BEFORE THE WAIKATO DISTRICT COUNCIL INDEPENDENT HEARING PANEL

IN THE MATTER	of Proposed Variation 3, under clause 16A of Schedule 1 of the Resource Management Act 1991, to the Proposed District Plan Change
AND	
IN THE MATTER	of submissions by Greig Developments No 2 Limited and Harrisville Twenty Three Limited, Tuakau.

To: **The Hearings Co-ordinator**
Waikato District Council

PRIMARY GEOTECHNICAL EVIDENCE OF ROBERT TILSLEY FOR HARRISVILLE TWENTY THREE LTD

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4 July 2023

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MAY IT PLEASE THE PANEL

1. Introduction

1.1 My full name is Robert Tilsley.

1.2 I am a Chartered Professional Engineer in Pukekohe. I hold a MNZM, BE Civil, CPEng and MIPENZ.

1.3 I am a member of Engineering New Zealand.

1.4 My relevant professional experience spans over 60 years in both the private and public sectors in New Zealand and overseas. My career started at the engineering services department of the Auckland City Council. I have held roles in local government as an Assistant County Engineer, Borough Engineer, Chief Executive, and an elected member of the Auckland Regional Council. I was an inaugural Board Member of Watercare. I have run my own Engineering Practice in Pukekohe since 1984.

1.5 Working in the urban and rural environment of Franklin and Waikato over the last 60 years, I have had a continuous association with both residential and rural activities and have a thorough understanding of issues within both environments and its interface. Most recently within the private sector, I have worked for a range of clients to obtain resource consents for large scale residential subdivisions and other development projects with my geotechnical expertise.

1.6 I confirm that I have read the 'Expert Witnesses Code of Conduct' contained in the Environment Court of New Zealand Practice Note 2023. My evidence has been prepared in compliance with that Code in the same way as I would if giving evidence in the Environment Court. In particular, unless I state otherwise, this evidence is within my sphere of expertise, and I have not omitted to consider material facts known to me that might alter or detract from the opinions I express.

2. Scope of Evidence

2.1 This evidence is provided in support of the submission made by Greig Developments and Harrisville Twenty-Three Limited on Variation 3 of the Proposed Waikato District Plan (**PWDP**). My evidence specifically addresses rezoning of land at 23A Harrisville Road to a to Medium Density Residential 2 Zone (**MDRZ 2**) through Variation 3 from a geotechnical perspective.

2.2 My evidence addresses whether rezoning the subject land to increase the density to incorporate MDRZ 2 zoning can be supported from a geotechnical perspective.

2.3 Previous specialist reporting has been prepared specifically in relation to a proposed subdivision consent at 23 and 23A Harrisville Road, Tuakau for the development of the site to create fourteen (14) residential lots. The specialist report that was previously prepared is enclosed as **Appendix A** and is titled **Preliminary Geotechnical Investigation for Proposed Residential Subdivision**, Job No: PG 16277/01, dated 5th July 2021.

2.4 This report is highly relevant to this evidence for rezoning the subject land with further statements included below based on the Concept Plan of Lots 9 & 10 DP 136581 - # 23 & 23A Harrisville Road Tuakau, dated June 2023, ref: J1257 Concept Plan 3-A – as attached to the Planning Evidence of Ms Addy.

2.5 The lot yield change based on a zone change would increase from seven (7) lots (current Rural-Residential / Large Lot Zoning) to approximately twenty-five (25) developable platforms (MDRS 2 zoning), being 18 additional lots/developable platforms. This is based on ultimate lot sizes of 350m^2 - 450m^2 as well as some larger lots and only utilising areas of land that do not present unfavourable contours.

2.6 It is noted that the difference in lot yield between the current proposed subdivision consent layout producing fourteen (14) lots and the potential yield under MDRS 2 zoning potentially generating twenty-five (25) developable platforms is eleven (11) additional lots/developable platforms.

2.7 In an addendum, to my original assessment for the fourteen (14) lot subdivision, and for the purpose of assessing the Harrisville submission relief on Variation 3, I can support the additional lots generated by a rezoning on the subject site as:

a) The additional lots are being located on land that has previously been assessed by us as capable of providing stable building platforms for lightweight timber frame structures with weatherboard or brick cladding as per NZS 3604:2011. Lot 19 is subject to further investigation and will require extensive foundation investigation and design in its current form. The structures are anticipated to be founded on reinforced concrete floor slabs or raised timber floors upon shallow pile foundations.

b) As per point 5 of the recommendations in TEL report dated 5 July 2021 reference PG 16277/01 it is recommended that a site-specific foundation assessment shall be undertaken for each house site as part of building consent.

3. Conclusion

3.1 Rezoning of the subject site to MDRZ 2 zone will increase the ability to generate a higher density of residential development. I can support the increase in density as per section 2.6.

Robert Tilsley

4 July 2023

APPENDIX A – PRELIMINARY GEOTECHNICAL INVESTIGATION

TILSLEY ENGINEERING LTD
CONSULTING ENGINEERS AND ENVIRONMENTAL CONSULTANTS

23 & 23A Harrisville Road, Tuakau

Preliminary Geotechnical Investigation for Proposed Residential Subdivision

Site Land Developments Limited
Job No: PG 16277/01
05 July 2021

23 & 23A Harrisville Road, Tuakau

Preliminary Geotechnical Investigation for Proposed Residential Subdivision

Client: Site Land Developments Limited

Prepared by

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05 July 2021

Job No: PG 16277/01

Quality Information

Rev	Revision Date	Details	Authorised	
			Prepared by	Reviewed by
01	05-July-2021	Issued to Client	Phil Guo (B.E. (Geotechnical), MEngSt(Civil) MEngNZ) Geotechnical Engineer	R Tilsley (CMEngNZ,CPEng) Director
			Phil Guo.	R Tilsley

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1.0 INTRODUCTION

1.1 Project Overview

Tilsley Engineering Ltd has been engaged by the client to provide a preliminary geotechnical investigation report for obtaining a subdivision and resource consent with the Council. The geotechnical assessment is undertaken at 23 & 23A Harrisville Road, Tuakau. The scope of this report encompasses geotechnical suitability, land stability and provides the foundation and site development recommendations in support of the subdivision scheme design and subdivision consent applications to the Council. The report provides preliminary geotechnical information and guidance for interested parties such as the builder, structural engineer, earthworks, and civil contractors. A site-specific geotechnical investigation report is required for each Lot at the building consent application stage. This report includes a summary of the investigations undertaken and provides an assessment of:

- Ground conditions.
- Groundwater conditions.
- Liquefaction
- Ground stability.
- Foundation.
- Other constraints and issues identified with the site.

The site location is shown in Figure 1 below.

Figure 1 Site Location

2.0 SITE INFORMATION

2.1 Site Description

The proposed subdivision project site is a part of the properties which are described legally as Lot 9 (26,170m^2 in area) & Lot 10 (851m^2 in area) DP 136581, as per Waikato District Council Map. The site is located on the north-western side of Harrisville Road. The site details and extent of the project are presented in Figure 2. Site photographs are presented in Appendix A.

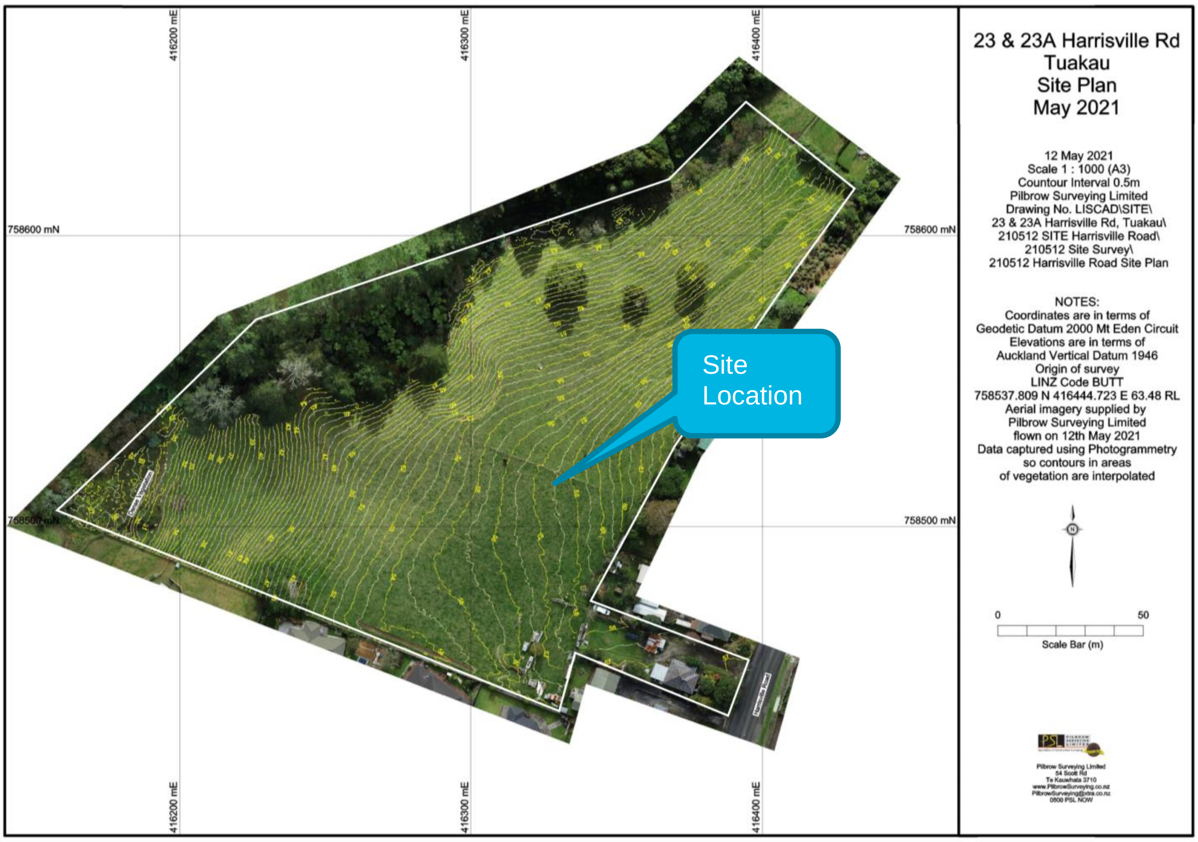

Figure 2 Extent of the Project

2.2 Proposed Site Development

The property is proposed to be subdivided into 14 new Lots. Tilsley Engineering only undertakes the preliminary geotechnical assessment for the site. The Lots will be accessed from proposed internal roads and access ways which are formed centrally within the site.

