TERRATEC

GEOTECHNICAL ENGINEERING INVESTIGATION REPORT

Proposed Shed and Future Dwelling

263 Wellsford Valley Road, Wellsford

Prepared for:

Michael & Diana Bassick
9 Siesta Terrace
Gulf Harbour
Auckland
0930

Ref No. J1222-1

TERRATEC

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PROPOSED SHED AND FUTURE DWELLING

263 WELLSFORD VALLEY ROAD, WELLSFORD

Job No:	J1222-1
Client(s):	Michael and Diana Bassick
Document Version:	1
Report Issue Date:	16th October 2023
Prepared By:	Aaron Fell *Director, CMEngNZ (PEngGeol)*
Signature:	
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Signature	

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1 INTRODUCTION

At the instruction of Michael and Diana Bassick, we have carried out a geotechnical engineering investigation for a proposed shed platform and suitable building platform area for a future residential dwelling to be constructed at 263 Wellsford Valley Road, Wellsford.

The scope of our investigation was to carry out a detailed site walkover inspection of the property and to explore subsurface conditions using hand operated equipment. The information obtained has been used to assess ground conditions, determine geohazards, create a geological ground model and carry out global stability analysis in order to provide geotechnical recommendations for the shed and preliminary geotechnical recommendations for a suitable building platform for a future dwelling.

We advise that aspects of stormwater disposal and land contamination are outside of our brief.

2 SITE DESCRIPTION AND PROPOSED DEVELOPMENT AREAS

The subject property at 263 Wellsford Valley Road, Wellsford, is legally described as Lot 5, DP 76439, with a total area of 1.05Ha. The property is irregular in plan shape, bounded by Wellsford Valley Road on the northern and eastern sides, a private rural property to the west and an easement to the south.

The property is presently vacant, with the exception of a gravel driveway running parallel with the western property boundary to the proposed development area. The approximate development area at the property is shown on Figure 1 below.

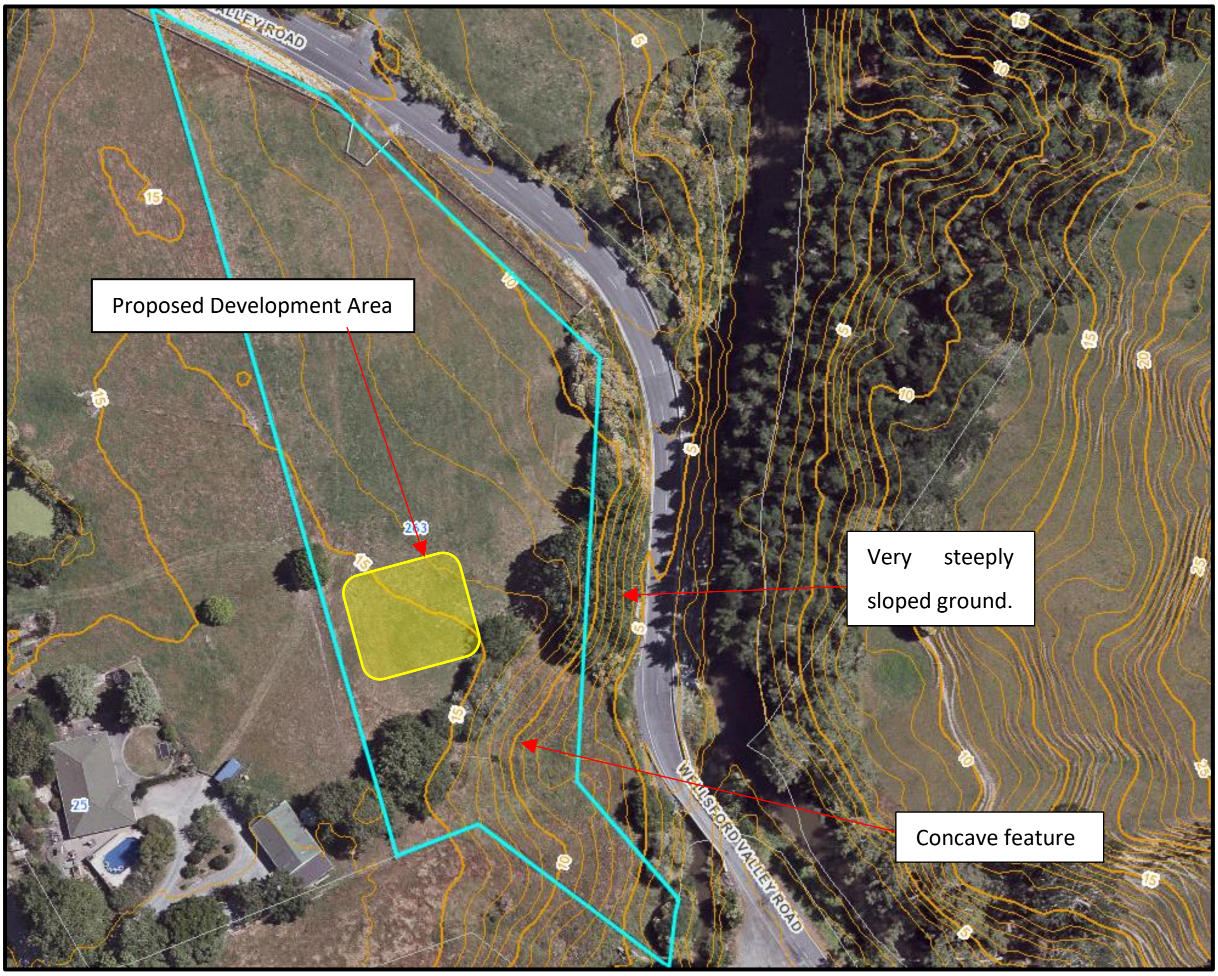


Figure 1: Subject Property and Proposed Development Area (Taken from Auckland Council GeoMaps; North Up Page)

The topography of the property is generally gently sloping, with gradients generally no steeper than 1 vertical on 12 horizontal (1V:12H) descending east and northeast. The exception to this is the southeastern portion of the property where a concave feature descending to a gully is present (to the east of an existing tree line). This concave feature descends at grades between 1V:3.5H and 1V:2.5H to Wellsford Valley Road.

Additionally, a steep to very steep (approx. 1V:2H) face is present immediately beyond the eastern property boundary, descending to the road (over a vertical height of approximately 6m). This steep face was likely formed during construction of Wellsford Valley Road.

To the east of Wellsford Valley Road, the land descends at moderately steep gradients over a vertical height of approximately 3m to Whakapirau Creek.

The steeply sloping concave feature extends southwards into neighbouring sites. During our walkover, we noted that in a property to the south (approximately 70m from the development area) had been subject to recent instability. A headscarp (estimated 1.5m depth) can be observed as shown on Figure 2 below.

Figure 2: Land Movement at Crest of Concave Slope in Neighbouring Property

Auckland Council (AC) GeoMaps shows no overland flowpath within the subject property. It also shows no buried public services within or adjacent to the property.

3 PROPOSED DEVELOPMENT

At the time of preparing this report, we have not been provided with proposed development plans. We understand from the client that it is proposed to construct a shed and future dwelling on the near level land at the southwestern portion of the property. We assume that the structures will be built in general accordance with NZS3604:2011. The land in the vicinity of the proposed building platform, which includes the concave feature to the southeast, is herein referred to as subject site.

As stated in Section 1 a suitable building platform area will be determined and preliminary geotechnical recommendations for a future residential dwelling will be provided.

4 DESKTOP STUDY

As part of our desktop study, we have reviewed relevant publicly available data including historic aerial photographs contained on the AC GeoMaps and Retrolens websites and published geology maps on GNS Science.

4.1 Published Geology

GNS Sciences web-based 1:250,000 geology map indicates that the land is underlain by Mahurangi Limestone of the Northland Allochthon. This unit is described as; *"Blue-grey to white, micritic, muddy limestone, commonly with glauconitic sandstone beds."*. The parent materials weather to a surficial capping silts, sandy silts and clays.

