Rogans Hill Park
1020 Melia Court, Castle Hill, NSW

Initial Geotechnical Assessment

Castle Hill Glen Pty Ltd

Reference: SYDGE321033-AC

8 November 2023

ROGANS HILL PARK, 1020 MELIA COURT, CASTLE HILL, NSW

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Restriction on Disclosure and Use of Data

Subsurface conditions can be complex and may vary over relatively short distances – and over time. The inferred preliminary geotechnical model and recommendations in this report are based on limited subsurface investigations at discrete locations. Further investigations will be required to support detailed planning and design.

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APPENDICES

1. INTRODUCTION

This report presents the results of an initial site geotechnical assessment carried out by Tetra Tech Coffey for the proposed Rogans Hill Park development at 1020 Melia Court, Castle Hill, NSW. This assessment was commissioned by Mr Basil Lim from EinV Group Pty Ltd, on behalf of Castle Hill Glen Pty Ltd (**CHG**), and was carried out in general accordance with our fee proposal (ref SYDGE318XXX, dated 23 March 2023)

The site is currently undeveloped. Ground conditions across the site comprise a deep soil creep landslide over shale bedrock. The cost of future landslide mitigation measures for the site was found to be uneconomical for conventional low density residential housing development. We understand that CHG is proposing to now submit a Development Application to the Hills Shire Council for the construction of a mix of low and medium density residential buildings at this site. The proposed development will include six 3-6 level unit blocks, together with an area of townhouses, internal access roads and central park. All buildings will have basement car parking.

The purpose of this initial geotechnical assessment is to review currently available geotechnical information for this site as a basis for comment on the perceived feasibility of this development, together with discussion on potential geotechnical design strategies for the construction the proposed development on this landslide site.

2. PROPOSED DEVELOPMENT

The currently proposed development scheme is shown in Figure 1 below.

Figure 1: Rogans Hill Park proposed residential development.

In summary the building area will occupy the central portion of the site and comprises the following:

- Five 3-6 level unit blocks (Buildings 1 to 5) to be located along the toe of the existing “upslope” hill side,
- A sixth unit block of 4 levels (Building 6) downhill from Buildings 1 and 2 and adjacent to the site road entrance,
- An area of 2 and 3 levels townhouse to the east of Building 6.

All buildings are to have basement car parks with lower basement floor finished level of between approximately RL 158m to RL167m requiring bulk excavations across the site of typically between 3 m to 8m depth below current site ground surface levels. The current concept design proposed that bulk excavations for the Buildings 1 to 5 basements will be retained on the uphill side with a large anchored shoring wall socketed into the underlying sound shale bedrock. All other bulk excavation across the site is expected to be stabilised with temporary cut batters where space permits.

3. CURRENT SITE DESCRIPTION

As part of our initial site assessment to observe current site conditions, a Tetra Tech Coffey geotechnical engineer carried out a site walkover visit on 4 May 2023. The weather at the time of our site visit was clear and sunny, no rainfall had occurred for at least three days prior to our site visit.

The attached Figure 2 shows the location of the site in Castle Hill. Figure 3 following this is an air photo plan of the site.

The site itself slope down to the south, from Melia Court to the north, down to a creek gully in the south. There is a change in elevation of approximately 60m north south across the site, from approximately RL 190m AHD along the northern site boundary down to RL 135m AHD at the southern end of site. This hillside can then be broadly divided into the following three distinct areas:

- The northern third of site is a moderately steep to steep hillside that is approximately 20m high and slopes down to the south at 22°-25°
- The central third of site is a gently to moderately sloping hillside that is approximately 15m high and slopes down to the south at 5°-8° with a change of elevation of approximately 20m.
- The lower third of site was again moderately steep with a slope of approximately 10°-15° degrees.

At the time of our site visit on the central third was readily accessible as it had been recently cleared of all weed and shrubs. The northern and southern thirds of site were heavily vegetated and was difficult to walk through due to this vegetation. There are no structures or paved areas on site.

In the central part of site, it was evident that the surficial soils had been disturbed by past track clearing and possibly some earthworks with a number of access tracks cut into the hill side, and low spoil mounds across the area. There were no bedrock exposures observed on, or around the site.

The land to the west of the site was largely vacant other than a large above ground Sydney Water reservoir tank located at the bottom of the hillside below the site.

The land to the east of the site, i.e., Doris Hirst Place, has been developed into a residential area comprising large single house lots, each with a large two level rendered brick house on it. From a brief walk through this area, it appears that these houses were in relatively good condition and are generally uncracked.

Above the subject site, i.e. at the top of the hill along the northern site boundary, the western half of this areas adjoins more neighbouring house lots, each with a single one or two levels house on it. At the time of our site visit we did not walk through this area. The eastern half of the sites along the northern boundary runs along

Melia Court and a private residential access road. The road pavement levels here were approximately 2.5m to 3m higher than the site ground level and is retained by a vertical concrete blockwork reinforced soil wall.