3.0 DESKTOP STUDY

- We are unaware of any previous geotechnical investigations which have been undertaken on the proposed building sites.

- Preliminary draft conceptual subdivision plan drawings have been prepared by The Surveying Company in June 2021 (Ref #: J1257).
- The New Zealand Geotechnical Database (NZGD) has been viewed and no geotechnical investigations have been identified at the proposed building sites.
- Aerial photographs available from Google Earth and Council GeoMaps GIS database dating from 2001 to 2020 were studied to observe the site over time and assess the geomorphological setting. Based on the review of historical aerial images and the site visit, there have not been any ground movements or soil instabilities at the location of the building sites. (Slip has been there since before 2001).

3.1 Published Geology

From the geological map of Auckland (Institute of Geological & Nuclear Sciences Ltd, 1:250000) in Figure 3 together with our experience of the surrounding areas, we infer the soils of the area are identified as ‘Qvs’ undifferentiated Kerikeri Volcanic Group basalt lava of South Auckland Volcanic Field. These are basaltic soils formed from scoria, lapilli, and ash originating from local volcanic events, which took place some 1 – 2 million years ago. The weathered surface soils are relatively free draining especially in the top 700 mm depth. The soils below 900 mm become more dense clays and are less well-draining.

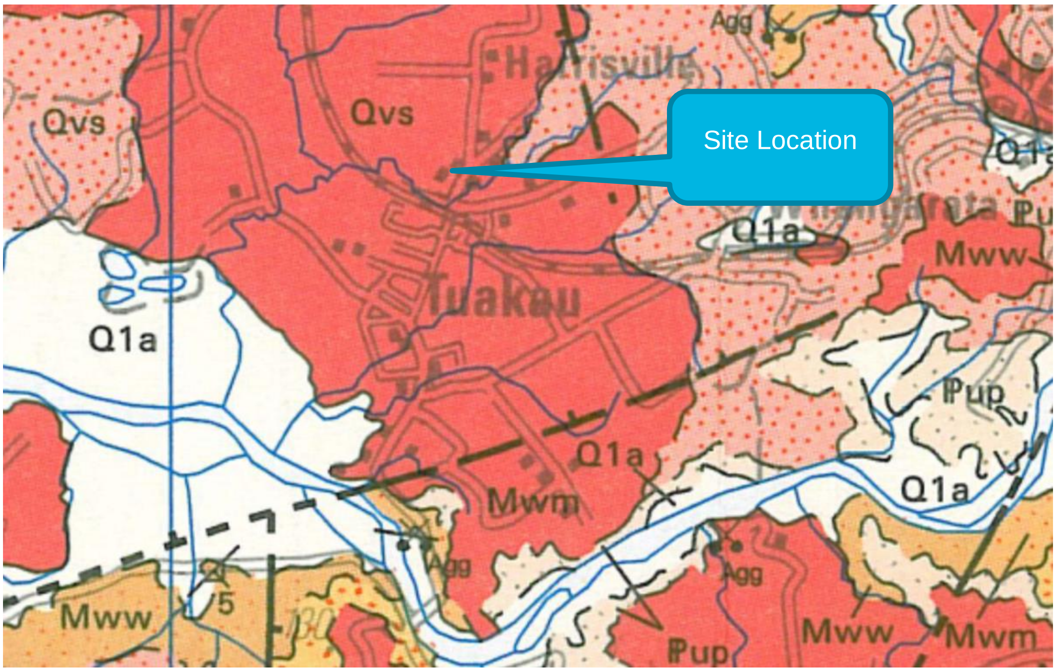

Figure 3 Extract from the Published GNS Geological Map

3.2 Faults

A review of the Institute of Geological and Nuclear Sciences Ltd (GNS) Active Fault Database (GNS Science, 2018) shows that no active faults occur within the immediate area of the site, although blind or unmapped faults may be present.

4.0 GEOTECHNICAL INVESTIGATION

4.1 Field Investigation

On the 25th May 2021, Tilsley Engineering Ltd conducted investigations for the project area, which consisted of the following:

- A detailed walkover inspection of the site.
- Drilling of 5 boreholes BH01 to BH05 with a spiral driller attachment using a digger, and drilling of 4 hand auger holes HA01 to HA04 within the proposed twelve Lots. **BH04 was drilled at the head of the slope failure due to the complex topography. This only gives the preliminary information of the residual soils in this Lot.**
- The locations of all field tests were measured by tape from existing site features and inferred boundaries without survey control and are therefore approximate only. Test locations are shown on the attached Site Plan – Appendix A.

Measurements of the undrained shear strengths were undertaken in the auger holes at intervals of depth by means of a handheld shear vane.

Visual-tactile field classification of the soils encountered during drilling was carried out in accordance with “Guidelines for the Field Classification and Description of Soil and Rock for Engineering Purposes”, issued by the New Zealand Geotechnical Society Inc. (2005). The test results are given on the attached test sheets.

4.2 Site Subsurface Conditions

Subsurface conditions encountered at the test locations are summarised below and a detailed description of the soils encountered during the investigations is given on the attached investigation logs. Conclusions and recommendations contained in this report are based on the results of our field investigation and in-situ testing within auger holes at point locations and information from geological maps. The nature and continuity of the soil conditions away from the test locations are inferred, however actual soil conditions could vary from the assumed model. This is particularly so where previous manmade disturbances and placement of non-engineered fill may have occurred in the past, typically associated with landscaping and/or previous construction activities. The subsurface conditions and groundwater underlying the site are summarised in Table 1 and Table 2 below:

Table 1 Summary of Hand-held Vane Shear Strength

Test Location	Termination Depth (m)	Depth of Topsoil (m)	Peak Shear Strength Range (kPa) Residual Soil	Residual Shear Strength Range (kPa) Residual Soil	Depth to Groundwater (m)
BH01	5.0	0.30	82-170	46-79	NE
BH02	5.0	0.30	73->200	42-74	NE
BH03	5.0	0.30	73->200	37-104	NE
BH04	5.0	0.45	102->200	34-96	NE
BH05	5.0	0.45	140->200	31-66	NE
HA01	2.0	0.25	141->200	31-68	NE
HA02	2.0	0.20	167->200	80-96	NE
HA03	2.0	0.20	155->200	57	NE
HA04	1.3	0.30	155->200	88	NE

All depths measured in meters below present ground level. NE = Not Encountered
* Fill not penetrated – obstruction encountered

Table 2 Recommended Geotechnical Units Strength

Unit	Material	Peak shear strength (kPa) Typical (min-max)	Residual shear strength (kPa) Typical (min-max)	Soil sensitivity
1	Topsoil	-	-	-
2	Residual Soils	100	30	3.33

Topsoil. Topsoil over the site generally is to depths of around 0.20m -0.45m below the existing ground level. It is described as non-engineered stiff to very stiff, clayey SILT to silty CLAY.

Fill. No non-engineered fill was encountered at the locations of the hand auger holes to depth between 0.30m to 5.0m.

Residual soil deposit.

- Weathered volcanic ash soil underlies the subdivision to a depth of at least 4.0m. The weathered volcanic ash soil deposits typically comprised clayey SILT to silty CLAY. Weaker layers were encountered at depths of around 3m to 4m below the existing ground level at the locations of BH01, BH02, and BH03. These weaker layers were identified along or close to the base of the subdivision gullies with a measured shear strength of between 70kPa and 80kPa. The weaker layers are typically associated with saturated soil conditions.

Scala Penetrometer Testing. No Scala penetrometer testing was carried out in the base of auger holes.

Groundwater. Groundwater was not encountered on site but is expected to be at a depth of approximately 7m. The groundwater table measured is considered to represent winter conditions.

4.3 Geomorphology

The subject site has an irregular shape. According to the topographical, site concept plan, and council contour map, the site comprised a grazed pastoral platform that falls gently to moderately to the northwest over the majority of the site. A deeply incised gully runs through northern boundary of the site with associated steep-sided slopes ranging from 18 to 30 degrees to the horizontal and extending over a vertical height of 6m to 21m. A major landslide scarp is located near the northern boundary of the site within the central part of the Lot. This comprises steep and hummocky ground topography inclined at gradients of up to 40 degrees to the horizontal and extending over a vertical height of up to 8m. The site location with contours from Waikato Regional Council maps is shown in Figure 4 and the drone contour plan can be found in Appendix A.

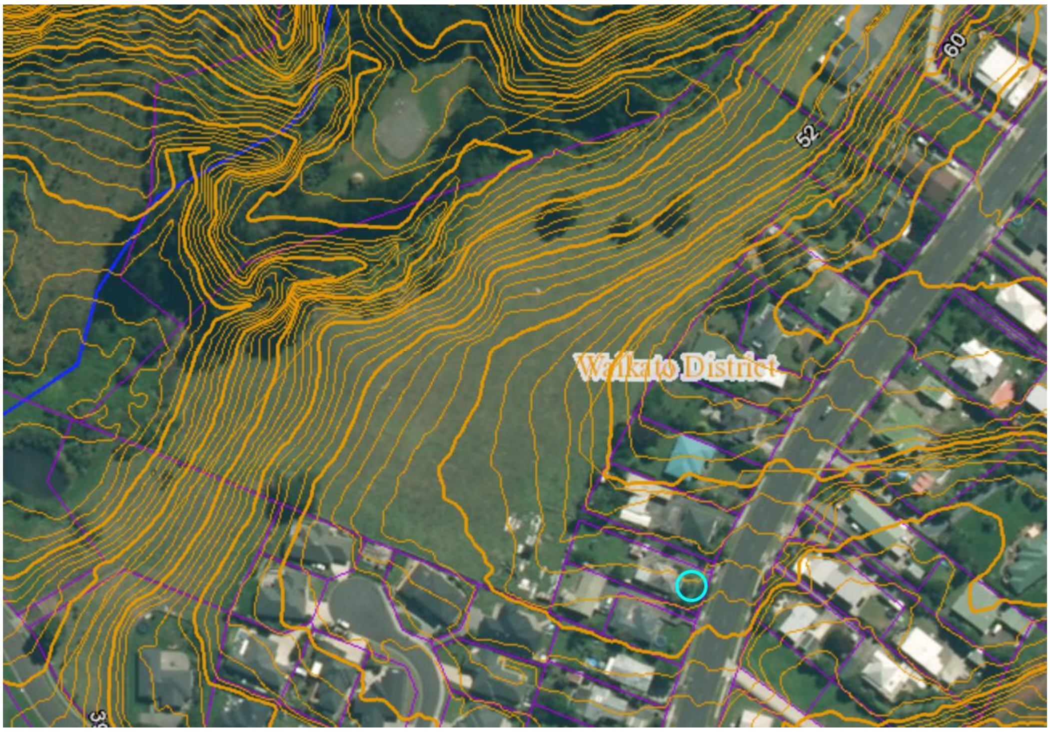

Figure 4 Project Location with Contours

4.3.1 Site Surface Water Features

There is an overland flow path along the northern boundary of the site and a permanent stream running along a gully further north as shown in Figure 4.