4.2 Historical Aerial Images

A review of historical aerial imagery contained on Retrolens and AC GeoMaps websites. Table 1 summarises our review of the images.

Table 1 – Historical Images Reviewed

DATE TAKEN	SOURCE	DESCRIPTION/NOTABLE FEATURES
1966	Retrolens	The property is vacant, with Wellsford Valley Road formed.
1976	Retrolens	Site remains vacant but it appears trees have been planted at the crest of the slopes which defines the concave feature in the southeastern portion of the property.
1982	Retrolens	No obvious changes at the property.
1992	Retrolens	No change within the subject site. The house and a pond at the property to the west is under construction.
2011	GeoMaps	No changes to subject site. Headscarp/exposed soils appear evident to the south of the subject property has occurred.
2015	GeoMaps	No changes to the subject site, but the aforementioned land movement appears to have extended northwards.

4.3 Liquefaction Vulnerability

We make reference to the AC GeoMaps viewer which includes a basic risk assessment for liquefaction in the Auckland region. This map is based on a regional scale and based on *"existing information (geological and topographic maps) and local knowledge".* However, it notes that the liquefaction vulnerability susceptibility should only be used as a first-pass assessment only and is not suitable for design.

Irrespective of the above, the map classifies the region of future development as "Very Low Liquefaction Vulnerability". We note that some of the land in the northern portion of the property is mapped as "Liquefaction Damage is Possible".

5 FIELD EXPLORATION

In order to carry out a slope stability assessment which is necessary to determine a suitable safe and stable future building platform area, and to assess geotechnical design parameters for future foundations, the ground conditions at the subject site were explored by drilling three geotechnical hand auger boreholes at the approximate locations shown on the Test Location Plan (Sheet 1). The boreholes (AH01 to AH03) were drilled on 4th October 2023 at locations selected to provide representative indications of the subsurface ground conditions within the future development area.

All boreholes were designated to be taken to 5m depth or refusal whichever was encountered first. A calibrated shear vane was used generally at 0.5m depth intervals in the boreholes to measure in-situ soil shear strengths. Scala penetrometer testing was designated to be carried out for an additional 2m maximum depth below the base of all hand auger boreholes.

All retrieved samples were logged by a Terratec Limited engineering geologist in accordance with 'The guidelines for the classification and description of soil and rock for engineering purposes' by New Zealand Geotechnical Society (2005). Borehole logs are presented on the attached Sheets 2A to 2C.

6 SUBSURFACE CONDITIONS

Subsurface ground conditions encountered in the exploratory boreholes are summarised below. For a full detailed description of the subsoils encountered on site, reference should be made to the attached borehole logs. For the purposes of this report, subsoil conditions on the site have been interpolated between the boreholes and it must be accepted that soil conditions can and do vary between borehole positions.

Non-Engineered Fill

A surficial veneer of non-engineered fill was encountered from the ground surface in borehole AH01 to 0.1m depth. This material had been recently placed on the ground surface following driveway formation.

Topsoil

Below the non-engineered engineered fill materials in borehole AH01 and from the ground surface in boreholes AH02 and AH03, a thin veneer of topsoil was identified.

Alluvial Soils

Below the topsoil at all test locations, alluvial soils were identified. At borehole AH01 and AH02 the soils were clay dominated (highly plastic) to depths of 4.4m and 5m respectively. Below the clayey materials in AH01, the soils comprised silts with some clay and some sand, to the target depth of 5m. Below the topsoil in AH03, silts and clayey silts were encountered to 3.2m, whereby a hard stratum was identified. The materials from 4.6m to 5m in AH02 comprised an organic clay with decomposed woody fragments.

At borehole AH01 and AH02, the upper 3m was very stiff and becoming firm to stiff below 3m. The materials within AH03 were generally stiff. In-situ undrained shear strength readings carried out in the alluvium ranged from 40kPa to 183kPa.

Scala penetrometer testing was carried out below the base of boreholes AH01 and AH02. Effective refusal (defined as six consecutive counts of 10 blows or greater per 50mm penetration of the Scala probe), was identified at a depth of 6.7m in borehole AH02. Scala testing was not carried out in AH03 due to the density of the material at immediate refusal of the hand auger.

Groundwater

Groundwater was measured in the boreholes at the conclusion of the investigation, which was undertaken during early spring (and following a historically wet Auckland year). Groundwater was encountered at 4.2m depth in AH01 and 3.0m depth in AH03, with AH02 dry.

7 GEOTECHNICAL PARAMETERS

We have assigned typical geotechnical soil strength parameters for the materials encountered at the site, taking into account the in-situ test data and typical industry adopted soil strength properties of similar soils. The parameters are given in Table 2 below. We have adopted these parameters for our stability analysis.

Table 2 – Geotechnical Design Parameters

Material Type	Unit Weight (γ) (kN/m³)	Undrained Shear Strength (S_u) (kPa)	Effective Shear Strength Parameters	
			Cohesion c′ (kPa)	Angle of Friction, ϕ′ (°)
Very Stiff Alluvium	18	80	4	28
Stiff to Very Stiff Alluvium	18	60	1	28
Firm Alluvium	18	25	1	18
Dense Materials *	18	100	4	28

* - Based on the geological maps, we consider that the dense materials may be less weathered Northland Allochthon.

8 GEOHAZARD ASSESSMENT

Table 3 below provides an unmitigated geohazard assessment risk register for the proposed future development at the subject site, based on our desktop study, review of relevant existing geotechnical information, walkover assessment, data obtained from our site-specific geotechnical investigation and our understanding of the development proposed for this property.

For each unmitigated geohazard, we have defined the likelihood and risk for each geohazard over the design life of the structure as follows, with the value in brackets the risk classification number. The risk classifications are multiplied together to determine the unmitigated risk rating for each geohazard.

Likelihood of Geohazard (over the 50-year design life of the building)

- Rare: Not expected to occur (1)
- Unlikely: Small chance (<10%) (2)
- Likely: At least 50% chance (3)
- Almost Certain: 90% Chance (4)

Consequence (over the 50-year design life of the building)

- Insignificant: Minimal economic loss & no risk of injury (1)
- Moderate: Structural damage &/or low risk of injury (2)
- Major: Significant structure damage &/or medium risk of injury (3)
- Extreme: Irreversible structure damage &/or high risk of injury (4)

Where the unmitigated geohazard risk rating is **4 or greater** an assessment and geotechnical recommendations is provided as part of this report in order to appropriately identify that a safe and stable building platform can be formed.

Table 3 – Geohazard Assessment Risk Register

Geohazard	Likelihood	Consequence	Risk Rating	Comments
Seismic Shaking	Rare (1)	Major (3)	3	Structural design will need to ensure that the structure(s) are able to tolerate movements under both a Serviceability Limit State (SLS) and Ultimate Limit State (ULS) earthquake motion events. We have assessed the subsoil seismic site class which may be used in design (Refer Section 9).
Expansive Soils	Likely (3)	Moderate (2)	6	The geotechnical investigation determined the presence of highly plastic alluvial clays and silty clays from the existing ground surface. Based on the plasticity of the near surface soils, we consider that the development must take into consideration for highly expansive soils in its foundation design.
Liquefaction	Rare (1)	Major (3)	3	The soils within the proposed development area comprise very stiff alluvium over the upper approximate 3.5m, overlying stiff silts and clays. No sand or loose soils were identified. Based on our experience, there is a thick crust of non-liquefiable soils (at least 4m) and therefore we do not anticipate that a future development to be affected by liquefaction in ULS and SLS seismic events.
Settlement	Rare (1)	Moderate (2)	2	The upper 3.5m to 4m of soils in the development area are very stiff, with firm to stiff soils encountered at depth. Based on our site assessment, any settlement of the natural soils under nominal building loads (typically lightweight residential structures) are anticipated to be within tolerable limits and less than 25mm; complying with the Building Code. Any identified non-engineered must be removed from beneath the building platform to expose natural soils.