Our pertinent site observation made during the site visit are as follows:

- We were able to locate and dip three existing standpipes at the site. The measured ground water levels were 1.5m, 3.5m and 4.5m below ground surface levels west to east across the site.
- Immediately downslope of the location where the standpipe was dipped at a water level of 1.5m, (western end of site) there was water pooled on the ground surface and boggy/soft ground. While no flow or point source was found, it is anticipated that this ponded water may be from near surface groundwater seepage. This would indicate that groundwater is still at a relatively shallow depth in the western part of the site.
- While the Glen Road pavement was not significantly cracked or distorted, the 3m to 4m high sandstone boulder wall along Glen Road which retains some of the properties above the subject site, had distinct outward bulge with part of the wall almost touching a street light pole at the base of the retaining wall. Additionally, some of the sandstone boulder in the area of the building are now cleanly cracked through the middle and there is also minor groundwater seepage from the wall face here at an estimated level of RL 176m. this would indicate that both groundwater seepage and soil creep movement is also occurring in the steeper hillside above the central part of site.
- Some sections of the blockwork wall supporting Melia Court and the private access road along the northern, i.e. uphill, site boundary was outwardly bulging and there was also a number of areas of transverse cracking, i.e. parallel to the hillside, along Melia Circuit. This could be an indication of creep movement at the crest of the hillside.
- During our walkabout in the residential area to the east of site, one of the landowners, who claimed to have been the builder of his house and the surrounding ones, suggested that all the houses in this area were founded on 10m to 15m deep bored piles to bedrock, as this had been a council requirement for construction here due to land sliding.

4. BACKGROUND GEOTECHNICAL INFORMATION

4.1 MECHANISM OF CREEP LANDSLIDE

The 100;000 scale geological map for Sydney indicates that site is underlain by Residual Soils and Ashfield Shale bedrock. However, we also note that there may be some remanent Bringelly Shale bedrock along the crest of the hill/ridge line up slope immediately to the north of the site. Hawkesbury Sandstone is likely to be present at lower elevations in the creek gully below, i.e. down slope to the south of the site.

Where the full sequence exists, Ashfield Shale is about 60 m to 70 m in thickness and consists of four siltstone and laminite sub-group members, comprising both siltstone and Siltstone laminite with fine-grained sandstone laminae. Fresh Ashfield Shale is typically of high and very high strength, however where it is located close to the ground surface, it can weather to depths of at least 6m to 10 into extremely weathered very low strength rock and Residual Silty and Shaley Clays of medium to high plasticity, and medium to high reactivity.

In the Castle Hill area, deep creep landslides like the one present on the subject site, are known to be present at numerous locations within the steeper western and southern facing slopes of the ridge lines along which Castle Hill Road, and Old Northern Road have been built. The mechanism of creep landslide is slow movement of the soil mass usually on relatively flat slope angles. The movements are triggered by cumulative rainfall that infiltrates into the bedrock from the uphill catchment, and then causes hydrostatic groundwater pressure at the interface between bedrock and the lower permeability clay soil above. The movements thus occur slowly and cumulatively over thousands of years and a slickenside surface then develops (i.e., polishing of the slide plane surface) thereby lowering the internal friction angle of the soil to residual values.

4.2 PREVIOUS GEOTECHNICAL INVESTIGATIONS AND PROPOSED LANDSLIDE MITIGATION MEASURES

At the subject site over the last 37 years several episodes of geotechnical investigations, geotechnical monitoring, and geotechnical modelling/design have been carried out by others as follows:

- 1986-1995, Golder Associates carried out a number of site investigation, geotechnical monitoring and site trial of trench drainage.
- 2003 to 2004, Douglas Partners carried out site investigation, geotechnical monitoring, and landslide back analysis for the design of network of deep trench drains across the site to stabilise the landslide.
- The Douglas Partners design solution comprised the construction of a deep trench drainage system through the central part of the site. The trench drains were to be spaced 15m apart and excavated to a minimum depth of 7m. After the construction of this drainages system, 5 years of site geotechnical monitoring was required to confirm that no future groundwater levels rise, or ground movement was occurring prior to any further site development works.
- 2015, Douglas Partners carried out additional site investigation and monitoring for the reassessment and revision their drainage design solution.
- 2017, Taylor Geotechnical Engineering carried out a geotechnical review of Douglas Partners 2015 site investigation and drainage design together with drilling their own site investigation pile holes around the site. from this they proposed site stabilisation using a two-tiered pile retaining wall together with the removal and replacement of the landslide material in the central part of the site.
- The Taylor Geotechnical design solution comprised the construction of a two-tiered pile retained wall along the toe of the northern site hillside. Both walls were to be socketed into sound bedrock and anchored for lateral restraint following this the central part of site was then to be excavated to the slightly weather to fresh shale and then filled with Engineered Fill.

4.2.1 Summary

For our assessment, we were only able to source some of the above documentation. However, in summary from the available information we note the following about the site:

- The site comprises a large creep landslide of between 8-10m depth, extending down through the upper layers of weathered shale bedrock to the top of sound shale bedrock, i.e., medium to high strength, slightly weathered to fresh shale and laminite.
- In the central part of the site, the ground profile comprises three strata as follows;
  - Clays soils of 4m to 5m thickness, over
  - Extremely to highly weathered and fractured shales of between 3m to 5m thickness, over
  - Slightly weathered to fresh, medium to high strength, sub horizontally bedded shale and laminite.
- Groundwater levels across the site are typically between 1m to 5m below ground surface levels but following high/extreme rainfall events these levels/pressures can rapidly rise to ground surface levels.
- It has not been possible to identify an exact location for a single slide plane within the soil profile. However, it is anticipated that the landslide actually comprises combination of slide planes within the residual soil and weathered shale, together with a large number of fractures zones, clay seams and clay filled or smooth joints, along which the soil mass is sliding.

