24 August 2018
Ref No. 30510LA3let-Rev1

Catholic Metropolitan Cemeteries Trust
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ATTENTION: Mr David De Angelis (NettCorp Pty Ltd)

Dear Sir

GEOTECHNICAL ASSESSMENT
VEGETATION REMOVAL AND LANDSLIP RISK
166-176 ST ANDREWS ROAD, VARROVILLE, NSW

1 INTRODUCTION

JK Geotechnics have been engaged by Catholic Metropolitan Cemeteries Trust (CMCT) as the geotechnical consultants for the proposed Macarthur Memorial Park project at 166-176 St Andrews Road, Varroville, NSW. A site location plan is presented as Figure 1. The detailed geotechnical investigations for the proposed development are in progress.

We have been requested to respond to the following Council Condition:

> ***Vegetation Removal/Land Slip Risk***
>
> *Insufficient information has been submitted to properly assess the risk of land slip in areas of moderate and high stability risk, particularly in regard to the extensive revegetation works proposed, which are not referred to in the Stability Assessment. An amended Stability Assessment and an Amended Vegetation Plan are required providing detailed recommendations in regard to vegetation removal and replanting areas, practices and timeframes and drainage control.*
>
> *Furthermore, additional information is required to assess the stability of the existing dams and if necessary, detailed recommendations for suitable stability measures must be prepared.*

We have been provided with the following reports:

1. 'Report on Preliminary Stability Assessment' prepared by Douglas Partners (Project 73732.01 dated March 2017).
2. 'Vegetation Management Plan' prepared by Travers Bushfire & Ecology (Ref. 18NETT02 dated July 2018).

In the sections below, we discuss our assessment procedure, provide our comments on the supplied reports, and our recommendations to address the geotechnical aspects of the supplied Council Condition.

For the purpose of this report, we have taken St Andrews Road to bound the site to the west, with 'Site North' shown on the attached Figure 2.

2 GEOTECHNICAL STABILITY ASSESSMENT

2.1 Assessment Procedure

Review of DP Report

The supplied Douglas Partners (DP) 'Preliminary Stability Assessment' report was reviewed and our comments are presented in Section 2.2 below.

Walkover Inspection

Our stability assessment was based upon a detailed walkover inspection of the topographic, surface drainage and geological conditions of the steepest portions of the site and their immediate environs. The inspection was carried out on 13 June 2018 by our Senior Associate level geotechnical engineer, Nicholas Smith. A summary of our observations is presented in Section 2.2 below.

Figure 2 presents a geotechnical site plan showing the principal geotechnical features present at the northern, steepest portion of the site. Figure 2 is based on the supplied survey plan prepared by Degotardi Smith & Partners (Ref. 34634A01, Issue B, dated 14/02/17) and the available 1961 aerial photograph of the site. Slope angles were measured using a hand held clinometer and the dimensions of features which were accessible were tape measured, otherwise they were estimated. The landslide features shown on Figure 2 were measured using differential GPS survey equipment, for which we expect the positional accuracy to be within 50mm.

Based on the results of our walkover inspection, five test pit locations (TP81 to TP84, and TP106) were nominated by our Senior Associate level geotechnical engineer to further investigate the nature of the landslide area. TP81, TP82 & TP84 were positioned along the obvious toe of the landslide area, whereas TP83 & TP106 were positioned within the landslide area itself.

Aerial Photographs

As part of our desktop study, we obtained what we understand to be the earliest available 'Historical Aerial Photograph' (HAP) of the site; that is, the 1961 image (Ref. 1043_43_103) from Department of Finance, Service and Innovation.

Subsurface Investigation

On 21 & 22 June 2018, four test pits (TP81 to TP84, and TP106) were excavated into the northern, steepest portion of the site to assess the nature of the subsurface profile. The test pits were excavated using a 35 tonne excavator, at the locations are shown on Figure 2, to maximum depths ranging between 4.0m and 6.6m below existing grade.

The surface RL and grid coordinates of each trench were measured by Degotardi Smith & Partners to the Australian Height Datum (AHD) and Map Grid Australia (MGA), respectively, and the information was provided to us on 7 August 2018.

The strength of the clay soil profile was assessed by hand penetrometer readings on recovered lump samples and by tactile examination. Groundwater observations were also made in each test pit. On completion of excavation, each test pit was backfilled using the excavated spoil and compacted in layers by tamping with the bucket. Excess spoil was mounded above the backfill and tracked over using the excavator. Further details of the methods and procedures employed in the investigation are presented in the attached Report Explanation Notes.

Our Senior Geotechnical Engineer (David Schwarzer) was present full-time during the fieldwork to nominate testing and sampling, and to prepare the attached test pit cross-sectional sketches, which are presented as Figures 8 to 12. The Report Explanation Notes define the logging terms and symbols used.

2.2 Results of the Assessment

Douglas Partners 'Preliminary Stability Assessment' Report

The pertinent details of the supplied DP report are summarised below:

- In Section 8.2, DP state that "*A review of aerial photographs of the area indicated signs of previous landslides on the northern side of the site along the south-facing slopes of Bunbury Curran Hill. The change of colour in the vegetation cover on this section of the site can be considered as an indicative* [sic.] *of ongoing soil creep*." For this site, we do not necessarily agree with the second sentence as the change of colour in vegetation cover could be caused by a change in geological conditions and/or more likely, previous land clearing where the African Olives (an aggressive woody weed) replaced the Cumberland Plain Woodland.

- The site observations provided in Section 8.3 are limited. Notwithstanding, DP state that "*signs of previous slope movements in the form of local surface irregularities were observed along the gently inclined south-facing slopes of Bunbury Curran Hill.*" The "*approximate extent of colluvium*" (ie. the landslide area) was plotted onto DP Figure 1.

- In Section 8.5, DP presented their 'Stability Risk Assessment' where they divided the site into "*three general risk of instability zones (low, moderate and high risk of instability)*". The zones were plotted onto DP Figure 2. Generally, the DP stability risk assessment adopted a simplified approach, based primarily on surface gradients.

Site Observations

The site is located on a gently to moderately sloping hillside, which generally grades down to the south. A maximum elevation relief of about 109m exists between the north-eastern corner of the site and the lower south-eastern corner. St Andrews Road bounds the site to the west.

The subject 114 hectare site extends around a 3 hectare battle-axe property at 196 St Andrews Road ('Varroville House'). The higher lying northern end of the site (Bunbury Curran Hill) is moderately to steeply sloping and covered in dense bushland. Elsewhere the site is essentially grass covered with scattered trees and 'pockets' of bushland. From the north-eastern corner of the site, a ridgeline extends mid-length along the eastern boundary and returns in a south-westerly direction towards Varroville House. The hillsides flanking the ridgeline are also moderately to steeply sloping. Elsewhere within the site, the hillsides which form gullies are generally gently to moderately sloping.

At the time of our inspection, the site was used to graze cattle. Ten farm dams were located within gullies across the site.

For this assessment, a detailed inspection was made of the northern, steepest portion of the site, which is the south facing slope of the Bunbury Curran Hill ridgeline, as well as the flanks of the secondary north-east to south-west trending ridgeline which extends towards Varroville House.

The south facing slope of the Bunbury Curran Hill ridgeline was densely vegetated. The western half predominantly comprised Cumberland Plain Woodland and the eastern half predominantly comprised African Olives. Access through the northern portion of the site was limited. Notwithstanding, detailed observations could be made along the ridgeline (northern site boundary) and below the tree line.

A summary of our observations is provided below:

<u>General observations along the Ridgeline:</u>

- The ridgeline along the northern site boundary generally graded down to the west at less than 5°, but was near level in some areas.
- The northern flank of the ridgeline generally graded down to the north between 2° and 8°, but predominantly between 2° and 5°.
- The southern flank of the ridgeline initially graded down to the south between 1° and 3°. A short distance down from the ridgeline was a notable break in slope, which gradually steepened to between 25° and 30°.
- Many of the breaks in slope were backscarps of former landslides. The backscarps were typically located 5m to 10m south of the tree line. In our opinion, there were several 'smaller' landslides across the northern steepest portion of the site, with some connected by regression.