Geohazard	Likelihood	Consequence	Risk Rating	Comments
Instability (Global)	Likely (3)	Major (3)	9	We make reference to the Auckland Unitary Plan (AUP) Definitions (J1), which state the following land may be prone to instability: (c) – Where the land is underlain by Holocene or Pleistocene sediments which has a slope angle greater than or equal to 1 vertical on 3 horizontal. The concave feature (which may represent a historic land movement) to the southeast of the proposed development area includes gradients between 1V:3.5H and 1V:2.5H, which, in accordance with the AUP, may be prone to instability. As noted in the Site Description section of this report, the slopes to the south of the subject property have been subject to recent instability in the neighbouring site. Based on this observation and the concave landform within the subject property, we consider these adjacent slopes to the proposed development area may be prone to future instability. In order to assess slope stability and to determine a safe building platform area, we have carried out a quantitative slope stability analysis.
Instability (Creep)	Unlikely (2)	Major (3)	6	In addition to the above, the land on the steep slopes is subject to naturally occurring shallow creep processes. The designated building platform is proposed to be sufficiently set back from the crest of the slope and therefore soil creep processes is unlikely to affect the proposed development area.
Rockfall	Rare (1)	Moderate (2)	2	There is no evidence of exposed rock within or in close proximity to the property that would result in a rockfall hazard.
Acid Sulphate Soils	Rare (1)	Moderate (2)	2	Acid sulphate soils occur in recent alluvial deposits and are a geotechnical issue only when sulphides are able to react with oxygen (such as groundwater lowering). Alluvial soils are present at this site; however, we do not anticipate excavations below the water table. Consequently, the acid sulphate geohazard is not an issue for this development.

9 SUBSOIL SEISMIC SITE CLASS

The geotechnical testing encountered stiff to very stiff natural soils, with Scala penetrometer refusal at two of the test locations.

The site seismic class has been assessed in accordance with Section 3.1.3 of NZS1170.5:2004 which provides a hierarchy for site classification methods.

Based on our site investigation, bedrock is anticipated to be less than 10m depth and therefore the site has been classified as Subsoil Class C.

10 EXPANSIVE SOILS

Expansive soil types are described as materials which can undergo shrinking and swelling due to cyclic soil moisture content variations occurring regularly through dry and wet weather periods. These cyclic soil moisture content variations result in cracking and heaving of the near surface soils which can pose significant risk to structures and their foundations if the expansive nature of the soils are not taken into consideration in their design.

The shrink-swell potential at the subject site has been carried out considering the design Serviceability Limit State (SLS), 500-year return period drought, as defined in NZBC B1/AS1 (November 2019 amendment) with reference to AS2870:2011.

Our preliminary assessment has been carried out based on the visual-tactile properties of the soils recovered from the geotechnical boreholes (logged by an engineering geologist in accordance with NZGS 2005 Guidelines) and our experience within these soil types.

Based on our assessment, we designate the natural soils as Highly Expansive, Class ‘H’, which can be assumed to have a SLS return drought characteristic surface movement (y_s) of up to 78mm.

Site specific Atterberg Limit or Shrink Swell Index testing could be carried out on the property to redesignate the expansive site class.

11 GLOBAL SLOPE STABILITY

As noted in the Site description and Geohazard Table, the concave feature to the southeast of the development may be prone to future instability, with recent instability being observed on the same topographic feature within the neighbouring property to the south.

The factor of safety (FoS) against instability for development must meet the minimum criteria as set out in the document, "The Auckland Code of Practice for Land Development and Subdivision - Chapter 2: Earthworks and Geotechnical", version 2.0 (AC CoP), May 2023.

In order to carry out the global stability assessment we have created a geological cross section through the development site and into the concave feature. The location of the geological cross section is shown on Sheet 1 as Section AA'. We have created a geological cross-section, attached as Sheet 3.

The surface profile has been generated based on the 2016 LIDAR Digital Elevation Model (DEM) data, publicly available on the LINZ website. The subsurface conditions have been extrapolated from our hand auger borehole investigation to create our ground model for the stability assessment; with geotechnical soil strength parameters detailed in Table 2 used for the ground profile.

Quantitative slope stability analysis has been carried out using the computer-based program SLOPE-W using the Spencer analysis method.

- Normal Groundwater Conditions – based on the results of our ground testing which was carried out in spring and during a historically wet year.
- Worst case groundwater – We have elevated the groundwater level within the ground model by approximately 1m and have also adopted an Ru coefficient of 0.25 for the upper very stiff alluvium to mimic partial saturation during a heavy rainfall period.
- Pseudo-static seismic loading using ULS PGA - 0.19g; as provided in Table A1 of the MBIE Guidelines – Module 1 for a ULS design level earthquake.

The minimum (critical) slip circle FoS values calculated in the slope stability analysis of Section AA', extending to the slope crest is shown in Table 4 below.

Table 4 - Slope stability assessment critical slip FoS values at building platform

Stability Model (Non-Circular)	FoS Results		
	Normal Groundwater	Worst Case Groundwater	Seismic
AC CoP minimum required FoS	**1.5**	**1.3**	**1.0**
Section AA'	1.29	1.05	1.45

As shown in Table 4, the minimum FoS against instability does not meet the AC CoP requirements for the measured and elevated groundwater modelled scenarios. Under the seismic scenario, the AC CoP minimum required FoS is achieved. We show the outputs within Appendix A of this report.

The static scenario outputs show all slip circles with less than the respective minimum required AC CoP requirements, which extend beyond the slope crest and partway into the near level land at the subject site. Based on the extent of the critical slip circles, we consider that a safe and stable building platform would need to be setback a minimum lateral distance of 13.5m back from the RL13m contour (approximately the crest of the concave feature). A restricted development line has been annotated on the Site Plan, (attached as Sheet 1) delineating this location, whereby developments behind the designated restricted development line are anticipated to be safe and stable from a geotechnical perspective.

We have also assessed the steep slope which is located beyond the eastern boundary and descending to Wellsford Valley Road. This appears to have been excavated historically as part of road formation. The slope appears to have performed well and comprises dense vegetation. Irrespective of this, our experience is that cut slopes can regress to a stable grade. There is a risk of erosion in the event of river flooding and therefore we have adopted a regression angle of this slope (assumed alluvium) of 1V:3H. The setback line continues above this slope, based on the anticipated slope regression.

The recommendations provided in this report are for a development located upslope (northwest and west) of the restricted development line only.

This does not preclude development extending into the restricted development setback zone; however, additional geotechnical analysis and recommendations will be required if any proposed development extends past the restricted development line.

It is likely that an in-ground palisade wall, designed for potential slope movement and associated loss of support is required if construction within the restricted development zone is preferred. We would recommend machine borehole drilling is required to confirm the quality of the dense soils at depth as part of a future investigation.

12 SITE FORMATION WORKS

12.1 Non-Engineered Fill

As we have previously described in this report, a surficial veneer of non-engineered fill has been recently placed at the property. This non-engineered fill comprises topsoil constituent and is not considered suitable to support foundations. All non-engineered fill must be removed beneath any ground bearing foundation system to expose natural soils and replaced with engineered fill. We recommend that a well graded aggregate fill (GAP40 or approved alternative) be used as the engineered material, with compacted fill subject to geotechnical testing.

Prior to filling, the subgrade must be checked by a geotechnical professional.

12.2 Floor Slab Preparation

As stated previously in Section 10, the natural soils at the site do not meet the criterion of “good ground” due to their expansive properties. Although the foundations will be specifically designed to consider the shrink-swell characteristics of the site soils, it is also vitally important (particularly when a concrete slab is proposed), to ensure the subgrade is prepared adequately before pouring. If the sub-slab soils are allowed to dry out during construction (particularly in summer or if kept open for an extended period of time), the natural soils can undergo shrinkage, with resultant reswelling in the months following the slab pour, which can result in ‘hogging’ and cause potentially irreversible damage to the foundations.