4.3.2 Flooding

The risk of flooding is low due to the elevation of the site.

4.4 Geotechnical Design Parameters

The selection of geotechnical strength parameters for use in building foundation design is based on the field investigation data, published correlations, and our experience with such materials. The recommended parameters for design are set out in Table 3.

Table 3 Recommended Geotechnical Design Parameters

Unit	Depth (m)	Material	Bulk density, ɣ (kN/m3)	Design undrained shear strength, Su (kPa)	Drained effective cohesion, c' (kPa)	Drained effective friction, ø' (deg.)
2	0.3-3	Residual Soils	18	70	3	28
2	3-4	Residual Soils (weak layer)	17	50	1	25
3	4-5	Transitional Soils	18	70	5	35

Notes: The lower bound values have been conservatively used for the design parameters, but can be further refined for specific structures with geotechnical data in the vicinity

5.0 ENGINEERING CONSIDERATIONS

5.1 General

The recommended parameters for the geotechnical aspects of the proposed development have been considered principally with the aim of demonstrating that safe and stable conditions for the proposed building site are presently available or are achievable with appropriate remedial works/constraints. This has been considered with respect to the following information, standards, guidelines, and codes:

- New Zealand Building Code: Clauses B1, E1, G12 & G13.
- MBIE Guidelines Modules " Earthquake geotechnical engineering practice series"
- NZS 3604:2011: "Timber-framed buildings".
- AS 2870:2011: "Residential slabs and footings".
- NZS 1170:2004: "Structural design actions".
- NZS 1170 Structural Design Action Part 5: Earthquake actions – New Zealand (2004);
- District and Regional Plan provisions on residential development.
- Council development codes, standards and guides on residential development.
- Robertson, P.K. and Wride, C.E., 1998. Cyclic Liquefaction and its Evaluation based on the CPT Canadian Geotechnical Journal, 1998, Vol. 35, August.

The proposed building sites are presently appropriate or are achievable with appropriate remedial works/constraints.

5.2 Site Subsoil Class

New Zealand Standard NZS 1170.5:2004 provides guidelines for determination of site subsoil class and includes classes which range from A to E. Based on the results of the site investigations and our experience in the area, in accordance with NZS 1170.5:2004, the Site is classified as a Class C shallow soil site.

5.3 Seismic Hazard

No site-specific seismic hazard assessment has been undertaken for this site. Therefore the methodology outlined in Module 1 of the New Zealand Geotechnical Society, Earthquake Geotechnical Engineering Practice Guidelines (NZGS, 2016) has been adopted for evaluation of seismic loading for geotechnical analysis.

Based on a review of the site data, site subsoil class C – "Shallow Soils" has been selected for the project site based on Table 3.2 of (NZS 1170.5, 2004). While localised areas of soft soil may correspond to having site subsoil class D – "Soft Soils", these areas are remote, and the use of class C based peak ground accelerations will be conservative.

The seismic loading for geotechnical design is dependent on the different importance levels and design lives assigned to the various structures and summarised in Table 4. The unweighted PGAs for liquefaction analyses following the Bridge Manual are obtained with the following equation:

$$PGA = C_{0,1000,Manukau\ City} \times \frac{R_u}{1.3} \times f \times g$$

The designed PGA value for the region is shown in Table 4. The PGA value shall be reviewed by the designer.

Table 4 Summary of Design PGA

Subsoil Class	Structure	Importance Level	Return Period (years)	ULS PGA (g)
Class C	Building Design Life 50Years	2	500 (ULS)	0.16

5.4 Soil Liquefaction Assessment

5.4.1 General

Soil liquefaction occurs when cyclic deformations generated by earthquakes cause an increase in pore water pressure in saturated, low-density sands, and silts. When the pore water pressure equals in-situ applied pressure, loss in strength occurs (liquefaction) leading to ground deformation and potentially, loss of bearing capacity. The presence of significant pore water pressure within the soil is essential for liquefaction and generally, the material above the water table is not susceptible to liquefaction.

The susceptibility of soil is a function of particle size distribution, groundwater level, soil density, loading, soil fabric, aging, and other factors. Typically, the fines content of the soil and plasticity play a key role in liquefaction susceptibility. During earthquake shaking, soil particles may dislodge and reorganise into a denser state, whether above or below the groundwater table, though typically effects are more pronounced below the groundwater table. Cyclic consolidation of discrete layers accumulated over the full depth of the soil profile can result in significant ground surface settlement which may be differential due to variability in underlying ground conditions. Settlements, particularly differential, can be damaging to facilities supported on individual shallow foundations. The soil layers comprise of stiff to very stiff cohesive material across the majority of the sites which will act to suppress the surface manifestation of liquefaction.

Due to the cohesive nature of the majority of the geotechnical units and low design level earthquake event, the risk of liquefaction is low to negligible. Therefore, no specific design is required in relation to liquefaction effects.

5.5 Expansive Soils

5.5.1 Classification

The site can be classified as Class "M" for the expansiveness of the soil for the foundation based on Table 2.1 of AS2870. Characteristic surface movement 20 <(ys) ≤40mm. Therefore, the soils are not considered to lie within the definition of "good ground" as per NZS3604, Specific engineering design shall be undertaken by a qualified engineer experienced in the design of footing systems.

5.5.2 Subgrade Preparation/Protection

Considering the importance of expansive soils, once the exposed subgrade has been inspected by a Geo-Professional, it shall be covered with 150mm of granular fill such as the GAP40 base course as soon as possible. The granular layer will not only protect from the drying effects of wind and sun, but the voids within it will also serve as a reservoir of additional moisture to recharge the subgrade, being careful to form a cross-fall on the subgrade to minimize undue ponding.

The footing inverts shall be poured as soon as possible once inspected by a Geo-Professional or covered with a protective layer of site concrete. If subgrade degradation occurs by:

- excessive drying out resulting in desiccation shrinkage cracking or
- subgrade softening after a period of wet weather,

It is recommended to undercut the depth of the degraded zone and replace that material immediately with granular fill.

5.6 Slope Stability Assessment

5.6.1 Qualitative Assessment

A major landslide scarp is located near the northern boundary of the site within the central part of the Lot 11 and Lot 12. No other recent large-scale or deep-seated rotational or translational instability features were observed within the subdivision at the time of our investigation. The series of gullies which have formed within the subdivision appear to be erosional features.

5.6.2 Quantitative Stability Assessment

We have undertaken a numerical slope stability assessment in order to determine the stability of the slope. Slope stability analysis has been performed using the general limit equilibrium Morgenstern-Price method of the proprietary software Slide2 2018. The analysis considers static, elevated water level, and seismic loading conditions. The cross-sections A-A' and B-B' were drawn through the area to contain the proposed development and are included in the Appendix. These cross-sections are considered to provide a range of representative slope conditions within the subdivision. The cross-sections were analyzed for modes of failure, given the deep soil profiles and the modes of failure associated with the existing instability features. Factors of safety against instability were assessed by worst-case scenario techniques. Worst-case scenarios involve the assessment of the theoretically worst groundwater levels for an existing slope and then using assumed realistic effective stress soil parameters to establish the lowest factor of safety for these conditions.

The assessment has been undertaken using existing ground profiles. A re-analysis of the slopes may be required where the slope geometry is altered with subdivision development works. This should be assessed on a site-by-site basis as part of a geotechnical completion report undertaken on the completion of subdivision development works.

5.6.2.1 Factors of Safety (FOS) – Requirements

The FOS as summarized in Table 5 is adopted as appropriate requirements for the various loading conditions:

Table 5 FOS Requirements

Modelled Loading Condition	FOS Required
Long Term (Normal Groundwater) Conditions	1.5
Short Term (Elevated Groundwater Surface) Conditions	1.3
Seismic Conditions (Ultimate Limit State)	1.1

Referring to the criteria provided in MBIE Module-1 and NZS1170.5:2004, a peak ground acceleration (PGA) of 0.16 was calculated for a design earthquake of 1 in a 150-year event for Importance Level 2 (residential) structures for seismic soil Class C.

For this analysis, the following table of effective stress soil parameters was selected:

Table 6 Soil Properties

Soil Description	Density y (kPa)	C' (kPa)	φ (degrees)
Residual Soils (Qvs)	18	3	28
Weak Layer (Qvs)	17	1	25
Transitional Soils (Qvs)	18	5	35

No groundwater was encountered by Tilsley Engineering at the time of the site visit. However, for conservative analysis, we assume that the groundwater table is at 7.0m below the ground level in normal conditions and 2.0m below the ground level in the short-term conditions (storm events).

Details of the stability analyses are shown in the Appendix and a summary of critical cases is presented in the table below:

Table 7 Details of the Slope Stability Analysis

Conditions of Analysis	Type of Failure	Factor of Safety	Meets Criteria
Cross-section A-A’			
Long Term (Normal Groundwater) Conditions	Circular	1.767	Yes
Short Term (Elevated Groundwater Surface) Conditions	Circular	1.199	No
Seismic Conditions	Circular	1.146	Yes
Cross-section B-B’			
Long Term (Normal Groundwater) Conditions	Circular	1.904	Yes
Short Term (Elevated Groundwater Surface) Conditions	Circular	1.483	Yes
Seismic Conditions	Circular	1.244	Yes

5.6.3 Discussion on Slope Stability Analyses

The results of the slope stability assessment are shown in Appendix B.

