Tony Head

127 Onga Onga Road, Waipawa, 4272

Preliminary Geotechnical Investigation Report

210418
4 November 2021

Tony Head

127 Onga Onga Road, Waipawa, 4272

Preliminary Geotechnical Investigation Report

Prepared by:

Lance Cowan
Engineering Geologist
BSc, MSc (Hons), PGDipSci

Reviewed and Approved for Release by:

Ian Jennings
Chartered Geotechnical Engineer
MEng Sc (Geotech), BSc (Geology), CPEng, CMEngNZ

Date:	4 November 2021
Reference:	210418
Status:	Final
Revision:	NA
Previous Revision Date:	NA

TABLE OF CONTENTS

1. INTRODUCTION

Cheal Consultants Ltd (Cheal) have been engaged by Tony Head (the client) to undertake a preliminary geotechnical investigation for a subdivision at 127 Onga Onga Road, Waipawa, legally described as SEC 72 BLK XI WAIPUKERAU SD.

The site is located on the southern side of the Waipawa River, approximately 1.2 km west of SH2 and 1.5 km from the Waipawa town centre (Figure 1).

The aim of this report is to:

1. Identify any geo-hazards that my impact the development
2. Determine the ground conditions across the site
3. Provide the site subsoil class and seismic parameters
4. Assess the bearing capacities for building platforms
5. Provide earthworks advice where applicable

This report has been prepared in general accordance with NZS 4404:2010 and is intended to support an application to the Central Hawke's Bay District Council (CHBDC) for resource consent for subdivision.

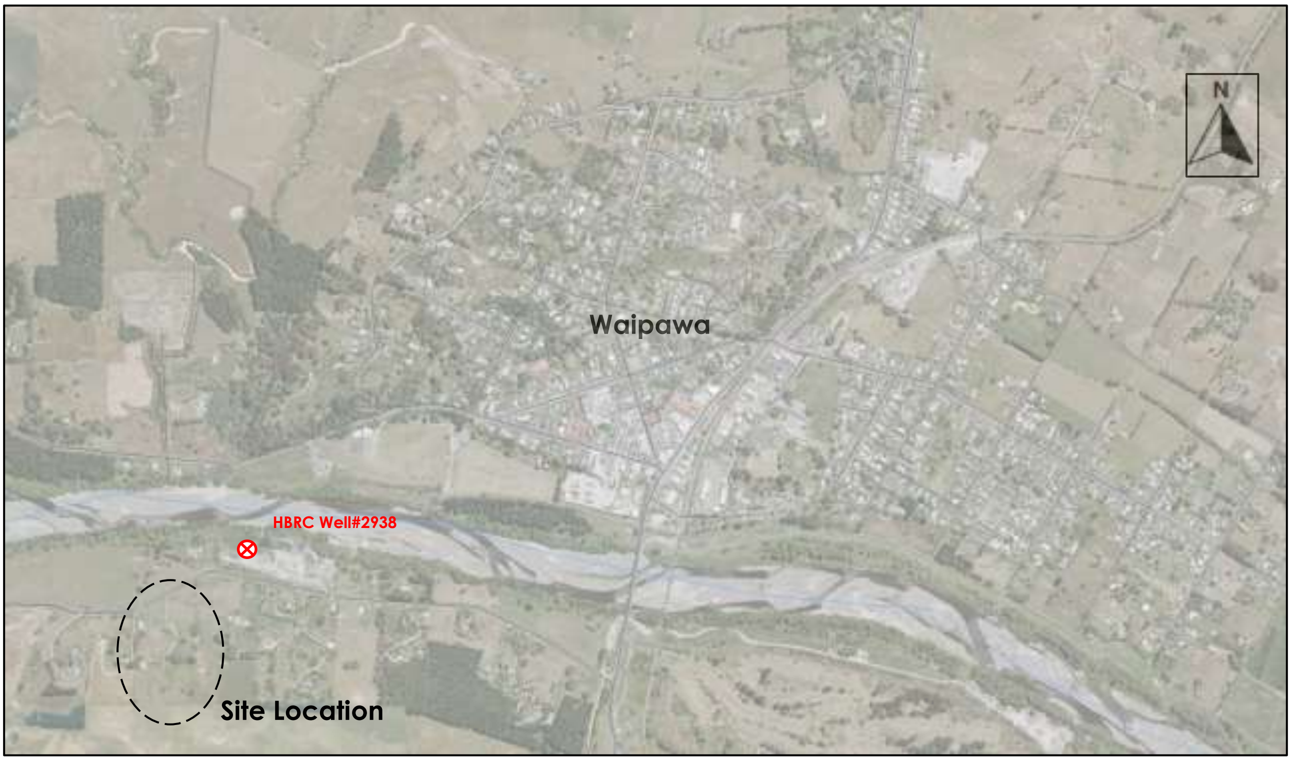


Figure 1: Site location

The proposed subdivision is shown in the Cheal scheme plan, labelled Dwg: 210418-SC001 Rev. A dated 14 May 2021. A cropped copy of the scheme plan is included as Figure 2.

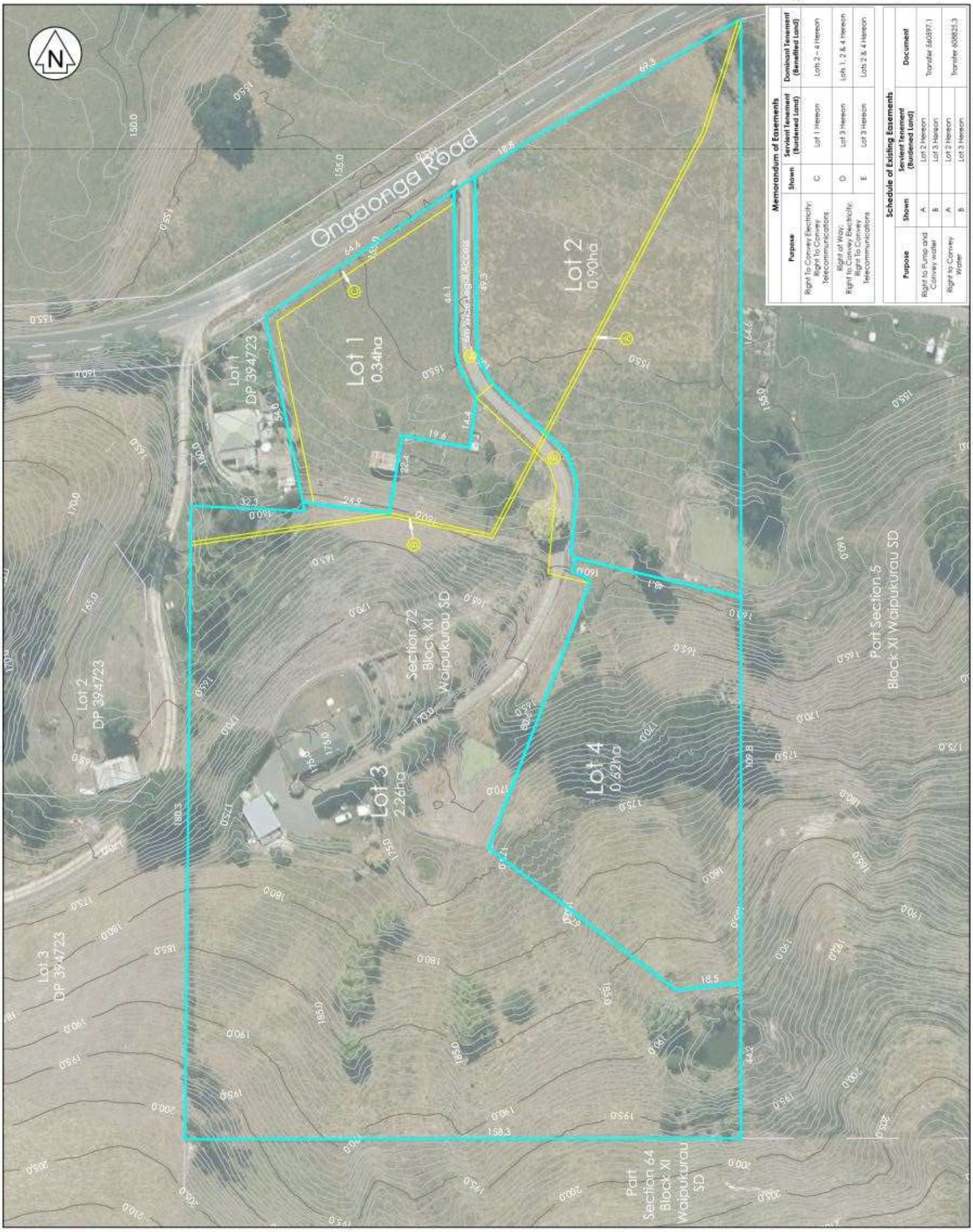


Figure 2: Cheal Scheme Plan (Cropped Copy)

In general, the site slopes down from the southern boundary and includes a flat strip approximately 160m – 180m wide along the southern boundary (Onga Onga Road). The site is currently grassed farmland but includes moderate to large trees, which are sporadically located across the site. An existing dwelling is located on the western side of the property and is accessed via a driveway that comes off Onga Onga Road. This dwelling will occupy Lot 3, which covers approximately half of the subdivision area (Figure 2).

A series of ponds are located on the site and two are located along the eastern subdivision boundary. The first is on the hill side towards the south-east corner of the subdivision, and the second is towards the toe of the hill slope. Two additional smaller ponds appear to have been formed in a gully system close to the existing house.

2. DESKTOP STUDY

2.1. Regional Geology

The 1:250,000 Geological and Nuclear Sciences (GNS) New Zealand Geological Web map has been reviewed and indicates the general area is underlain by the Wanstead (Egw) and Whangai (Kiw) Formations (Figure 3). The Wanstead Formation is described as *claystone, greensand, sandstone and marl*, whereas the Whangai Formation typically consists of *alternating sandstone and mudstone*. The northern side of the site (flat area) is expected to be underlain by Holocene river deposits (mQa).

As seen in Figure 3, the site is close to the boundary of the Wanstead and Whangai Formations, which is marked by the active Glendevon Fault. This fault is approximately 50m west of the subdivision boundary and a review of the Hawke's Bay Regional Council's (HBRC) on-line Hazard Portal indicates the subdivision is not within the Fault Avoidance Zone (FAZ).

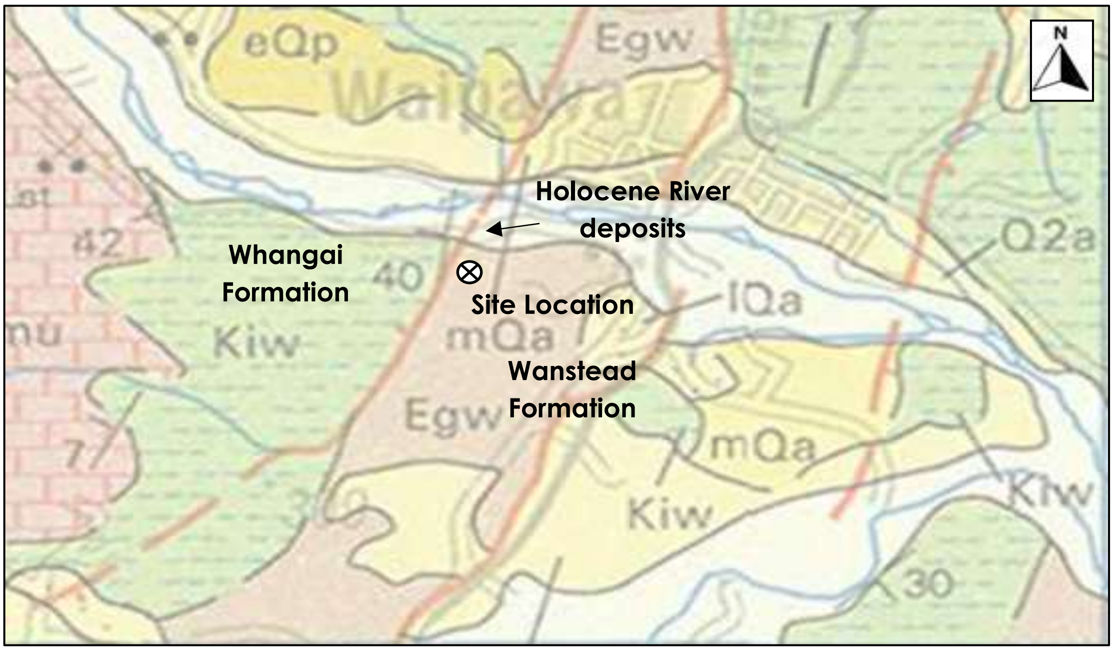


Figure 3: GNS 1:250000 Geological web map

2.2. Geological Hazards

A review of the (HBRC) Hazard Portal shows that potential for liquefaction across the flat area of the subdivision is very low, and the only significant geo-hazard that may affect the subdivision is slope stability. The portal shows the slopes that form the southern part of the site have a moderate earthflow risk.

