GEOSOLVE

GEOTECHNICAL

WATER RESOURCES

PAVEMENTS

Geotechnical Report

Lots 9 & 10, Block 3/DP 28

60 Kauri Street

Ravensbourne, Dunedin

Report prepared for:

David Walsh

Report prepared by:

GeoSolve Limited

Distribution:

David Walsh

GeoSolve Limited (File)

November 2018

GeoSolve Ref: 180239

Revision	Issue Date	Purpose	Author	Reviewed
1	**8/11/2018**	Client issue	RS	MTW

GEOTECHNICAL WATER RESOURCES PAVEMENTS

Table of Contents

1 Introduction

1.1 General

This report presents the results of geotechnical investigations carried out by GeoSolve Ltd in order to determine subsoil conditions and provide geotechnical inputs for a proposed subdivision at 60 Kauri Street in Ravensbourne, Dunedin.

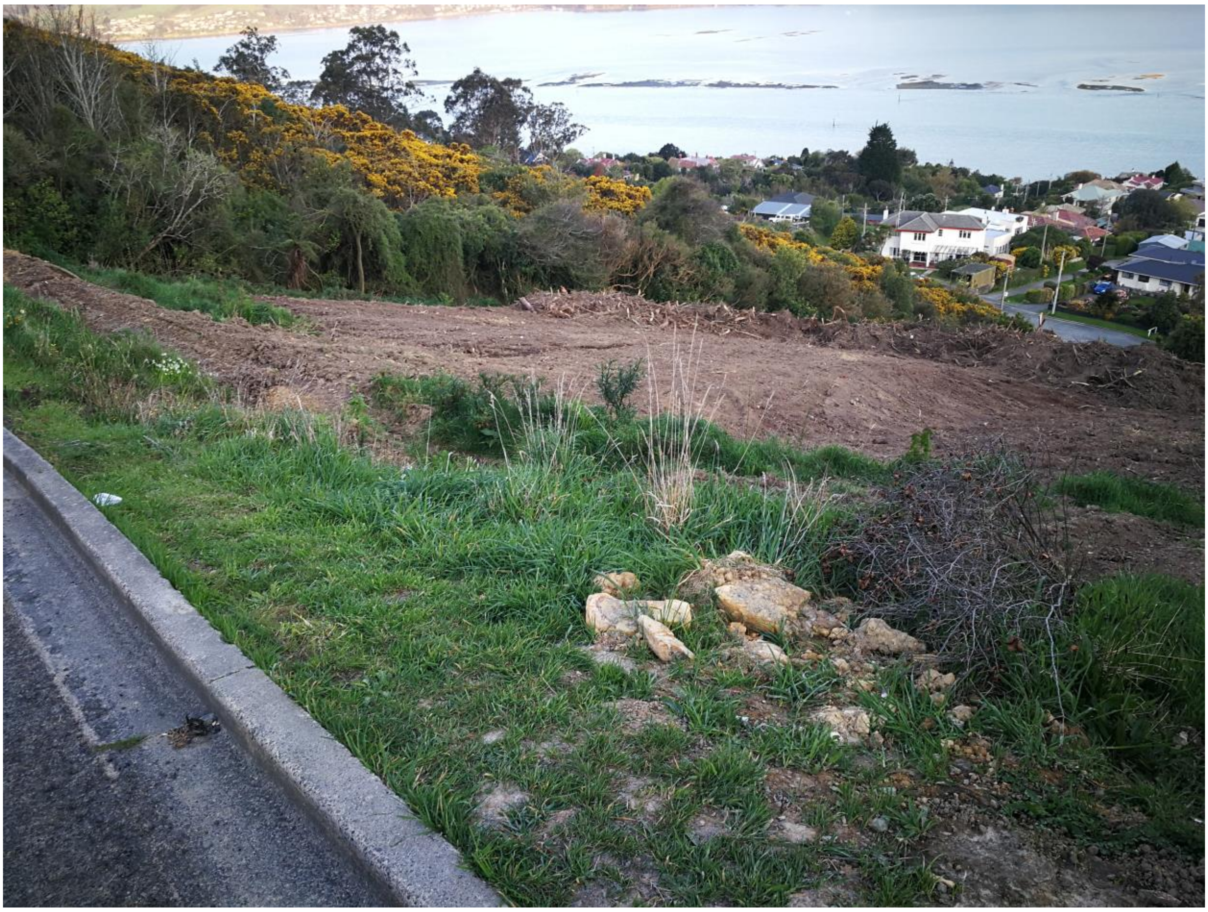

Photo 1 – Proposed development site, 60 Kauri Street

The investigations were carried out for David Walsh in accordance with GeoSolve Ltd's proposal dated 14 August 2018, which outlines the scope of work and conditions of engagement.

1.2 Development

We understand the proposed development is for subdivision of the ½-acre site into three smaller residential lots and an access lot. We understand that the current proposal is to establish access along the eastern extents of the site to provide a right-of-way to all three lots.

2 Site Description

2.1 General

The subject property is located at Ravensbourne which is situated approximately 5 km east of central Dunedin. The site is currently undeveloped, having only recently been cleared of thick vegetation comprising trees, shrubbery, grasses and gorse.

The property is accessed from the north by Kauri Street and is bounded to the south and west by ¼ acre residential lots. An undeveloped road (Michie Street) is situated to the east, where a partially culverted, unnamed stream receives stormwater draining from the flanks of Signal Hill to the north.

2.2 Topography and Surface Drainage

The development site has been surveyed and the site topography is illustrated in plans by TL Survey (ref 18-197, dated 30 October 2018) which is included in Appendix A.

The development site occupies a south-facing slope which is formed at a moderate gradient of approximately 19°. The northern end of the site near Kauri Street is locally steeper upslope of the cut formed to provide access to the site; gradients range from 38-70° in this area over short distances.

The difference in elevation between the highest and lowest surveyed parts of the site is approximately 24 m.

The site is naturally free draining and no spring flows were evident during the investigations.

3 Geotechnical Investigations

An engineering geological site appraisal has been undertaken with confirmatory subsurface investigations. GeoSolve Ltd visited the subject property on 4 October 2018, undertaking geotechnical investigations comprising five test pits which were advanced to a maximum depth of 2.4 m. Dynamic cone (Scala) penetrometer tests were undertaken at 4 of the 5 test pit locations advancing to at least 1.3 m below existing ground level.

Test pit and Scala penetrometer locations and logs are contained in Appendices A and B respectively. Test pit and Scala locations were marked with stakes prior to the topographical survey included in Figure 1, Appendix A; thus (for example) ‘Stake 1’ refers to the location of both ‘TP1’ and ‘SC1’.

The investigations described in this report are sufficient for the purposes of assessment of slope stability and foundation considerations to a level suitable for subdivision consent. Further site-specific investigations are required for detailed design and for individual dwellings prior to the building consent stage.

4 Subsurface Conditions

4.1 Geological Setting

4.1.1 Regional Geology

The geology of the Dunedin area is dominated by volcanic rock types of basaltic to andesitic composition that were intruded through pre-existing marine sediments during Miocene times. Extensive volcanism at that time produced lava flows and bedded volcanoclastic materials were widely distributed by eruptions. The generalised stratigraphic profile comprises schist at depth, overlain by a Cretaceous to Tertiary-age sequence; initially by thin non-marine sediments and then a thick accumulation of marine sediments including sandstones and mudstones. The volcanic rock types cross cut these sediments where vents were present and extensively mantle them where lava flows or volcanic ejecta were deposited.