<u>General Observations within the Landslide Area:</u>

- Some backscarps appeared to have eroded to a flatter grade, however some were near vertical and up to about 5m high.
- Below the backscarps, the ground surface was hummocky. Sandstone boulders and large 'plates' of sandstone were observed within the hummocky area.
- No apparent leaning of trees nor curvatures of trunks (ie. indicators of creep movement) were observed.
- Within the upper portion of the landslide area, surface gradients generally ranged between 25° and 30°. Within the lower portion, surface grades typically flattened to about 10°.
- Deeply incised erosion channels, exacerbated by a walking track and cattle movements, were observed towards the eastern end of the landslide area.

- Based on surface indicators, the landslide area appeared to extend into the neighbouring property to the east. Notwithstanding, the eastern boundary fence line appeared straight (ie. no alignment curvature which would indicate creep movement).

<u>Observations along Toe of Landslide Area:</u>

- The toes of the former landslides were generally well defined, with toe bulges up to about 2m high.
- The former landslides generally extended further downslope in gully areas rather than along spurs.
- The toe was typically at, or close to, the tree line.
- As can be seen on Figure 2, our interpreted extent of the landslide area is larger than shown in the DP report.

<u>Secondary Ridgeline Extending into the Site:</u>

- North-east to south-west trending ridgeline, with the ridge surface grading down generally to the south-west at between about 3° and 5°.
- The flanks and end of the ridge generally graded between 10° and 12°.
- Some earthworks had been completed in areas along the ridge including:
    - Access track cuttings, up to 1m deep, exposing residual clay soils.
    - Localised cuttings along the north-western flank of the ridge, towards the crest, to a maximum depth of about 0.5m.
    - A maximum 3m deep borrow pit at the south-western end of the ridge, exposing residual clay soils and weathered siltstone bedrock.
- Surface erosion had occurred within gullies descending from the ridgeline and associated with the above mentioned earthworks.
- Shallow contour drains had been formed along the flanks of the ridgeline, and the area to the south of the ridge had been terraced.
- There was no evidence of natural hillside instability along the flanks of the secondary ridgeline. All disturbance, apart from erosion, appeared to be man-made.

Review of 1961 Aerial Photograph

The available 1961 HAP was overlain onto the survey drawing, along with our survey reference points. The HAP indicates a landslide feature on the subject site.

The backscarps and toe of the landslide area as can be seen on the 1961 HAP appear very close to, or at, the same locations mapped by JK Geotechnics. Furthermore, the location of the toe bulge appears unchanged between the 1961 HAP and the current Google Earth Pro image of the site.

Subsurface Conditions

The 1:100,000 series geological maps of Wollongong-Port Hacking (Geological Survey of NSW, Geological Series Sheet 9029-9129) and Penrith (Geological Series Sheet 9030) indicate the site to be underlain by Bringelly Shale of the Wianamatta Group.

All five test pits were excavated within the landslide area. Generally, all test pits encountered topsoil overlying colluvial silty clay. In TP106, the silty clay was underlain by colluvial gravel. Slip surfaces were observed within the colluvial silty clay in TP81, TP82 & TP83 at various depths.

Siltstone bedrock was encountered in TP81, TP82, TP84 & TP106 below the colluvium. From what could safely be observed from outside the sides of the test pits, the siltstone appeared to be sub-horizontally bedded (ie. potentially not disturbed landslide material). Siltstone bedrock was not encountered in TP83 within the maximum 6.6m excavation depth.

The test pits were 'dry' during and on completion of excavation. We note that the groundwater levels may not have stabilised within the limited observation period.

Reference should be made to the attached Figures 8 to 12 for specific details at each test pit location.

3 <u>VEGETATION MANANAGEMENT PLAN</u>

The purpose of the supplied 'Vegetation Management Plan' (VMP) prepared by Travers Bushfire & Ecology (Travers), was partly to "*protect and regenerate the Cumberland Plain Woodland*" and to "*maximise native vegetation cover and species diversity within the site*". For the VMP, the site was divided into six zones; that is, Management Zone A to Management Zone F.

Management Zone A (Escarpment Reserve) covers the northern, steepest portion of the site, and was assigned 'Moderate to High Priority' by Travers. For ease of cross-reference, we have reproduced Section 3.1.1 of the Travers report below:

> *"This management zone contains is a [sic.] remnant Cumberland Plain Woodland* [CPW] *on the lower, mid and upper slopes. The extent of weed infestation within this zone means that it will have a very long restoration program of 20 plus years to remove weeds, revegetate and regenerate in a mosaic manner.*

To ensure the slopes remain stable at all times the weed control and revegetation works are to be undertaken in strategic locations to progressively consolidate the better condition areas and expand into the existing densely weed infested areas.

A staged and mosaic approach should be undertaken to reduce the area of bare soil susceptible to erosion at any one time:

- *Woody weeds such as African Olive should be controlled using cut-and-paint or drill-and-fill herbicide techniques to minimise soils disturbance, and the stumps and roots left in situ to provide soil stabilisation.*
- *An initial seeding of sterile or non-invasive cover grasses (e.g. Japanese Millet) is to be used to provide a quickly-established ground cover to reduce erosion and supress weeds.*
- *A covering of biodegradable erosion protection is to be laid following seeding of the cover grass. This may be either hydromulch or pegged and overlapped jute matting.*
- *Planting of CPW species is to follow establishment of the cover grass.*

Whilst there is remnant native vegetation present the area is to be progressively restored to a natural state over an estimated 20 plus years within with [sic.] ongoing maintenance for the life of the project."

Management Zone E (Eastern Woodland Reserve – Cumberland Plain Woodland) covers a portion of the secondary ridgeline which extends into the site, and was assigned 'Low Priority' by Travers. Regeneration of CPW is proposed in this area, with no weed removal.

We concur with the staged methodology proposed by Travers for Management Zone A (Escarpment Reserve) of replacing the woody weeds with CPW. Notwithstanding, our preference is that each area of revegetation be kept as small as practical so as to limit any potential impacts on the landslide area. As such, the sizing of each area should be a collaborative process between CMCT, Travers and JK Geotechnics.

The proposed regeneration of the CPW within Management Zone E (Eastern Woodland Reserve) should have no effect on the stability of the ridgeline flanks.

4 COMMENTS AND RECOMMENDATIONS

Based on the results of our assessment and review of the VMP, it is our opinion that the increased risk of instability arising from the staged weed control and revegetation works nominated by Travers within the northern, steepest portion of the site will be negligible. This is on condition that size of each revegetation area is kept as small as practical.

With respect to the stability of the existing dams, JK Geotechnics have already completed a geotechnical assessment with the findings and advice presented in report Ref. '30510ZArpt' dated 10 January 2018. In summary, we considered that all ten 'farm' dams were in poor condition for the following primary reasons:

1. Presence of trees on the embankments (all dams);
2. Presence of burrows and ant hills (all dams);
3. Presence of over-steep upstream and downstream shoulders (all dams);
4. Evidence of seepage through the foundation material (at 8 of the 10 dams);
5. Evidence of scour and erosion along the toe of the downstream shoulder (at 2 of the 10 dams);
6. Evidence of dispersive soils (all dams);
7. Lack of formal spillways (all dams).

It was our recommendation that the dams which were to be retained as part of the proposed development be replaced with properly engineered water retention structures.

5 GENERAL COMMENTS

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

Should you require any further information regarding the above, please do not hesitate to contact the undersigned.