To mitigate against these effects, we recommend a layer of compacted hardfill (no less than 100mm in thickness) be placed on the exposed subgrade immediately following building platform preparation. We also suggest that the hardfill is wetted down prior to pouring of concrete should a period of dry weather be encountered leading up to slab construction.

13 PRELIMINARY GEOTECHNICAL RECOMMENDATIONS FOR FOUNDATIONS

As noted in Section 3, we have not been provided with proposed development plans. Consequently, the recommendations provided below must be treated as preliminary only. Once development plans become available, they must be subject to review by a geotechnical consultant familiar with the contents of this report to confirm that the geotechnical recommendations remain valid.

We provide the following recommendations on the basis that the building platform is located west/upslope of the building restriction setback zone as defined by the results of the stability analysis and our site assessment.

We have assumed that the proposed development will comprise NZS3604:2011 type structures (i.e., typically lightweight construction materials).

Shed and Future Building Platform

- The natural soils are considered suitable to support the loads from the proposed shed and future dwelling. A conventional concrete slab-on grade floor with perimeter strip footings or a waffle type concrete floor slab, specifically designed for the highly expansive Class H soils, with associated characteristic surface movement (y_s) of up to 78mm, as specified in Section 10, are considered suitable at the site. Alternatively, a suspended timber floor supported on piles could also be a suitable foundation type.
- Where a slab-on grade concrete floor is proposed, the perimeter strip footings will need to be taken to a minimum foundation depth of 1.2m below the finished adjacent ground surface to ensure that potential effects from shrink-swell settlements are within tolerable limits as specified int the Building Code. The same foundation embedment depths apply to a structure fully supported on piles.
- For preliminary design purposed for shallow foundations, an ultimate unfactored bearing capacity of **300kPa** may be assumed.

In addition to the above, we recommend that no filling or installation of any structures that surcharge the ground (such as water tanks) be placed within the restricted development area unless specific geotechnical input is carried out. Additionally, on site wastewater discharge should not occur below the development restriction zone.

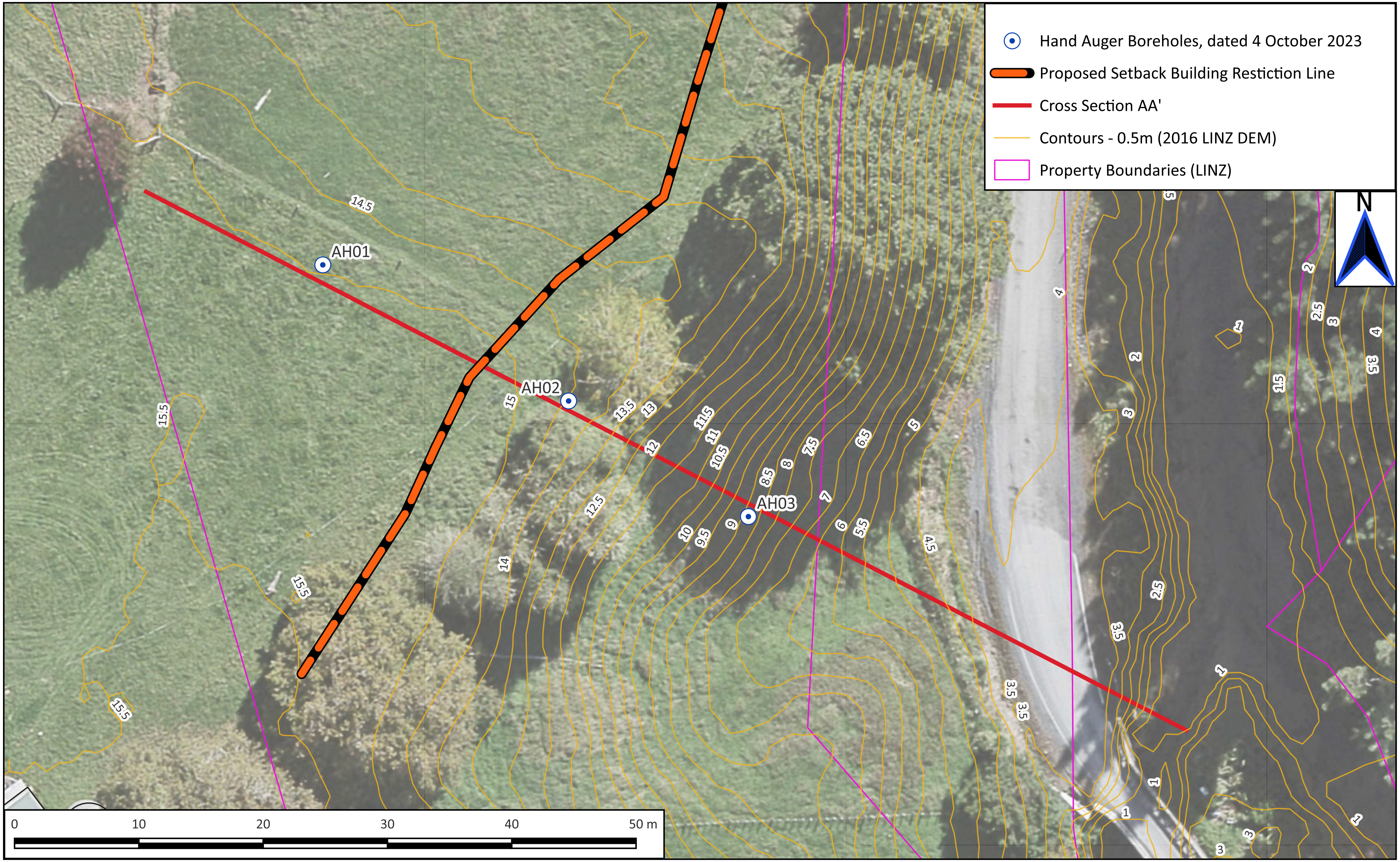
14 STORMWATER COLLECTION AND DISPOSAL

Stormwater from all hard standing areas for the proposed development must be collected and discharged in a controlled manner in accordance with current Council requirements. To ensure that stormwater does not adversely affect site stability, it is important that all stormwater runoff are collected by means of sealed pipes and discharged away from the steep slopes. If planning rules allow, stormwater should be discharged to the base of the gully via a T-bar (or similar).

15 ADDITIONAL GEOTECHNICAL INPUT

At the time of writing this geotechnical investigation report, we have not been provided with development drawings. We have designated a restricted development setback zone as shown on our Site Plan. If building beyond this zone, we recommend additional geotechnical testing and analysis be carried out to determine the quality of the dense materials at depth and to determine appropriate geotechnical slope stability mitigation measures to ensure a safe and stable development can be constructed.

If building upslope/west of the setback line, a geotechnical review of drawings must be carried out by a geotechnical professional familiar with the contents of this report to determine if any additional site testing is required or to confirm that the geotechnical assumptions provided in this report remain valid.



Location of features are approximate only. This drawing is for information only and should not be used to scale from. It is NOT intended to be used for construction purposes.