We have reviewed historical aerial photography from retrolens.nz, and Figure 4 is a photo flown in 1943 (cropped). We have identified two landslide features affecting the site, but the photo shows these features pre-date 1943. These areas are indicated in red on Figure 4. Recent aerial photographs (i.e., Figure 2) show that there has been no significant change to the landslide features, but a pond has been developed at the top of one of the debris lobes.

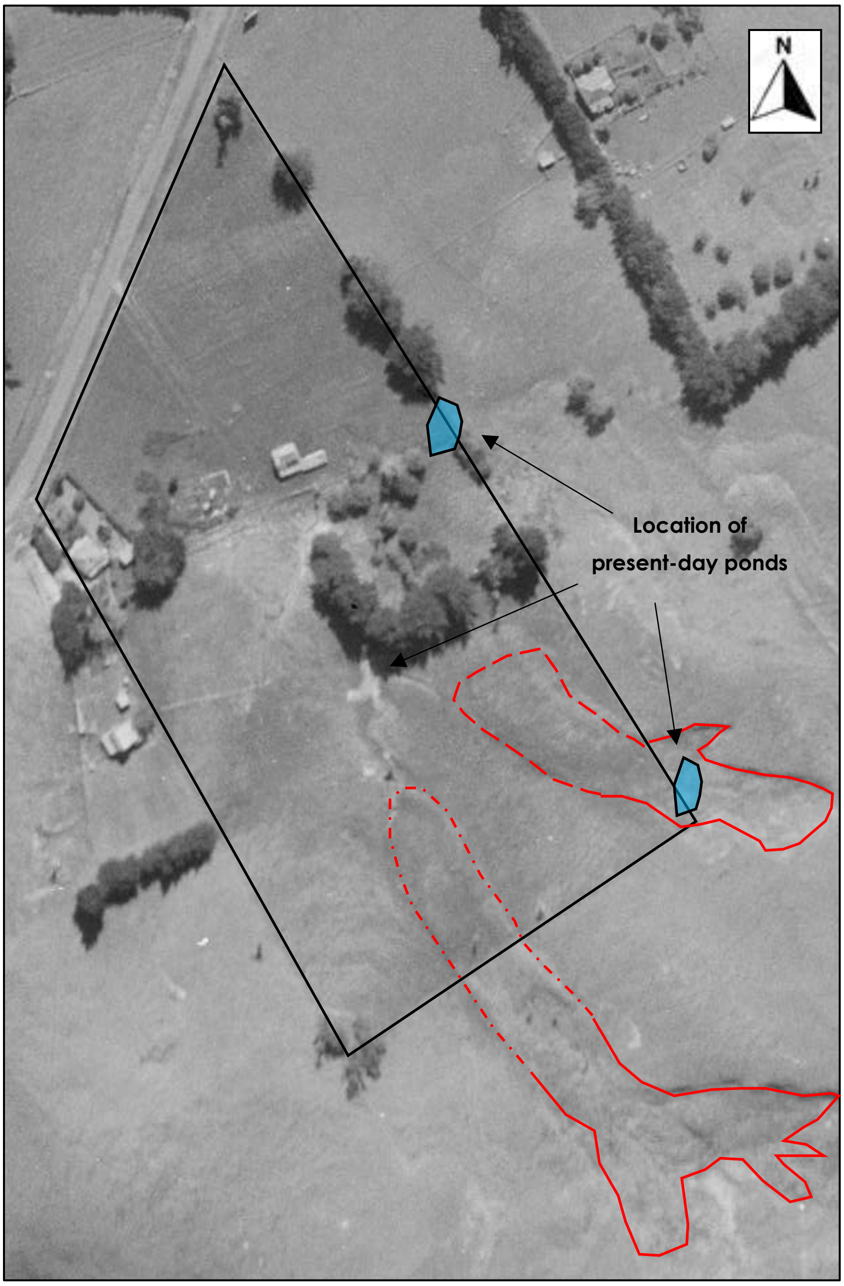


Figure 4: 1943 aerial photography (Retrolens.nz)

2.3. Existing Investigations

The HBRC Well Log data base has been reviewed and we have identified well log 2938 approximately 150m north-east of the site (see Figure 1).

The well log indicates that the Holocene alluvial deposits include an initial 2m thickness of clays and silts, which are sitting on a thin (approx. 1.0m) band of gravel. Below the gravels, clays extend to the termination depth of the well at 31m.

3. GEOTECHNICAL INVESTIGATIONS

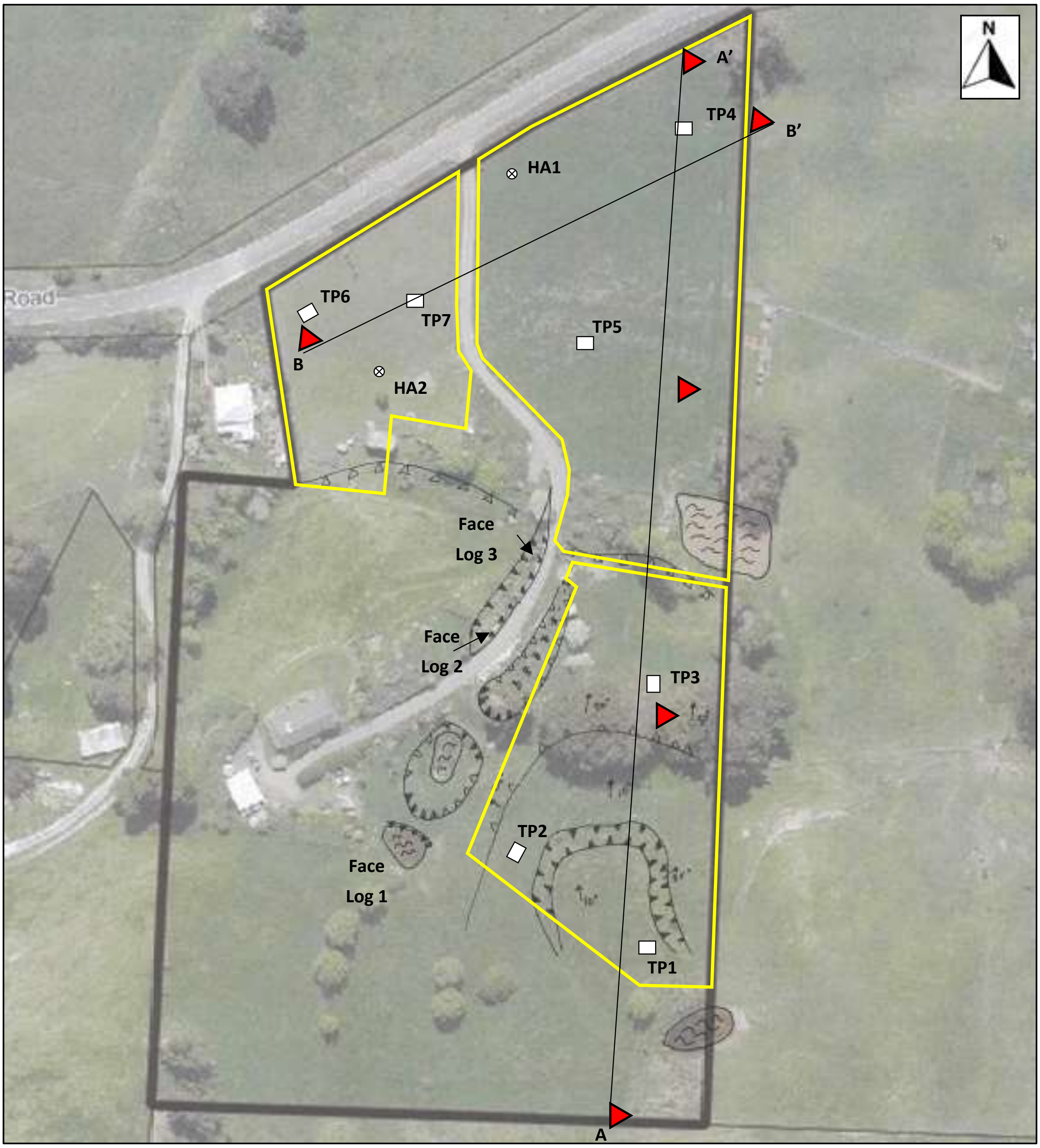
Site-specific field investigations were undertaken by Cheal on 29 June 2021 in clear conditions and included:

- 7 x Digger-excavated Test Pits (TP)
- 2 x Hand Augered boreholes (HA)
- 3 x Face Logs (FL)
- Geomorphic mapping of the site

The HA and TP locations were set out and undertaken by a Cheal Engineering Geologist and soils were described in general accordance with the New Zealand Geotechnical Society (NZGS) 'Guidelines for field description of Soil and Rock' (Dec. 2005).

Test locations are indicated on the Geotechnical Plan (Figure 5) and investigation logs are included in Appendix 1.

Geomorphic mapping was carried out by the Engineering Geologist and the main geomorphic features are include on the Geotechnical Plan using standard symbols (see Key). Typical slope angles were measured using a smart level. Natural exposures of soils were logged, and the location of 3 FLs are shown on the plan. The FLs are included in Appendix 1.



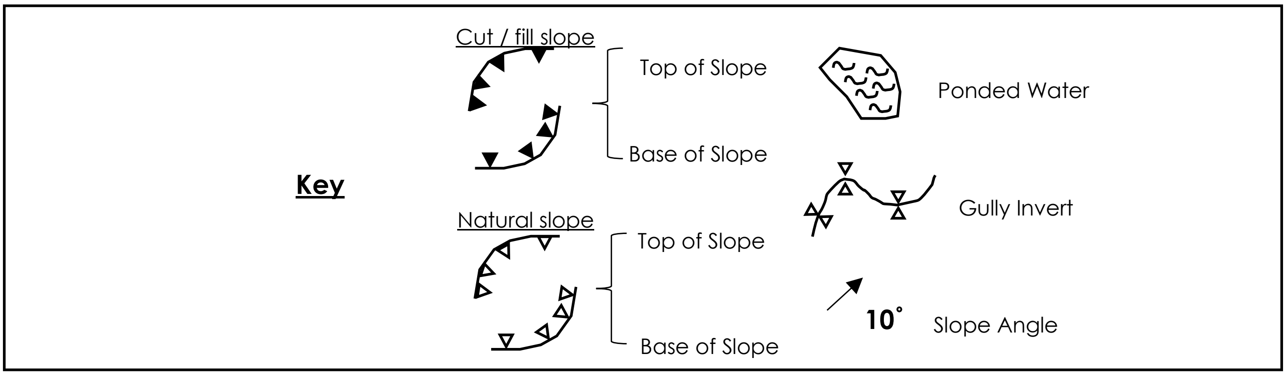


Figure 5: Geotechnical plan

4. GROUND MODEL

Based on our field investigation and background geological study, we have developed a ground model for the site, and this is shown in a series of Geotechnical Sections. Figure 6 is a section subparallel to the eastern subdivision boundary that has been extended through the riverbed to give the wider context of the ground conditions. Figures 6a and 6b are smaller scale sections (Section Aa & Section Ab) to show the more detailed ground conditions through the subdivision.