Lot 1, 2, 3, & 4: These Lots are considered to provide stable building areas and suitable conditions for subdivision development with only minor development constraints.

Lot 5, 6, 7, 8, 9, & 10: The risk of deep-seated slope instability within these Lots is low. The main risk within these Lots is shallow soil creep and debris erosion. This will be mitigated by landscaping, construction retaining structures, or deep pile foundations.

Lot 11 & 12: Due to the historical land sliding within these two Lots, it is recommended to construct two 3m deep counterfort drains. The locations of the counterfort drains are shown on the attached site plan. These drains provide a cut-off to ground water from higher ground and an elevated ground water table developing and reduce pore water pressures. Due to the limited distance between the proposed building area and land sliding, the proposed building area will be safe and stable with construction of Palisade walls(In-ground barrier timber pile retaining walls). The Palisade wall will be 300mm(SED) in diameter, 5m(min) embedded, 1.05m c/c spacing. The Palisade wall locations are shown on the attached site plan. The Palisade wall piles will be drilled. The proposed buildings within these Lots shall have a minimum of 5m setback from the Palisade walls. 2m soil creep shall be considered in the design of the Piles. It should be expected that piles will need to be embedded at a minimum depth as stated above, however, structural design may require deeper embedment. The pile structure design shall be undertaken by a Chartered Professional Engineer.

Lot 13 & 14: According to the slope stability analysis, we consider that a geotechnical Building Line Restriction (defined as 10m from the furthest upslope extent of any unsatisfactory slip surface) be imposed for Lot 13 and 14 in order to maintain safe setback from the slope to the proposed developments. All building structures requiring building consent within Lot 13 and 14 shall be located entirely on the upslope side of the BRL unless supported by a further geotechnical investigation by a suitably experienced Chartered Professional Engineer.

5.7 Foundation Conditions

5.7.1 Lot 1, 2, 3 & 4

The ground conditions are not considered to comply with NZS3604 in terms of seasonal shrink/swell.

5.7.1.1 Shallow Foundation:

The design strength of the shallow foundation for the project site is set out in Table 8. Topsoil shall be stripped beneath and 1m from the building footprint and the building platform checked by a Chartered Professional Engineer or their representative who is experienced in geo-mechanics.

Table 8 Shallow Foundation Design Bearing Capacity

Location	Type	Bearing capacity applicable Depth	Ultimate bearing capacity, qult	Dependable bearing capacity, qdbs	Allowable bearing capacity, qabs
Zone A	Raft/ Strip/Pad	0.3m-2.5m	300kPa	150kPa	100kPa
Notes: Design bearing strength shall be determined using suitable strength reduction factors. Based on B1/VM4 recommendation for shallow foundations.					

5.7.1.2 Pile Foundations

The recommended geotechnical capacities are as set out in Table 9. The design capacities are based on a minimum foundation embedment depth of 500mm from cleared ground level.

Table 9 Pile Design Parameter

Design Parameter Applicable Depth(m)	Ultimate (failure) geotechnical pile shaft side friction capacity	Ultimate (failure) geotechnical pile end bearing capacity	Strength reduction factor	Pile design shaft side friction strength	Pile design end bearing capacity
0-0.5	N/A	300kPa	0.5	N/A	150kPa
0.5-2.5	30kPa	300kPa	0.5	15kPa	150kPa

Design Parameter Applicable Depth(m)	Ultimate (failure) geotechnical pile shaft side friction capacity	Ultimate (failure) geotechnical pile end bearing capacity	Strength reduction factor	Pile design shaft side friction strength	Pile design end bearing capacity

Notes:

1. It should be noted that the upper 0.5m of the pile embedment should be ignored for friction resistance calculations to allow for any loss of contact due to soil shrinkage, and creep movement.
2. End bearing design strengths shall be determined using suitable strength reduction factors. Based on B1/VM4. Strength reduction factor selection may be refined using the risk-based procedure set out in AS 2159.
3. Shaft side friction design strengths shall be determined using suitable strength reduction factors, based on B1/VM4 for bored piles. Strength reduction factor selection may be refined using the risk-based procedure set out in AS 2159.
4. Pile bases need to be cleaned and free of sediment or softened rock prior to concreting.
5. Pile end bearing capacity shall be checked using effective stress parameters provided in Table 9 and standard design pile methods; however calculated capacities exceeding the above values are not recommended unless proven by site-based instrumented pile load testing.
6. Pile load testing would enable adoption of higher strength reduction factors as per the risk based procedure set out in AS 2159.
7. Shallow pile i.e Anchor Pile, Braced piles shall be designed as per NZS 3604 and NZD AS2870. The shallow piles shall be designed by a Chartered Professional Engineer who is familiar with AS2870 and NZS3604.

5.7.2 Lot 5, 6, 7, 8, 9, 10, 11, 12, 13, & 14

These Lots are not considered to provide “good ground” according to NZS 3604 and as such will require a specific geotechnical report for engineered foundation design.

5.8 Retaining Walls

The parameters presented in table 3 shall be used for design of retaining walls for the site. Where possible, it is recommended that building foundations be embedded below or offset horizontally from the 45° zone of influence of the retaining wall to avoid surcharging the retaining wall with building loads. Specific geotechnical assessment shall be required for any proposed foundations located in the zone of influence.

6.0 DEVELOPMENT RECOMMENDATIONS

We consider all Lots are geotechnically suitable for the proposed conventional residential Lot development. This opinion is furnished on the condition that the following recommendations are implemented during design and construction.

6.1 Earthworks

The earthworks will need to be undertaken in accordance with any applicable Council guidelines and the following requirements. No earthworks to be undertaken that increase the slope angle in areas identified with instability features.

The ground is acceptable for shallow and deep foundations. The foundations shall be designed by a Chartered Professional Engineer who is familiar with the contents of this report.All pipework entering the building must enter the foundation at 90° and shall not run parallel to the foundation within 1m from the building perimeter. All earthworks shall be undertaken during the summer works period. No earthworks to be undertaken during winter works unless approved by a Chartered Professional Engineer who is experienced in geo-mechanics and is familiar with the contents of this report.

6.1.1 Fill

In areas where structural fill is to be placed to carry building loads, we recommend that all earthworks procedures and compaction testing are carried out in accordance with NZS4404 and NZS4431. Compaction of cohesive fill shall be carried out in loose layers no greater than 150mm thickness. Compaction testing shall be carried out after each 150mm of fill placement. All fill materials shall be clear of unsuitable materials as outlined above. Cohesive soils shall be suitably moisture conditioned prior to compaction so that once compacted they achieve a minimum vane shear strength of 120kPa and maximum air voids of 10%. A Geotechnical Engineer familiar with the findings of this report shall carry out compaction testing during construction to ensure the correct level of compaction is being achieved.All un-retained fill batters shall be not steeper than 1V:4H and shall not be higher than 0.6m. Fill batter faces shall be compacted as a separate operation or, alternatively, overfilled and cut back. Any engineered fill higher than 0.6m shall be assessed, supervised and approved by a Chartered Professional Engineer experienced in Civil Earthworks.

6.1.2 Cut

All new un-retained cut batters shall be graded at a maximum slope of 1V: 3H and shall not be higher than 0.6m. Cut batters shall be located at a distance of at least the cut batter height from the building platform site. All cuts over 0.6m shall be assessed, supervised and approved by a Chartered Professional Engineer experienced in Civil Earthworks, or their representative The cut area shall be compacted and tested to a minimum allowable bearing strength of 100kPa. Retaining walls within the zone of influence for the building or surcharge load or neighbouring properties shall be designed by a structural Chartered Professional Engineer who is familiar with the contents of this report.

6.1.3 Earthworks Limitation

We recommend that all earthworks procedures and compaction testing be carried out in accordance with NZS4404 and NZS4431. Any cut/fill steeper than the recommended slope may require the construction of a retaining wall specifically designed according to site conditions by a suitably qualified Structural Engineer. No cut or fill depths greater than mentioned in the above sections shall be undertaken without the approval in writing of a Chartered Professional Engineer who is experienced in geo-mechanics and is familiar with the contents of this report. This is because such works may disturb existing equilibrium conditions.

6.2 Adverse Effects on Foundations

Factors that can adversely influence the ground shrinkage and swelling within the vicinity of the foundations are site drainage, watering of gardens, planting trees, and leaking plumbing. For the life of the foundations, no watering shall be undertaken within 3 m of the foundations. No trees to be placed within 3 m of the foundations or further away from foundations for larger trees. Site drainage and plumbing to be regularly and well maintained. All piping shall be kept in a stiff soil layer to avoid possible differential settlement.

6.3 Driveways and Services

No substantial problems are foreseen in relation to driveway construction on the Lot. A minimum CBR of 5 is anticipated. However, we recommend that further investigation of penetration resistance testing shall be conducted when the driveway is being developed to its final level, conforming to designed CBR values.

6.4 Construction Observation

A Geotechnical Engineer familiar with the findings of this report shall be engaged to carry out inspections during earthworks, to confirm soil conditions are consistent with those summarized within this report. It is in the interests of all parties that Tilsley Engineering are retained to inspect earthworks during construction, so that ground conditions can be compared with those assumed in formulating this report. In any event, we shall be notified of any variations in ground conditions from those described or assumed to exist. Any subgrade covered with fill or concrete prior to geotechnical inspection will be specifically excluded from completion certification.