More recently (Pleistocene times), the hills of Dunedin have been extensively mantled by windblown loess to depths of up to several metres. Watercourses and tidal embayments such as Otago Harbour have locally deposited alluvial, estuarine and marine deposits and generally modified the volcanic landscape by deep incision and sedimentation. Fill and refuse has been placed locally during post-settlement times. Landslips have occurred on steeper hillsides particularly where springs emerge or where fills have been placed.

4.1.2 Seismicity

Dunedin has traditionally been considered to have lower than average seismic activity when compared to other areas in New Zealand, however nearby active faults are known and strong shaking is certain to occur periodically.

McCahon et al[1] states that the earthquake hazard in Dunedin is dominated by relatively infrequent moderate to large earthquakes (magnitude up to M_w 7.5) in eastern Otago, and large to very large earthquakes in the much more seismically active Fiordland and Westland regions.

The nearest active faults with demonstrated Late Quaternary movement history are the Green Island Fault and the Akatore Fault. The Green Island Fault is currently considered to be the cause of the 1974 earthquake that caused damage in Dunedin. It is mapped approximately 12 km to the southwest of the subject site, but its projection is believed to continue through South Dunedin and may run northeast up the harbour in which case it would pass within about 2 km of the site. The Akatore Fault has also been projected beneath South Dunedin; the nearest mapped trace of the fault is truncated about 6 km southwest of the site, but the fault likely continues beneath South Dunedin and may run northeast up the harbour as well. It should be noted the fault terminations shown on fault trace maps are often approximations (owing to lack of data) and the presence of other active faults may be unknown because they may be obscured by overburden soils. Both of these faults are likely to be capable of generating magnitude 7.5 earthquakes in Dunedin.

[1] McCahon, I.F., Yetton, M.D., Cook, D.R.L. (1993). The Earthquake Hazard in Dunedin. EQC report 91/56.

Other known faults that have some potential to cause strong shaking in Dunedin are the Titri Fault and the North Taieri Fault, located roughly 11 km and 14 km west/northwest of the site, respectively.

The above faults are not included in Table 3.6 of NZS 1170.5:2004 as major faults requiring near fault factors when assessing structural design actions. Recent events in Canterbury have highlighted the issue that previously unidentified faults may be very significant factors in the actual future risk applying to any particular site.

Strong ground shaking throughout the South Island is likely to be associated with a rupture of the Alpine Fault, located along the West Coast of South Island. There is a high probability an earthquake with an expected magnitude of over 7.5 will occur along the Alpine Fault within the next 50 years.

Average return periods for shaking intensity are: MM 7 = 100 years, MM 8 = 450 years and MM 9 = >2,500 years. The most recent major earthquake to affect Dunedin occurred in 1974 and produced damage consistent with MM 7 intensity.

4.2 Stratigraphy

An engineering geological model for the site is shown in Figure 2, Appendix A. More detailed geotechnical description of soils is provided in the test pit logs contained in Appendix B.

Apart from the thin layer of surficial topsoil, the site is underlain by various alluvial and colluvial deposits to a depth of at least 2 m below the surface of the slope.

Topsoil comprises soft – firm organic SILT with rootlets.

Underlying this surficial topsoil layer across the majority of the site is **colluvium** which comprises mottled light brown and orange, silty CLAY with occasional boulders which is firm to stiff. Boulders were found to be up to 0.6 m in diameter in the colluvium. **Alluvium** was identified near test pits TP3 and TP5, comprising silty CLAY with a trace of sand and gravel, which was found to be soft to firm in consistency. Occasional to frequent boulders found in the alluvium were up to 0.5 m in diameter. All of the test pits were terminated in colluvium or alluvium.

Some isolated **uncontrolled fill** was observed on the steep slope adjacent to Kauri Street at the northern margins of the site in TP1, likely comprising cut-to-fill colluvium from the formation of Kauri Street; the fill was up to 0.4 m in thickness at TP1 but is likely to be up to 1.3 m thick locally along the northern boundary. Some recently placed uncontrolled fill was also observed downslope of the recently-formed access track. **Buried topsoil** associated with the placement of this uncontrolled fill was also noted in TP1.

Bedrock was not encountered but is expected to lie at moderate depth below the site, likely comprising Dunedin Volcanic Group basalt bedrock which is expected to extend to significant depths.

4.3 Groundwater

While the soils observed were predominantly moist in condition, minor groundwater seepage was observed in TP5 and some locally saturated zones were noted in the alluvial deposits. Perched groundwater may develop on the contact between any fill and the underlying in-situ soils, and where frequent boulders are found during times of high rainfall.

4.4 Slope Stability

No evidence of slope instability was identified on the site at the time of inspection.

The area has been mapped by Benson[2] as being underlain by Cossyritic phonolite (basalt), a strong rock type with little susceptibility to land instability on moderate slopes. The subdivision is not in close proximity to any areas mapped in a 2017 GNS Science report[3] as having been subject to current or historical land instability. No indications of potential global slope instability were noted during review of available published hazard mapping.

No evidence of slope instability was noted during the investigations.

The cut/fill slopes at the northern margins of the site are at overall batters of up to 70° and may become unstable if they become saturated or are subjected to seismic loading. Providing this cut is re-battered or retention and drainage is redesigned by a Chartered Professional Engineer, the risk of landslip activity affecting the site is likely to be very low providing the other recommendations of this report are adhered to. This is supported by the lack of groundwater and the firm to stiff or better condition of the soils observed.

[2] Benson, W.N. (1968). Dunedin District, 1:50,000. NZGS Miscellaneous Series Map 1. Department of Scientific and Industrial Research.

[3] Barrell DJA, Smith Lyttle B, Glassey PJ. (2017). Revised landslide database for the coastal sector of the Dunedin City district. Lower Hutt (NZ): GNS Science. 29 p. (GNS Science consultancy report; 2017/41).

5 Engineering Considerations

5.1 General

The recommendations and opinions contained in this report are based upon ground investigation data obtained at discrete locations and historical information held on the GeoSolve database. The nature and continuity of subsoil conditions away from the investigation locations is inferred and cannot be guaranteed.

5.2 Excavations

We recommend that all excavations be inspected by a geotechnical practitioner during earthworks construction.

Minimal seepage was encountered during test pitting and hence significant groundwater flows are unlikely to be encountered during excavations. However, a geotechnical practitioner should inspect any seepage, spring flow or under-runners that may be encountered during construction.

Recommendations for permanent batters are as follows.

Table 5.1 – Recommended batters for permanent cuts up to 3 m in height

Material Type	Recommended Maximum Batter for Permanent Cuts Less than 3 m High (horizontal to vertical)	
	Dry Ground	Wet Ground
Fill, Topsoil	2 : 1	3 : 1
Colluvium / Alluvium (stiff in consistency)	1.5 : 1	3 : 1
Colluvium / Alluvium (firm in consistency)	2 : 1	3 : 1

Temporary cuts may be formed at steeper angles (see below) subject to geotechnical advice.

An earthworks consent may be required prior to construction and this should be checked with Dunedin City Council.

The subsurface materials should be relatively easy to excavate by conventional methods; however, boulders are likely to be encountered within the colluvium and alluvium. Dunedin Volcanic Group basalt rock is expected to be present at moderate depth.

5.3 Fill Slopes

For landscaping or trenching purposes (where loads are not applied to fills), certification is generally not required but we recommend that a compaction methodology should be specified to minimise future settlement and landslip risk in subdivision areas.

All fill slopes less than 3 m in height should be constructed with a maximum batter of 2:1 (horizontal to vertical) or flatter, if well drained.