Figure 7 cuts across the flat ground at the northern end of the subdivision subparallel to Onga Onga Road. This section only shows the upper 5m of the soil profile, as this is likely to represent the depth of influence from foundation loads.

The lines of Sections Aa, Ab and Section B are included on Figure 5.

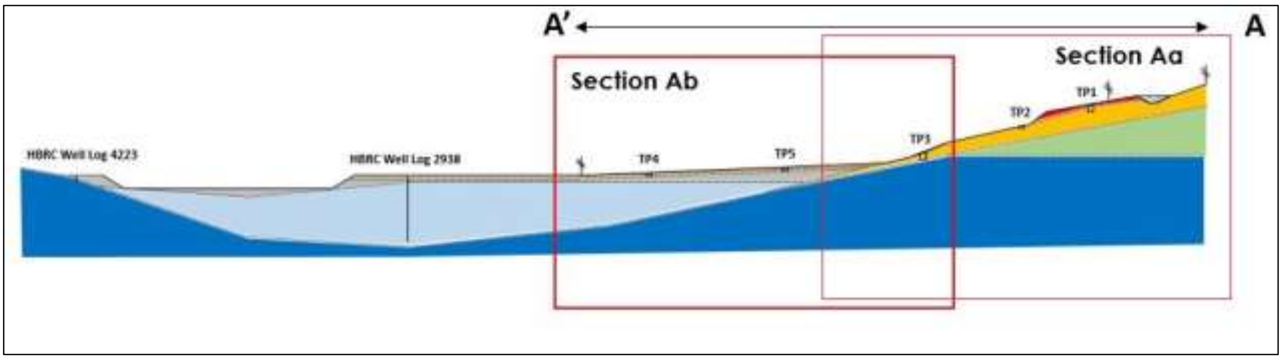


Figure 6: Geotechnical Section A'A

In general, the northern (flat) part of the site has been built up with alluvial deposits that would have on-lapped the hill slopes that form the main area of the subdivision. The alluvial deposits would tend to become thicker towards the centre of the valley and river channel. Alluvial deposits typically consist of clays and gravel, although a silty hardpan was encountered nearer to the surface. The hill slopes generally consist of clays and include areas where mobilised landslide material has been identified.

Descriptions of the main Geotechnical Units are outlined in the following sections (Section 4.1).

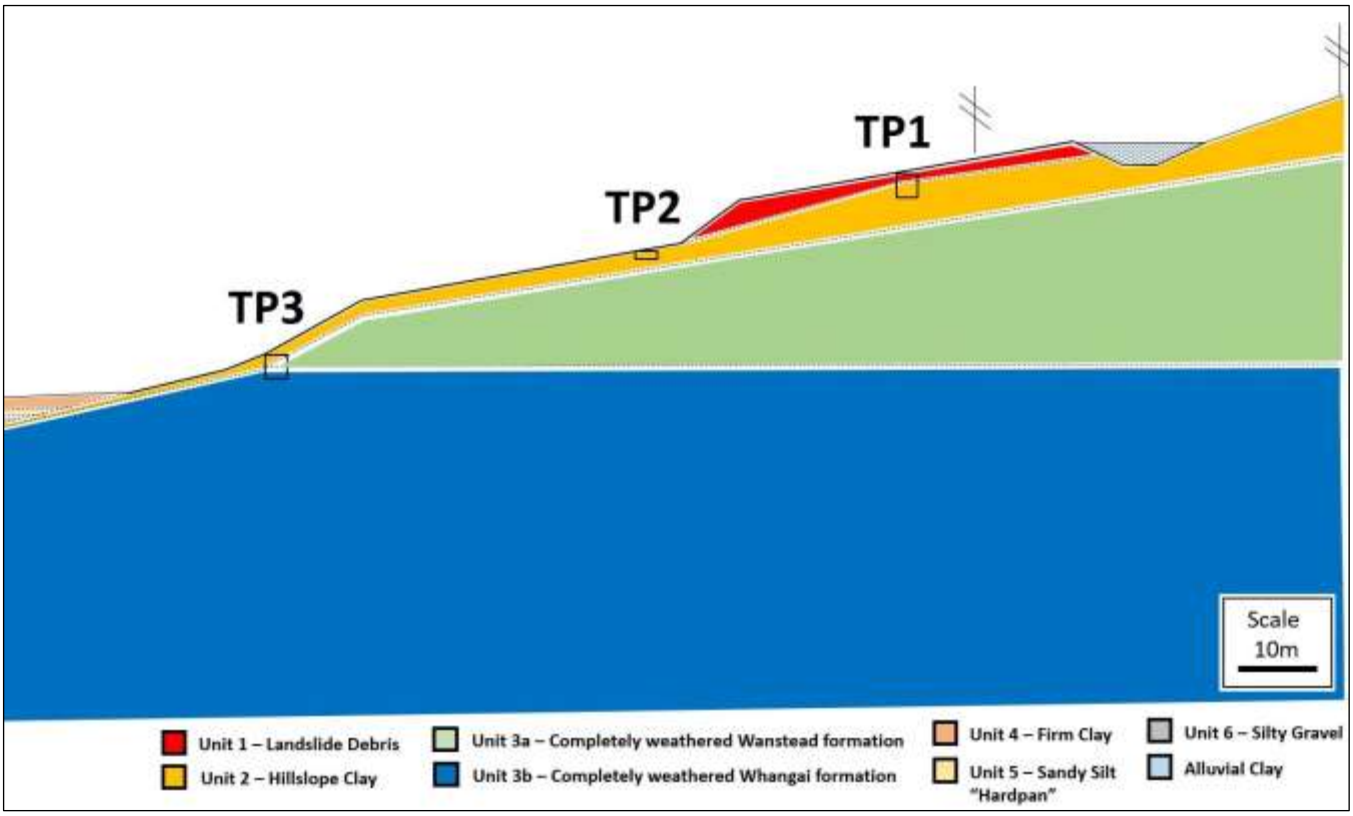


Figure 6a: Geotechnical Section Aa

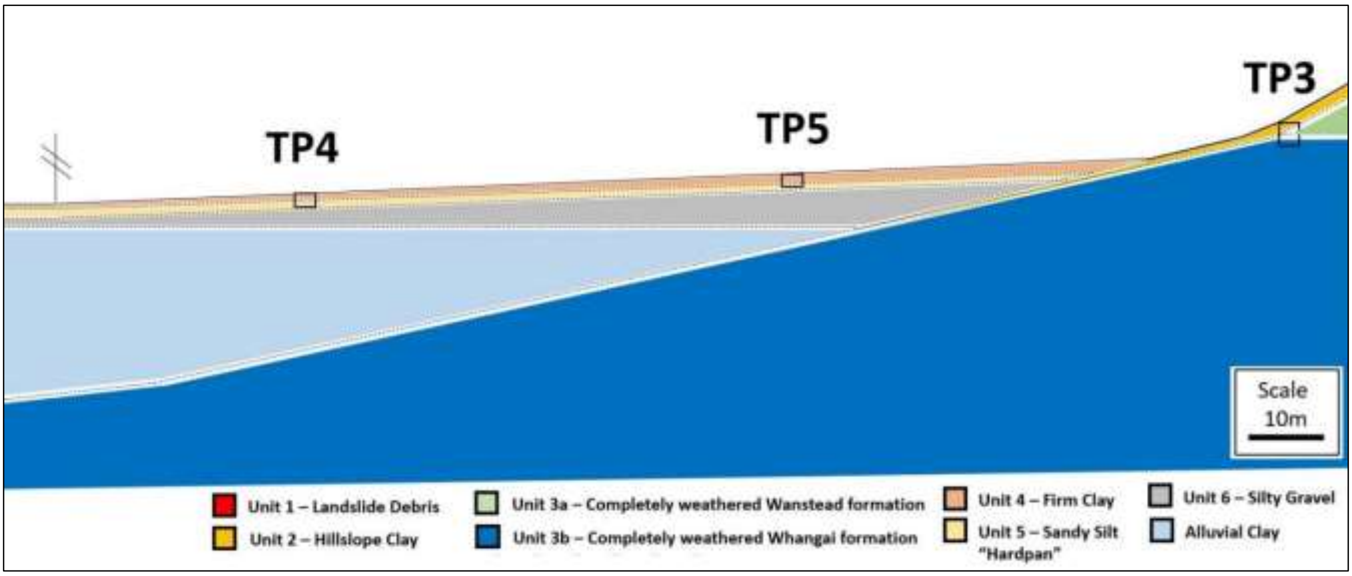


Figure 6b: Geotechnical Section Ab

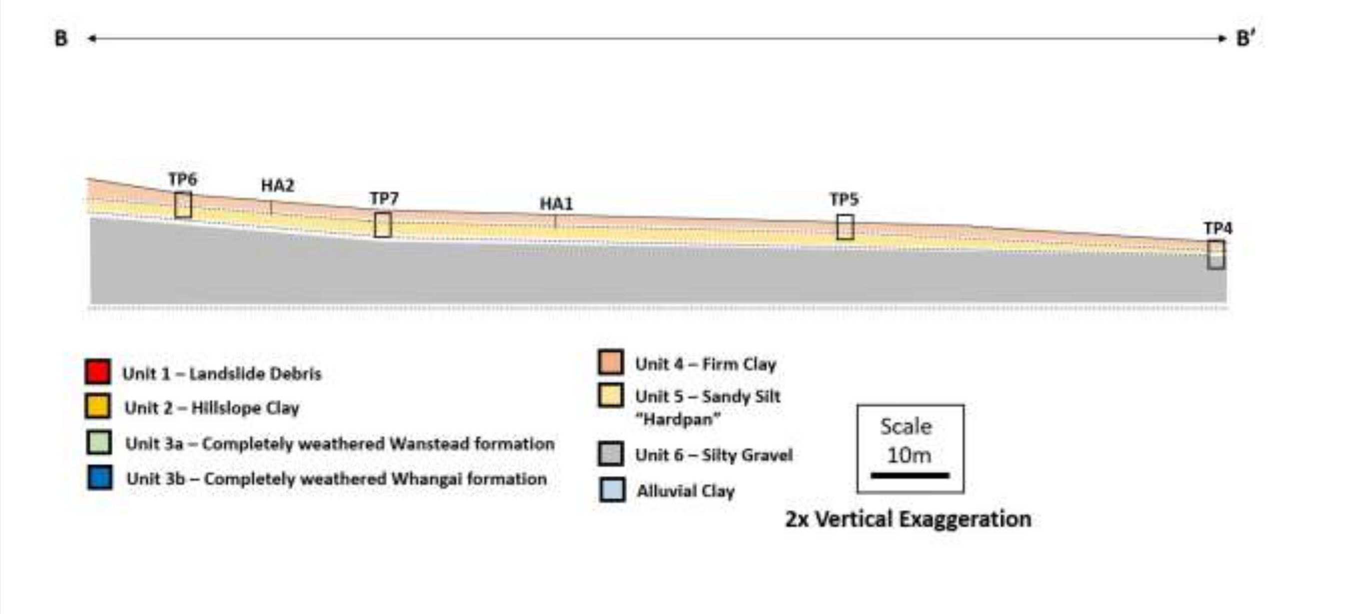


Figure 7: Geotechnical Section B'B

4.1. Geotechnical Units

Based on our site testing, we have identified 6 Geotechnical Units. These units may be separated into two subgroups based on their geomorphic environment. The first subgroup (Units 1 – 3) represent ground conditions on the hill slopes, whilst the second subgroup (Units 4-6) represent the alluvial deposits that make up the flatter area at the northern side of the site.