7.0 NATURAL HAZARDS RISK ASSESSMENT

In accordance with Section 106 of the Resource Management Act (RMA), a qualitative natural hazards risk assessment is conducted for the proposed building site. The natural hazard consequence and likelihood of occurrence have been assessed by risk matrix as shown in Table 10, with the risk classifications defined in Table 11

Table 10 Risk Matrix

Potential Consequences	Likelihood				
	Very Unlikely (0-5%)	Unlikely (5-45%)	Possible (45-55%)	Likely (55-95%)	Almost Certain (95-100%)
Severe	Low	Low	Moderate	High	Very High
Moderate	Negligible	Low	Moderate	Moderate	High
Minor	Negligible	Low	Low	Moderate	Moderate
Negligible	Negligible	Negligible	Negligible	Low	Low

Table 11 Risk Classification

RATING SCALE	SECTION 106 COMPLIANCE	DISCUSSION
VERY HIGH	Non-compliant	There is a high probability that severe damage to the proposed building site could arise from an identified source without appropriate remedial action
HIGH	Non-compliant	The proposed building site is likely to experience significant damage from an identified source without remedial action
MODERATE	Non-compliant	May be tolerated in certain circumstances, but requires investigation, planning and implementation of treatment options to reduce the risk to Low. Treatment options to reduce to Low risk shall be implemented as soon as practicable
LOW	Compliant	Usually acceptable
NEGLIGIBLE	Compliant	Acceptable.

Table 12 indicates the risk classification for the identified natural hazards is low to negligible for all risks apart from "soil shrink/swell" and "slope instability" where appropriate mitigation measures can be reasonably provided. As such, we consider the proposed building site fulfills Section 106 of the Resource Management Act.

Table 12 Risk Register

Risk	Potential consequences	Likelihood	Risk class	Comment	Mitigation measures
SLOPE INSTABILITY	Moderate	Possible	Moderate	As per section 5.6	Specific remedial measures such as palisade wall, retaining wall, and counterfort drains.
SOIL EXPANSIVITY	Moderate	Possible	Moderate	Class M soils.	SED Foundations
GROUND SUBSIDENCE	Severe	Unlikely	Low	Low risk of settlement from Liquefaction	N/A
EARTHQUAKE	Severe	Unlikely	Low	Remote from active fault	N/A
FLOODING	Moderate	Unlikely	Low	Elevated site	N/A
TSUNAMI	Severe	Very Unlikely	Negligible	Remote from Ocean	N/A
VOLCANIC ERUPTION	Severe	Very Unlikely	Negligible	Remote from active volcanos	N/A

8.0 RECOMMENDATIONS

- Due to the potential risks of slope instability, we have undertaken a numerical slope stability analysis. Please see section 5.6 for detailed information.

- The ground within the Lots is safe and stable in its current state. A suitably qualified Chartered Professional Engineer (CPEng) or their representative shall confirm the foundations meet the required ultimate bearing capacity at the building consent stage.

- The assessed AS 2870 expansive Site Class for Lot is M and this needs to be considered in conjunction with the foundation design parameters presented in the report.

- The proposed building foundation recommendations are stated in section 5.7 of this report. The soil is **not considered** as “good ground” in terms of NZS 3604:2011 due to the soil expansivity class.

- It is recommended that a site-specific foundation assessment shall be undertaken for each house site as part of building consent.

- Any future Cut/fill under and around the foundations to be inspected by a suitably qualified engineer. All works shall comply with the Earthworks section of this report.

- The proposed building developments within the Lot area shall be required to satisfy the requirements of the Building Act, as discussed in Section 5.0 of this report.

- Due to the cohesive nature of the soils and the relatively low seismicity exhibited in the project area, the site is considered to have a low susceptibility to liquefaction under a design-level earthquake event.

9.0 LIMITATIONS

This report has been prepared solely for the benefit of our client with respect to a particular brief given to us, and data or opinions in it may not be used in other contexts, by any other party or for any other purposes. To the maximum extent permitted by law, Tilsley Engineering Ltd disclaims all liability and responsibility (in contract or tort, including negligence, or otherwise) for any loss or damage whatsoever which may be suffered as a result of any reliance by any third party on this report, whether that loss is caused by any fault or negligence on the part of Tilsley Engineering Ltd or otherwise.

Council is able to rely on this report for processing the resource or building consent only for the site mentioned within this report. It may not be used for any other use or purpose without permission from Tilsley Engineering Ltd.

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Notice to Reader/ User of this document

Should you be in any doubt as to the applicability of this report and/or its recommendations for the proposed development as described herein, and/or encounter materials on site that differ from those described herein, it is essential that you discuss these issues with the authors before proceeding with any work based on this document.

The recommendations and opinions contained in this report are based upon site observations of the investigation area. Inferences about ground conditions over the site are made using geological principles and engineering judgment, however, it is possible that conditions over the site may vary and therefore it is not possible to guarantee the continuity of ground conditions away from investigation locations and visible areas.

Furthermore, the logs are provided presenting descriptions of the soils and geology based on field observations of the samples recovered in the fieldwork and may not be truly representative of the actual underlying conditions.

Recommendations and opinions in this report are based on data obtained from the investigations and site observations as detailed in this report. The nature and continuity of subsoil conditions at locations other than the investigation bores and tests are inferred and it shall be appreciated that actual conditions could vary from the assumed model.

It is recommended that construction activity shall be undertaken in the dry season. If construction activities are undertaken in wet seasons there is a potential risk of reduction of soil strength. Tilsley Engineering Ltd is not responsible for the reduction in soil strength due to construction activities.

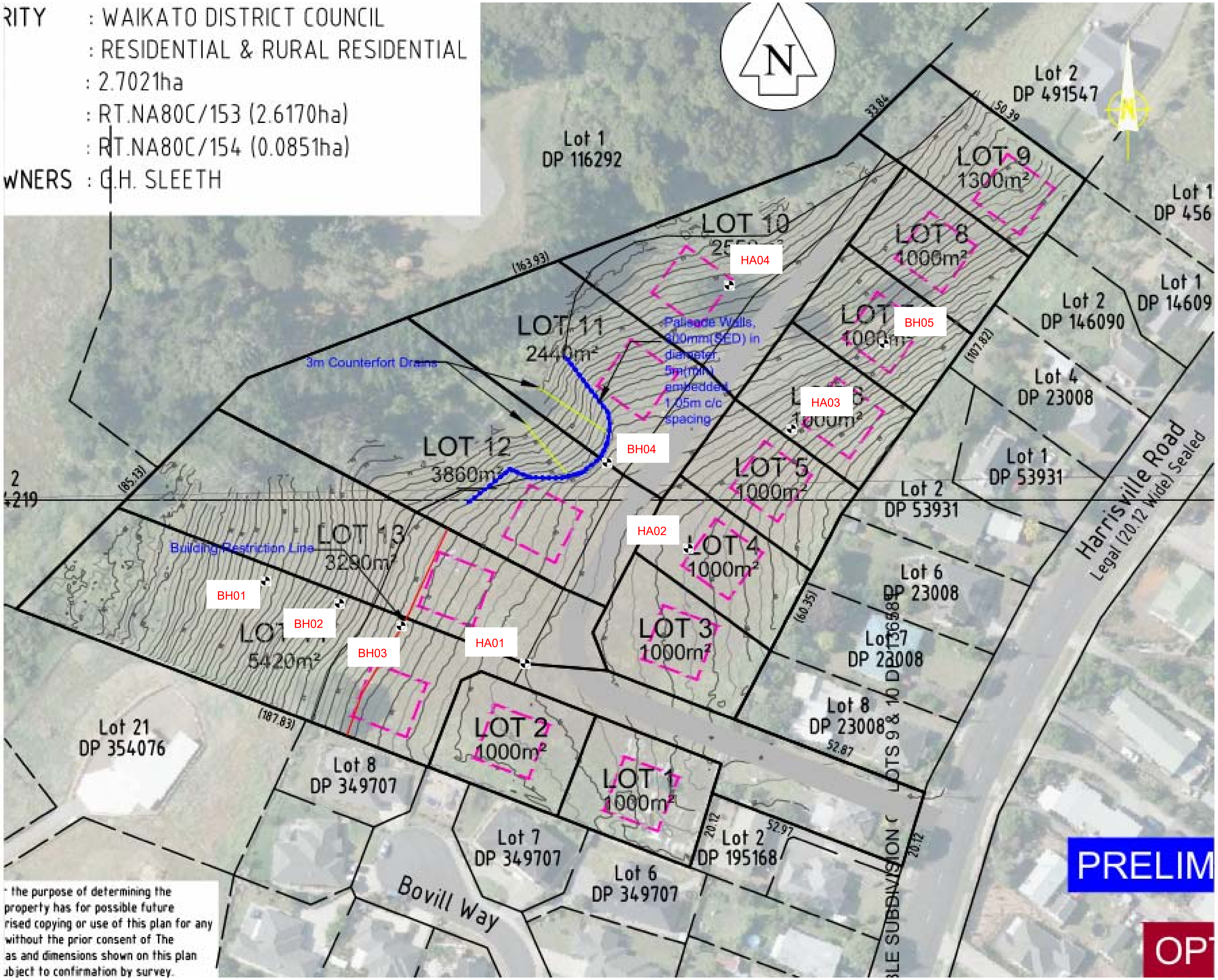
Ground conditions can vary with time, fill may be placed on a site and pollutants may migrate with time. Because a report is based on conditions that existed at the time of subsurface exploration, decisions shall not be based on a report whose adequacy may have been affected by time. Tilsley Engineering Ltd to be advised how time may have impacted on the project.

This report is based on the assumption that the site conditions as revealed through investigation stages are indicative of actual conditions throughout an area. This assumption cannot be substantiated until project implementation has commenced and therefore this report recommendations can only be regarded as preliminary. Only Tilsley Engineering Ltd, 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 shall 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 Tilsley Engineering Ltd cannot be held responsible for such misinterpretation.