- Even during extended dry periods there were areas of constant seepage at the ground surface, particularly in the western part of site.
- When groundwater levels rise in response to high rainfall levels, between 3m to 10mm of landslide movement was recorded by borehole inclinometers.
- The Douglas Partners 2004 back analysis of the landslide estimated that the failure surface has a friction angle of between 12° to 16°.
- The Taylor Geotechnical Engineering 2017 back analysis of the landslide estimated the failure surface has a friction angle of between 10.5° to 14°.

It is also important to note that all previous site investigation was only carried out in the accessible central part of the site. No geotechnical investigation has been carried out in the steeper northern or southern thirds of the site. All previous design assessments have assumed a shallow uphill bedrock profile with a relatively horizonal groundwater level back into the slope.

5. ANTICIPATED SITE GROUND CONDITIONS

The attached Figures 4, 5 and 6 present our summary sketch long sections through the subject site developed from the available survey, borehole, and test pit information together with our observations on site. These sketch sections show that that there is currently no ground information for both the northern and southern thirds of site. however, in the central third of the site ground conditions are expected to comprise;

Silty clays; 5m (upslope) to 3m (down slope) of silty clays. The investigation to date has not clearly defined whether theses soils are residual or colluvium, it is expected that there will varying degrees of each throughout the site.

Extremely to highly weathered shale, of between 4m to 6m thickness. This material has been assessed to be typically of extremely low to very low strength, and in places high fracture with numerous clays seams and sub horizontal jointing.

Slightly weathered to fresh shale and laminite, underlies the soils and weathered rock at depth of between 8m to 13m across the central part of site, i.e. below an approximate level of RL155m to RL160m AHD dipping east to west across the central section of site. this shale was of medium and high strength.

Groundwater was observed at depths of between 1.5m to 5m west to east across the central porting of site. However, it is noted that during extreme rainfall events groundwater levels/pressures can rapidly rise to ground surfaces level in some parts of the site.

Land slip surface, while the soil and weathered rock may actually contain numerous slide planes, it should be assumed that the overall slide surface exists at the weathered rock/slightly weathered rock interface. Back analysis by others suggest that the friction angle of this slide surface could range from 10.5° to 16°

6. DISCUSSION

6.1 LANDSLIDE MANAGEMENT AND CONSTRUCTION STRATEGIES

Typical strategies for the remediation and/or construction on existing creep landslide sites often included any or all of the following:

- Support of the slide mass with engineering structures such as retaining walls and/or toe berms so it is fully retained.
- Remove and/or replace the slide mass and slide surfaces with Engineered Fill, structures can then be constructed on the Engineered Fill.
- Construct a permanent subsurface drainage network so that no excess groundwater pressure can build up on the slide surface and in the soil mass.
- Reduce the slide mass by cutting back the slope.
- Reinforce the slide mass with cast insitu elements such as piles or jet grout columns on a closely spaced grid pattern across the slide footprint.

For an initial understanding of basement/building depths in relation to existing ground conditions, the estimated basement profiles have also been included on our sketch geotechnical long sections, Figures 4, 5 and 6. However, it is important to note that these sections are preliminary sketches for initial information only, they are not for detailed project planning or design.

6.1.1 Buildings 1, 2, 3, 4 and 5

At this stage we understand that it is currently proposed to construct a large anchored shoring wall around the northern perimeter of the proposed development footprint to enable excavation and construction of the Building 1 to 5 basements. From our initial understanding of ground conditions there, it is expected that this wall will have to be socketed into the slightly weathered to fresh shale at depths of 10 m to 13 m along the toe of the current hillside. Allowing for suitable pile sockets and toe restraint, this would then result in a typical pile depth of 15 m to 20 m. Several rows of ground anchors to be installed back into the hillside would be necessity for lateral restraint. Once the full depth of soil mass upslope is retained, bulk earthworks could then be carried out using conventional excavation methods within the existing landslide materials. Downslope excavation stability could be achieved by battering and benching back the excavation face where space permits. From a geotechnical perspective this is expected to be a suitable approach for the construction of the building basements, however we note that further geotechnical site investigation and geotechnical analysis would be required for design and sizing of piles and anchor elements. In addition to the retaining wall, permanent drainage of the upslope and under building areas will also be necessary. It is expected that this would comprise the following:

- Strip drains built into the retaining wall, i.e. between piles,
- Subhorizontal borehole drains drilled back into the hillside.
- A continuous drainage layer/blanket and possibly deep trench drains under each building footprint.