4.1.1. Unit 1 – Landslide Debris

This unit represents mobilised landslide material (debris lobes) and is described as highly plastic clays, with a trace of gravel. The gravels comprised 'intact' mudstone clasts, and the unit as a whole was stiff and moist. Scala measurements through this unit were 1-2 blows/100mm, however as this material has been remobilised, we consider that the strength may be highly variable.

4.1.2. Unit 2 – Hillslope Clay

Unit 2 represents the residual soil that is the natural weathering product of the underlying rock units. This unit is typically a stiff moderately-highly plastic clay. The clay extends to depths of between 3m – 7m, and Scala measurements were typically between 6-8 blows/100mm.

These higher blow counts are likely to represent the difference between the insitu peak strength of the clay, as opposed to the remoulded strength as measured in Unit 1 (Landslide Debris).

4.1.3. Unit 3 – Mudstone

Unit 3 is the moderately weathered rock mass of the Wanstead and/or Whangai Formations. Relatively unweathered materials are exposed in areas across the site (see Figure 5) and were descriptions range from hard low plastic sandy silt (Unit 3a), to extremely weak highly fractured mudstone (Unit 3b).

4.1.4. Unit 4 – Firm Clay

Highly plastic clay is present across the northern (flat) side of the site and extends to a depth of 0.7m – 1.2m. The clay is light grey with orange streaks and was described as firm. Scala measurements through Unit 4 were typically 1-3 blows/100mm.

4.1.5. Unit 5 – Sandy Silt "Hardpan"

Unit 5 is a yellowish grey 'Hardpan' and consists of a weakly cemented, low plastic sandy Silt. This unit is typically 0.8m thick and Scala measurements in this unit exceeded 20 blows/100mm. Unit 5 appears to extend across the majority the alluvial area, however is notably absent in TP4.

4.1.6. Unit 6

Unit 6 is a very dense silty gravel that was unable to be excavated with a mechanical excavator. The gravels are rounded-subrounded, and poorly graded. The gravels range from fine – coarse sized, with an estimated D_{50} of 30mm.

The base of this unit was not encountered during site testing, however the HBRC well log 2938 indicates the gravels are approximately 1m thick. Given the general shape of the alluvial deposits it is inferred that the base of this unit is between 4m - 7m below existing ground level.

The well log indicates the gravels are likely underlain by alluvial clay deposits, and it is assumed these are relatively consistent with Unit 4.

4.2. Groundwater

Groundwater was encountered in TP4 at a depth of 1.3m. Assuming a sub-horizontal groundwater level, this may increase to a depth of 4.8m (approx.) near the toe of the hill slopes.

4.3. Geotechnical Parameters

Design parameters for the main Geotechnical Units are summarised in Table 1. Values are based on our field testing, local experience, and typical ranges for similar soils. Topsoil has been excluded.

Table 1: Geotechnical Strength Parameters

Unit	Soil description	γ' (kN/m³)	c' (kPa)	ø' (°)	S_u (kPa)
1	Remoulded Clay (Landslide debris)	16	3	28	40
2	Clay (CW Mudstone)	17	7	28	80
3	Mudstone (Sandy Silt)	18	50	34	N/A
4	Clay (Alluvial)	17	5	28	50
5	Sandy SILT (Hardpan/Alluvial)	17	2	32	100
6	Silty GRAVEL (Alluvial)	19	0	36	N/A

4.4. Seismic Loading

Un-weighted Peak Ground Accelerations (PGA) and effective magnitudes have been calculated based on the NZTA Bridge Manual method (3rd edition; 2014) and the MBIE/NZGS guidelines on earthquake geotechnical engineering practice, Module 1 (March 2016), and are summarised in Table 2.

Calculations are based on:

- Site Subsoil Class 'C' (Shallow Soil) in terms of NZS 1170.0:2002
- $C_{0,1000}$ = 0.40 (Waipawa)
- f = 1.33

Return periods for an Importance Level (IL2) structure with design life of 50 years (AS/NZS 1170.0:2002 – Structural Design Actions) are:

- Serviceability Limit State (SLS1) – 1/25 years: R_U = 0.25
- Ultimate Limit State (ULS) – 1/500 years: R_U = 1.0

Table 2: Ground Motion Parameters

Limit State	Earthquake Magnitude	PGA
SLS1	6.25	0.10g
ULS	6.75	0.41g

This method is recommended for liquefaction and slope stability. For structural actions in terms of NZS 1170.0: 2004, PGAs should be recalculated using a Hazard Factor (z) of 0.41 for Waipawa.

5. GEOTECHNICAL DESIGN ADVICE

Based on the Cheal scheme plan (Figure 2), it is likely that building platforms for Lot 1 or 2 will be formed on the flat area along the southern extent of the subdivision, or possibly the relatively gentle slopes that form the toe of the hillside. Lot 4 covers steeper slopes and it is likely that earthworks will be required to form this platform and access.

It is our understanding that no further development of Lot 3 will occur as part of the subdivision process.

The following advice can be used for preliminary design. However, we consider that each lot will need Specific Engineered Design (SED) at building consent stage, and this may require additional investigation once the location of the final building platform (and access) are determined.

5.1. Lots 1 & 2

Field investigations carried out over the flat area adjacent to Onga Onga Road indicate there is approximately 300mm of topsoil, then clay down to a typical depth of 700mm – 800mm, followed by a hard pan and gravels. Scala results through the clay are generally 2-3 blows/100mm, which indicates this area cannot be considered 'good ground' in terms of NZS 3604:2011, and any foundation design will require SED.

Based on Stockwells correlation (1977) the Scala results indicate the clays have an Allowable Bearing Capacity (All. BC) of 70 kPa. This level of capacity may not be suitable for typical NZS 3604;2011 shallow foundations, but may be suitable for typical low demand cellular foundation systems (i.e., Firth Ribraft, Cupolex or similar) that use a modified (lower) requirement for 'good ground'.

Where structures are to be founded on piles, these could be founded on the sandy silts (Hard pan) that start at a depth of approximately 700mm – 800mm. Scala results indicate a minimum All. BC of 200 kPa. This unit is relatively thin, and although it sits on a (thin) layer of gravel, we consider that 'punching failure' should be checked, and additional site-specific investigation may be required.

For driveway access across the flat area, we recommend pavement design is based on a Californian Bearing ration (CBR) of 4. However, it should be noted that any topsoil will need to be removed and subgrade is likely to be at a depth of 300mm.

5.1.1. Slopes

Where a building platform is located on the sloping ground along the south of the lots, it is likely that earthworks will be required. SED will be required for any earthworks and are likely to require additional test pits once the final location has been established. However, for preliminary design we consider that the advice provided in Section 6 can be used as a guide but needs to be verified by a geotechnical engineer for detailed design.

5.2. Lots 4

Lot 4 is located on a hill slope and contouring on the Cheal scheme plan (Figure 2) indicates the slopes range from 10° - 15° across the lot. However, there is an area of landslide debris (Unit 1) covering the upper (southern) portion of the lot (see Figures 4 & 5). We consider that this area is not suitable for residential development and recommend that no earthworks are carried out on this feature.

In general, the insitu clays across the slope (Unit 2) are stiff and Scala test results were typically between 6-8 blows/100mm, indicating these areas are equivalent to 'good ground' in terms of NZS 3604:2011.

5.2.1. Cut Batters

We have carried out a slope stability analysis using design charts for rotation failure (Mitchel 1983) and the soil parameters for Unit 2 (Clay) in Table 1. Our analysis does not allow for seismic loading, and for cut batters over 1.5m high, a more rigorous stability analysis using software such as SLIDE 2/SlopeW (or similar) should be carried out for detailed design.

The design charts for dry conditions (r_U = 0) and partially saturated conditions (r_U = 0.3) have been used and the cut batters and maximum hights that achieve a Factor of Safety (FOS) of 1.5 are listed in Table 3.

Due to the vicinity of the slump debris, and the pond(s) further up the slope, we recommend the ground conditions are assumed to be partially saturated and values for r_U=0.3 are used.

Table 3: Cut Batters

Batter	Max Height (m)	
	r_U=0.0	r_U=0.3
1.5H:1.0V	7.4	4.0
2.0H:1.0V	18	5.5
2.5H:1.0V	55	9.0

5.2.2. Fill

Any engineered fill shall be carried out in accordance with NZS 4431:1989 (Code of Practice: Earthfill for Residential Development).

It is likely that any fill will be associated with cut and the material would be consistent with the clay (Unit 2) outlined in Section 4.1.2. This material should be suitable, but care will be required to compact the material close to its Optimum Moisture Content (OMC).

All topsoil shall be stripped from the site prior to any fill being placed. Where the fill is placed on a slope greater than 10° the footprint will need to be benched, and where the total thickness of the fill is greater than 2.5m, we recommend a shear key is cut along the toe of the fill. This key should be a minimum of 1.2m deep and wide enough that compaction equipment can operate effectively along the base.

Where any sign of seepage is detected, a subsoil pipe with a filter sock should be installed on the stiped/benched surface.

The clay shall be spread in lifts not exceeding 200mm thick (loose) and compacted to 95% of MDD, and within 2.5% of OMC. For construction, sampling and laboratory testing will be required to provide Maximum Dry Densities (MDD) and OMCs for compaction control testing (Compaction Curve – NZ Std Comp.). Additional samples and testing will be required where more than one soil type is used.

During construction, fill density shall be measured using a Nuclear Densometer (NDM) with 5 individual readings taken across each lift. NDM testing shall be carried out at 0.6m lifts measured from the toe of the fill/top (or the base of the shear key), and on the finished surface. Once densities are achieved and a constant compaction methodology is established, a correlation between density (NDM) and Shear Strength (Hand Held Sheal Vane) could be established for monitoring the fill.

It is likely that the compacted clay will have strength parameters similar to those outlined in Table 1 for Unit 2 (Clay) and therefore based on the stability assessment carried out for the cut batters we consider that fill batters of 2.0H:1.0V up to a hight of 4m would be appropriate. Note, these represent a maximum slope and the landscaping and ease-of-access across the batter may require that the batter is reduced.

Our analysis doesn't allow for seismic loading and final fill batters higher than 1.5m may require a more rigorous stability analysis as outlined above (Section 5.2.1).

5.2.3. Bearing Capacity - Earthworks

Any excavation into the slope is likely to expose the clays (Unit 2) of the underlying weathered mudstone (Unit 3). As outlined above, Scala testing through the clays exceed the usual level for 'good ground' in terms of NZS 3604:2011 (5 blows/100mm), and we would expect the weathered mudstone to be better still.

Where fill has been carried out in accordance with NZS 4431:1989 (and the advice above) it could be certified and would be considered equivalent to 'good ground' in terms of NZS 3604:2011.

cheal

Where an access is being formed across the slope by cut and fill, we consider a CBR of 10 can be used. However, it should be noted that this represents a 'drained' value, and the formation will require a good level of water control to maintain this state. Where the subgrade could become wet, we recommend using a 'soaked' CBR of 3, but this should be confirmed at detailed design by laboratory testing on site specific samples.

5.2.4. Settlement

Load-induced settlement from placing fill will be dependent on the location of the fill and the intended thickness. Given the likely extent of the fill and the stiff nature of the clay (Unit 2) on the hill slopes, we do not consider that settlement will be a significant issue. However, the designer will need to consider this further during the development of the final earthworks design.

5.3. Infiltration

No infiltration testing has been carried out for wastewater or stormwater purposes, but in general we would expect the clayey soils to be relatively impervious.

We have assessed the likely soil texture and soil category in accordance with AS/NZS 1547:2012 by comparing our field engineering descriptions with a standard Soil Textural Triangle (Figure 8). We have considered the topsoil and soil units in the upper 1.0m of the soil profile, which typically consist clays of silty clay.