Yours faithfully
For and on behalf of
JK GEOTECHNICS

Andrew Jackaman
Principal | Geotechnical Engineer

Figure 1: Site Location Plan
Figure 2: Geotechnical Site Plan
Figure 8: Test Pit 81 Cross Sectional Sketch
Figure 9: Test Pit 82 Cross Sectional Sketch
Figure 10: Test Pit 83 Cross Sectional Sketch
Figure 11: Test Pit 84 Cross Sectional Sketch
Figure 12: Test Pit 106 Cross Sectional Sketch
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This Report (which includes all attachments and annexures) has been prepared by JK Geotechnics (JKG) for its Client, and is intended for the use only by that Client.

This Report has been prepared pursuant to a contract between JKG and its Client and is therefore subject to:

a) JKG's proposal in respect of the work covered by the Report;

b) the limitations defined in the Client's brief to JKG;

c) the terms of contract between JKG and the Client, including terms limiting the liability of JKG.

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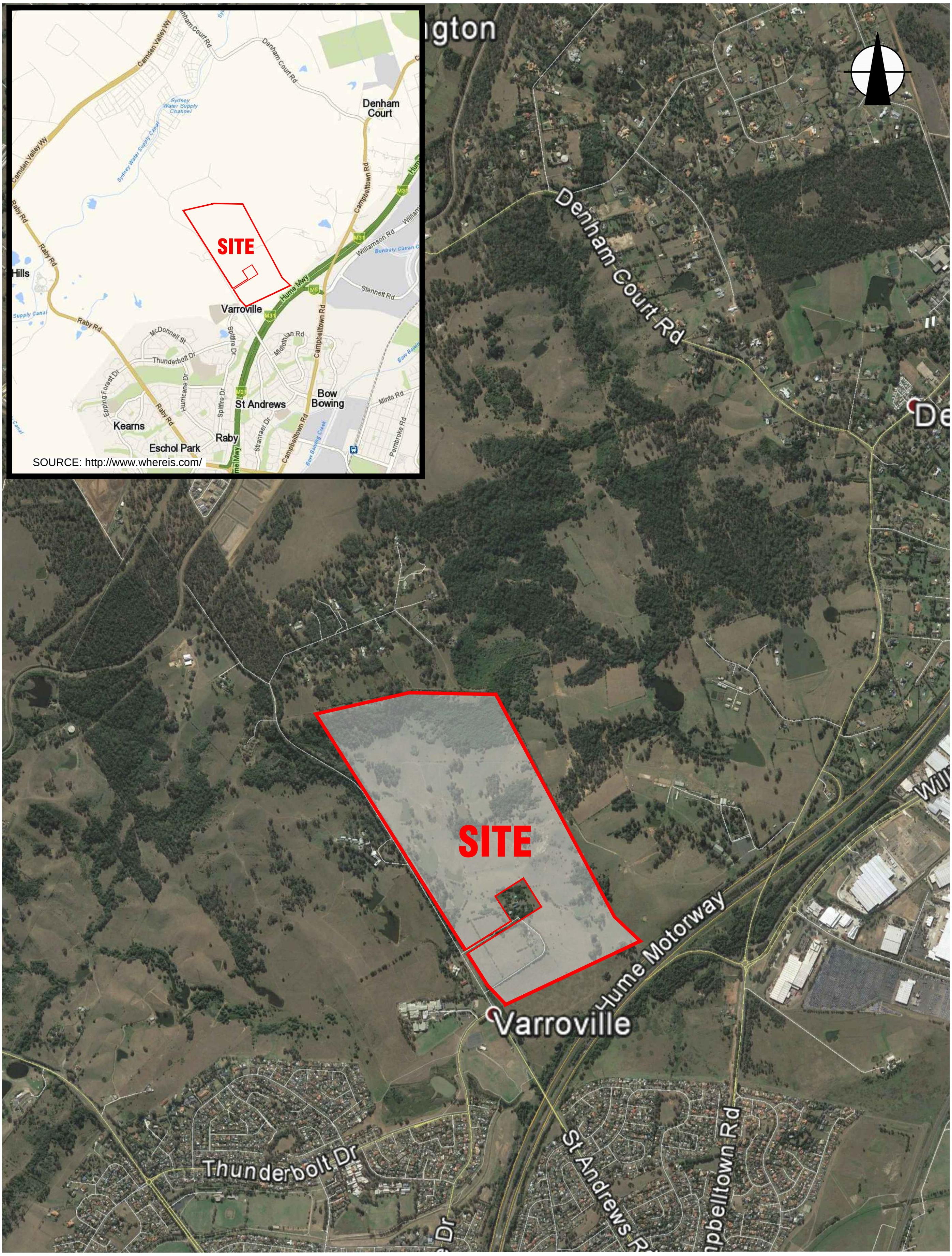

AERIAL IMAGE SOURCE: GOOGLE EARTH PRO 7.1.5.1557
AERIAL IMAGE ©: 2015 GOOGLE INC.

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

Title:	**SITE LOCATION PLAN**
Location:	MACARTHUR MEMORIAL PARK ST ANDREWS ROAD, VARROVILLE, NSW
Report No: 30510ZA3	Figure No: 1

JK Geotechnics

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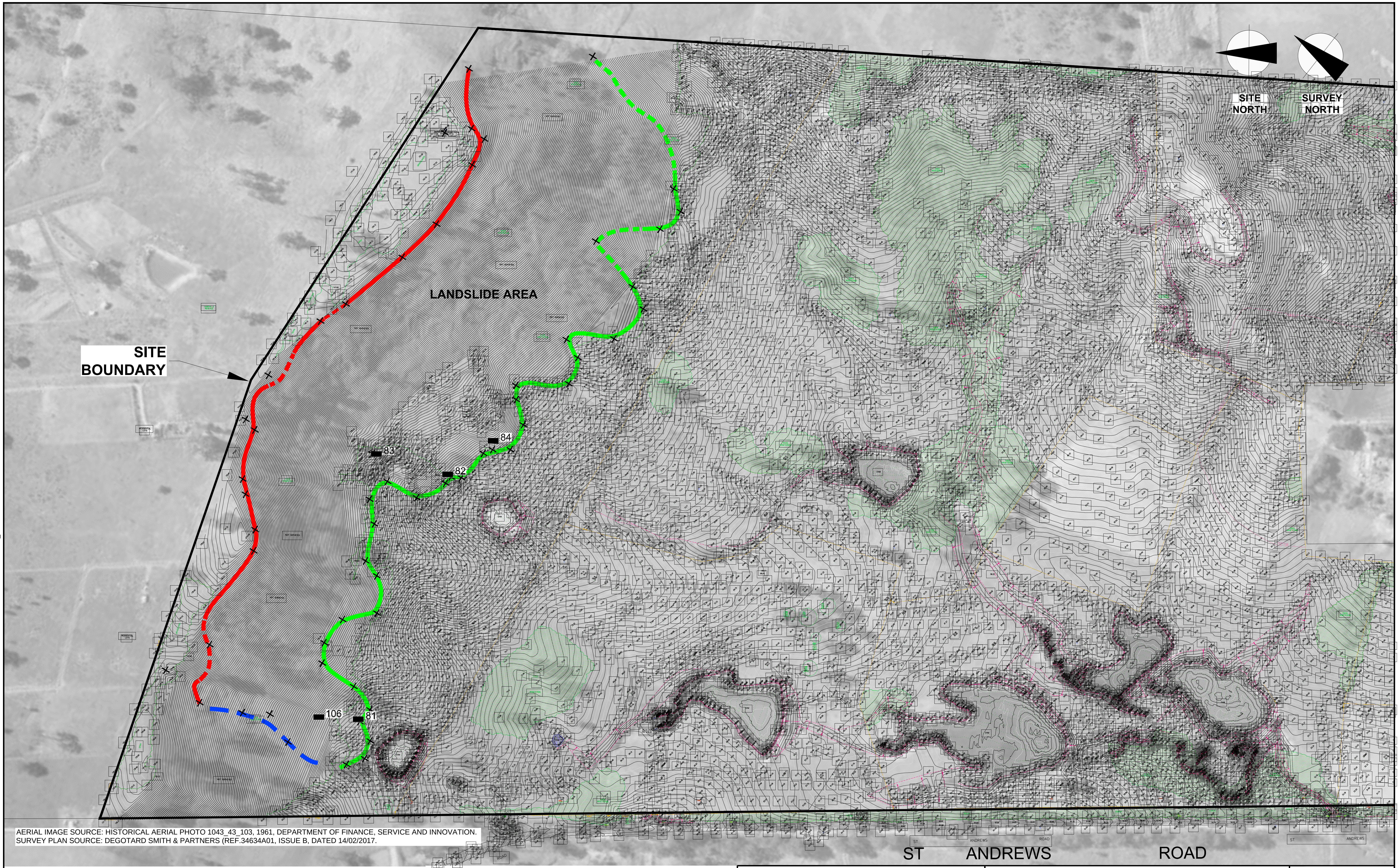