For the currently proposed lower basement level of approximately RL 161 to RL162 m AHD, there may be approximately 2 m to 5 m of slide material/extremely weather shale remaining above the slightly weathered to fresh Shale. As a result, all buildings and basement floor slabs will be founded on pile footings or blade walls socketed into the “sound” slightly weathered to fresh shale. At this stage it is currently expected that where a limited depth of slide material remained under the building footprint it can be over-excavated and removed down to a sound base and then replaced with Engineered Fill, or alternatively additional piles to sound rock and the use of trench drains can be used to stabilise/drain the remaining “slide” materials.

Any backfilling of bulk excavations on the down-hill side of each building must be carried out using Engineered Fill taken down to a sound based, and having appropriate permanent drainage measured built into it.

6.1.2 Building 6 and the Townhouses

It is understood that the lower basement floor levels for Building 6, and the townhouses will be between RL 158 m and 162 m. It is expected that between 3 m to 9 m of soils and slide material would then remain beneath these buildings following bulk excavations. It is expected that the lowest cost solution here may be to then leave all remaining slide material insitu and found the building, including lowest basement slabs on pile footings or shear walls (which would also be utilised to support the buildings) socketed into the slightly weathered to fresh shale. From a geotechnical perspective this may be feasible if sufficient permanent subsoil drainage can be constructed within the remaining soil mass preventing any future water pressure build-up and potential sliding. However, to properly assess the feasibility of this option, additional site investigation and detailed geotechnical/groundwater modelling will be required. Analysis of slope movement and bending and shear forces on the piles will be required to assess pile or shear wall reinforcement. If this is not feasible then the alternative construction strategies for these areas could include one or a combination the following:

- Over-excavation and replacement of the slide materials with Engineered Fill.
- Reinforcement of the remaining slide material using bored or jet grout piles on a grid pattern under each building footprint.

It is expected that each of the above would also require a permanent subsoil drainage system to reduce potential build-up of groundwater water pressure in the retained soil.

6.1.3 Remaining areas of open space

In addition to the proposed buildings, this development will include areas of open spaces for internal roads and landscaping. Where it can be determined that soil creep movements are not of consequence, (e.g. open space/playgrounds), then no landslide remediation or stabilisation measures would be required. However, in those areas where this is not the case (e.g. internal roads), then it is expected that the soil mass could then be stabilised using permanent subsoil drainage such as deep trench drains or structural elements such as those described in Section 6.1.2 above.

6.2 BUILDING FOOTINGS

6.2.1 Buildings 1 to 5

It is recommended that Buildings 1 to 5 be founded on the slightly weathered to fresh shale using either shallow pad, or pile footings, or blade walls depending on the final design basement levels, depth to the top of this rock unit and geotechnical stability assessment of the remaining materials

As a guide for initial pad, plie or wall design, footings socketed into the slightly weathered to fresh shale of at least medium strength may be initially designed for an indicative allowable end bearing pressure of between 3000 kPa and 3500 kPa. However, this will have to confirmed by further site investigation and rock core borehole drilling to at least 3 m below proposed founding levels in the shale.

6.2.2 Building 6 and the Townhouses

As per Buildings 1 to 5, it is also recommended that Building 6 and the townhouses be supported on footings socketed into the slightly weathered to fresh shale. Specifically, footings for Building 6 and the townhouses will depend on the landslide mitigation measures to be adopted. Where structural elements such as piles, shear walls, or jet grout columns are proposed to stabilise the landslide, these elements that are socketed into slightly weathered to fresh shale may be incorporated as the support for the buildings.

Where the risk of landslide is to be managed by the use of subsoil drainage, buildings should be supported on piles socketed into the slightly weathered to fresh shale.

6.3 ADDITIONAL GEOTECHNICAL INVESTIGATION AND DESIGN

We note the suitability and design of the proposed building strategies will have to be further assessed and confirmed by additional detailed site geotechnical investigation and geotechnical design in close consultation with the project structural designers and review of construction costs. Specifically, it is expected that the following would be required;

- Additional detailed site geotechnical investigation.
- Ongoing groundwater level monitoring.
- Detailed geotechnical design support and assessments including groundwater, geotechnical and landslide Finite Element modelling.

As a guide for addition site investigation, it is expected that the following would be required:

- Borehole drilling in the northern third of the site to characterise ground conditions for retaining wall design and the installation of groundwater observations wells. Some form of cross slope track construction would be required for drill rig access here.
- Rock core borehole drilling in the central part of the site to fully assess the strength and nature of defects within the slightly weathered to fresh shale and laminite, this will be required for building and retaining wall footing/socket design. It is expected that core boreholes would have to extend to depth of at least 3m below footing base or pile toes.
- Excavation of deep test pits near the top of the slope with the aim of identification of existing slide planes, and if found, sampling and laboratory shear box testing should be carried out to better assist its peak and residual shear strengths.
- Ongoing groundwater levels monitoring and comparison with rainfall records to assess current site groundwater levels, fluctuations and flow directions for detailed design groundwater modelling. The required standpipes can be installed as part of the building foundation and upslope borehole drilling investigation works.