The topsoil plots as 'Silt' on a soil textural triangle and is likely to be Soil Category 3 – Loam and have an indicative permeability (k_{sat}) of 1.0 – 1.5 m/day (AS/NZS 1547 – Table 5.1).

The underlying soil units plot as 'Clay' and are likely to be Soil Category 6 – Medium to Heavy Clays. For preliminary design we recommend an indicative permeability (k_{sat}) of 0.06 m/day, but this should be verified with field testing once the location of any disposal field are determined.

Based on local experience the risk of shrink/swell behaviour for the clays is low.

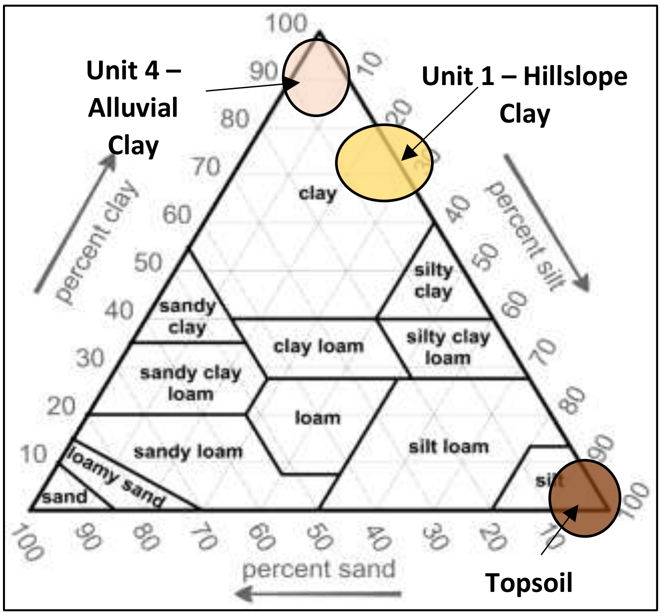


Figure 8: Soil Textural categorisation

6. SUMMARY & RECOMMENDATIONS

127 Onga Onga Road is located on the southern side of the Waipawa River, approximately 1.5 km from the Waipawa town centre (Figure 1). The site is mainly sloping but has a strip of flat ground adjacent to Onga Onga Road. The intention is to subdivide the property into 4 Lots (Figure 2), with the existing house occupying the largest lot (Lot 3).

This report has been prepared for the purpose of resource consent application, and it is anticipated that further geotechnical investigation and assessment will be required on a lot-by-lot basis for detailed design and building consent application.

Geotechnical investigation has been carried out and indicates the hillsides are predominantly mudstones associated with the Wanstead and/or Whangai Formations (Figure 3). The soil profile consists mainly stiff clays (residually weathered mudstone), although there are two large historic landslide features, and the slump debris is a weaker remoulded clay.

The flat ground next to Onga Onga Road is the edge of the Waipawa River alluvial deposits and comprise softer clays at the surface underlay by a silty hard pan and gravels. Nearby well logs indicate this sequence may extend to depths of 4m – 7m and are sitting on a thick unit of clay that extends to a depth of at least 31m.

In terms of NZS 1170.0:2002, we consider that Site Subsoil Class 'C' (Shallow Soil) is appropriate for the site.

Lots 1 and 2 are located on the flat ground and our testing shows that this area cannot be considered 'good ground' in terms of NZS 3604:2011. SED will be required for foundations, although low-demand cellular foundation systems, such as Firth Ribraft or Copulex (or similar), are likely to be suitable.

Where building platforms are to be excavated into the slopes, we have provided a range of batter slopes and maximum batter heights that satisfy a FOS of 1.5. These have been determined using design charts and do not include seismic loading. All earthworks will require SED, and where batters are greater than 1.5m high more rigorous analysis will be required for detailed design.

We recommend that no residential development is carried out on the large historic landslide features identified at the site.

Any fill should be constructed to NZS 4431:1989, and where the construction advice in this report is followed could be certified as engineered fill.

In general, the insitu clays across the slopes are stiff and our testing indicates these will provide 'good ground' in terms of NZS 3604:2011. Where platforms are cut into the slopes, and the clays are used as fill, we would anticipate that these would provide 'good ground' (once certified).

For pavement design we recommend a CBR of 4 for the clays across the flat areas (Unit 4) of the subdivision and 10 for the insitu clays on the hill sides. Where it is anticipated that the subgrade will become wet, we recommend a CBR of 3 for the hill slope clays.

No infiltration testing has been carried out but based on textural descriptions and indicative permeabilities listed in AS/NZS 1547:2012 the soil category is likely to be 6 (Clay) and the ground is relatively impervious.

7. DISCLAIMER

This Report has been prepared solely for the use of our client with respect to the particular brief given to Cheal Consultants Limited (Cheal).

No liability is accepted in respect of its use for any other purpose or by any other person or entity. All future owners of this property should seek professional geotechnical advice to satisfy themselves as to its ongoing suitability for their intended use.

The opinions, recommendations and comments given in this Report are the result from the application of accepted industry methods of site investigations. As factual evidence over much of the site has been obtained solely from Hand Augers, Test Pits and Scala tests, which by their nature only provide information about that exact location, there may be special conditions pertaining to this site which have not been identified by the investigation and which have not been taken into account in the report. Any groundwater levels measured during the investigation may change over time.

If variations in the subsoils occur from those described or assumed to exist, then the matter should be referred back to Cheal immediately.

CHEAL CONSULTANTS LIMITED
4 November 2021

Appendix 1

Investigation Logs

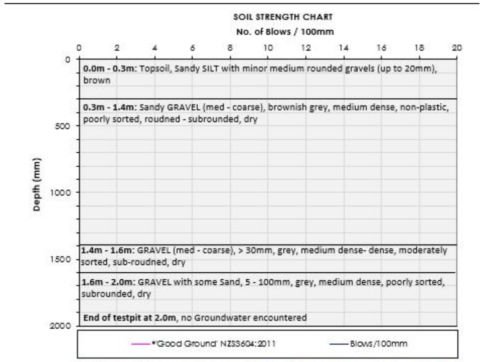
cheal

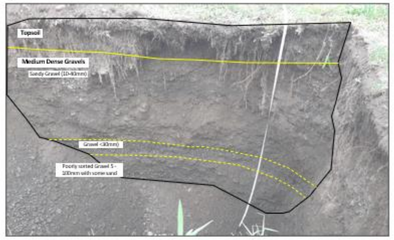
SCALA PENETROMETER TEST SHEET

CLIENT: Tony Head
JOB LOCATION: 127 Onga Onga Road, Waipawa
JOB DESCRIPTION:

JOB No: 210418
DATE:
TEST BY: LHC

DEPTH: Millimetres Below Ground Level (BGL)	TEST No. / Approximate surface RL															
	TP1	TP2	TP3	TP4	TP5	TP6	TP7	HA1	HA2	10	11	12	13	14	15	16
	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
0 - 100	1	1	2	1	1		0.5	1	1							
100 - 200	1	2	3	2	2		0.5	1	2							
200 - 300	3	2	6	1	2		1	1	3							
300 - 400	6	1	11	2	2		2	3	2							
400 - 500	20	1	9	2	1		3	2	1							
500 - 600		1	10	3	3		10	2	2							
600 - 700		2	7	9	2		20	4	5							
700 - 800		2	8	13	2			15	10							
800 - 900		5	8	20	2			20	20							
900 - 1000	5	5	6		5											
1000 - 1100	7	7	6		8											
1100 - 1200	8	8	4		12											
1200 - 1300	5	6	6		16											
1300 - 1400	9	7	8		20											
1400 - 1500	7	7	15													
1500 - 1600	7	8	11													
1600 - 1700	5	9	6													
1700 - 1800		7	9													
1800 - 1900			10													
1900 - 2000			12													
2000 - 2100			9													
2100 - 2200			10													
2200 - 2300			12													
2300 - 2400			13													
2400 - 2500			13													
2500 - 2600			14													
2600 - 2700			10													
2700 - 2800			14													
2800 - 2900			13													
2900 - 3000																
3000 - 3100																
3100 - 3200																
3200 - 3300																
3300 - 3400																
3400 - 3500																
3500 - 3600																
3600 - 3700																
3700 - 3800																
3800 - 3900																
3900 - 4000																



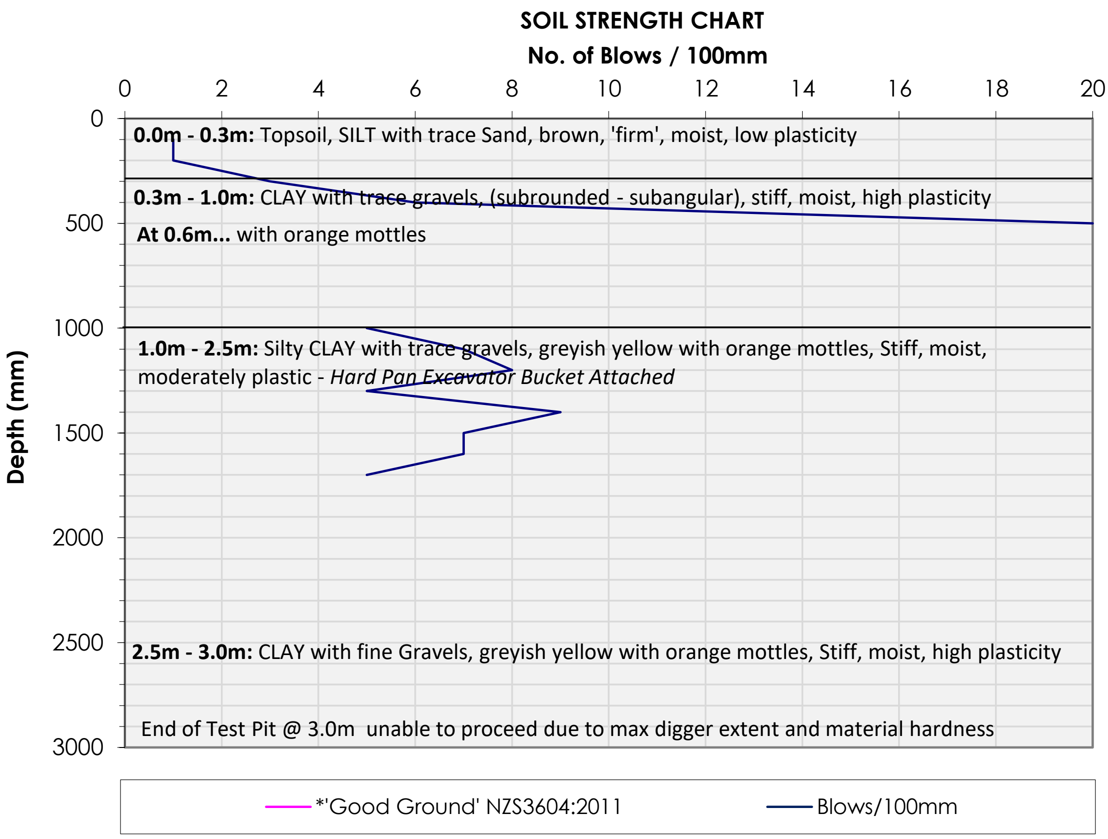


Notes:

Minimum Test Depth as per NZS 3604....................................1.0m (1000mm) or 2x width of footing below bottom of footing
Blow count for Good Ground as per NZS 3604...................5 blows/100mm to a depth of 1.0m or 2x width of footing (whichever is the greater)
Blow count for Good Ground as per NZS 3604...................3 blows/100mm below the above depths
Blow count for 100kPa Allowable Bearing Pressure...........4 blows/100mm