TERRATEC

Address:
263 Wellsford Valley Road
Wellsford

Title:
Test Location Plan
Geotechnical Investigation

Client:
Diana and Michael Bassick

Date: 13/10/2023
Drawn: MW
Rev: 0
Scale: 1:400
Job No: J1222
Sheet No: 1

HAND AUGER BOREHOLE LOG

TERRATEC

HOLE NO.: AH01

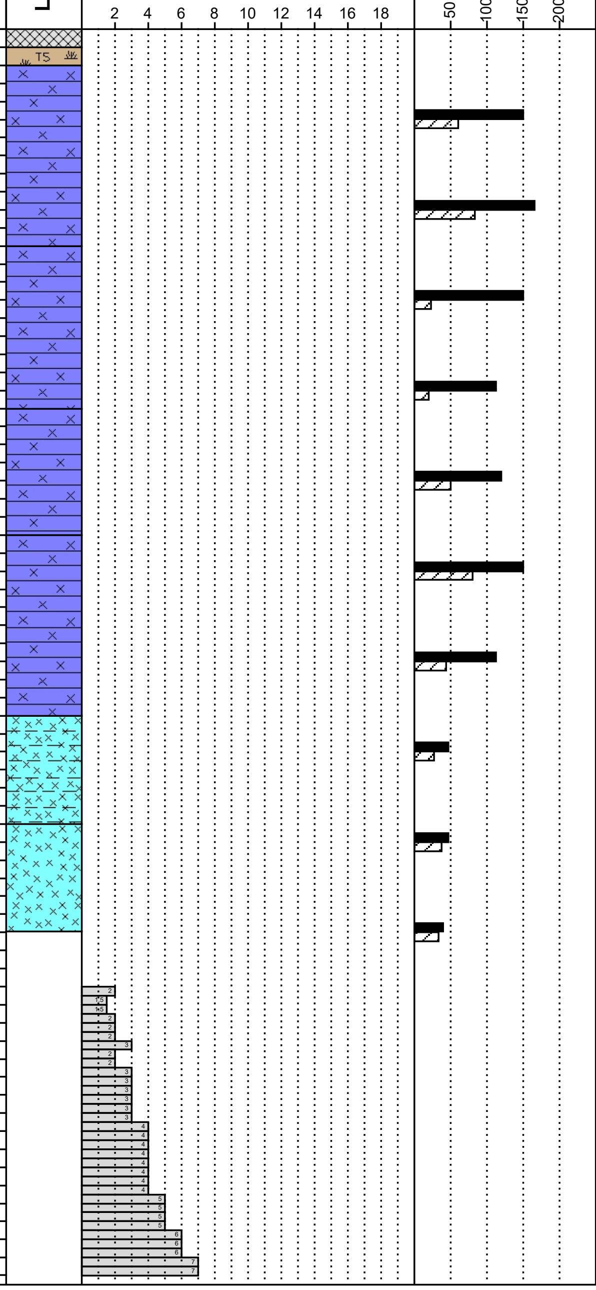
CLIENT: Diana and Michael Bassick
PROJECT: Geotechnical Investigation
JOB NO.: J1222

SITE LOCATION: 263 Wellsford Road, Wellsford
START DATE: 04/10/2023
CO-ORDINATES:
ELEVATION: Ground
END DATE: 04/10/2023
CONTRACTOR:
RIG:
DRILLER:
LOGGED BY: AF

MATERIAL DESCRIPTION (See Classification & Symbology sheet for details)
TOPSOIL & SILT; brown. Loose; non-plastic; FILL.
TOPSOIL; brown.
Silty CLAY; orange and light grey. Very stiff; moist; high plasticity; ALLUVIUM.
Silty CLAY; orange and light grey. Very stiff; moist to wet; high plasticity; ALLUVIUM.
Silty CLAY; orange and light grey. Stiff to very stiff; moist to wet; high plasticity; ALLUVIUM.
Silty CLAY; grey streaked orange and brown. Stiff to very stiff; moist to wet; high plasticity; ALLUVIUM.
Clayey SILT; bluish grey. Firm to stiff; wet; low plasticity; ALLUVIUM.
SILT, with some clay and sand; bluish grey. Firm; wet to saturated; low plasticity; sand, fine; ALLUVIUM.
End Of Hole: 5.00m (Target Depth)

SAMPLES | DEPTH (m) | LEGEND | SCALA PENETROMETER (Blows / 50mm) | VANE SHEAR STRENGTH (kPa) Vane: | WATER

Depth (m): 0.2, 0.4, 0.6, 0.8, 1.0, 1.2, 1.4, 1.6, 1.8, 2.0, 2.2, 2.4, 2.6, 2.8, 3.0, 3.2, 3.4, 3.6, 3.8, 4.0, 4.2, 4.4, 4.6, 4.8, 5.0, 5.2, 5.4, 5.6, 5.8, 6.0, 6.2, 6.4, 6.6, 6.8



Vane shear Values
150 / 60
166 / 83
150 / 23
113 / 20
120 / 50
150 / 80
113 / 44
47 / 27
47 / 38
40 / 33

Water: 04/10/2023

PHOTO(S)

REMARKS

WATER

- Standing Water Level
- Out flow
- In flow

INVESTIGATION TYPE

- [x] Hand Auger
- [ ] Test Pit

TERRATEC

HAND AUGER BOREHOLE LOG

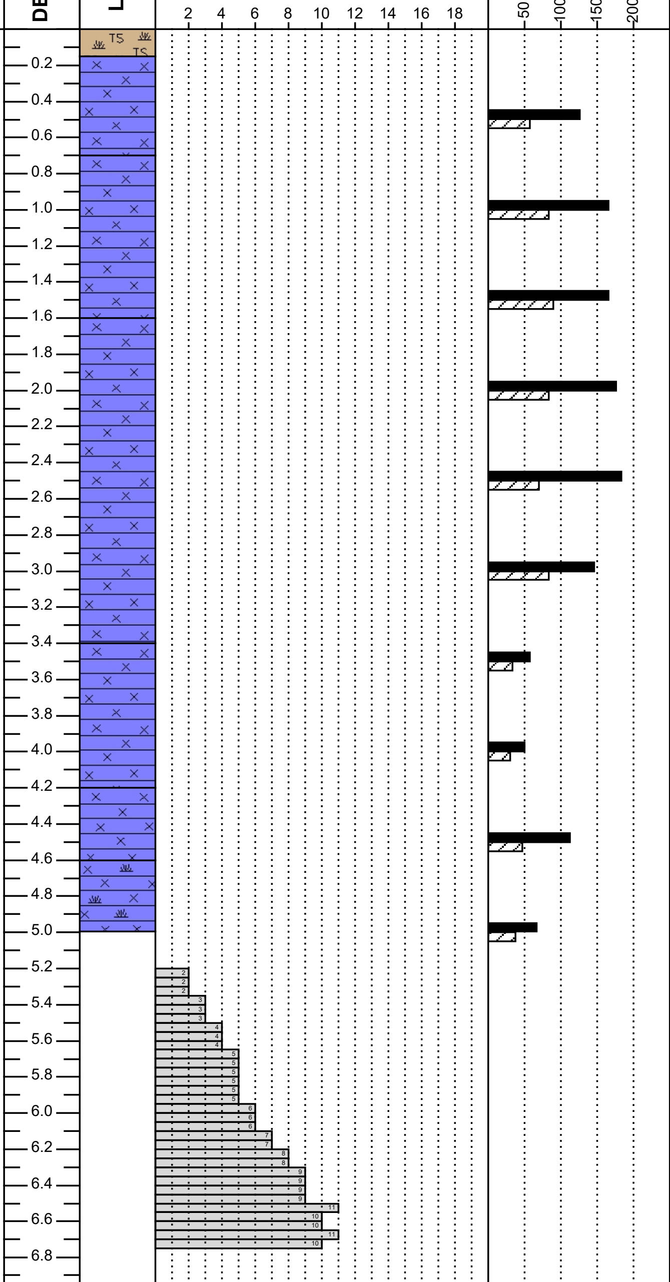
HOLE NO.: AH02

CLIENT: Diana and Michael Bassick
PROJECT: Geotechnical Investigation
JOB NO.: J1222

SITE LOCATION: 263 Wellsford Road, Wellsford
START DATE: 04/10/2023
CO-ORDINATES:
ELEVATION: Ground
END DATE: 04/10/2023
CONTRACTOR:
RIG:
DRILLER:
LOGGED BY: MW