APPENDIX A

INVESTIGATION LOCATIONS, LOGS AND SITE PICTURES

PROJECT PLAN AND BOREHOLE LOCATIONS



Typical Picture of Subsoil

Typical Picture of Subsoil

Project Area

Project Area

Steep Slopes within Lot

Steep Slopes within Lot

APPENDIX B

INVESTIGATION BORE LOGS

BORE LOG

TILSLEY ENGINEERING LIMITED
27 Roulston Street, Pukekohe
info@teng.co.nz
PO Box 392, Pukekohe
Phone: 09 238 3245

Job number: 16277
Client: Site Land Development Ltd
Location: 23 Harrisville Road, Tuakau
Soil type Qvs

Date: 25/05/2021
Time: 10:00am
Augered by: Digger
Logged by: JR
Bore No: BH1

COMMENTS

Depth (m)	Material Description	Soil Symbol	Soil Sensitivity	Moisture	Water	Shear Vane Test / Reform (kPa)	Scala Penetrometer Test (blows/150mm - kPa)
0.2	Topsoil, brown, trace rootlets present.			moist			
0.4 0.6 0.8 1 1.2 1.4 1.6 1.8 2 2.2 2.4	Silty CLAY, traces of sand and volcanic ash, orange-brown, very stiff, low plasticity.		2.15 2.33 2.05 1.69	moist very moist		170/79 170/73 127/62 115/68	
2.6 2.8 3 3.2 3.4 3.6 3.8	Clayey SILT, traces of sand and volcanic ash, red and light brown, with grey streaks, stiff, moderate plasticity.		1.73 2.20 1.78			88/51 112/51 82/46	
4 4.2 4.4 4.6 4.8	Clayey SILT, trace gravels and sand present, red, cohesive, stiff, moderate to high plasticity.		2.00	very moist		88/58 102/51	
5 5.2	EOB = 5m						

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BORE LOG

TILSLEY ENGINEERING LIMITED
27 Roulston Street, Pukekohe
info@teng.co.nz
PO Box 392, Pukekohe
Phone: 09 238 3245

Job number: 16277
Client: Site Land Development Ltd
Location: 23 Harrisville Road, Tuakau
Soil type Qvs

Date: 25/05/2021
Time: 10:45am
Augered by: Digger
Logged by: JR
Bore No: BH2

COMMENTS

Depth (m)	Material Description	Soil Symbol	Soil Sensitivity	Moisture	Water	Shear Vane Test / Reform (kPa)	Scala Penetrometer Test (blows/150mm - kPa)
0.0–0.3	Topsoil, brown, friable.			moist			
0.5	Clayey SILT, traces of sand and volcanic ash, red and light brown, very stiff, friable, low to moderate plasticity. Dark brown layer at 3.5 to 4m.		-			>200	
1.0			2.40	moist		156/65	
1.5			1.91			141/74	
2.0			3.29			112/34	
2.5			2.04	very moist		102/50	
3.0			1.58			93/59	
3.5			1.88			79/42	
4.0			1.52			73/48	
4.0–4.5	Silty CLAY, traces of sand and volcanic ash, orange-red, stiff, moderate plasticity.			very moist			
4.5–5.0	Clayey SILT, traces of sand and volcanic ash, orange-red stiff, moderate plasticity.		1.56			114/73	
5.0	EOB = 5m						

Disclaimer Soil type and strength only applicable at location of bore hole.

BORE LOG

TILSLEY ENGINEERING LIMITED
27 Roulston Street, Pukekohe
info@teng.co.nz
PO Box 392, Pukekohe
Phone: 09 238 3245

Job number: 16277
Client: Site Land Development Ltd
Location: 23 Harrisville Road, Tuakau
Soil type Qvs

Date: 25/05/2021
Time: 11:30am
Augered by: Digger
Logged by: JR
Bore No: BH3

COMMENTS

Depth (m)	Material Description	Soil Symbol	Soil Sensitivity	Moisture	Water	Shear Vane Test / Reform (kPa)	Scala Penetrometer Test (blows/150mm - kPa)
0.2	Topsoil, brown, friable.			dry			
0.4	Clayey SILT, traces of sand and volcanic ash, orange-brown, very stiff, friable, low to moderate plasticity.		-			>200	
0.6							
0.8							
1			1.63	moist		170/104	
1.2							
1.4			1.92			127/66	
1.6							
1.8							
2			2.14			141/66	
2.2							
2.4			2.49	moist		112/45	
2.6	Silty CLAY, traces of sand and volcanic ash, red with light orange streaks, stiff, moderate plasticity.						
2.8							
3			1.96			88/45	
3.2	Clayey SILT, traces of sand and volcanic ash, red and light brown, very stiff, friable, low to moderate plasticity.						
3.4			1.97			73/37	
3.6							
3.8							
4			1.55			96/62	
4.2	Silty CLAY, traces of sand and volcanic ash, red with light orange streaks, stiff, moderate plasticity.						
4.4				very moist			
4.6							
4.8			1.70			102/60	
5	EOB = 5m						
5.2							

Disclaimer Soil type and strength only applicable at location of bore hole.

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BORE LOG

TILSLEY ENGINEERING LIMITED
27 Roulston Street, Pukekohe
info@teng.co.nz
PO Box 392, Pukekohe
Phone: 09 238 3245

Job number: 16277
Client: Site Land Development Ltd
Location: 23 Harrisville Road, Tuakau
Soil type Qvs

Date: 25/05/2021
Time: 12pm
Augered by: Digger
Logged by: JR
Bore No: BH4

COMMENTS

Depth (m)	Material Description	Soil Symbol	Soil Sensitivity	Moisture	Water	Shear Vane Test / Reform (kPa)	Scala Penetrometer Test (blows/150mm - kPa)
0.2	Topsoil, brown, friable.			dry			
0.4			-				
0.6	Silty CLAY, traces of sand and volcanic ash, red/brown with light orange streaks, stiff to very stiff, low to moderate plasticity.					>200	
0.8							
1			1.77			170/96	
1.2							
1.4							
1.6							
1.8							
2			2.38	dry		155/65	
2.2							
2.4			2.12			155/73	
2.6							
2.8							
3			3.04			155/51	
3.2							
3.4			3.18	moist		108/34	
3.6							
3.8							
4			2.27	wet		102/45	
4.2							
4.4			3.35			114/34	
4.6							
4.8							
5			2.00			102/51	
5.2	EOB = 5m						

Disclaimer Soil type and strength only applicable at location of bore hole.

produced by ESlog.ESdat.net on 01 Jun 2021

BORE LOG

TILSLEY ENGINEERING LIMITED
27 Roulston Street, Pukekohe
info@teng.co.nz
PO Box 392, Pukekohe
Phone: 09 238 3245

Job number: 16277
Client: Site Land Development Ltd
Location: 23 Harrisville Road, Tuakau
Soil type Qvs

Date: 25/05/2021
Time: 12:30pm
Augered by: Digger
Logged by: JR
Bore No: BH5

COMMENTS

Depth (m)	Material Description	Soil Symbol	Soil Sensitivity	Moisture	Water	Shear Vane Test / Reform (kPa)	Scala Penetrometer Test (blows/150mm - kPa)
0.0–0.45	Topsoil, brown, friable.			dry			
0.5	Silty CLAY, traces of sand and volcanic ash and pumiceous inclusions, orange-brown/red, stiff to very stiff, low to moderate plasticity, dry to moist.		-			>200	
1.0			2.30			152/66	
1.5			-			>200	
2.0			-	moist		>200	
2.5			3.81			141/37	
3.0			2.76			141/51	
3.5	Silty CLAY, highly weathered rock, traces of sand and volcanic ash, yellow, stiff, high plasticity.			moist			
4.0			3.78			140/37	
5.0			4.55			141/31	
	EOB = 5m						

Disclaimer Soil type and strength only applicable at location of bore hole.

BORE LOG

TILSLEY ENGINEERING LIMITED
27 Roulston Street, Pukekohe
info@teng.co.nz
PO Box 392, Pukekohe
Phone: 09 238 3245

Job number: 16277
Client: Site Land Development Ltd
Location: 23 Harrisville Road, Tuakau
Soil type Qvs

Date: 25/05/2021
Time: 3:30pm
Augered by: PG
Logged by: JR
Bore No: HA3

COMMENTS

Depth (m)	Material Description	Soil Symbol	Soil Sensitivity	Moisture	Water	Shear Vane Test / Reform (kPa)	Scala Penetrometer Test (blows/150mm - kPa)
0 – 0.2	Topsoil, brown, friable.			dry			
0.2 – 2.0	Clayey SILT, traces of sand and volcanic ash, orange-brown/red, very stiff, low to moderate plasticity, dry.						
0.5			-			>200	
1.2			2.72			155/57	
1.5			-			>200	
1.75				moist			
2			-			>200	
	EOB = 2m						

Disclaimer Soil type and strength only applicable at location of bore hole.

produced by ESlog.ESdat.net on 01 Jun 2021

BORE LOG

TILSLEY ENGINEERING LIMITED
27 Roulston Street, Pukekohe
info@teng.co.nz
PO Box 392, Pukekohe
Phone: 09 238 3245

Job number: 16277
Client: Site Land Development Ltd
Location: 23 Harrisville Road, Tuakau
Soil type Qvs

Date: 25/05/2021
Time: 2pm
Augered by: PG
Logged by: JR
Bore No: HA1

COMMENTS

Depth (m)	Material Description	Soil Symbol	Soil Sensitivity	Moisture	Water	Shear Vane Test / Reform (kPa)	Scala Penetrometer Test (blows/150mm - kPa)
0.0–0.25	Topsoil, brown, friable.			dry			
0.25–2.0	Silty CLAY, traces of sand and volcanic ash, orange-brown, very stiff, low to moderate plasticity, dry to moist.						
0.5			2.28			155/68	
1.0			-			>200	
1.125				moist			
1.5			-			>200	
1.75			4.55			141/31	
2.0			3.78			170/45	
2	EOB = 2m						

Disclaimer Soil type and strength only applicable at location of bore hole.

produced by ESlog.ESdat.net on 01 Jun 2021

BORE LOG

TILSLEY ENGINEERING LIMITED
27 Roulston Street, Pukekohe
info@teng.co.nz
PO Box 392, Pukekohe
Phone: 09 238 3245

Job number: 16277
Client: Site Land Development Ltd
Location: 23 Harrisville Road, Tuakau
Soil type Qvs

Date: 25/05/2021
Time: 2:30pm
Augered by: PG
Logged by: JR
Bore No: HA2

COMMENTS

Depth (m)	Material Description	Soil Symbol	Soil Sensitivity	Moisture	Water	Shear Vane Test / Reform (kPa)	Scala Penetrometer Test (blows/150mm - kPa)
0.0–0.2	Topsoil, brown, friable.			dry			
0.2–2.0	Silty CLAY, traces of sand and volcanic ash, orange-brown/red, very stiff, low to high plasticity, dry to moist.						
0.5			1.96			167/85	
1			-			>200	
1.1				moist			
1.5			1.83			176/96	
1.8			2.30			184/80	
2			-			>200	
	EOB = 2m						

Disclaimer Soil type and strength only applicable at location of bore hole.