Test Pit 1

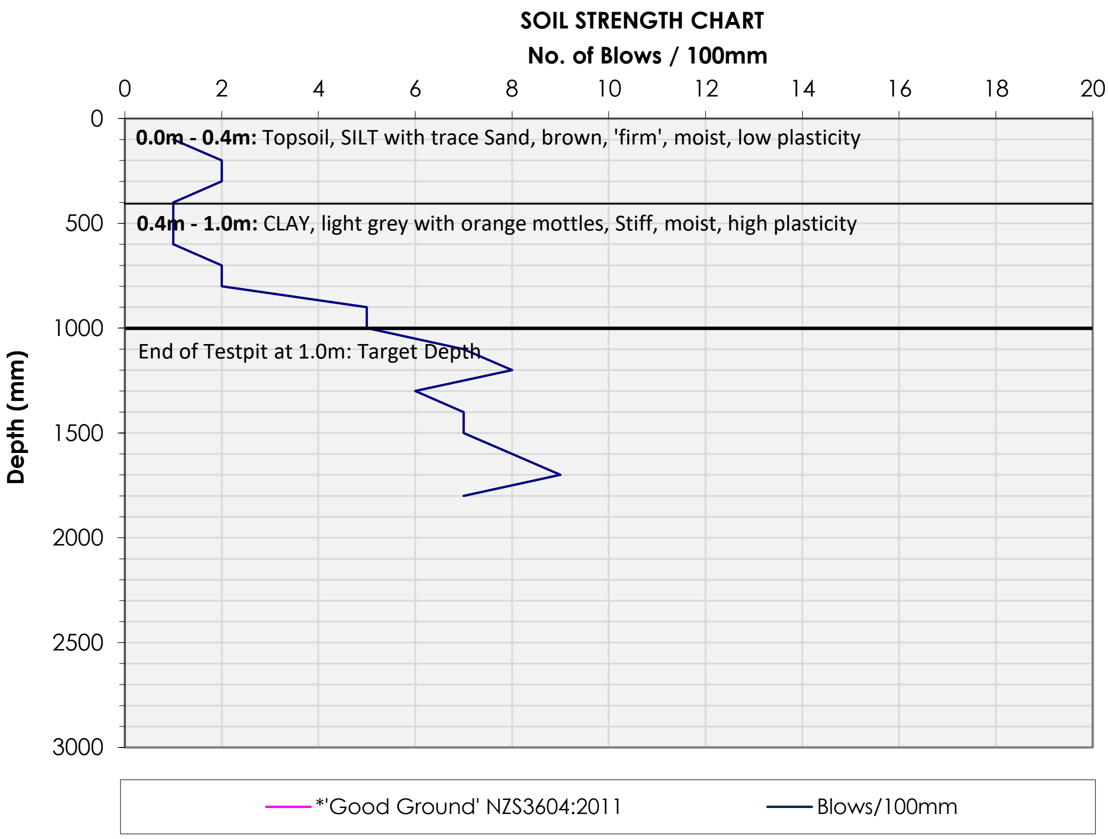
Job No.	210418 | Tony Head		
Location:	127 Onga Onga Road, Waipawa		
Test No.	TP1	Starting Depth:	
Date:	0/01/1900	Test by:	LHC



Test Pit 2

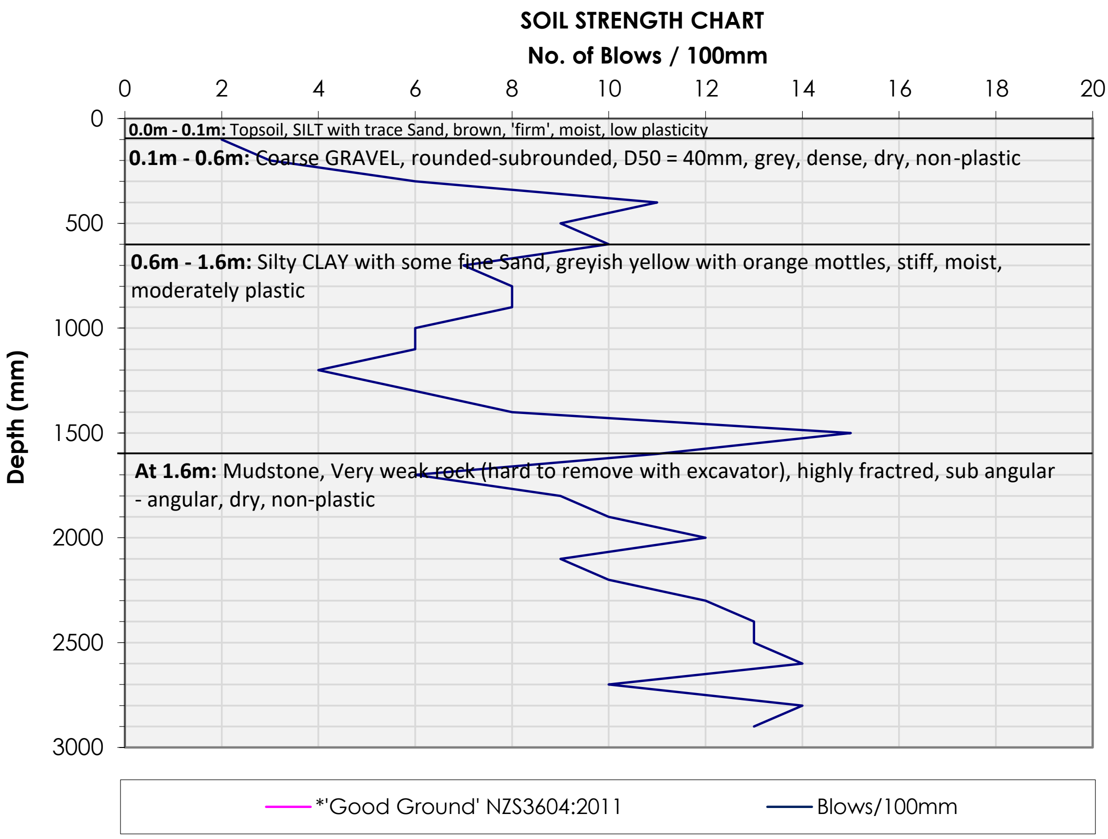
cheal

Job No.	210418 | Tony Head		
Location:	127 Onga Onga Road, Waipawa		
Test No.	TP2	Starting Depth:	
Date:	0/01/1900	Test by:	LHC



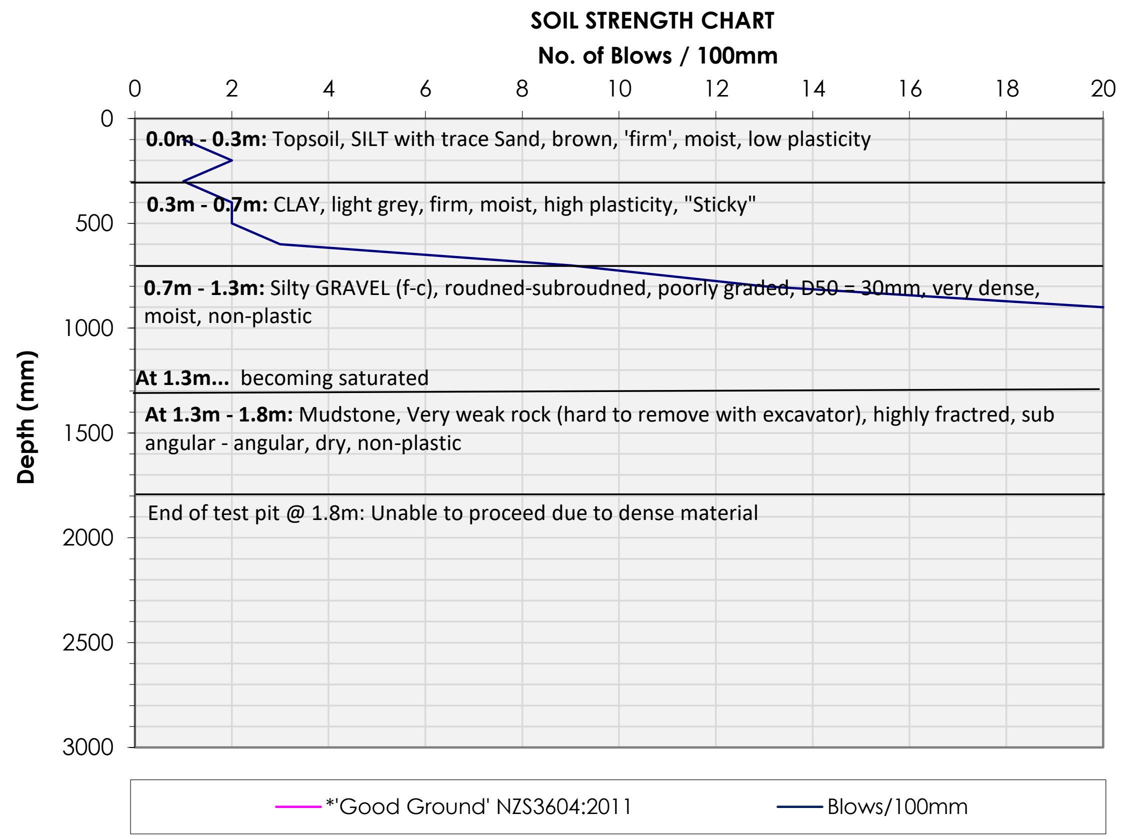
Test Pit 3

Job No.	210418 | Tony Head		
Location:	127 Onga Onga Road, Waipawa		
Test No.	TP3	Starting Depth:	
Date:	0/01/1900	Test by:	LHC



Test Pit 4

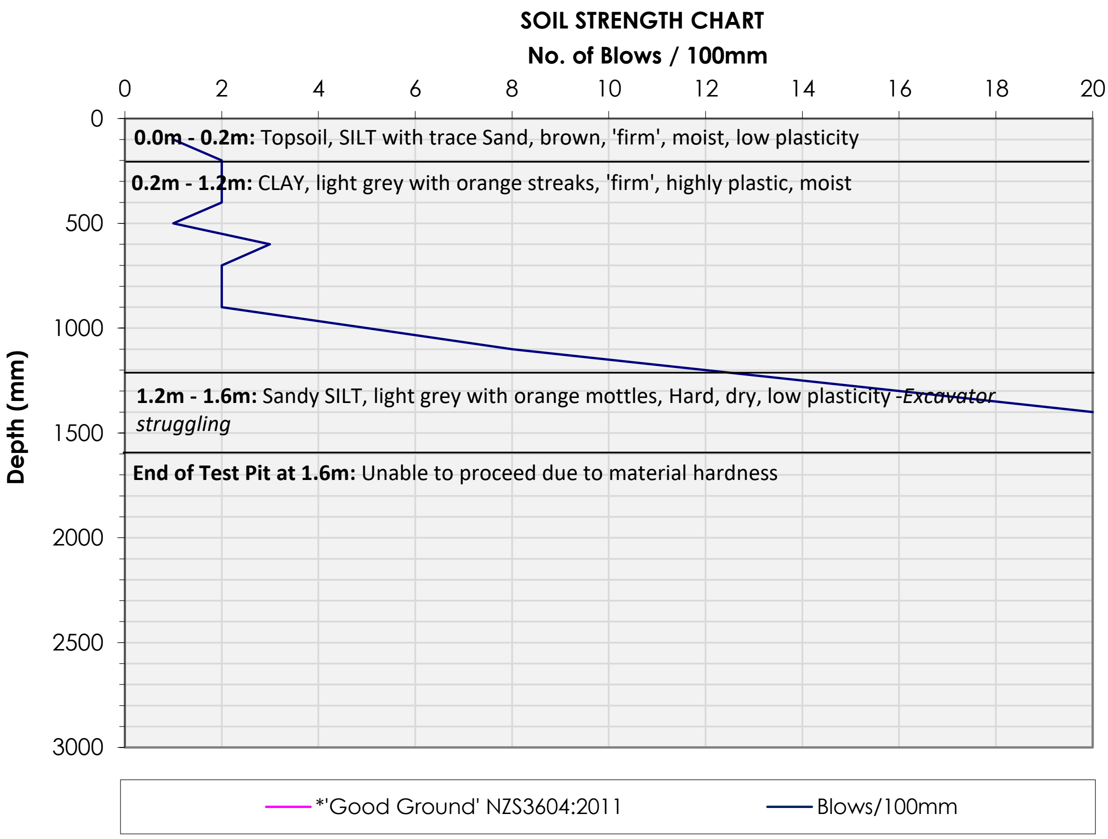
Job No.	210418 | Tony Head		
Location:	127 Onga Onga Road, Waipawa		
Test No.	TP4	Starting Depth:	
Date:	0/01/1900	Test by:	LHC