LEGEND

- TEST PIT
- LANDSLIDE FEATURE REFERENCE LOCATIONS USING DIFFERENTIAL GPS SURVEY EQUIPMENT
- OBSERVED BACK SCARP(S)
- BREAK IN SLOPE
- OBVIOUS TOE BULGE OF LANDSLIDE AREA
- POORLY DEFINED TOE BULGE
- INFERRED WESTERN SIDE OF LANDSLIDE AREA

0 35 70 105 140 175

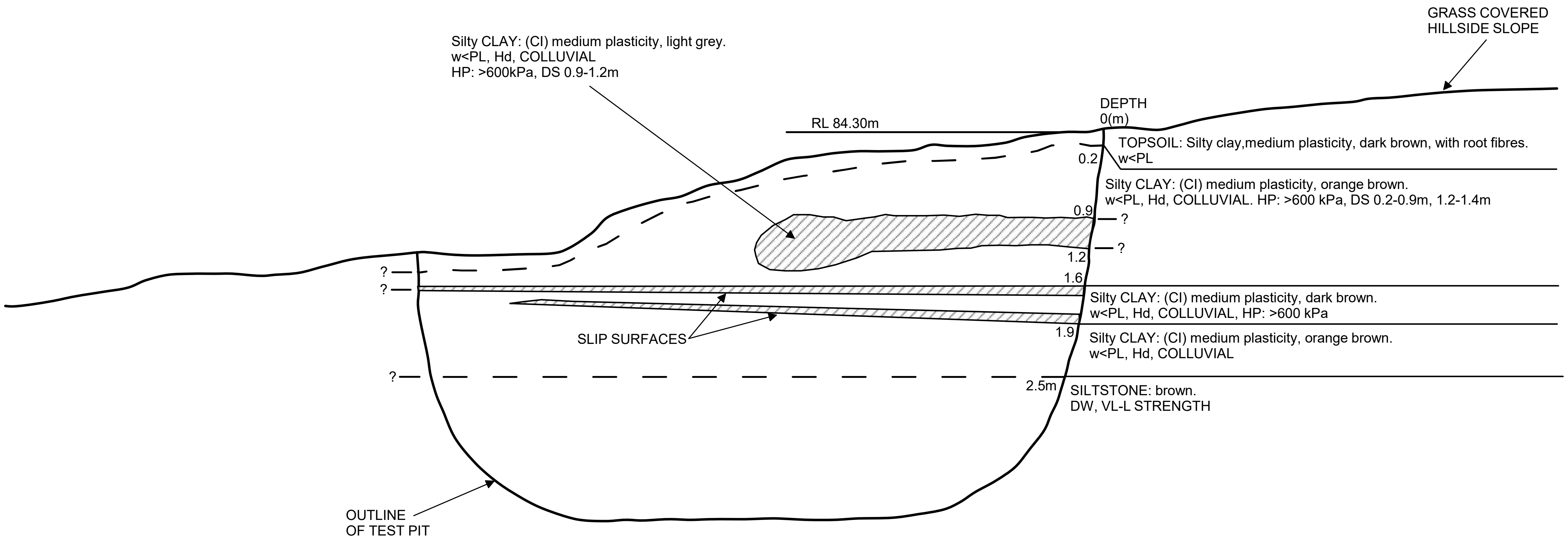
SCALE 1:3500 @A3 METRES

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

Title:	**GEOTECHNICAL SITE PLAN**
Location:	166-176 ST ANDREWS ROAD VARROVILLE, NSW
Report No: 30510ZA3	Figure No: 2

JK **Geotechnics**

PLOT DATE: 24/08/2018 11:38:26 AM DWG FILE: S:\6 GEOTECHNICAL\6F GEOTECHNICAL JOBS\30000'S\30510ZA VARROVILLE\CAD\30510ZA3_F2.DWG



DRY ON COMPLETION
MAXIMUM DEPTH OF TEST PIT WAS 4.0m

TEST PIT 81
CROSS SECTIONAL SKETCH
LOOKING WEST

0 5
Scale 1:50 @A3 Metres

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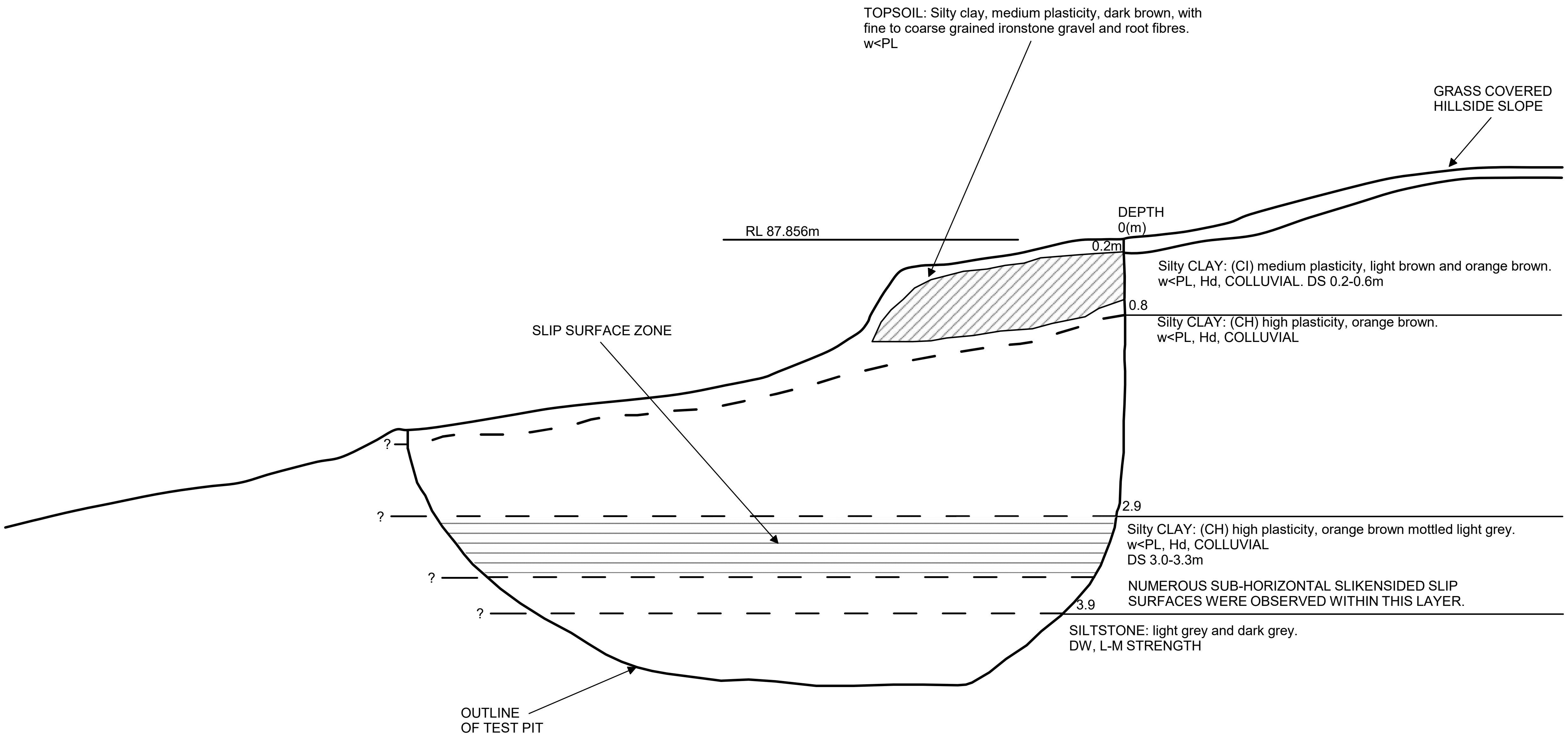
GEOTECHNICAL & ENVIRONMENTAL ENGINEERS
Report No. 30510ZA3 Figure No. 8