To minimise erosion, effective vegetation cover should be established on fill batters and no water flows should be directed to these slopes. Thicker or steeper fills will require specific engineering assessment and design.

The subgrade of any proposed fills will need to be sub-horizontal (with benching of slopes as required) to ensure stability.

If any re-use of the excavated soils on the site as engineered fill is required then an earthfill specification can be supplied on request. All fill that is utilised as bearing for foundations should be placed and compacted in accordance with the recommendations of NZS 4431:1989 and certification provided to that effect. Laboratory testing and verification will be required and boulders and cobbles over 75 mm in size will need to be screened from engineered fill sources. For engineered fills, the contractor will need to submit a sample of the proposed fill materials (possibly more than one) to obtain laboratory compaction curves, and in-situ Nuclear Density Meter (NDM) testing of the fill will need to be arranged. An engineer will need to specify the fill methodology and review the lab results to ensure that a statement of suitability for the fills can be issued; this will be required for code compliance.

Earthworks should only be carried out in the summer or during a period of forecast, prolonged dry weather as the alluvium and colluvium soils present on site are susceptible to becoming excessively moist and could rapidly become unsuitable for placement if they get wet.

Maintaining the moisture content of any cohesive fill soils to achieve the required compaction will need to be addressed by the contractor. It is recommended that cut to fill soils be placed and compacted immediately as they are excavated, as stockpiling and reworking is highly likely to degrade the compaction properties of these soils.

5.4 Ground Retention

We understand that no retention is currently proposed for the subdivision. Any retaining walls proposed should be designed by a chartered professional engineer.

Any retaining walls should be designed by a chartered professional engineer, adopting the geotechnical parameters outlined above. Further specific subsurface investigations for wall design may be required depending on final development requirements.

Due allowance should be made during the detailed design of all retaining walls for any additional loads upslope of the wall (i.e. surcharge due to backslope or vehicle loadings).

All temporary slopes for retaining wall construction should be battered at 1:1 (horizontal to vertical) provided these are within dry, stiff soils. Wet or softer soils will require specific inspection and assessment.

Groundwater seepages identified in some test pits had minimal flow rates, but these rates are likely to increase as a result of heavy or prolonged rainfall. To ensure potential groundwater seeps and flows are properly controlled behind retaining walls, the following recommendations are provided:

- A minimum 0.3 m width of durable free draining granular material should be placed behind all retaining structures;
- A heavy duty non-woven geotextile cloth, such as Bidim A14, should be installed between the natural ground surface and the free draining granular material to prevent siltation and blockage of the drainage media; and
- A heavy-duty (TNZ F/2 Class 500) perforated pipe should be installed within the drainage material at the base of all retaining structures to minimise the risk of excessive groundwater pressures developing. This drainage pipe should be connected to the permanent piped stormwater system.

The safety implications of working under temporary cuts will need to be adequately addressed.

5.5 Groundwater Issues

The regional watertable is expected to lie well below the indicated finished floor levels. Groundwater-related construction issues and dewatering are therefore unlikely to be required. It is important that GeoSolve be contacted should there be any seepage, spring flow or under-runners encountered during construction. There is potential for under-runners in these soil types; if any voids are detected during construction then further engineering assessment should be carried out.

5.6 Slope Stability

No instability was identified during the time of inspection; however, any excavations should be inspected by a geotechnical practitioner to ensure that no shear surfaces or other unstable features are exposed.

The existing cut/fill slopes at the northern margins of the site are oversteepened and may become unstable if they become saturated or are subjected to seismic loading. These batters should be re-formed in accordance with the recommendations in Table 5.1 above, or retention should be provided.

Owing to the relatively steep slopes and evidence of landslide activity on adjacent hillsides, care will be required to ensure that the development does not promote slope instability. Placement of uncontrolled side-cast fill should be avoided on the slopes and adequate setbacks should be defined for structures and areas of surcharge adjacent to moderate or steep slopes. All sources of slope saturation should be eliminated by cutoff drains upslope of the cuts and no stormwater or wastewater should be discharged to these slopes.

All cuts should be subject to inspection during construction and if higher than 3 metres should be subject to specific design.

5.7 Provision of Services

It is expected that services will be conveyed to or from each lot via subsurface cabling or piping, with most of these services emplaced beneath the accessway. Care will be required to ensure trenching and subsequent filling of these trenches do not promote slope instability. Where possible, services should be conveyed in separate parts of a single trench.

As the excavation of a trench on these high gradient slopes is likely provide a preferred conduit for groundwater, we recommend emplacement of a subsoil drain at the lowest point of the trench. A filter-wrapped, perforated drain pipe (e.g. 65 mm Novaflow) should be installed within free-draining gravels at the lowest point of the trench to minimise the risk of the trench backfill soils becoming saturated or unstable. This drainage pipe should be connected to a settling tank (mudtank) and discharged to a council-approved stormwater discharge point.

Trench backfill should be placed and compacted in accordance with industry best practice for services on slopes.

5.8 Conceptual Foundation Options

Preliminary Scala penetrometer testing indicates that the depth to 'Good Ground' (as defined by NZS 3604:2011) is likely to be between 600 mm and 1 m depth across much of the site. Depths to Good Ground in areas on the eastern or northern margins of the site (areas found to contain either uncontrolled fill or softer alluvium) were found to be deeper, typically beyond 1-1.5 m. Foundation design for any dwellings constructed on the proposed new lots will need to account for the moderate gradient and the depth to good ground.

Pole style dwellings formed of lightweight materials and supported on bored and cast piles are likely to be most cost-effective on the new lots, as this will minimise any earthworks and retention required. Piles will need to be embedded well into stiff soils.

A concrete slab-on-grade formed on cut (or cut and fill) platform is also likely to be appropriate in this setting, though earthworks will require careful planning to ensure adequate space for temporary cuts and permanent batters. This option, while likely to be more costly than bored piles owing to the earthworks and retention likely to be required, should provide more space within height plane envelopes permitted by the local District Plan.

Deep cuts with basement retaining walls are also likely to be feasible, subject to specific investigation and design.

It is recommended that detailed site specific geotechnical investigations for foundation design should be carried out on the individual lots by a suitably qualified and experienced geotechnical specialist once development requirements have been established, but prior to the building consent application stage.

5.9 Accessway

We understand the main access to the subdivision is proposed to extend along the eastern margin of the existing lots. Given the existing (overall) gradient from the northeastern margins of the site to the southeastern margins is approximately 20° (approximately 1 in 2.75, or 36%), an accessway aligned directly along the eastern margins of the site would be very steep and would probably need to be formed of concrete and specifically designed.

Depending on the location of the accessway, some earthworks and/or retention may be required. This should be assessed by a geotechnical practitioner once final plans become available in order to determine whether any retention is required.

We recommend that a surveyor should be engaged to determine the most appropriate alignment for the accessway. Cross-sections at critical locations should be provided by the surveyor showing cut and fill profiles. These cross-sections should be checked by a geotechnical practitioner to enable advice on any physical support requirements.

The accessway should be contoured appropriately to allow surface runoff to fall to a contour drain or equivalent in order to intercept any surface runoff, and provision should be made to divert any overflows from both the accessway and the kerb and channel in Kauri Street to an appropriate secondary flow path.

Under-drainage below the concrete surface may also be required.

Construction of the accessway should be carried out under the supervision of a geotechnical practitioner. Any seepage encountered will require appropriate drainage measures during the earthworks.

