instability (GeoGuide LR7). Only building techniques intended to maintain, or reduce, the overall level of landslide risk should be considered. Examples of good hillside construction practice are illustrated below.

WHY ARE THESE PRACTICES GOOD?

Roadways and parking areas - are paved and incorporate kerbs which prevent water discharging straight into the hillside (GeoGuide LR5).

Cuttings - are supported by retaining walls (GeoGuide LR6).

Retaining walls - are engineer designed to withstand the lateral earth pressures and surcharges expected, and include drains to prevent water pressures developing in the backfill. Where the ground slopes steeply down towards the high side of a retaining wall, the disturbing force (see GeoGuide LR6) can be two or more times that in level ground. Retaining walls must be designed taking these forces into account.

Sewage - whether treated or not is either taken away in pipes or contained in properly founded tanks so it cannot soak into the ground.

Surface water - from roofs and other hard surfaces is piped away to a suitable discharge point rather than being allowed to infiltrate into the ground. Preferably, the discharge point will be in a natural creek where ground water exits, rather than enters, the ground. Shallow, lined, drains on the surface can fulfil the same purpose (GeoGuide LR5).

Surface loads - are minimised. No fill embankments have been built. The house is a lightweight structure. Foundation loads have been taken down below the level at which a landslide is likely to occur and, preferably, to rock. This sort of construction is probably not applicable to soil slopes (GeoGuide LR3). If you are uncertain whether your site has rock near the surface, or is essentially a soil slope, you should engage a geotechnical practitioner to find out.

Flexible structures - have been used because they can tolerate a certain amount of movement with minimal signs of distress and maintain their functionality.

Vegetation clearance - on soil slopes has been kept to a reasonable minimum. Trees, and to a lesser extent smaller vegetation, take large quantities of water out of the ground every day. This lowers the ground water table, which in turn helps to maintain the stability of the slope. Large scale clearing can result in a rise in water table with a consequent increase in the likelihood of a landslide (GeoGuide LR5). An exception may have to be made to this rule on steep rock slopes where trees have little effect on the water table, but their roots pose a landslide hazard by dislodging boulders.

Possible effects of ignoring good construction practices are illustrated on page 2. Unfortunately, these poor construction practices are not as unusual as you might think and are often chosen because, on the face of it, they will save the developer, or owner, money. You should not lose sight of the fact that the cost and anguish associated with any one of the disasters illustrated, is likely to more than wipe out any apparent savings at the outset.

ADOPT GOOD PRACTICE ON HILLSIDE SITES

AUSTRALIAN GEOGUIDE LR8 (CONSTRUCTION PRACTICE)

WHY ARE THESE PRACTICES POOR?

Roadways and parking areas - are unsurfaced and lack proper table drains (gutters) causing surface water to pond and soak into the ground.

Cut and fill - has been used to balance earthworks quantities and level the site leaving unstable cut faces and added large surface loads to the ground. Failure to compact the fill properly has led to settlement, which will probably continue for several years after completion. The house and pool have been built on the fill and have settled with it and cracked. Leakage from the cracked pool and the applied surface loads from the fill have combined to cause landslides.

Retaining walls - have been avoided, to minimise cost, and hand placed rock walls used instead. Without applying engineering design principles, the walls have failed to provide the required support to the ground and have failed, creating a very dangerous situation.

A heavy, rigid, house - has been built on shallow, conventional, footings. Not only has the brickwork cracked because of the resulting ground movements, but it has also become involved in a man-made landslide.

Soak-away drainage - has been used for sewage and surface water run-off from roofs and pavements. This water soaks into the ground and raises the water table (GeoGuide LR5). Subsoil drains that run along the contours should be avoided for the same reason. If felt necessary, subsoil drains should run steeply downhill in a chevron, or herring bone, pattern. This may conflict with the requirements for effluent and surface water disposal (GeoGuide LR9) and if so, you will need to seek professional advice.

Rock debris - from landslides higher up on the slope seems likely to pass through the site. Such locations are often referred to by geotechnical practitioners as "debris flow paths". Rock is normally even denser than ordinary fill, so even quite modest boulders are likely to weigh many tonnes and do a lot of damage once they start to roll. Boulders have been known to travel hundreds of metres downhill leaving behind a trail of destruction.

Vegetation - has been completely cleared, leading to a possible rise in the water table and increased landslide risk (GeoGuide LR5).

DON'T CUT CORNERS ON HILLSIDE SITES - OBTAIN ADVICE FROM A GEOTECHNICAL PRACTITIONER

More information relevant to your particular situation may be found in other Australian GeoGuides:

- GeoGuide LR1 - Introduction
- GeoGuide LR2 - Landslides
- GeoGuide LR3 - Landslides in Soil
- GeoGuide LR4 - Landslides in Rock
- GeoGuide LR5 - Water & Drainage
- GeoGuide LR6 - Retaining Walls
- GeoGuide LR7 - Landslide Risk
- GeoGuide LR9 - Effluent & Surface Water Disposal

GeoGuide LR10 - Coastal Landslides

- GeoGuide LR11 - Record Keeping

The Australian GeoGuides (LR series) are a set of publications intended for property owners; local councils; planning authorities; developers; insurers; lawyers and, in fact, anyone who lives with, or has an interest in, a natural or engineered slope, a cutting, or an excavation. They are intended to help you understand why slopes and retaining structures can be a hazard and what can be done with appropriate professional advice and local council approval (if required) to remove, reduce, or minimise the risk they represent. The GeoGuides have been prepared by the Australian Geomechanics Society, a specialist technical society within Engineers Australia, the national peak body for all engineering disciplines in Australia, whose members are professional geotechnical engineers and engineering geologists with a particular interest in ground engineering. The GeoGuides have been funded under the Australian governments' National Disaster Mitigation Program.