MATERIAL DESCRIPTION (See Classification & Symbology sheet for details)
TOPSOIL; dark brown. Moist.
CLAY, with some silt; brownish grey streaked orange. Very stiff; moist; high plasticity; ALLUVIUM.
CLAY, with some silt; light grey streaked orange. Very stiff; moist; high plasticity; ALLUVIUM.
CLAY, with trace silt; light grey streaked orange. Very stiff; moist; high plasticity; ALLUVIUM.
CLAY, with trace silt; light grey streaked orange. Firm to stiff; moist; high plasticity; ALLUVIUM.
CLAY, with trace silt; bluish grey. Stiff to very stiff; moist; high plasticity; ALLUVIUM.
Silty CLAY, with some carbonaceous; brownish dark grey. Firm to very stiff; moist; low plasticity; ALLUVIUM.
End Of Hole: 5.00m (Target Depth)

SAMPLES | DEPTH (m) | LEGEND | SCALA PENETROMETER (Blows / 50mm) | VANE SHEAR STRENGTH (kPa) Vane: | Values | WATER



Values
126 / 57
166 / 83
166 / 90
176 / 83
183 / 70
146 / 83
57 / 33
50 / 30
113 / 47
67 / 37

WATER: Groundwater Not Encountered

PHOTO(S)

REMARKS

WATER

- Standing Water Level
- Out flow
- In flow

INVESTIGATION TYPE

- [x] Hand Auger
- [ ] Test Pit

HAND AUGER BOREHOLE LOG

TERRATEC

HOLE NO.: AH03

CLIENT: Diana and Michael Bassick

PROJECT: Geotechnical Investigation

JOB NO.: J1222

SITE LOCATION: 263 Wellsford Road, Wellsford

START DATE: 04/10/2023

CO-ORDINATES:

ELEVATION: Ground

END DATE: 04/10/2023

CONTRACTOR:

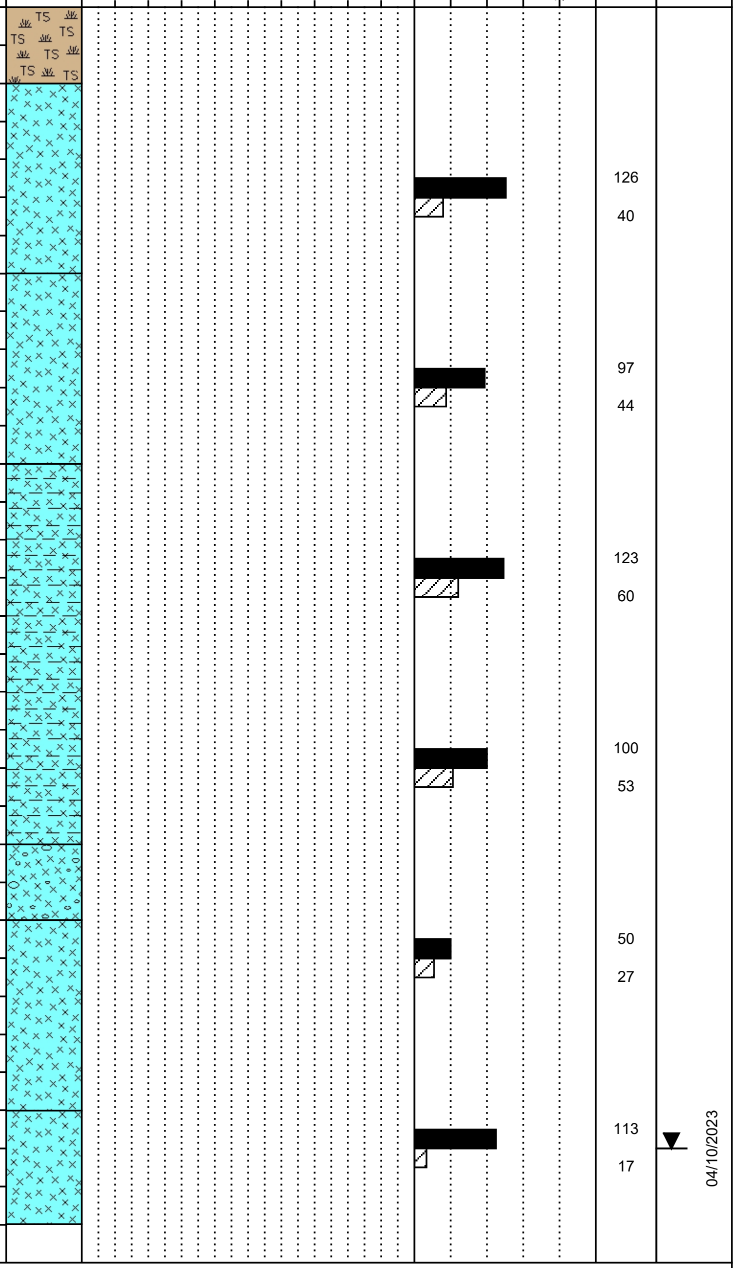
RIG:

DRILLER:

LOGGED BY: AF

MATERIAL DESCRIPTION (See Classification & Symbology sheet for details)	DEPTH (m)
TOPSOIL & SILT; brown. Firm; dry to moist; non-plastic.	0.0 – 0.2
SILT, with some clay; brownish orange. Very stiff; moist; low plasticity; ALLUVIUM.	0.2 – 0.7
SILT, with some clay; light grey and orange. Stiff to very stiff; moist; low plasticity; ALLUVIUM.	0.7 – 1.2
Clayey SILT; light grey and orange. Stiff to very stiff; moist; low plasticity; ALLUVIUM.	1.2 – 2.2
SILT, with some gravel; dark orange and brown. Medium dense; moist; non-plastic; gravel, fine, subround to subangular, moderately weathered, oxidised; ALLUVIUM.	2.2 – 2.4
SILT; greyish orange. Very stiff; moist; non-plastic; ALLUVIUM.	2.4 – 2.8
SILT; greyish orange. Firm to stiff; moist; non-plastic; ALLUVIUM.	2.8 – 3.2

End Of Hole: 3.20m (Too Dense to Auger)



Depth (m) (approx.)	Vane Shear Values (kPa)
0.5	126 / 40
1.0	97 / 44
1.5	123 / 60
2.0	100 / 53
2.5	50 / 27
3.0	113 / 17

Water: Standing water level at approx. 3.0 m (04/10/2023)

PHOTO(S)