BORE LOG

TILSLEY ENGINEERING LIMITED
27 Roulston Street, Pukekohe
info@teng.co.nz
PO Box 392, Pukekohe
Phone: 09 238 3245

Job number: 16277
Client: Site Land Development Ltd
Location: 23 Harrisville Road, Tuakau
Soil type Qvs

Date: 25/05/2021
Time: 4pm
Augered by: PG
Logged by: JR
Bore No: HA4

COMMENTS

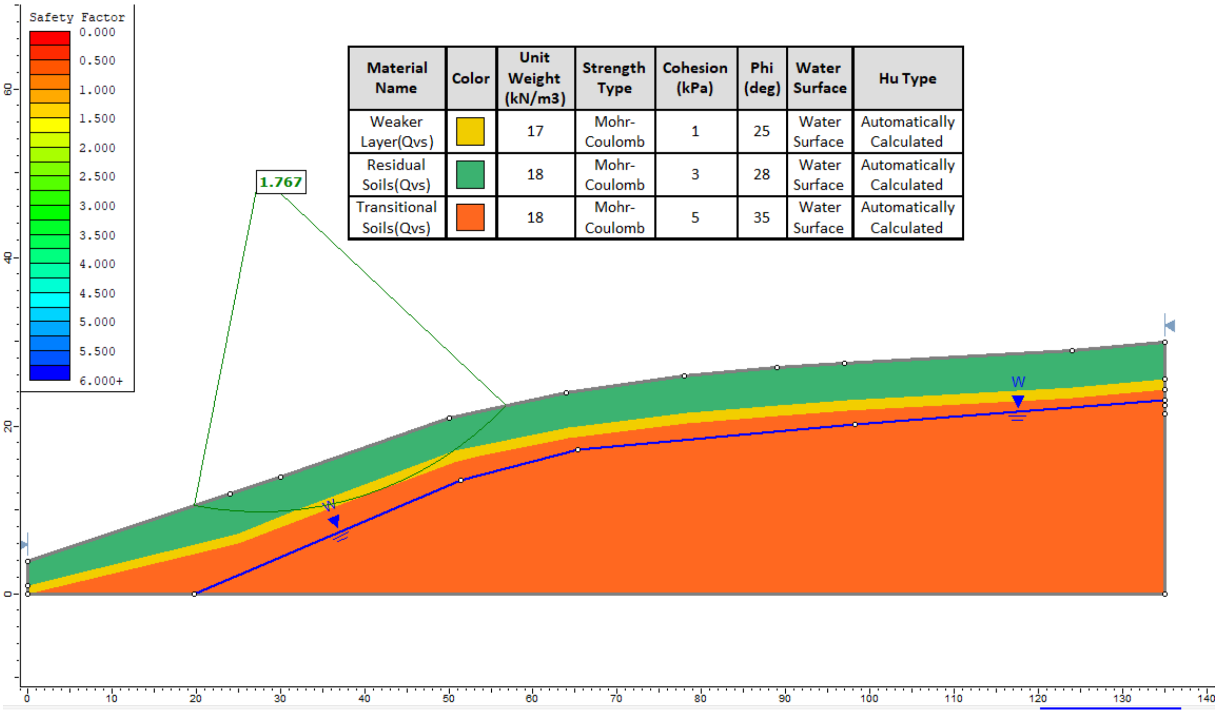
Depth (m)	Material Description	Soil Symbol	Soil Sensitivity	Moisture	Water	Shear Vane Test / Reform (kPa)	Scala Penetrometer Test (blows/150mm - kPa)
0.0–0.3	Topsoil, brown, friable.			dry			
0.3–1.3	Silty CLAY, traces of sand and volcanic ash, orange-brown/red with light yellow streaks, stiff to very stiff, low to moderate plasticity, dry to moist.						
0.5			1.76			155/88	
1			1.89			167/88	
1.1				moist			
1.3	EOB =1.3m		-			>200	

Disclaimer Soil type and strength only applicable at location of bore hole.

APPENDIX C

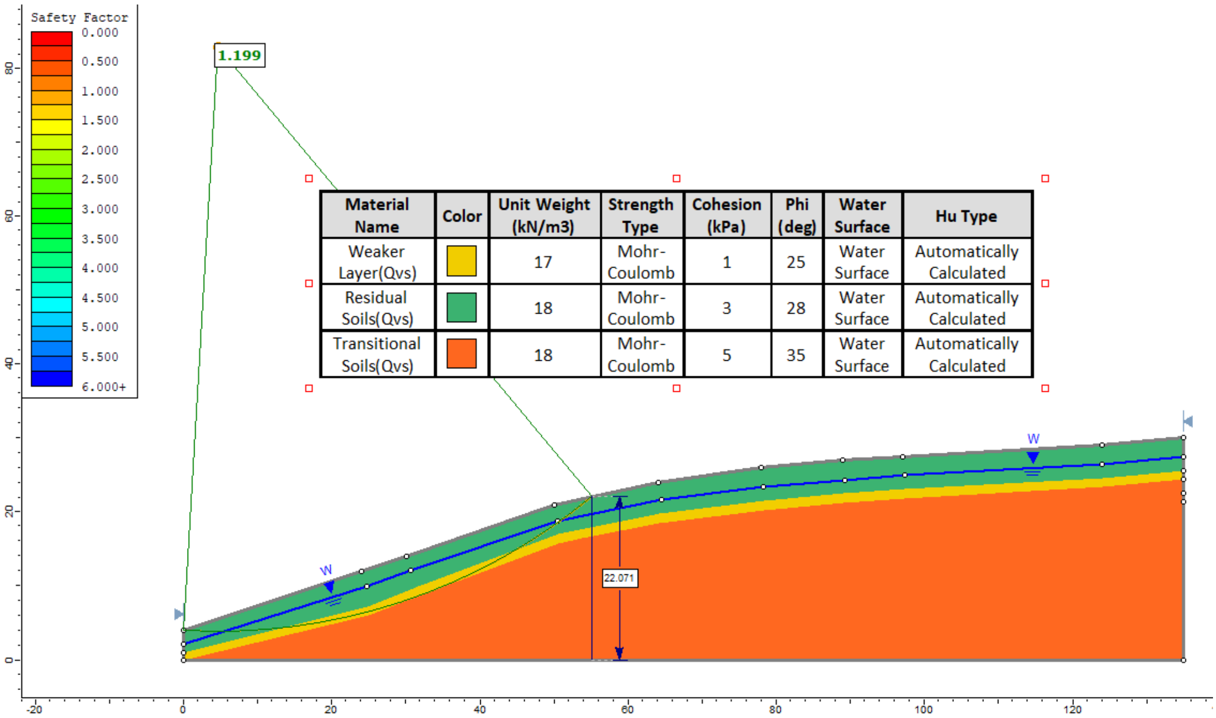
SLOPE STABILITY ANALYSIS

Cross-section A-A’



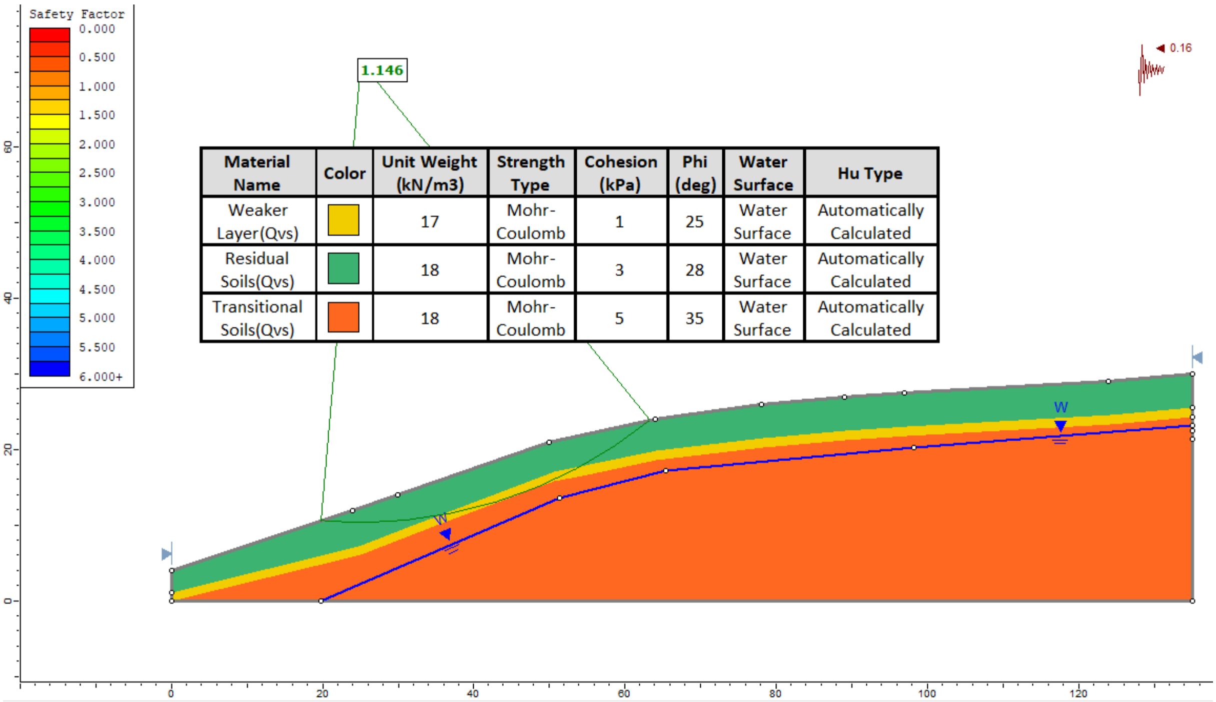
Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)	Water Surface	Hu Type
Weaker Layer(Qvs)		17	Mohr-Coulomb	1	25	Water Surface	Automatically Calculated
Residual Soils(Qvs)		18	Mohr-Coulomb	3	28	Water Surface	Automatically Calculated
Transitional Soils(Qvs)		18	Mohr-Coulomb	5	35	Water Surface	Automatically Calculated