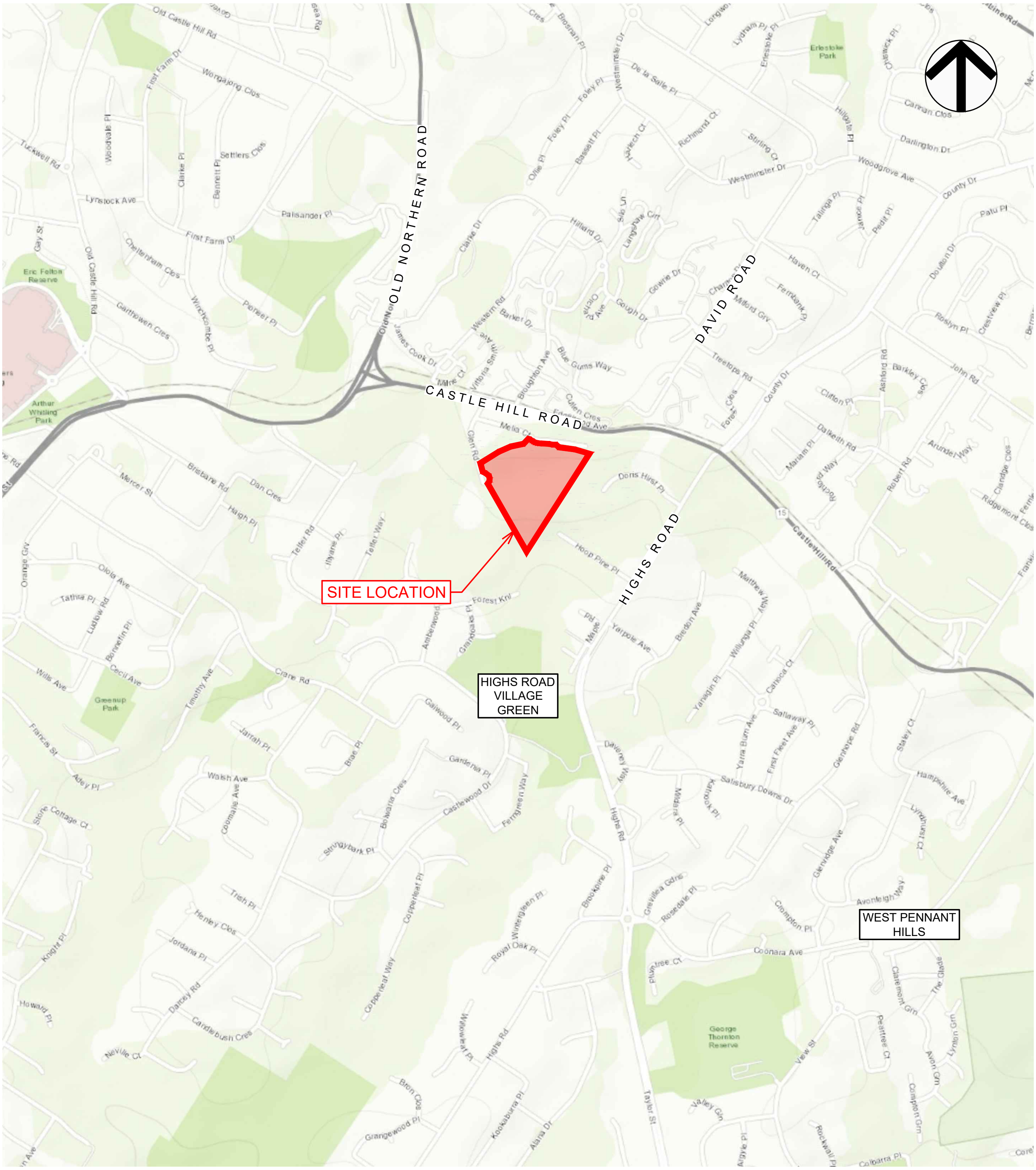
7. CONCLUSIONS

Based on our site observations, preliminary geotechnical model, and experience on similar projects, the proposed development is considered feasible from a geotechnical perspective. Appropriate additional site investigation, design assessments, and construction monitoring normally associated with this type of development would need to be carried out.

8. CLOSURE

Subsurface conditions can be complex and may vary over relatively short distances – and over time. The inferred preliminary geotechnical model and recommendations in this report are based on limited subsurface investigations at discrete locations. Further investigations will be required to confirm our current recommendations and support detailed planning and design.

APPENDIX A: FIGURES



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Scale (metres) 1:15000

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PRELIMINARY NOT FOR DESIGN

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approved	S.P.	project:	Rogans Hill Park 1020 Melia Court CASTLE HILL, NSW
date	8-11-2023	title:	SITE LOCATION PLAN
scale	AS SHOWN	project no:	754-SYDGE321033
original size	A4	figure no:	FIGURE 2
		rev:	A

TETRA TECH COFFEY

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revision no.	description	drawn	approved	date
A	ORIGINAL ISSUE	-	-	-

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date	8-11-2023
scale	AS SHOWN
original size	A3