6 Neighbouring Structures/Hazards

6.1 Natural Hazards

A risk of seismic activity has been identified for the region as a whole and appropriate allowance should be made for seismic loading during detailed design of the proposed development, but there are no site-specific constraints.

Owing to the consistency and type of soil encountered and no occurrences of groundwater on site, the risk of liquefaction is expected to be very low.

A review of The Otago Regional Council's Natural Hazards Database and GeoSolve's archives did not find any records of mapped slope instability or landslide features in the vicinity of the subject site. The recommendations of this report should be followed in order to mitigate the risk associated with landslip and subterranean erosion.

Flood hazard has not been assessed in this study but is unlikely in this hillslope setting, provided that upslope flow paths are well controlled.

Provided the recommendations within this report are adhered to, development of the site is unlikely to accelerate, worsen, or result in material damage by erosion, falling debris, subsidence, slippage or inundation from any source, on the subject site or any other property.

7 Conclusions and Recommendations

- Apart from the thin layer of surficial topsoil, the site is underlain by various alluvial and colluvial deposits to a depth of at least 2 m below the surface of the slope.
- Bedrock was not encountered but is expected to lie at moderate depth below the site, likely comprising Dunedin Volcanic Group basalt bedrock which is expected to extend to significant depths.
- The subsurface materials should be relatively easy to excavate by conventional methods; however, boulders are likely to be encountered within the colluvium and alluvium.
- The unretained cut/fill slope at the northern margins of the site is oversteepened and may be become unstable if it becomes saturated or is subject to seismic loading. Once re-graded, this slope should not exceed the recommended batters in Table 5.1.
- Based on the nature of the soils encountered on site, a lack of common landslide indicators and the firm or better condition of the soils observed, we consider the future landslip hazard at the site to be very low, provided the recommendations in this report are adhered to.
- Care will be required to ensure that the development does not promote slope instability. Placement of uncontrolled side-cast fill should be avoided on the slopes and adequate setbacks should be defined for structures and areas of surcharge adjacent to moderate or steep slopes.
- Permanent, dry cut batters in stiff alluvium or colluvium should be formed at 1.5 : 1 (horizontal to vertical) if cuts are less than 3 m. Specific design will be required for higher cuts.
- All fill slopes less than 3 m in height should be constructed with a maximum batter of 2:1 (horizontal to vertical) or flatter, if well drained. Thicker or steeper fills will require specific engineering assessment and design.
- The subgrade of any proposed fills will need to be sub-horizontal (with benching of slopes as required) to ensure stability.
- Any retaining walls should be designed by a chartered professional engineer. Further specific subsurface investigations for wall design may be required depending on final development requirements.
- Due allowance should be made during the detailed design of all retaining walls for any additional loads upslope of the wall (i.e. surcharge due to backslope or vehicle loadings).
- All temporary slopes for retaining wall construction should be battered at 1:1 provided these are within soils stiff in consistency and under the supervision of a geotechnical practitioner. Adequate working space at the toe of cuts should be provided.

- A geotechnical practitioner should inspect all excavations and additionally any seepage, spring flow or under-runners that may be encountered during construction.
- Care will be required to ensure trenching and subsequent filling of trenches conveying services do not promote slope instability.
- A filter-cloth lined subsoil drain pipe should be emplaced at the lowest point of any service trench to minimise the risk of the trench backfill soils becoming saturated or unstable. This drainage pipe should be connected to a settling tank and discharged to a council-approved stormwater discharge point.
- All service trench backfill should be placed and compacted in accordance with industry best practice for services on slopes.
- Dwellings formed of lightweight materials and supported on bored piles are likely to be most cost-effective on the new lots, given foundation systems such as these tend to minimise any earthworks and retention required.
- A concrete slab-on-grade formed on cut (or cut and fill) platform or basement retention are also likely to be appropriate methods of construction in this setting, though earthworks will require careful planning to ensure adequate space for temporary cuts and permanent batters.
- It is recommended that detailed investigations for foundation design are carried out on the individual lots by a suitably qualified and experienced geotechnical specialist once development requirements have been established, but prior to the building consent application stage.
- Given the existing site gradient, an accessway aligned directly along the eastern margins of the site would be very steep and would most likely need to be formed of concrete. Specific engineering design is recommended.
- A surveyor should be engaged to determine the most appropriate alignment for the accessway. Cross-sections at critical locations should be provided by the surveyor showing cut and fill profiles, with these cross-sections checked by a geotechnical practitioner to enable advice on any physical support requirements.
- The accessway should be contoured appropriately to allow surface runoff to fall to a contour drain or equivalent in order to intercept any surface runoff, and provision should be made to divert any overflows from both the accessway and the kerb and channel in Kauri Street to an appropriate secondary flow path. Under-drainage below the concrete surface may also be required.

8 Applicability

This report has been prepared for the benefit of David Walsh with respect to the particular brief given to us and it may not be relied upon in other contexts or for any other purpose without our prior review and agreement.

It is important that we be contacted if there is any variation in subsoil conditions from those described in this report.

Report prepared by:

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Rob Stuff
Engineering Geologist

Reviewed for GeoSolve Ltd by:

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Mark Walrond
Senior Engineering Geologist

Authorised for GeoSolve Ltd by:

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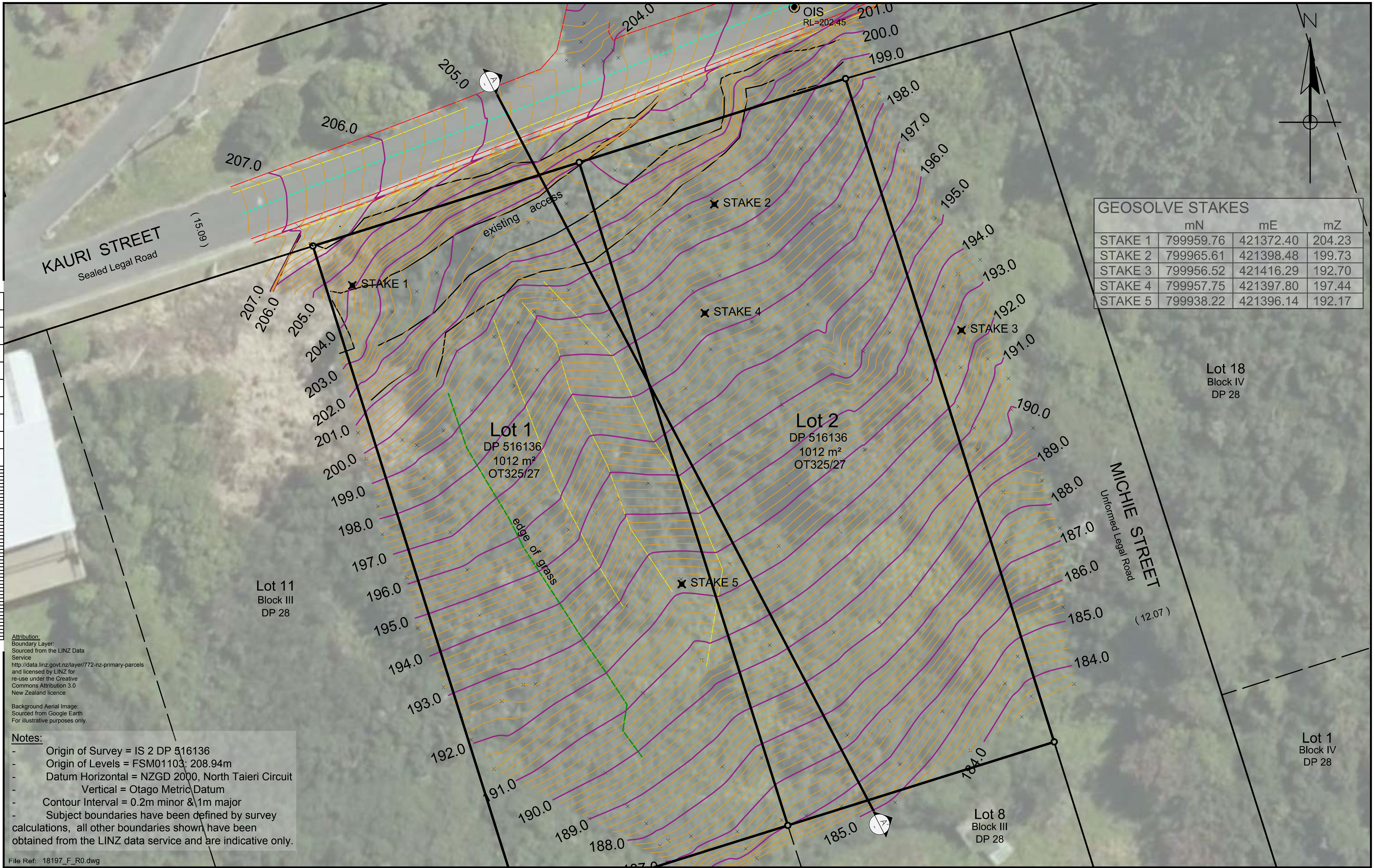
Colin Macdiarmid
Geotechnical Group Director