DRY ON COMPLETION
MAXIMUM DEPTH OF TEST PIT WAS 4.6m

TEST PIT 82
CROSS SECTIONAL SKETCH
LOOKING WEST

0 5
Scale 1:50 @A3 Metres

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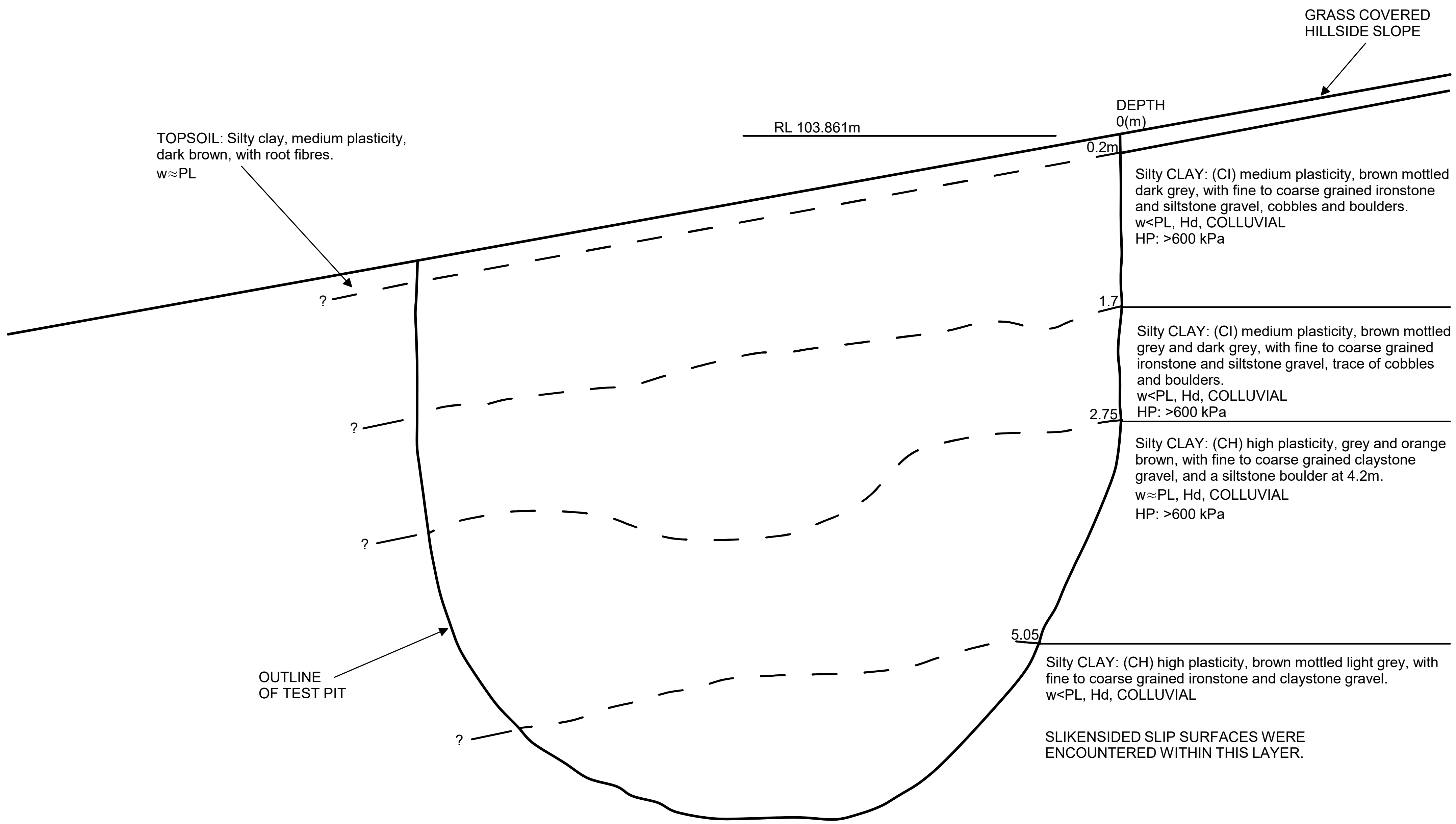
Report No. 30510ZA3 Figure No. 9