TETRA TECH COFFEY

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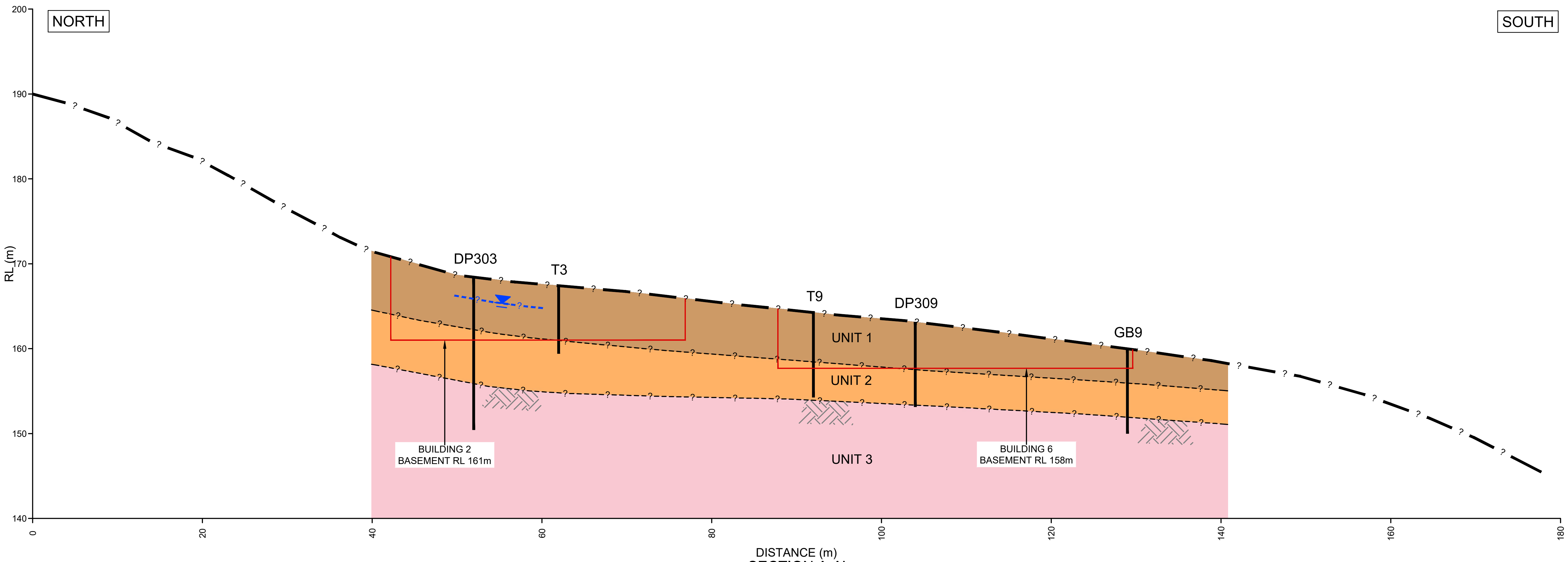
project: Rogans Hill Park
1020 Melia Court
CASTLE HILL, NSW

title: SITE PLAN

project no: 754-SYDGE321033 | figure no: FIGURE 3 | rev: A



SECTION A-A'

UNIT LEGEND

- UNIT 1 - SILTY CLAYS
- UNIT 2 - EXTREMELY TO HIGHLY WEATHERED SHALE
- UNIT 3 - SLIGHTLY WEATHERED TO FRESH SHALE

SECTION LEGEND

- —?— INFERRED GROUND SURFACE
- INFERRED BASEMENT PROFILE
- — ? — INFERRED GEOLOGICAL BOUNDARY
- — ? — INFERRED WATER TABLE
- WATER LEVEL

PRELIMINARY NOT FOR DESIGN

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revision

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Vertical Scale (metres) 1:500 (5 0 5 15 25)

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TETRA TECH COFFEY

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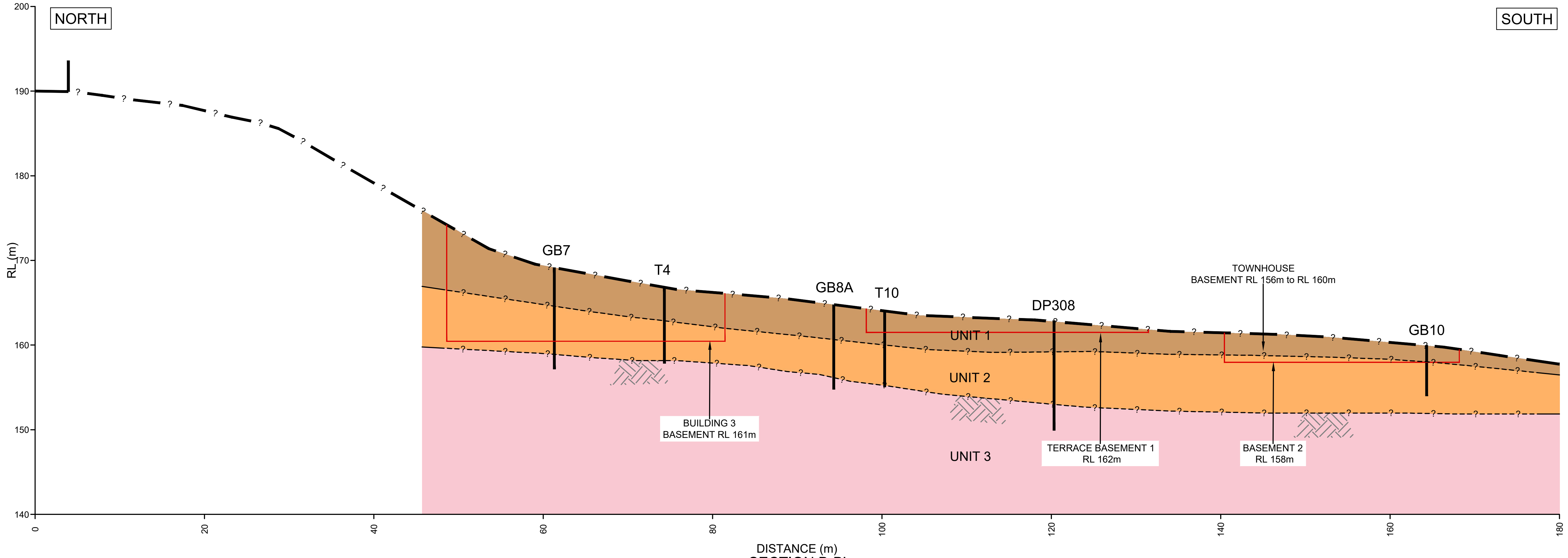
project no:	754-SYDGE321033
figure no:	FIGURE 4
rev:	A