Appendix A: Site Plan & Cross-section



GEOSOLVE STAKES

	mN	mE	mZ
STAKE 1	799959.76	421372.40	204.23
STAKE 2	799965.61	421398.48	199.73
STAKE 3	799956.52	421416.29	192.70
STAKE 4	799957.75	421397.80	197.44
STAKE 5	799938.22	421396.14	192.17

Attribution:
Boundary Layer:
Sourced from the LINZ Data Service
http://data.linz.govt.nz/layer/772-nz-primary-parcels
and licensed by LINZ for re-use under the Creative Commons Attribution 3.0 New Zealand licence

Background Aerial Image:
Sourced from Google Earth
For illustrative purposes only.

Notes:
- Origin of Survey = IS 2 DP 516136
- Origin of Levels = FSM01103: 208.94m
- Datum Horizontal = NZGD 2000, North Taieri Circuit
- Vertical = Otago Metric Datum
- Contour Interval = 0.2m minor & 1m major
- Subject boundaries have been defined by survey calculations, all other boundaries shown have been obtained from the LINZ data service and are indicative only.

File Ref: 18197_F_R0.dwg

Prepared For:
David Walsh

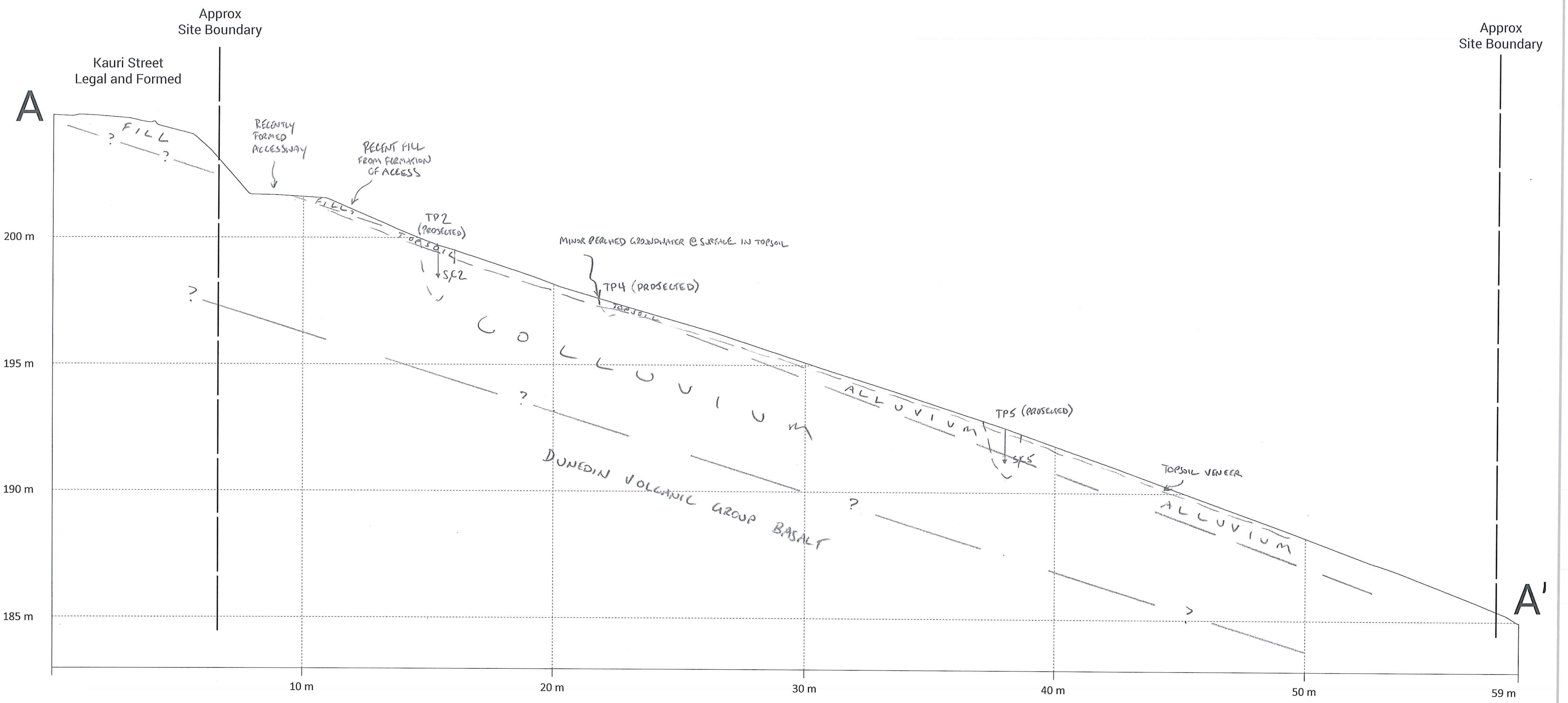
No.	Amendments	Drawn	Date
0	Original Issue	CH	30.10.18

60 Kauri Street - Topographical Survey
Ravensbourne, DUNEDIN

Site Contours & Geo-Stake Locations
Lots 1 & 2 DP 516136 - Record of Title OT325/27

Project No.: 18-197	Surveyed: CH
Scale: 1:500	Designed: CH
Date: 30-10-2018	Drawn: CH
Sheet: 1	Checked: BWS

Survey Services
Surveying Consultants
TL Survey Services Limited
P.O. Box 901 DUNEDIN
Phone (03) 477 1133



Key

CADFILE:	Cross-section A-A'.ai	DRAWN	RS	11/2018
SCALE (AT A3 SIZE):	1 : 150	DRAFTING CHECKED	RS	11/2018
PROJECT No:	180239	APPROVED	MTW	11/2018

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GEOTECHNICAL
WATER RESOURCES
PAVEMENTS

David Walsh
Proposed Subdivision
60 Kauri Street, Ravensbourne, Dunedin
Cross-section A-A'