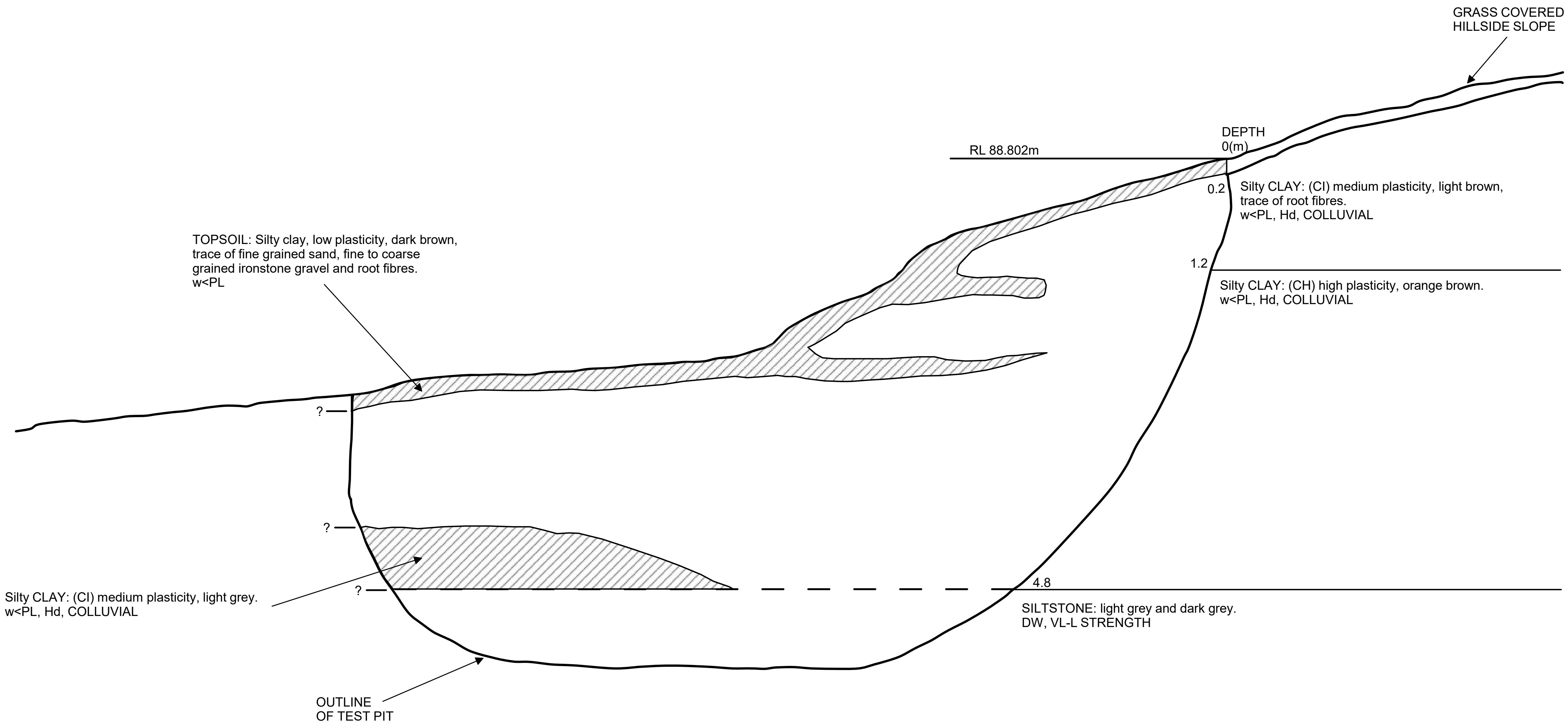


DRY ON COMPLETION
MAXIMUM DEPTH OF TEST PIT WAS 6.6m

TEST PIT 83
CROSS SECTIONAL SKETCH
LOOKING WEST

0 5
Scale 1:50 @A3 Metres

JK Geotechnics
GEOTECHNICAL & ENVIRONMENTAL ENGINEERS
Report No. 30510ZA3 Figure No. 10



DRY ON COMPLETION
MAXIMUM DEPTH OF TEST PIT WAS 5.6m

TEST PIT 84
CROSS SECTIONAL SKETCH
LOOKING WEST

0 5
Scale 1:50 @A3 Metres

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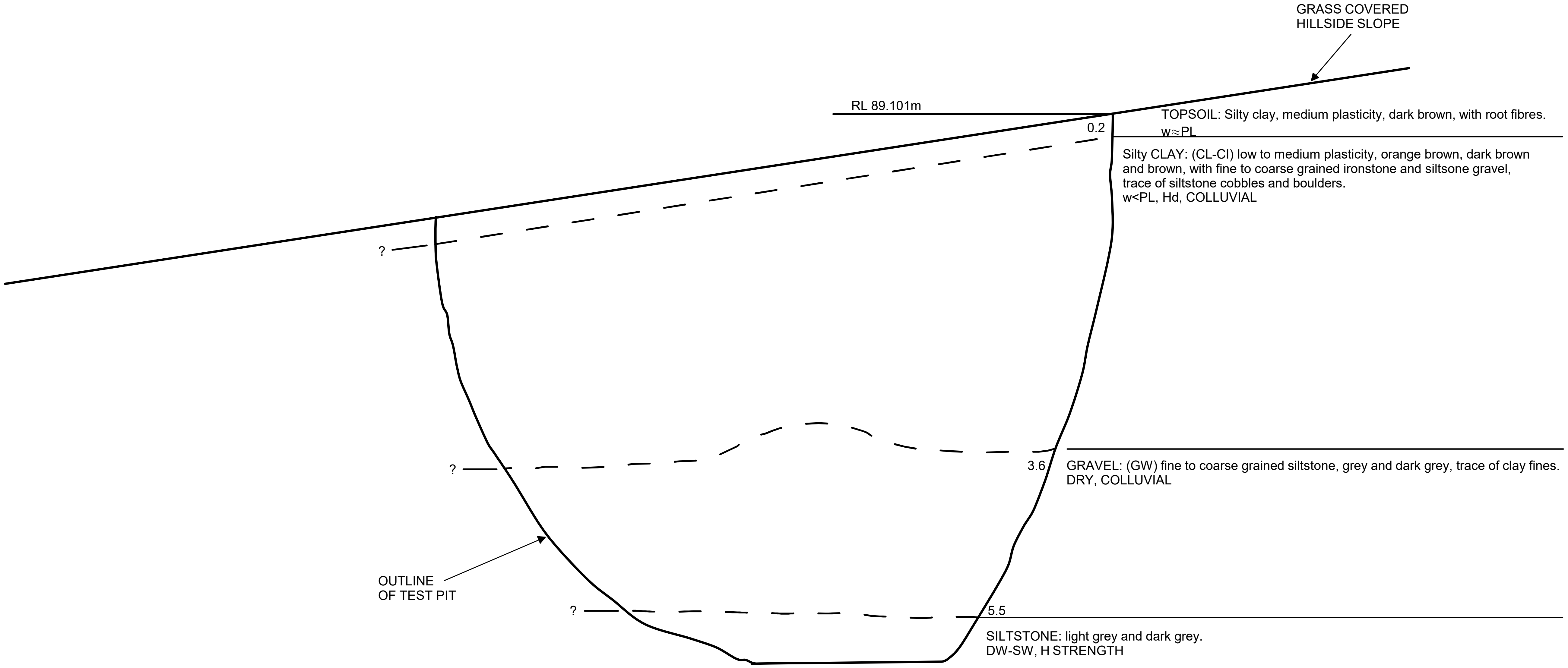
Report No. 30510ZA3 Figure No. 11



DRY ON COMPLETION
MAXIMUM DEPTH OF TEST PIT WAS 6.0m

TEST PIT 106
CROSS SECTIONAL SKETCH
LOOKING WEST

0 5
Scale 1:50 @A3 Metres

JK Geotechnics
GEOTECHNICAL & ENVIRONMENTAL ENGINEERS
Report No. 30510ZA3 Figure No. 12

REPORT EXPLANATION NOTES

INTRODUCTION

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

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

DESCRIPTION AND CLASSIFICATION METHODS

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

Soil types are described according to the predominating particle size and behaviour as set out in the attached soil classification table qualified by the grading of other particles present (eg. sandy clay) as set out below:

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

Non-cohesive soils are classified on the basis of relative density, generally from the results of Standard Penetration Test (SPT) as below:

Relative Density	SPT 'N' Value (blows/300mm)
Very loose (VL)	< 4
Loose (L)	4 to 10
Medium dense (MD)	10 to 30
Dense (D)	30 to 50
Very Dense (VD)	> 50

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

Classification	Unconfined Compressive Strength (kPa)	Indicative Undrained Shear Strength (kPa)
Very Soft (VS)	≤ 25	≤ 12
Soft (S)	> 25 and ≤ 50	> 12 and ≤ 25
Firm (F)	> 50 and ≤ 100	> 25 and ≤ 50
Stiff (St)	> 100 and ≤ 200	> 50 and ≤ 100
Very Stiff (VSt)	> 200 and ≤ 400	> 100 and ≤ 200
Hard (Hd)	> 400	> 200
Friable (Fr)	Strength not attainable – soil crumbles	

Rock types are classified by their geological names, together with descriptive terms regarding weathering, strength, defects, etc. Where relevant, further information regarding rock classification is given in the text of the report. In the Sydney Basin, 'shale' is used to describe fissile mudstone, with a weakness parallel to bedding. Rocks with alternating inter-laminations of different grain size (eg. siltstone/claystone and siltstone/fine grained sandstone) is referred to as 'laminite'.

SAMPLING

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

Disturbed samples taken during drilling provide information on plasticity, grain size, colour, moisture content, minor constituents and, depending upon the degree of disturbance, some information on strength and structure. Bulk samples are similar but of greater volume required for some test procedures.

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

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

INVESTIGATION METHODS

The following is a brief summary of investigation methods currently adopted by the Company and some comments on their use and application. All methods except test pits, hand auger drilling and portable Dynamic Cone Penetrometers require the use of a mechanical rig which is commonly mounted on a truck chassis or track base.