SECTION B-B'

UNIT LEGEND	SECTION LEGEND	
UNIT 1 - SILTY CLAYS	INFERRED GROUND SURFACE	INFERRED WATER TABLE
UNIT 2 - EXTREMELY TO HIGHLY WEATHERED SHALE	INFERRED BASEMENT PROFILE	WATER LEVEL
UNIT 3 - SLIGHTLY WEATHERED TO FRESH SHALE	INFERRED GEOLOGICAL BOUNDARY	

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TETRA TECH COFFEY

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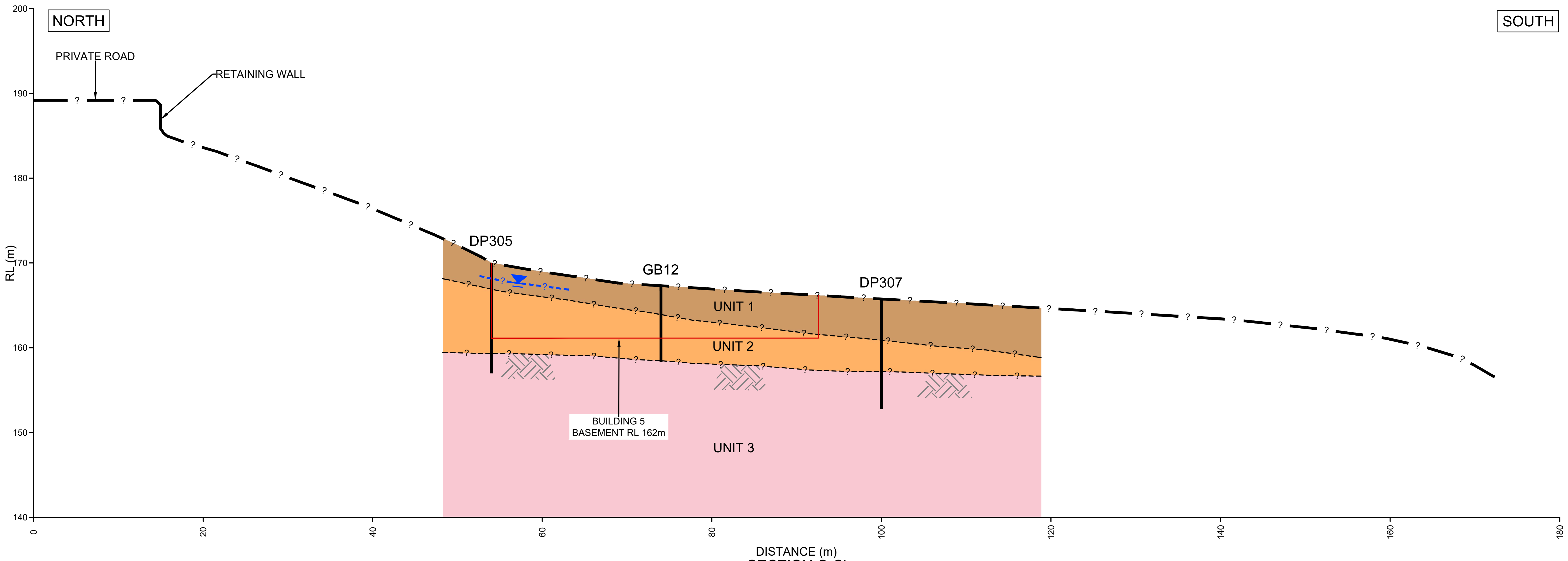
rev:	A



SECTION C-C'

UNIT LEGEND

- UNIT 1 - SILTY CLAYS
- UNIT 2 - EXTREMELY TO HIGHLY WEATHERED SHALE
- UNIT 3 - SLIGHTLY WEATHERED TO FRESH SHALE

SECTION LEGEND

- INFERRED GROUND SURFACE
- INFERRED BASEMENT PROFILE
- INFERRED GEOLOGICAL BOUNDARY
- INFERRED WATER TABLE
- WATER LEVEL

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TETRA TECH COFFEY

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title:	SKETCH GEOTECHNICAL SECTION C-C'
project no:	754-SYDGE321033
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rev:	A

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IMPORTANT INFORMATION ABOUT YOUR TETRA TECH COFFEY REPORT

As a client of Tetra Tech Coffey you should know that site subsurface conditions cause more construction problems than any other factor. These notes have been prepared by Tetra Tech Coffey to help you interpret and understand the limitations of your report.