FIG No: FIGURE 2

REV. 1

Appendix B: Investigation Data

EXCAVATION LOG

EXCAVATION NUMBER:
TP 1

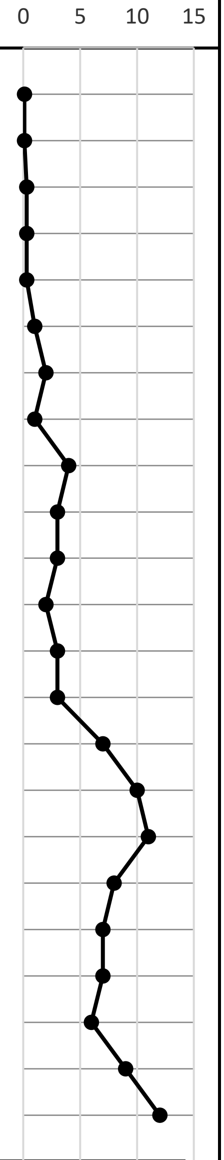
PROJECT:	KAURI60	JOB NUMBER:	180239
LOCATION:	See Site Plan - Stake 1 INCLINATION: Vertical		

EASTING:		mE	EQUIPMENT:	7T Excavator	OPERATOR:	Matt
NORTHING:		mN	INFOMAP NO.		COMPANY:	Craig Myhill Contracting
ELEVATION:		m	DIMENSIONS:		HOLE STARTED:	4-Oct-18
METHOD:			EXCAV. DATUM:		HOLE FINISHED:	4-Oct-18

DEPTH (m)	SOIL / ROCK TYPE	GRAPHIC LOG	DESCRIPTION	USCS GROUP	GROUNDWATER / SEEPAGE
0.2	TOPSOIL		Dark brown, organic SILT. Roots and rootlets. Very soft. Moist.		
0.4	FILL		Light brown, silty CLAY. Frequent rootlets. Low plasticity. Very soft. Moist.		
0.5	BURIED TOPSOIL		Brown, organic clayey SILT. Low plasticity. Very soft. Moist.		
0.8	COLLUVIUM		Mottled light brown and orange, silty CLAY with trace of boulders. Occasional rootlets, boulders and cobbles to 0.3 m diameter, subrounded to rounded. Low plasticity. Soft. Moist.		
2.4	COLLUVIUM		Mottled light brown and orange, silty CLAY with trace of boulders. Occasional rootlets, boulders and cobbles to 0.3 m diameter, subrounded to rounded. Low plasticity. Firm to stiff. Moist.		NO SEEPAGE

SCALA PENETROMETER

Blows per 100mm



Total Depth = 2.4 m

COMMENT: Unable to penetrate further, scraping on boulders.	Logged By: RS
	Checked Date: 9/10/2018
	Sheet: 1 of 1

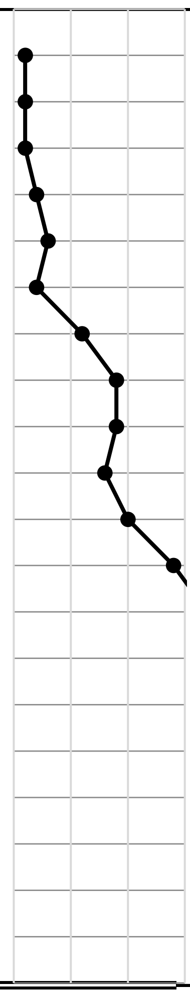
EXCAVATION LOG

EXCAVATION NUMBER: **TP 2**

PROJECT:	KAURI60	JOB NUMBER:	180239
LOCATION:	See Site Plan - Stake 2 INCLINATION: Vertical		

EASTING:		mE	EQUIPMENT:	7T Excavator	OPERATOR:	Matt
NORTHING:		mN	INFOMAP NO.		COMPANY:	Craig Myhill Contracting
ELEVATION:		m	DIMENSIONS:		HOLE STARTED:	4-Oct-18
METHOD:			EXCAV. DATUM:		HOLE FINISHED:	4-Oct-18

DEPTH (m)	SOIL / ROCK TYPE	GRAPHIC LOG	DESCRIPTION	USCS GROUP	GROUNDWATER / SEEPAGE	SCALA PENETROMETER Blows per 100mm 0 5 10 15
0.3	TOPSOIL		Dark brown, organic SILT. Roots and rootlets. Soft to firm. Moist.			
2.1	COLLUVIUM		Mottled light brown and orange, silty CLAY with trace of boulders. Occasional rootlets, boulders and cobbles to 0.3 m diameter, subrounded to rounded. Low plasticity. Firm to very stiff. Moist.		NO SEEPAGE	



Total Depth = 2.1 m

COMMENT: Unable to penetrate further, scraping on boulders.	Logged By: RS
	Checked Date: 9/10/2018
	Sheet: 1 of 1

EXCAVATION LOG

EXCAVATION NUMBER: **TP 3**

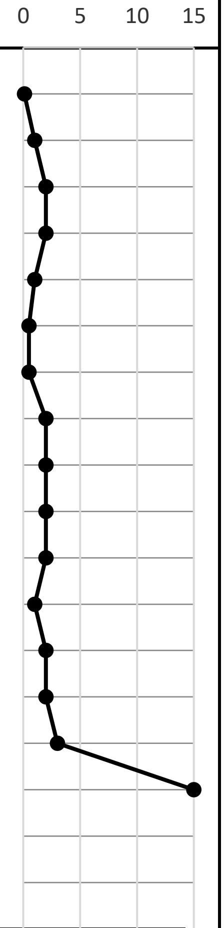
PROJECT:	KAURI60	JOB NUMBER:	180239
LOCATION:	See Site Plan - Stake 3 INCLINATION: Vertical		

EASTING:		mE	EQUIPMENT:	7T Excavator	OPERATOR:	Matt
NORTHING:		mN	INFOMAP NO.		COMPANY:	Craig Myhill Contracting
ELEVATION:		m	DIMENSIONS:		HOLE STARTED:	4-Oct-18
METHOD:			EXCAV. DATUM:		HOLE FINISHED:	4-Oct-18

DEPTH (m)	SOIL / ROCK TYPE	GRAPHIC LOG	DESCRIPTION	USCS GROUP	GROUNDWATER / SEEPAGE
0.3	TOPSOIL		Dark brown, organic SILT. Roots and rootlets. Very soft. Saturated at surface, becoming moist.		
1.3	ALLUVIUM		Light brown, silty CLAY with trace of sand. Sand is firm to medium, subrounded. Low plasticity. Soft. Moist.		
1.9	ALLUVIUM		Light brown and orangey brown, silty CLAY with minor sand, gravel and boulders. Sand is fine to coarse, subangular to subrounded. Gravel is fine, subangular to subrounded. Frequent boulders to 0.5 m diameter. Low plasticity. Soft to firm. Moist to wet with isolated zones of saturation.		NO SEEPAGE

SCALA PENETROMETER

Blows per 100mm



Total Depth = 1.9 m

COMMENT: Unable to penetrate further, scraping on boulders.	Logged By: RS
	Checked Date: 9/10/2018
	Sheet: 1 of 1

EXCAVATION LOG

EXCAVATION NUMBER:
TP 4

PROJECT:	KAURI60	JOB NUMBER:	180239
LOCATION:	See Site Plan - Stake 4 INCLINATION: Vertical		

EASTING:		mE	EQUIPMENT:	7T Excavator	OPERATOR:	Matt
NORTHING:		mN	INFOMAP NO.		COMPANY:	Craig Myhill Contracting
ELEVATION:		m	DIMENSIONS:		HOLE STARTED:	4-Oct-18
METHOD:			EXCAV. DATUM:		HOLE FINISHED:	4-Oct-18

DEPTH (m)	SOIL / ROCK TYPE	GRAPHIC LOG	DESCRIPTION	USCS GROUP	GROUNDWATER / SEEPAGE	SCALA PENETROMETER
0.2	TOPSOIL		Dark brown, organic SILT. Roots and rootlets. Very soft. Saturated at surface, becoming moist.		NO SEEPAGE	
0.5	COLLUVIUM		Mottled light brown and orange, silty CLAY with trace of boulders. Occasional rootlets, boulders and cobbles to 0.3 m diameter, subrounded to rounded. Low plasticity. Soft. Moist.			