Test Pits: These are normally excavated with a backhoe or a tracked excavator, allowing close examination of the insitu soils and 'weaker' bedrock if it is safe to descend into the pit. The depth of penetration is limited to about 3m for a backhoe and up to 6m for a large excavator. Limitations of test pits are the problems associated with disturbance and difficulty of reinstatement and the consequent effects on close-by structures. Care must be taken if construction is to be carried out near test pit locations to either properly recompact the backfill during construction or to design and construct the structure so as not to be adversely affected by poorly compacted backfill at the test pit location.

Hand Auger Drilling: A borehole of 50mm to 100mm diameter is advanced by manually operated equipment. Refusal of the hand auger can occur on a variety of materials such as obstructions within any fill, tree roots, hard clay, gravel or ironstone, cobbles and boulders, and does not necessarily indicate rock level.

Continuous Spiral Flight Augers: The borehole is advanced using 75mm to 115mm diameter continuous spiral flight augers, which are withdrawn at intervals to allow sampling and insitu testing. This is a relatively economical means of drilling in clays and in sands above the water table. Samples are returned to the surface by the flights or may be collected after withdrawal of the auger flights, but they can be very disturbed and layers may become mixed. Information from the auger sampling (as distinct from specific sampling by SPTs or undisturbed samples) is of limited reliability due to mixing or softening of samples by groundwater, or uncertainties as to the original depth of the samples. Augering below the groundwater table is of even lesser reliability than augering above the water table.

Rock Augering: Use can be made of a Tungsten Carbide (TC) bit for auger drilling into rock to indicate rock quality and continuity by variation in drilling resistance and from examination of recovered rock cuttings. This method of investigation is quick and relatively inexpensive but provides only an indication of the likely rock strength and predicted values may be in error by a strength order. Where rock strengths may have a significant impact on construction feasibility or costs, then further investigation by means of cored boreholes may be warranted.

Wash Boring: The borehole is usually advanced by a rotary bit, with water being pumped down the drill rods and returned up the annulus, carrying the drill cuttings. Only major changes in stratification can be assessed from the cuttings, together with some information from "feel" and rate of penetration.

Mud Stabilised Drilling: Either Wash Boring or Continuous Core Drilling can use drilling mud as a circulating fluid to stabilise the borehole. The term 'mud' encompasses a range of products ranging from bentonite to polymers. The mud tends to mask the cuttings and reliable identification is only possible from intermittent intact sampling (eg. from SPT and U50 samples) or from rock coring, etc.

Continuous Core Drilling: A continuous core sample is obtained using a diamond tipped core barrel. Provided full core recovery is achieved (which is not always possible in very low strength rocks and granular soils), this technique provides a very reliable (but relatively expensive) method of investigation. In rocks, NMLC or HQ triple tube core barrels, which give a core of about 50mm and 61mm diameter, respectively, is usually used with water flush. The length of core recovered is compared to the length drilled and any length not recovered is shown as NO CORE. The location of NO CORE recovery is determined on site by the supervising engineer; where the location is uncertain, the loss is placed at the bottom of the drill run.

Standard Penetration Tests: Standard Penetration Tests (SPT) are used mainly in non-cohesive soils, but can also be used in cohesive soils, as a means of indicating density or strength and also of obtaining a relatively undisturbed sample. The test procedure is described in Australian Standard 1289.6.3.1–2004 (R2016) *'Methods of Testing Soils for Engineering Purposes, Soil Strength and Consolidation Tests – Determination of the Penetration Resistance of a Soil – Standard Penetration Test (SPT)'*.

The test is carried out in a borehole by driving a 50mm diameter split sample tube with a tapered shoe, under the impact of a 63.5kg hammer with a free fall of 760mm. It is normal for the tube to be driven in three successive 150mm increments and the 'N' value is taken as the number of blows for the last 300mm. In dense sands, very hard clays or weak rock, the full 450mm penetration may not be practicable and the test is discontinued.

The test results are reported in the following form:

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

  N = 13
  4, 6, 7

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

  N > 30
  15, 30/40mm

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

A modification to the SPT is where the same driving system is used with a solid 60° tipped steel cone of the same diameter as the SPT hollow sampler. The solid cone can be continuously driven for some distance in soft clays or loose sands, or may be used where damage would otherwise occur to the SPT. The results of this Solid Cone Penetration Test (SCPT) are shown as 'N_c' on the borehole logs, together with the number of blows per 150mm penetration.

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

In the tests, a 35mm or 44mm diameter rod with a conical tip is pushed continuously into the soil, the reaction being provided by a specially designed truck or rig which is fitted with a hydraulic ram system. Measurements are made of the end bearing resistance on the cone and the frictional resistance on a separate 134mm or 165mm long sleeve, immediately behind the cone. Transducers in the tip of the assembly are electrically connected by wires passing through the centre of the push rods to an amplifier and recorder unit mounted on the control truck. The CPT does not provide soil sample recovery.

As penetration occurs (at a rate of approximately 20mm per second), the information is output as incremental digital records every 10mm. The results given in this report have been plotted from the digital data.

The information provided on the charts comprise:

- Cone resistance – the actual end bearing force divided by the cross sectional area of the cone – expressed in MPa. There are two scales presented for the cone resistance. The lower scale has a range of 0 to 5MPa and the main scale has a range of 0 to 50MPa. For cone resistance values less than 5MPa, the plot will appear on both scales.
- Sleeve friction – the frictional force on the sleeve divided by the surface area – expressed in kPa.
- Friction ratio – the ratio of sleeve friction to cone resistance, expressed as a percentage.

  The ratios of the sleeve resistance to cone resistance will vary with the type of soil encountered, with higher relative friction in clays than in sands. Friction ratios of 1% to 2% are commonly encountered in sands and occasionally very soft clays, rising to 4% to 10% in stiff clays and peats. Soil descriptions based on cone resistance and friction ratios are only inferred and must not be considered as exact.

Correlations between CPT and SPT values can be developed for both sands and clays but may be site specific.

Interpretation of CPT values can be made to empirically derive modulus or compressibility values to allow calculation of foundation settlements.

Stratification can be inferred from the cone and friction traces and from experience and information from nearby boreholes etc. Where shown, this information is presented for general guidance, but must be regarded as interpretive. The test method provides a continuous profile of engineering properties but, where precise information on soil classification is required, direct drilling and sampling may be preferable.

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

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

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

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

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

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

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

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

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

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

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

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

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

LOGS

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

The terms and symbols used in preparation of the logs are defined in the following pages.

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

GROUNDWATER

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

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

More reliable measurements can be made by installing standpipes which are read after the groundwater level has stabilised at intervals ranging from several days to perhaps weeks for low permeability soils. Piezometers, sealed in a particular stratum, may be advisable in low permeability soils or where there may be interference from perched water tables or surface water.

FILL

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

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

LABORATORY TESTING

Laboratory testing is normally carried out in accordance with Australian Standard 1289 *'Methods of Testing Soils for Engineering Purposes'* or appropriate NSW Government Roads & Maritime Services (RMS) test methods. Details of the test procedure used are given on the individual report forms.

ENGINEERING REPORTS

Engineering reports are prepared by qualified personnel and are based on the information obtained and on current engineering standards of interpretation and analysis. Where the report has been prepared for a specific design proposal (eg. a three storey building) the information and interpretation may not be relevant if the design proposal is changed (eg. to a twenty storey building). If this happens, the Company will be pleased to review the report and the sufficiency of the investigation work.

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

- Unexpected variations in ground conditions – the potential for this will be partially dependent on borehole spacing and sampling frequency as well as investigation technique.
- Changes in policy or interpretation of policy by statutory authorities.
- The actions of persons or contractors responding to commercial pressures.
- Details of the development that the Company could not reasonably be expected to anticipate.