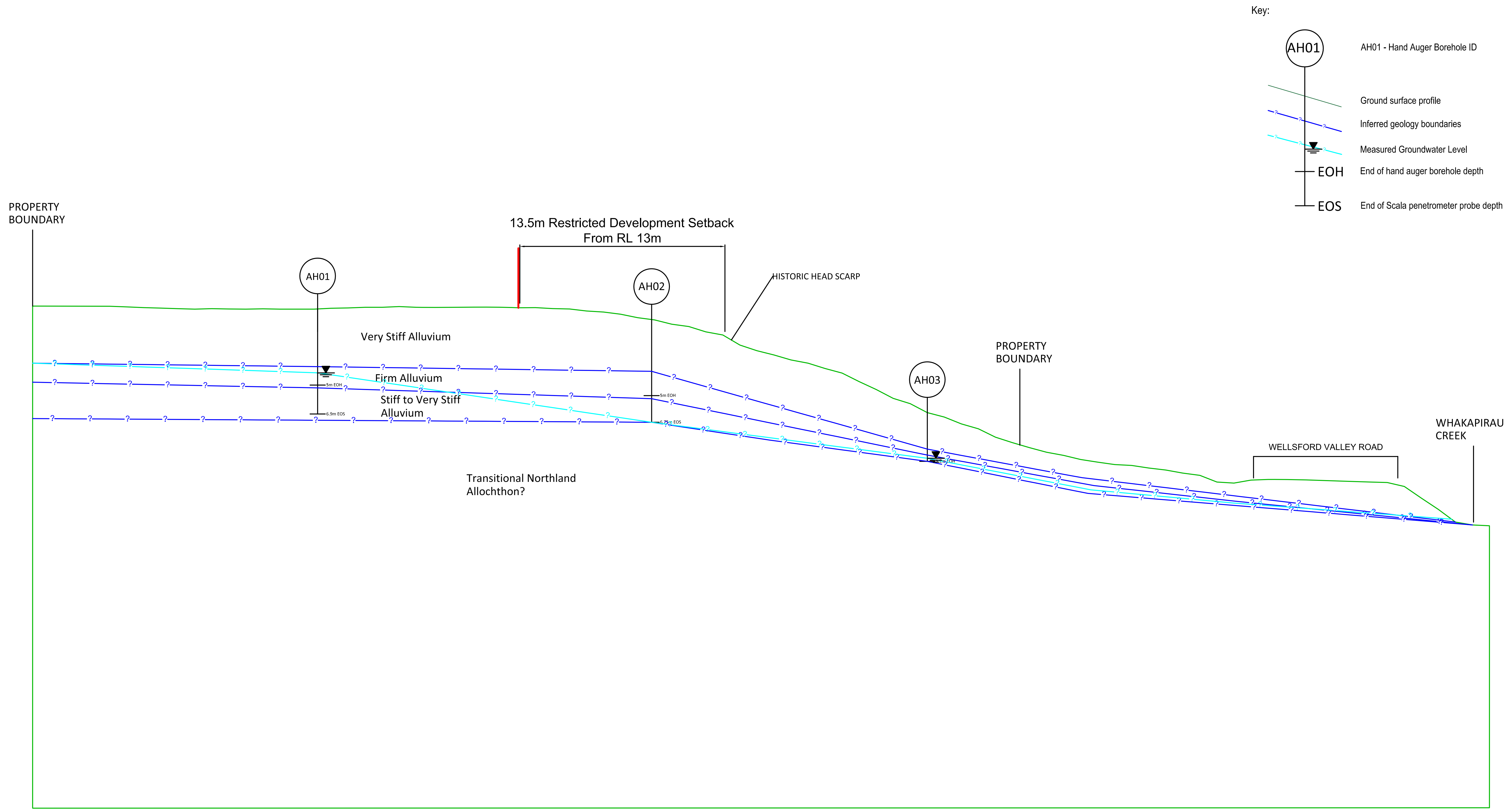
REMARKS

WATER

- Standing Water Level
- Out flow
- In flow

INVESTIGATION TYPE

- [x] Hand Auger
- [ ] Test Pit



Notes:

1. Locations of features approximate only.
2. Ground profile based on 2016 LIDAR aerial survey.
3. For detailed subsurface conditions, refer to borehole logs.

SCALE (1:250)

0 5 10 15 20 25 metres

TERRATEC

Address:
263 Wellsford Valley Road
Wellsford

Title:
Geological Cross Section AA'