Total Depth = 0.5 m

COMMENT: Target depth reached, no springs at depth. Patches of standing water at surface.	Logged By: RS
	Checked Date: 9/10/2018
	Sheet: 1 of 1

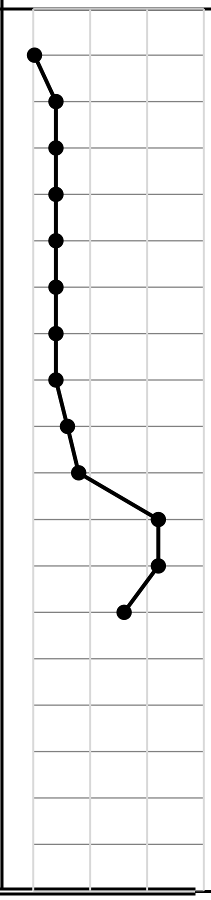
EXCAVATION LOG

EXCAVATION NUMBER:
TP 5

PROJECT:	KAURI60	JOB NUMBER:	180239
LOCATION:	See Site Plan - Stake 5 INCLINATION: Vertical		

EASTING:		mE	EQUIPMENT:	7T Excavator	OPERATOR:	Matt
NORTHING:		mN	INFOMAP NO.		COMPANY:	Craig Myhill Contracting
ELEVATION:		m	DIMENSIONS:		HOLE STARTED:	4-Oct-18
METHOD:			EXCAV. DATUM:		HOLE FINISHED:	4-Oct-18

DEPTH (m)	SOIL / ROCK TYPE	GRAPHIC LOG	DESCRIPTION	USCS GROUP	GROUNDWATER / SEEPAGE	SCALA PENETROMETER Blows per 100mm (0 5 10 15)
0.2	TOPSOIL		Dark brown, organic SILT. Roots and rootlets. Soft. Moist.			
1.1	ALLUVIUM		Light brown, silty CLAY with trace of sand and gravel. Sand is medium to coarse. Gravel is fine to medium, subangular to subrounded. Low plasticity. Soft to firm. Moist.			
1.9	COLLUVIUM		Light brown and orangey brown, silty CLAY with some cobbles and boulders, trace of sand and gravel. Sand is medium to coarse. Gravel is fine to medium, subangular to subrounded. Boulders to 0.6 m diameter, subrounded, increasingly frequent. Low plasticity. Firm. Moist, isolated saturated patches.		Seepage @ 1.3 m	



Total Depth = 1.9 m

COMMENT: Unable to penetrate further, scraping on boulders. Groundwater seepage ~0.05 L/min.	Logged By: RS
	Checked Date: 9/10/2018
	Sheet: 1 of 1

GEOSOLVE

GEOSOLVE LTD

SCALA PENETROMETER LOG

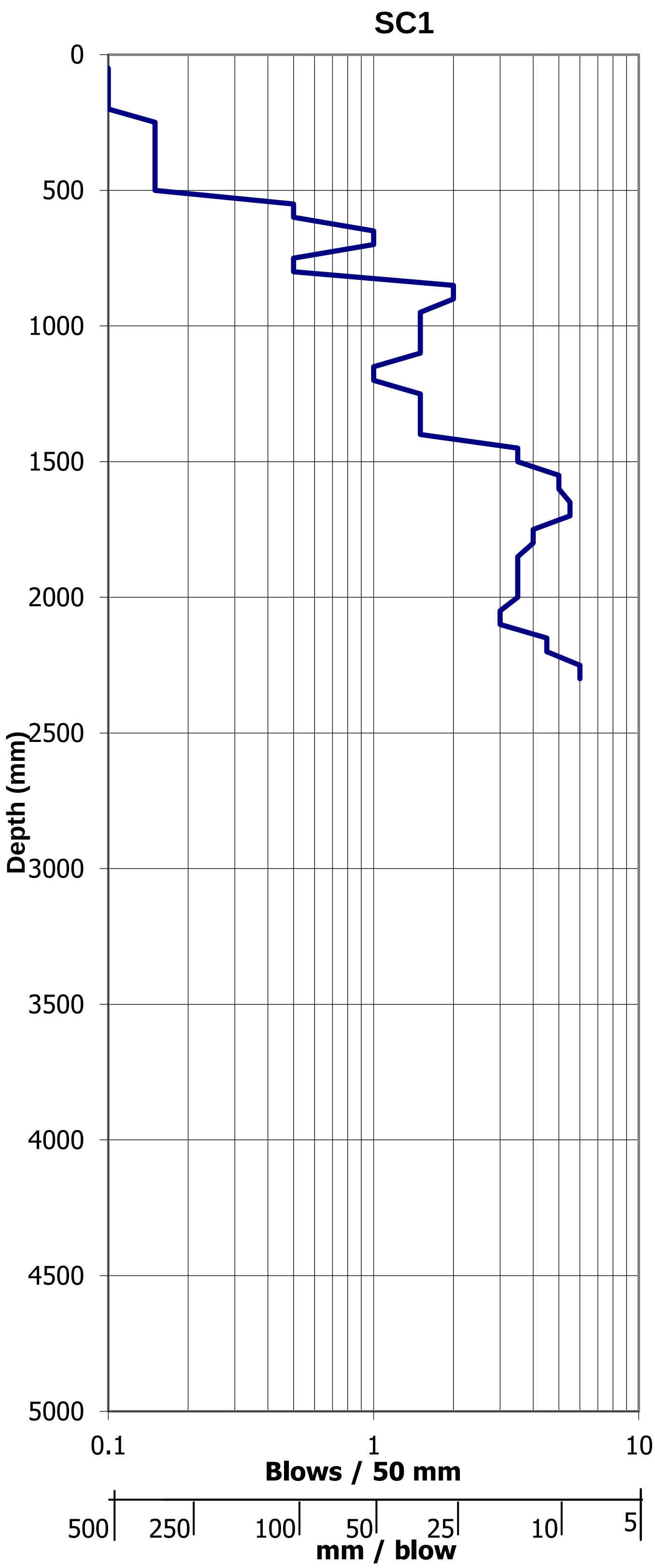
Job No: 180239
Project: KAURI60
Location: TP1
RL: GL

Date: 4/10/2018
Operated by: RS
Logged by: RS
Inferred Soil Type:

Test No. SC1

Sheet 1 **of** 1

SC1 mm Driven	SC1 No. of Blows	SC1 cont... mm Driven	SC1 cont... No. of Blows
50	0.1	2550	
100	0.1	2600	
150	0.1	2650	
200	0.1	2700	
250	0.15	2750	
300	0.15	2800	
350	0.15	2850	
400	0.15	2900	
450	0.15	2950	
500	0.15	3000	
550	0.5	3050	
600	0.5	3100	
650	1	3150	
700	1	3200	
750	0.5	3250	
800	0.5	3300	
850	2	3350	
900	2	3400	
950	1.5	3450	
1000	1.5	3500	
1050	1.5	3550	
1100	1.5	3600	
1150	1	3650	
1200	1	3700	
1250	1.5	3750	
1300	1.5	3800	
1350	1.5	3850	
1400	1.5	3900	
1450	3.5	3950	
1500	3.5	4000	
1550	5	4050	
1600	5	4100	
1650	5.5	4150	
1700	5.5	4200	
1750	4	4250	
1800	4	4300	
1850	3.5	4350	
1900	3.5	4400	
1950	3.5	4450	
2000	3.5	4500	
2050	3	4550	
2100	3	4600	
2150	4.5	4650	
2200	4.5	4700	
2250	6	4750	
2300	6	4800	
2350		4850	
2400		4900	
2450		4950	
2500		5000	



Test Method Used: NZS 4402:1988 Test 6.5.2 Dynamic Cone Penetrometer

GeoSolve Ltd

SCALA PENETROMETER LOG

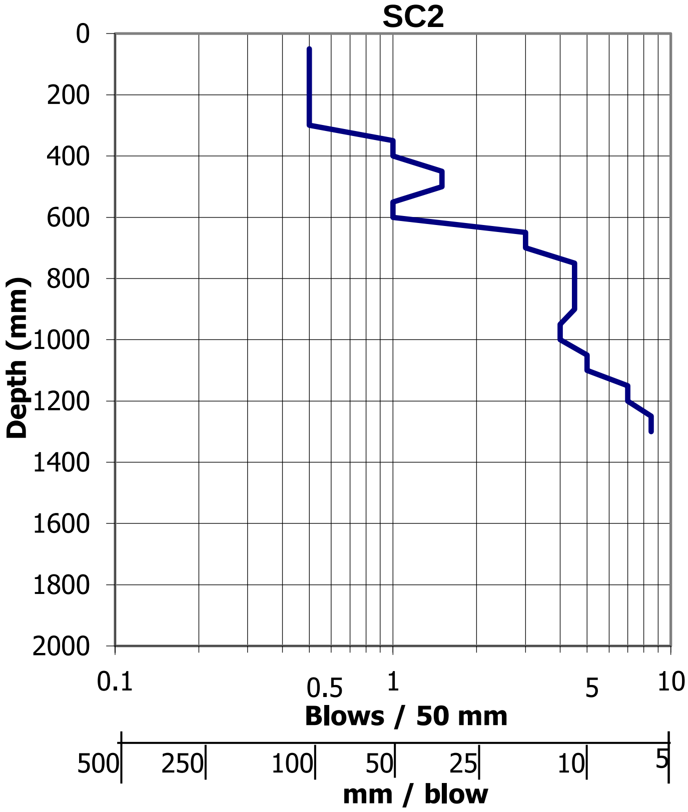
Job No: *180239*
Project: *KAURI60*

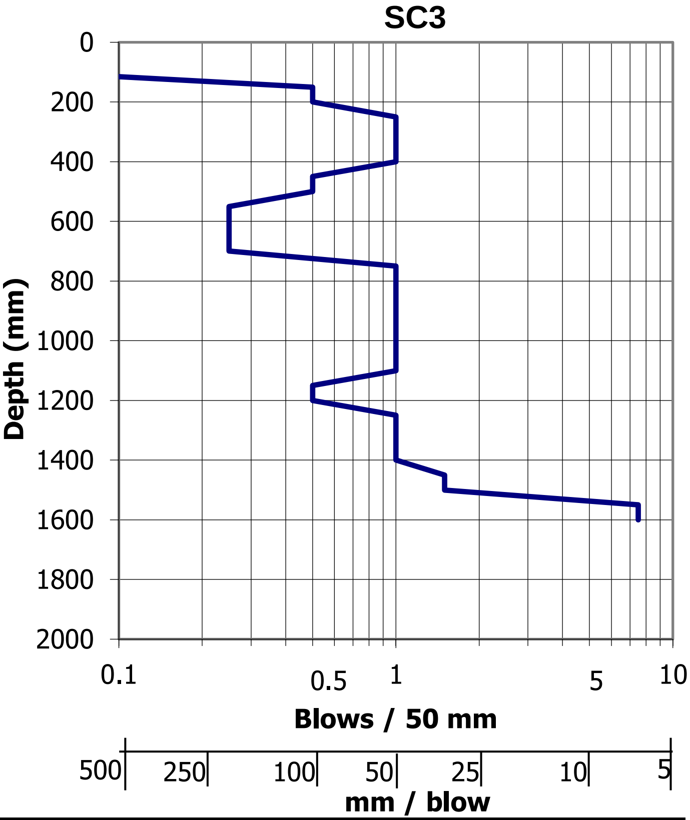
Date: *4/10/2018*
Operated by: *RS*
Logged by: *RS*

Test Number *SC2 & SC3*
Sheet *1* **of** *1*

SC2 Location: TP2 RL: GL	
mm Driven	No. of Blows
50	0.5
100	0.5
150	0.5
200	0.5
250	0.5
300	0.5
350	1
400	1
450	1.5
500	1.5
550	1
600	1
650	3
700	3
750	4.5
800	4.5
850	4.5
900	4.5
950	4
1000	4
1050	5
1100	5
1150	7
1200	7
1250	8.5
1300	8.5
1350	
1400	
1450	
1500	
1550	
1600	
1650	
1700	
1750	
1800	
1850	
1900	
1950	
2000	
Inferred Soil Type	
Watertable Depth	

SC3 Location: TP3 RL: GL	
mm Driven	No. of Blows
50	0.05
100	0.05
150	0.5
200	0.5
250	1
300	1
350	1
400	1
450	0.5
500	0.5
550	0.25
600	0.25
650	0.25
700	0.25
750	1
800	1
850	1
900	1
950	1
1000	1
1050	1
1100	1
1150	0.5
1200	0.5
1250	1
1300	1
1350	1
1400	1
1450	1.5
1500	1.5
1550	7.5
1600	7.5
1650	
1700	
1750	
1800	
1850	
1900	
1950	
2000	
Inferred Soil Type	
Watertable Depth	





GEOSOLVE

GeoSolve Ltd

SCALA PENETROMETER LOG

Job No: *180239*
Project: *KAURI60*

Date: *4/10/2018*
Operated by: *RS*
Logged by: *RS*

Test Number	*SC5*
Sheet	*1*
of	*1*

SC5 Location: TP5 RL: GL	
mm Driven	No. of Blows
50	*0.1*
100	*0.1*
150	*1*
200	*1*
250	*1*
300	*1*
350	*1*
400	*1*
450	*1*
500	*1*
550	*1*
600	*1*
650	*1*
700	*1*
750	*1*
800	*1*
850	*1.5*
900	*1.5*
950	*2*
1000	*2*
1050	*5.5*
1100	*5.5*
1150	*5.5*
1200	*5.5*
1250	*4*
1300	*4*
1350	
1400	
1450	
1500	
1550	
1600	
1650	
1700	
1750	
1800	
1850	
1900	
1950	
2000	
Inferred Soil Type	
Watertable Depth	

Location: RL:	
mm Driven	No. of Blows
50	
100	
150	
200	
250	
300	
350	
400	
450	
500	
550	
600	
650	
700	
750	
800	
850	
900	
950	
1000	
1050	
1100	
1150	
1200	
1250	
1300	
1350	
1400	
1450	
1500	
1550	
1600	
1650	
1700	
1750	
1800	
1850	
1900	
1950	
2000	
Inferred Soil Type	
Watertable Depth