Test Pit 5

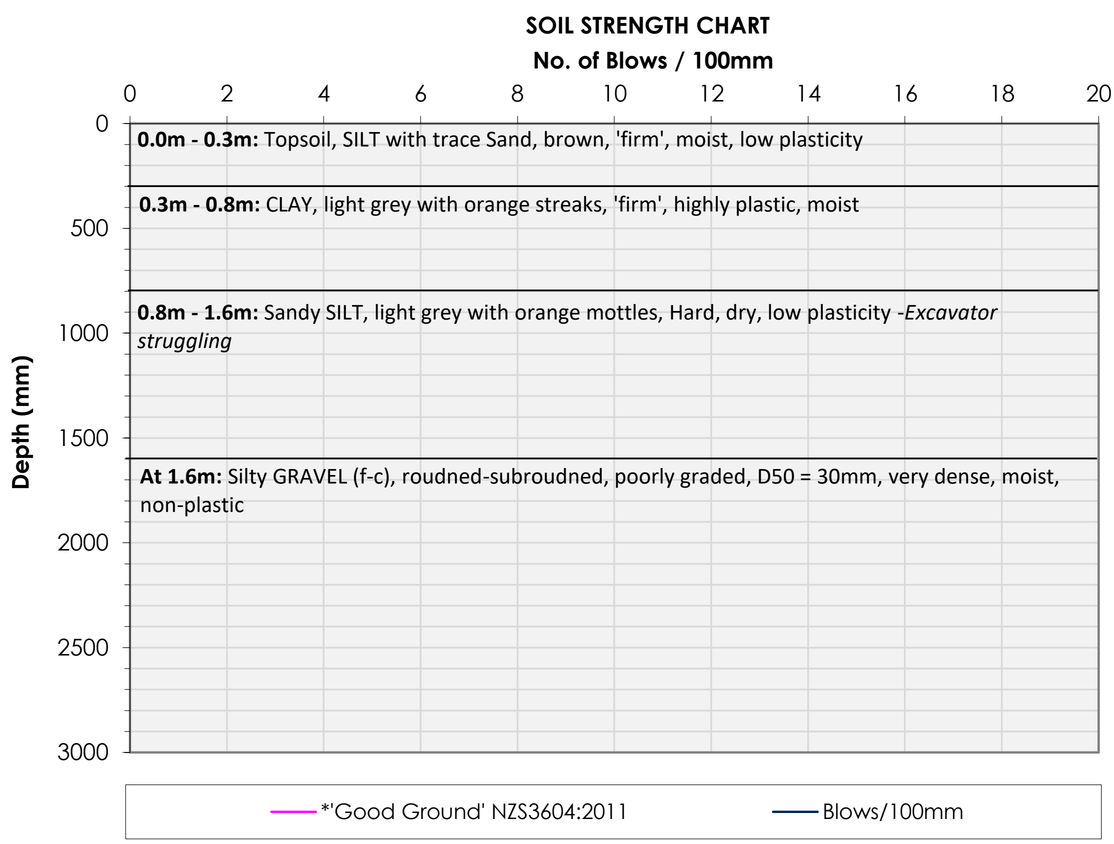
cheal

Job No.	**210418 | Tony Head**		
Location:	**127 Onga Onga Road, Waipawa**		
Test No.	**TP5**	**Starting Depth:**	
Date:	**0/01/1900**	**Test by:**	**LHC**



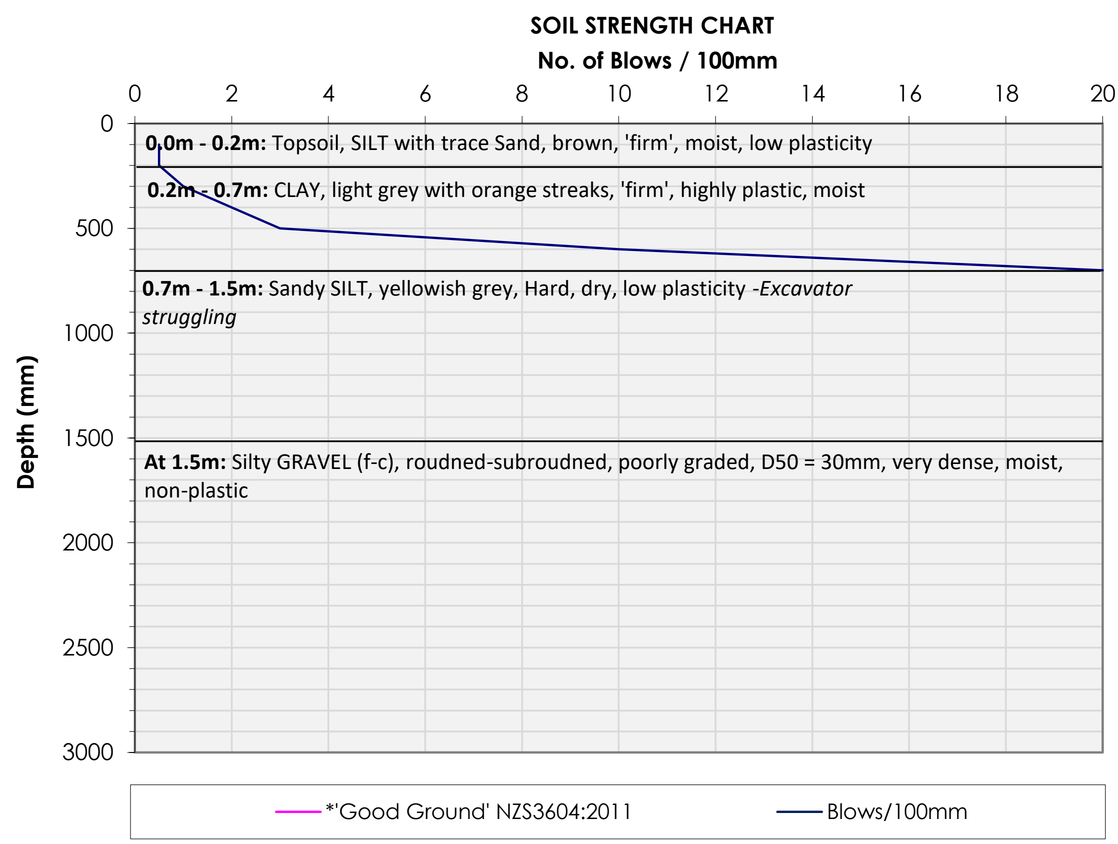
Test Pit 6

Job No.	210418 | Tony Head		
Location:	127 Onga Onga Road, Waipawa		
Test No.	TP6	Starting Depth:	
Date:	0/01/1900	Test by:	LHC



Test Pit 7

Job No.	210418 | Tony Head		
Location:	127 Onga Onga Road, Waipawa		
Test No.	TP7	Starting Depth:	
Date:	0/01/1900	Test by:	LHC

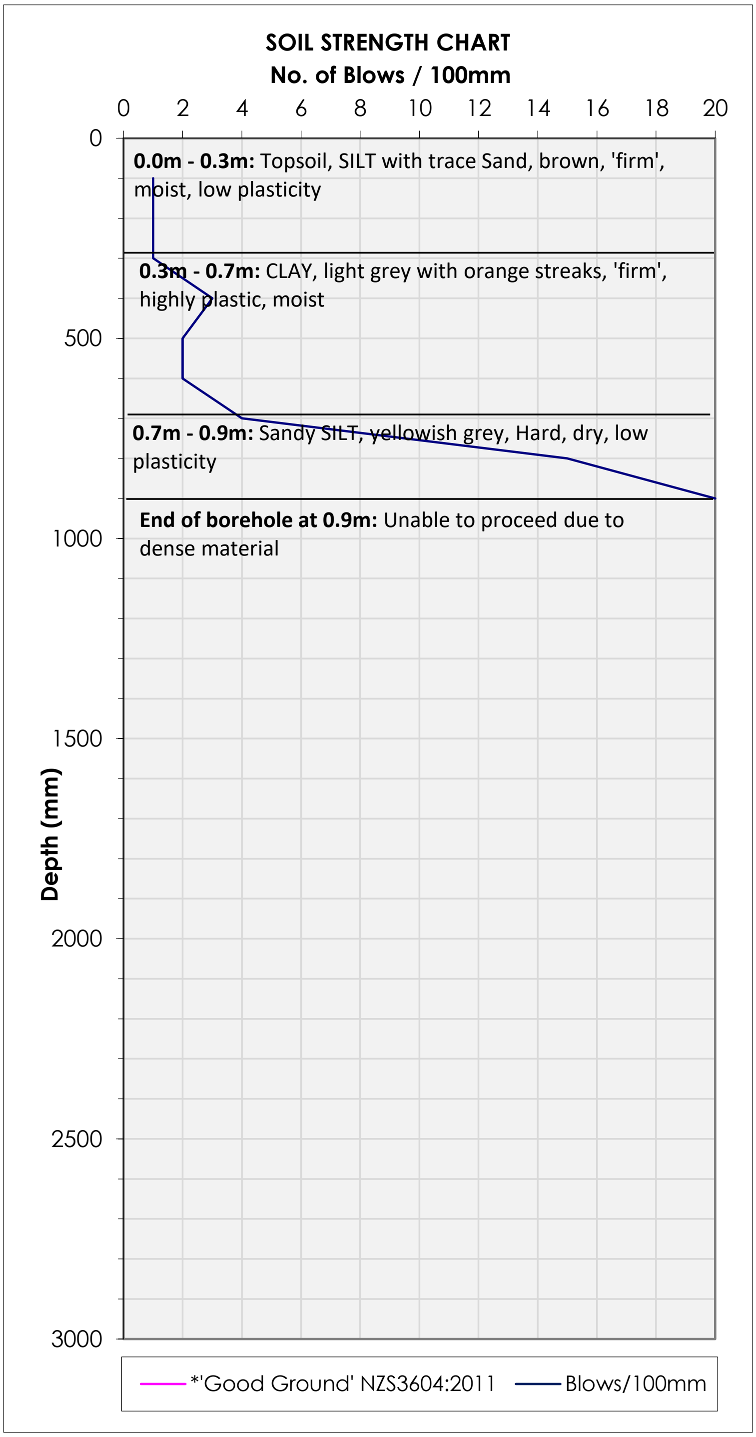


cheal

Hand Auger 1

Job No.	210418 | Tony Head		
Location:	127 Onga Onga Road, Waipawa		
Test No.	HA1	Starting Depth:	
Date:	0/01/1900	Test by:	LHC

Depth (mm)			Blows	Average Blows / 300mm
0	-	100	1	1.0
100	-	200	1	1.0
200	-	300	1	1.0
300	-	400	3	1.7
400	-	500	2	2.0
500	-	600	2	2.3
600	-	700	4	2.7
700	-	800	15	7.0
800	-	900	20	13.0