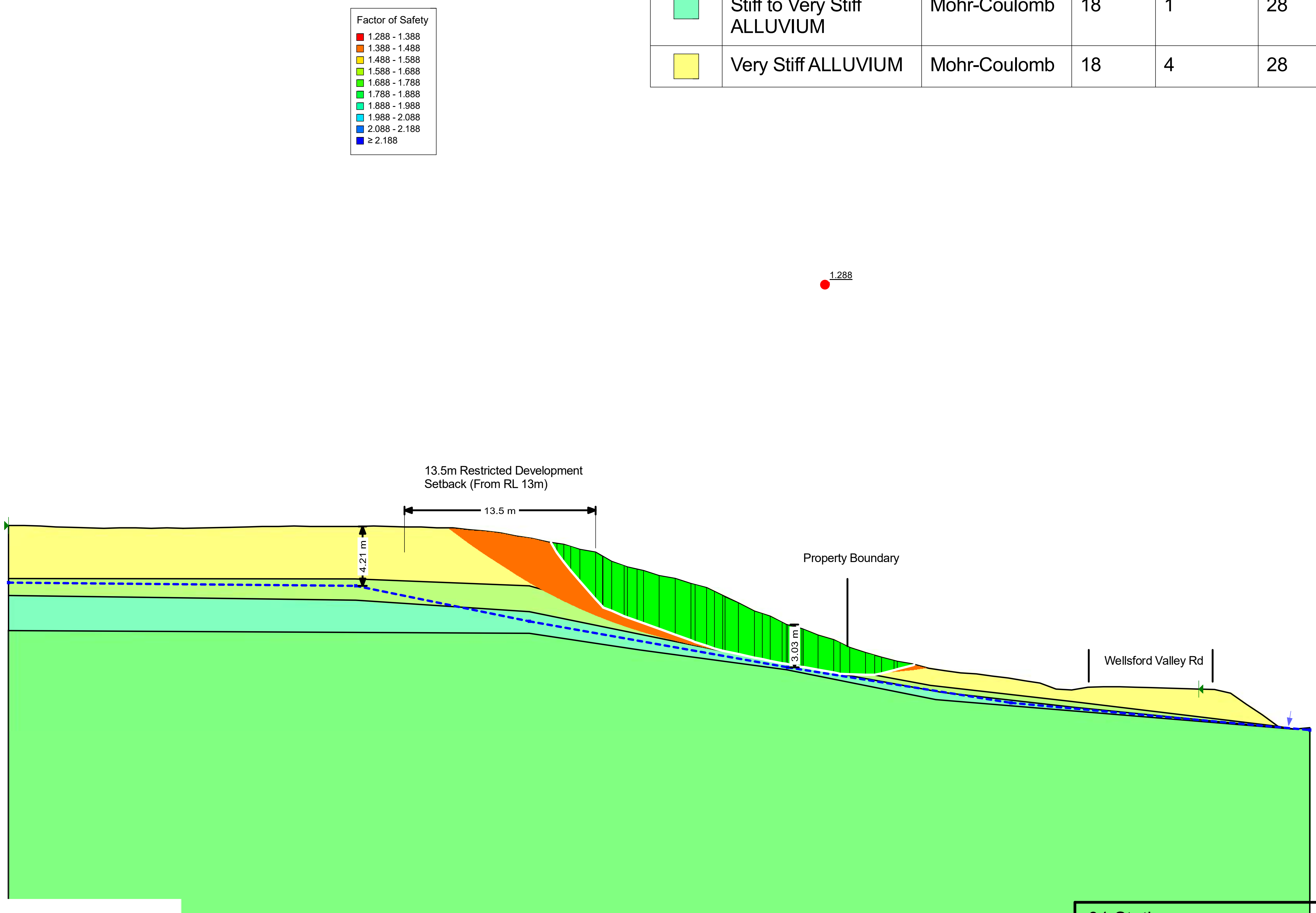
Client:
Diana and Michael Bassick

Date: 05/10/2023
Drawn: MW
Rev:0
Scale: 1:250
Job No: J1222
Sheet No: 3

Appendix A

Slope Stability Outputs

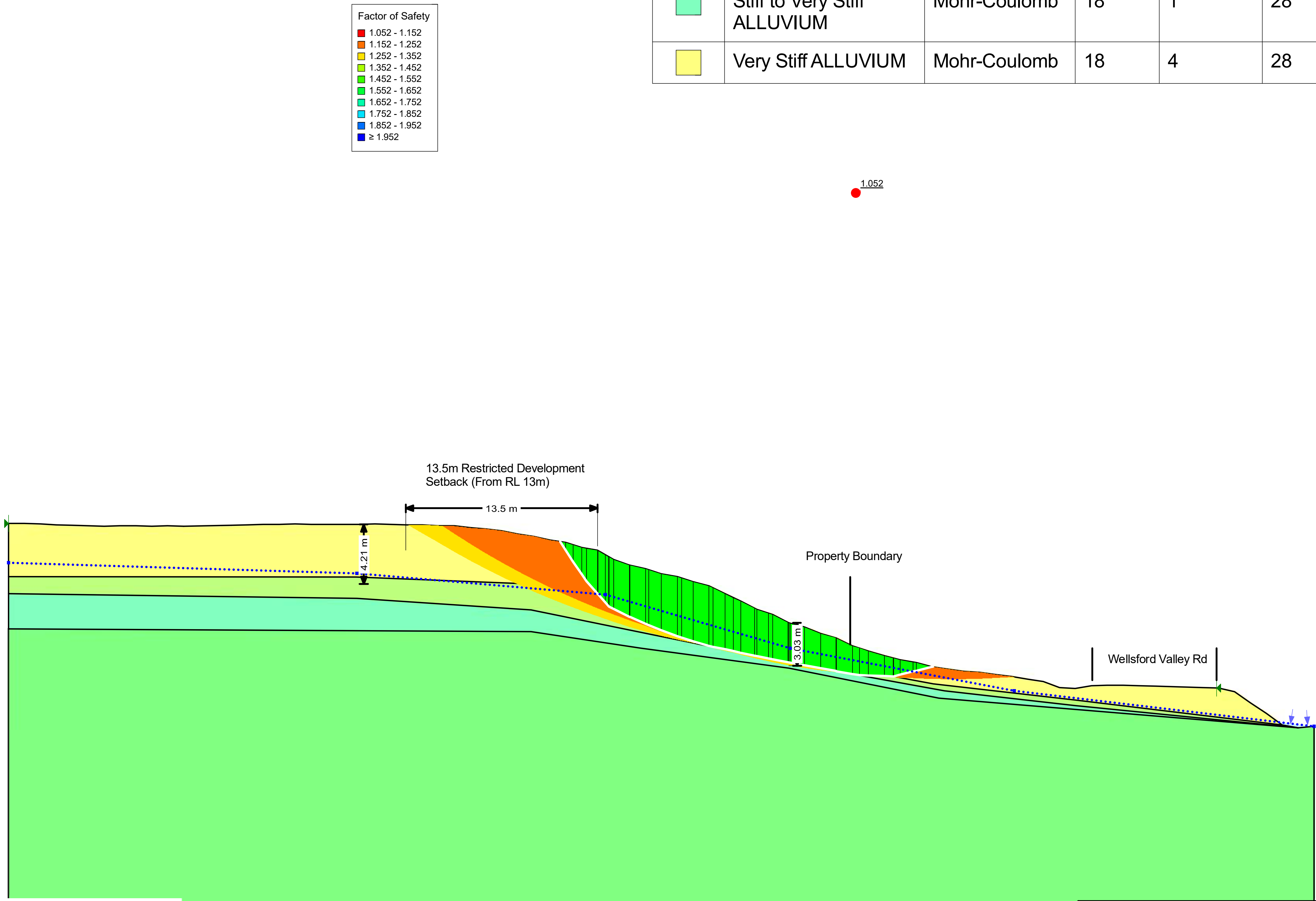
Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Phi-B (°)	Piezometric Surface
	Dense NORTHLAND ALLOCTHON?	Mohr-Coulomb	18	4	28	0	1
	Firm ALLUVIUM	Mohr-Coulomb	18	1	18	0	1
	Stiff to Very Stiff ALLUVIUM	Mohr-Coulomb	18	1	28	0	1
	Very Stiff ALLUVIUM	Mohr-Coulomb	18	4	28	0	1



TERRATEC

01 Static	
J1222.gsz	
14/10/2023	1:325

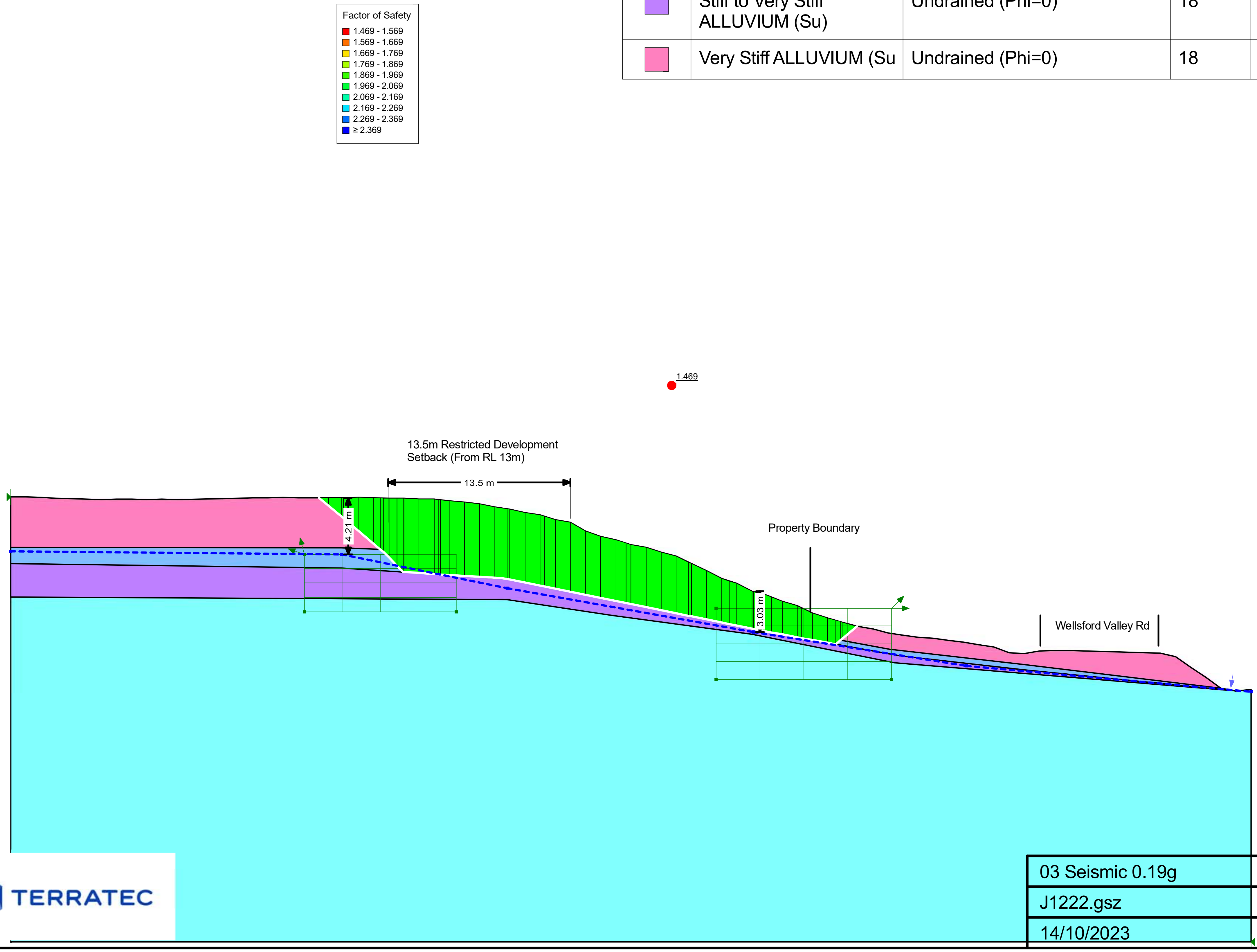
Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Phi-B (°)	Piezometric Surface	Ru	Include Ru in PWP
	Dense NORTHLAND ALLOCTHON?	Mohr-Coulomb	18	4	28	0	1		No
	Firm ALLUVIUM	Mohr-Coulomb	18	1	18	0	1		No
	Stiff to Very Stiff ALLUVIUM	Mohr-Coulomb	18	1	28	0	1		No
	Very Stiff ALLUVIUM	Mohr-Coulomb	18	4	28	0	1	0.25	Yes



TERRATEC

02 Elevated Groundwater	
J1222.gsz	
14/10/2023	1:325

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Total Cohesion (kPa)	Piezometric Surface
(cyan)	Dense NORTHLAND ALLOCTHON? (Su)	Undrained (Phi=0)	18	100	1
(light blue)	Firm ALLUVIUM (Su)	Undrained (Phi=0)	18	25	1
(purple)	Stiff to Very Stiff ALLUVIUM (Su)	Undrained (Phi=0)	18	60	1
(pink)	Very Stiff ALLUVIUM (Su	Undrained (Phi=0)	18	80	1



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03 Seismic 0.19g	
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Appendix B

Terratec Limitations

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