Your report is based on project specific criteria

Your report has been developed on the basis of your unique project specific requirements as understood by Tetra Tech Coffey and applies only to the site investigated. Project criteria typically include the general nature of the project; its size and configuration; the location of any structures on the site; other site improvements; the presence of underground utilities; and the additional risk imposed by scope-of-service limitations imposed by the client. Your report should not be used if there are any changes to the project without first asking Tetra Tech Coffey to assess how factors that changed subsequent to the date of the report affect the report's recommendations. Tetra Tech Coffey cannot accept responsibility for problems that may occur due to changed factors if they are not consulted.

Subsurface conditions can change

Subsurface conditions are created by natural processes and the activity of man. For example, water levels can vary with time, fill may be placed on a site and pollutants may migrate with time. Because a report is based on conditions which existed at the time of subsurface exploration, decisions should not be based on a report whose adequacy may have been affected by time. Consult Tetra Tech Coffey to be advised how time may have impacted on the project.

Interpretation of factual data

Site assessment identifies actual subsurface conditions only at those points where samples are taken and when they are taken. Data derived from literature and external data source review, sampling and subsequent laboratory testing are interpreted by geologists, engineers or scientists to provide an opinion about overall site conditions, their likely impact on the proposed development and recommended actions. Actual conditions may differ from those inferred to exist, because no professional, no matter how qualified, can reveal what is hidden by earth, rock and time. The actual interface between materials may be far more gradual or abrupt than assumed based on the facts obtained. Nothing can be done to change the actual site conditions which exist, but steps can be taken to reduce the impact of unexpected conditions. For this reason, owners should retain the services of Tetra Tech Coffey through the development stage, to identify variances, conduct additional tests if required, and recommend solutions to problems encountered on site.

Your report will only give preliminary recommendations

Your report is based on the assumption that the site conditions as revealed through selective point sampling are indicative of actual conditions throughout an area. This assumption cannot be substantiated until project implementation has commenced and therefore your report recommendations can only be regarded as preliminary. Only Tetra Tech Coffey, who prepared the report, is fully familiar with the background information needed to assess whether or not the report's recommendations are valid and whether or not changes should be considered as the project develops. If another party undertakes the implementation of the recommendations of this report there is a risk that the report will be misinterpreted and Tetra Tech Coffey cannot be held responsible for such misinterpretation.

Your report is prepared for specific purposes and persons

To avoid misuse of the information contained in your report it is recommended that you confer with Tetra Tech Coffey before passing your report on to another party who may not be familiar with the background and the purpose of the report. Your report should not be applied to any project other than that originally specified at the time the report was issued.

Interpretation by other design professionals

Costly problems can occur when other design professionals develop their plans based on misinterpretations of a report. To help avoid misinterpretations, retain Tetra Tech Coffey to work with other project design professionals who are affected by the report. Have Tetra Tech Coffey explain the report implications to design professionals affected by them and then review plans and specifications produced to see how they incorporate the report findings.

Data should not be separated from the report

The report as a whole presents the findings of the site assessment and the report should not be copied in part or altered in any way. Logs, figures, drawings, etc. are customarily included in our reports and are developed by scientists, engineers or geologists based on their interpretation of field logs (assembled by field personnel) and laboratory evaluation of field samples. These logs etc. should not under any circumstances be redrawn for inclusion in other documents or separated from the report in any way.

Geoenvironmental concerns are not at issue

Your report is not likely to relate any findings, conclusions, or recommendations about the potential for hazardous materials existing at the site unless specifically required to do so by the client. Specialist equipment, techniques, and personnel are used to perform a geoenvironmental assessment. Contamination can create major health, safety and environmental risks. If you have no information about the potential for your site to be contaminated or create an environmental hazard, you are advised to contact Tetra Tech Coffey for information relating to geoenvironmental issues.

Rely on Tetra Tech Coffey for additional assistance

Tetra Tech Coffey is familiar with a variety of techniques and approaches that can be used to help reduce risks for all parties to a project, from design to construction. It is common that not all approaches will be necessarily dealt with in your site assessment report due to concepts proposed at that time. As the project progresses through design towards construction, speak with Tetra Tech Coffey to develop alternative approaches to problems that may be of genuine benefit both in time and cost.

Responsibility

Reporting relies on interpretation of factual information based on judgement and opinion and has a level of uncertainty attached to it, which is far less exact than the design disciplines. This has often resulted in claims being lodged against consultants, which are unfounded. To help prevent this problem, a number of clauses have been developed for use in contracts, reports and other documents. Responsibility clauses do not transfer appropriate liabilities from Tetra Tech Coffey to other parties but are included to identify where Tetra Tech Coffey's responsibilities begin and end. Their use is intended to help all parties involved to recognise their individual responsibilities. Read all documents from Tetra Tech Coffey closely and do not hesitate to ask any questions you may have.