Long Term (Normal Groundwater) Conditions



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)	Water Surface	Hu Type
Weaker Layer(Qvs)		17	Mohr-Coulomb	1	25	Water Surface	Automatically Calculated
Residual Soils(Qvs)		18	Mohr-Coulomb	3	28	Water Surface	Automatically Calculated
Transitional Soils(Qvs)		18	Mohr-Coulomb	5	35	Water Surface	Automatically Calculated

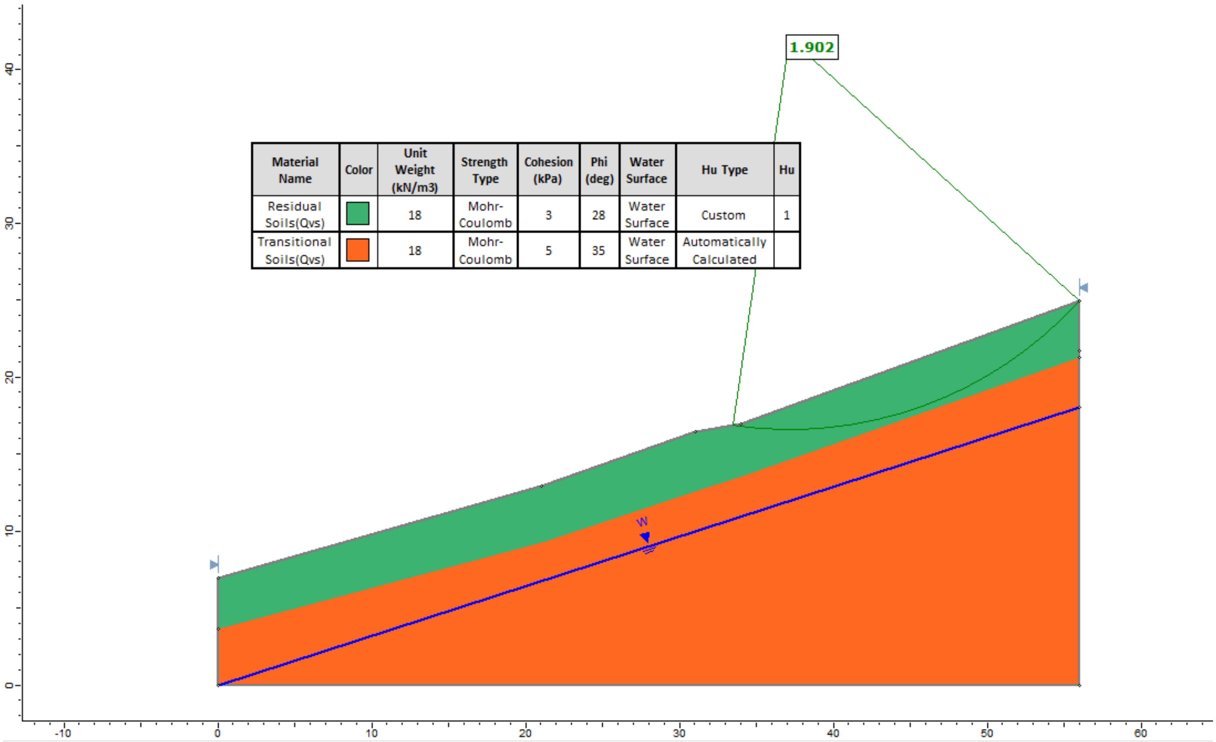
Short Term (Elevated Groundwater Surface) Conditions



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)	Water Surface	Hu Type
Weaker Layer(Qvs)		17	Mohr-Coulomb	1	25	Water Surface	Automatically Calculated
Residual Soils(Qvs)		18	Mohr-Coulomb	3	28	Water Surface	Automatically Calculated
Transitional Soils(Qvs)		18	Mohr-Coulomb	5	35	Water Surface	Automatically Calculated

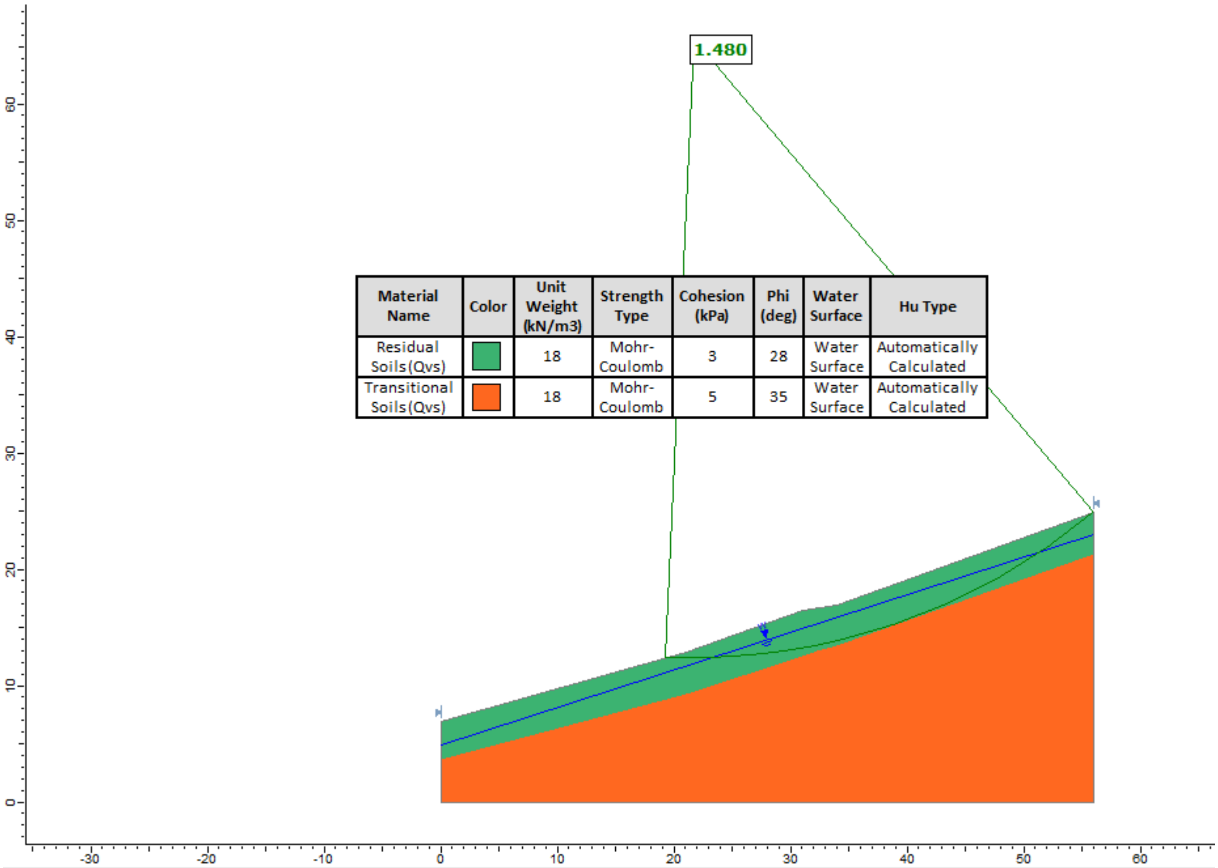
Sesmic Conditions

Cross-section B-B'

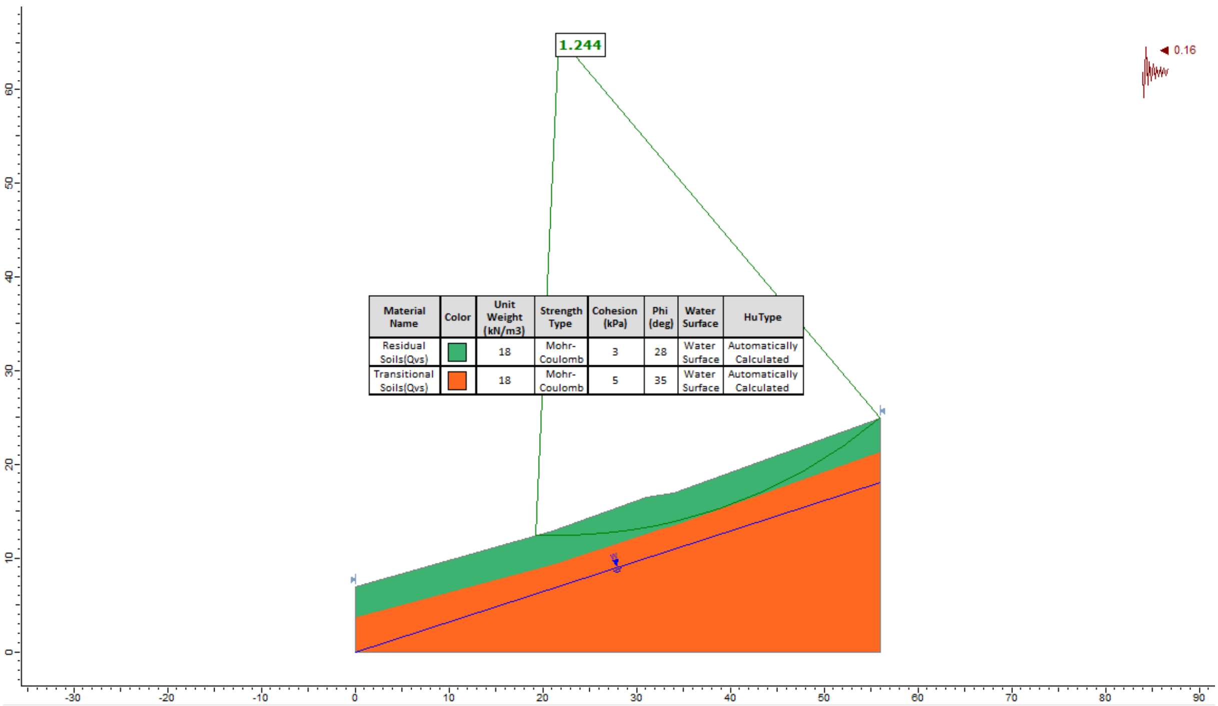


Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)	Water Surface	Hu Type	Hu
Residual Soils(Qvs)		18	Mohr-Coulomb	3	28	Water Surface	Custom	1
Transitional Soils(Qvs)		18	Mohr-Coulomb	5	35	Water Surface	Automatically Calculated	

Long Term (Normal Groundwater) Conditions



Short Term (Elevated Groundwater Surface) Conditions



Sesmic Condition