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

SITE ANOMALIES

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

REPRODUCTION OF INFORMATION FOR CONTRACTUAL PURPOSES

Where information obtained from this investigation is provided for tendering purposes, it is recommended that all information, including the written report and discussion, be made available. In circumstances where the discussion or comments section is not relevant to the contractual situation, it may be appropriate to prepare a specially edited document. The Company would be pleased to assist in this regard and/or to make additional report copies available for contract purposes at a nominal charge.

Copyright in all documents (such as drawings, borehole or test pit logs, reports and specifications) provided by the Company shall remain the property of Jeffery and Katauskas Pty Ltd. Subject to the payment of all fees due, the Client alone shall have a licence to use the documents provided for the sole purpose of completing the project to which they relate. Licence to use the documents may be revoked without notice if the Client is in breach of any obligation to make a payment to us.

REVIEW OF DESIGN

Where major civil or structural developments are proposed <u>or</u> where only a limited investigation has been completed <u>or</u> where the geotechnical conditions/constraints are quite complex, it is prudent to have a joint design review which involves an experienced geotechnical engineer/engineering geologist.

SITE INSPECTION

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

Requirements could range from:

i) a site visit to confirm that conditions exposed are no worse than those interpreted, to

ii) a visit to assist the contractor or other site personnel in identifying various soil/rock types and appropriate footing or pile founding depths, or

iii) full time engineering presence on site.

SYMBOL LEGENDS

SOIL

FILL

TOPSOIL

CLAY (CL, CI, CH)

SILT (ML, MH)

SAND (SP, SW)

GRAVEL (GP, GW)

SANDY CLAY (CL, CI, CH)

SILTY CLAY (CL, CI, CH)

CLAYEY SAND (SC)

SILTY SAND (SM)

GRAVELLY CLAY (CL, CI, CH)

CLAYEY GRAVEL (GC)

SANDY SILT (ML, MH)

PEAT AND HIGHLY ORGANIC SOILS (Pt)

ROCK

CONGLOMERATE

SANDSTONE

SHALE/MUDSTONE

SILTSTONE

CLAYSTONE

COAL

LAMINITE

LIMESTONE

PHYLLITE, SCHIST

TUFF

GRANITE, GABBRO

DOLERITE, DIORITE

BASALT, ANDESITE

QUARTZITE

OTHER MATERIALS

BRICKS OR PAVERS

CONCRETE

ASPHALTIC CONCRETE

CLASSIFICATION OF COARSE AND FINE GRAINED SOILS

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

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

Laboratory Classification Criteria

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

$$C_u = \frac{D_{60}}{D_{10}} \quad \text{and} \quad C_c = \frac{(D_{30})^2}{D_{10}\ D_{60}}$$

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

NOTES:

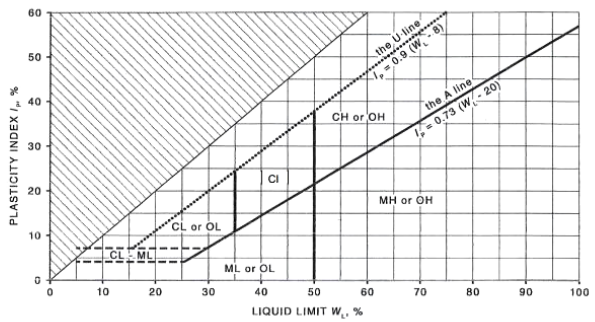
1 For a coarse grained soil with a fines content between 5% and 12%, the soil is given a dual classification comprising the two group symbols separated by a dash; for example, for a poorly graded gravel with between 5% and 12% silt fines, the classification is GP-GM.

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

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

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

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



LOG SYMBOLS

<table>
<tr><th>Log Column</th><th>Symbol</th><th>Definition</th></tr>
<tr><td rowspan="3">Groundwater Record</td><td>▼</td><td>Standing water level. Time delay following completion of drilling/excavation may be shown.</td></tr>
<tr><td>C</td><td>Extent of borehole/test pit collapse shortly after drilling/excavation.</td></tr>
<tr><td>▶</td><td>Groundwater seepage into borehole or test pit noted during drilling or excavation.</td></tr>
<tr><td>Samples</td><td>ES
U50
DB
DS
ASB
ASS
SAL</td><td>Sample taken over depth indicated, for environmental analysis.
Undisturbed 50mm diameter tube sample taken over depth indicated.
Bulk disturbed sample taken over depth indicated.
Small disturbed bag sample taken over depth indicated.
Soil sample taken over depth indicated, for asbestos analysis.
Soil sample taken over depth indicated, for acid sulfate soil analysis.
Soil sample taken over depth indicated, for salinity analysis.</td></tr>
<tr><td rowspan="3">Field Tests</td><td>N = 17
4, 7, 10</td><td>Standard Penetration Test (SPT) performed between depths indicated by lines. Individual figures show blows per 150mm penetration. ‘Refusal’ refers to apparent hammer refusal within the corresponding 150mm depth increment.</td></tr>
<tr><td>N_c = 5
7
3R</td><td>Solid Cone Penetration Test (SCPT) performed between depths indicated by lines. Individual figures show blows per 150mm penetration for 60° solid cone driven by SPT hammer. ‘R’ refers to apparent hammer refusal within the corresponding 150mm depth increment.</td></tr>
<tr><td>VNS = 25
PID = 100</td><td>Vane shear reading in kPa of undrained shear strength.
Photoionisation detector reading in ppm (soil sample headspace test).</td></tr>
<tr><td>Moisture Condition
(Fine Grained Soils)

(Coarse Grained Soils)</td><td>w > PL
w ≈ PL
w < PL
w ≈ LL
w > LL
D
M
W</td><td>Moisture content estimated to be greater than plastic limit.
Moisture content estimated to be approximately equal to plastic limit.
Moisture content estimated to be less than plastic limit.
Moisture content estimated to be near liquid limit.
Moisture content estimated to be wet of liquid limit.
DRY – runs freely through fingers.
MOIST – does not run freely but no free water visible on soil surface.
WET – free water visible on soil surface.</td></tr>
<tr><td>Strength (Consistency)
Cohesive Soils</td><td>VS
S
F
St
VSt
Hd
Fr
( )</td><td>VERY SOFT – unconfined compressive strength ≤ 25kPa.
SOFT – unconfined compressive strength > 25kPa and ≤ 50kPa.
FIRM – unconfined compressive strength > 50kPa and ≤ 100kPa.
STIFF – unconfined compressive strength > 100kPa and ≤ 200kPa.
VERY STIFF – unconfined compressive strength > 200kPa and ≤ 400kPa.
HARD – unconfined compressive strength > 400kPa.
FRIABLE – strength not attainable, soil crumbles.
Bracketed symbol indicates estimated consistency based on tactile examination or other assessment.</td></tr>
<tr><td>Density Index/
Relative Density
(Cohesionless Soils)</td><td>VL
L
MD
D
VD
( )</td><td><table><tr><th></th><th>Density Index (I_D) Range (%)</th><th>SPT ‘N’ Value Range (Blows/300mm)</th></tr><tr><td>VERY LOOSE</td><td>≤ 15</td><td>0 – 4</td></tr><tr><td>LOOSE</td><td>> 15 and ≤ 35</td><td>4 – 10</td></tr><tr><td>MEDIUM DENSE</td><td>> 35 and ≤ 65</td><td>10 – 30</td></tr><tr><td>DENSE</td><td>> 65 and ≤ 85</td><td>30 – 50</td></tr><tr><td>VERY DENSE</td><td>> 85</td><td>> 50</td></tr></table>Bracketed symbol indicates estimated density based on ease of drilling or other assessment.</td></tr>
<tr><td>Hand Penetrometer Readings</td><td>300
250</td><td>Measures reading in kPa of unconfined compressive strength. Numbers indicate individual test results on representative undisturbed material unless noted otherwise.</td></tr>
</table>

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

Classification of Material Weathering

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

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

Rock Material Strength Classification

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

Abbreviations Used in Defect Description

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