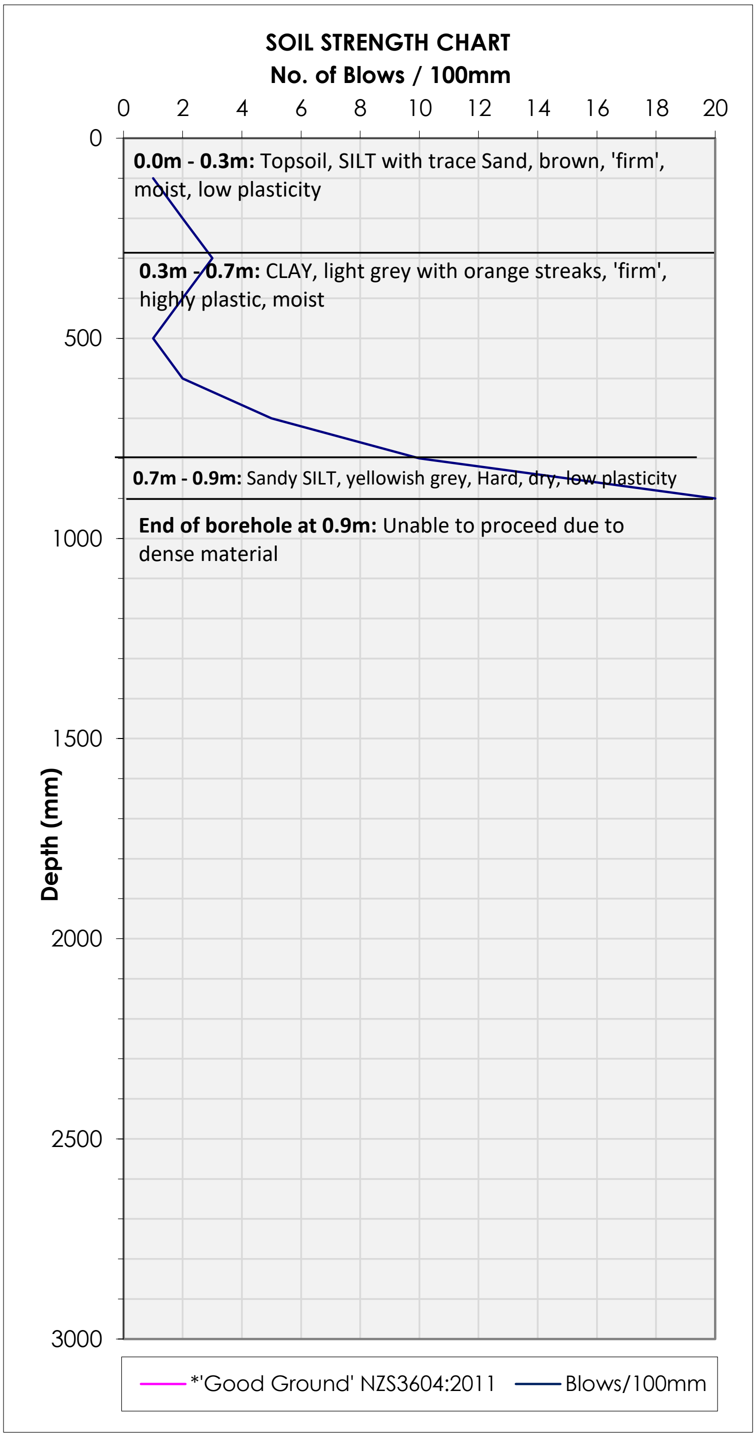


Hand Auger 2

cheal

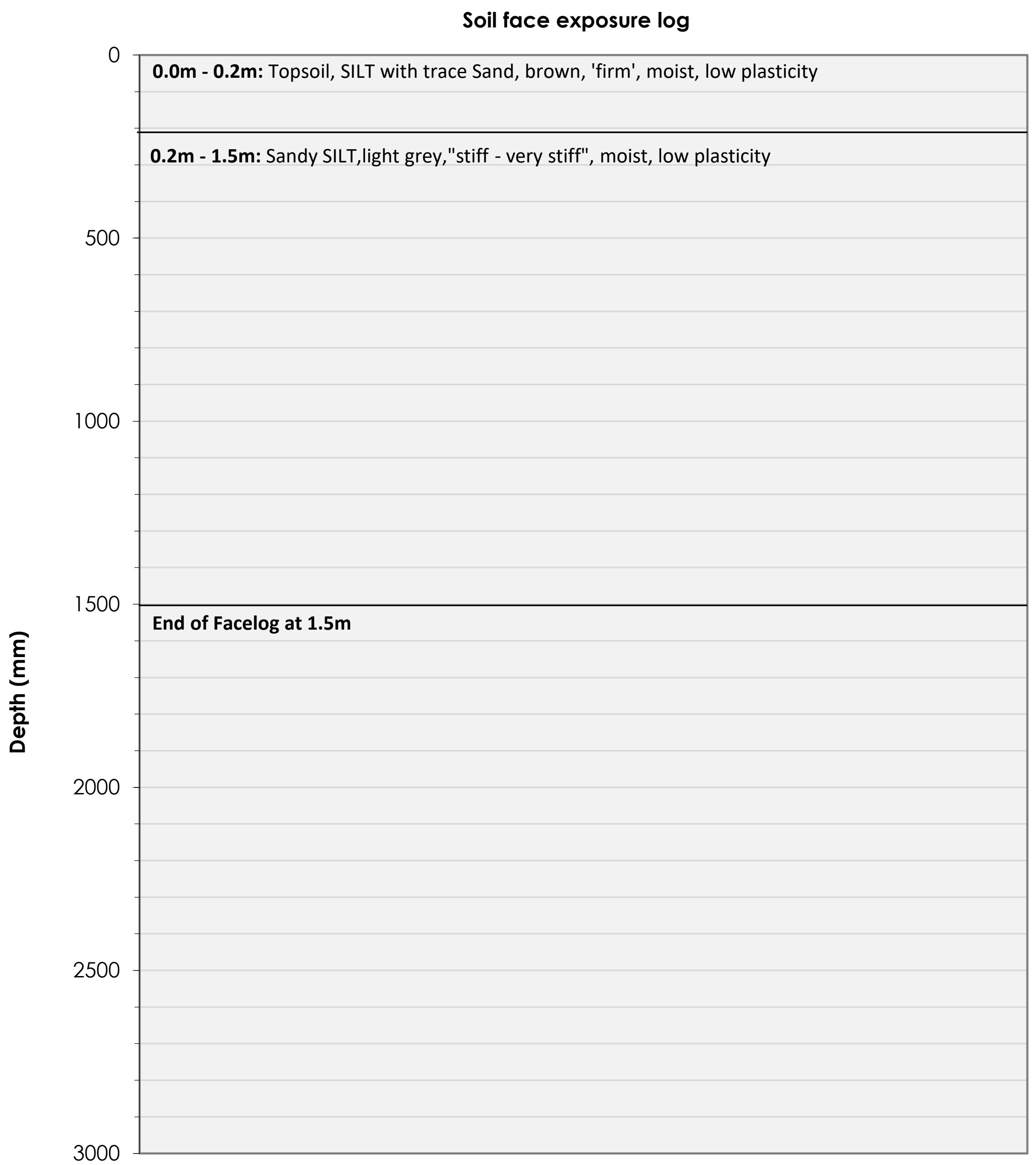
Job No.	210418 | Tony Head		
Location:	127 Onga Onga Road, Waipawa		
Test No.	HA2	Starting Depth:	
Date:	0/01/1900	Test by:	LHC

Depth (mm)			Blows	Average Blows / 300mm
0	-	100	1	1.0
100	-	200	2	1.5
200	-	300	3	2.0
300	-	400	2	2.3
400	-	500	1	2.0
500	-	600	2	1.7
600	-	700	5	2.7
700	-	800	10	5.7
800	-	900	20	11.7



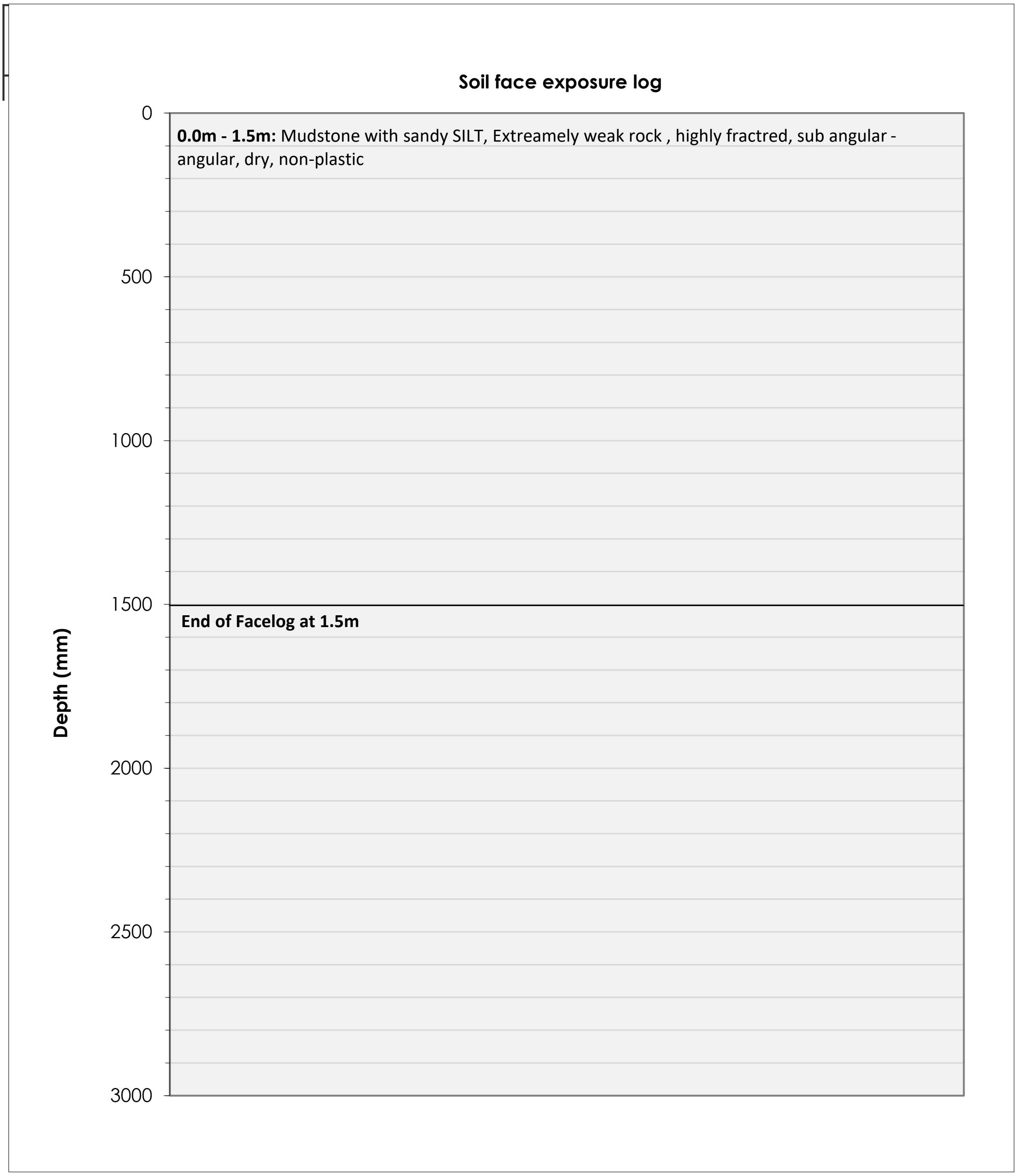
FL1

Job No.	210418 | Tony Head		
Location:	127 Onga Onga Road, Waipawa		
Test No.	FL1	Starting Depth:	
Date:	0/01/1900	Test by:	LHC



FL2

Job No.	210418 | Tony Head		
Location:	127 Onga Onga Road, Waipawa		
Test No.	FL2	Starting Depth:	
Date:	0/01/1900	Test by:	LHC



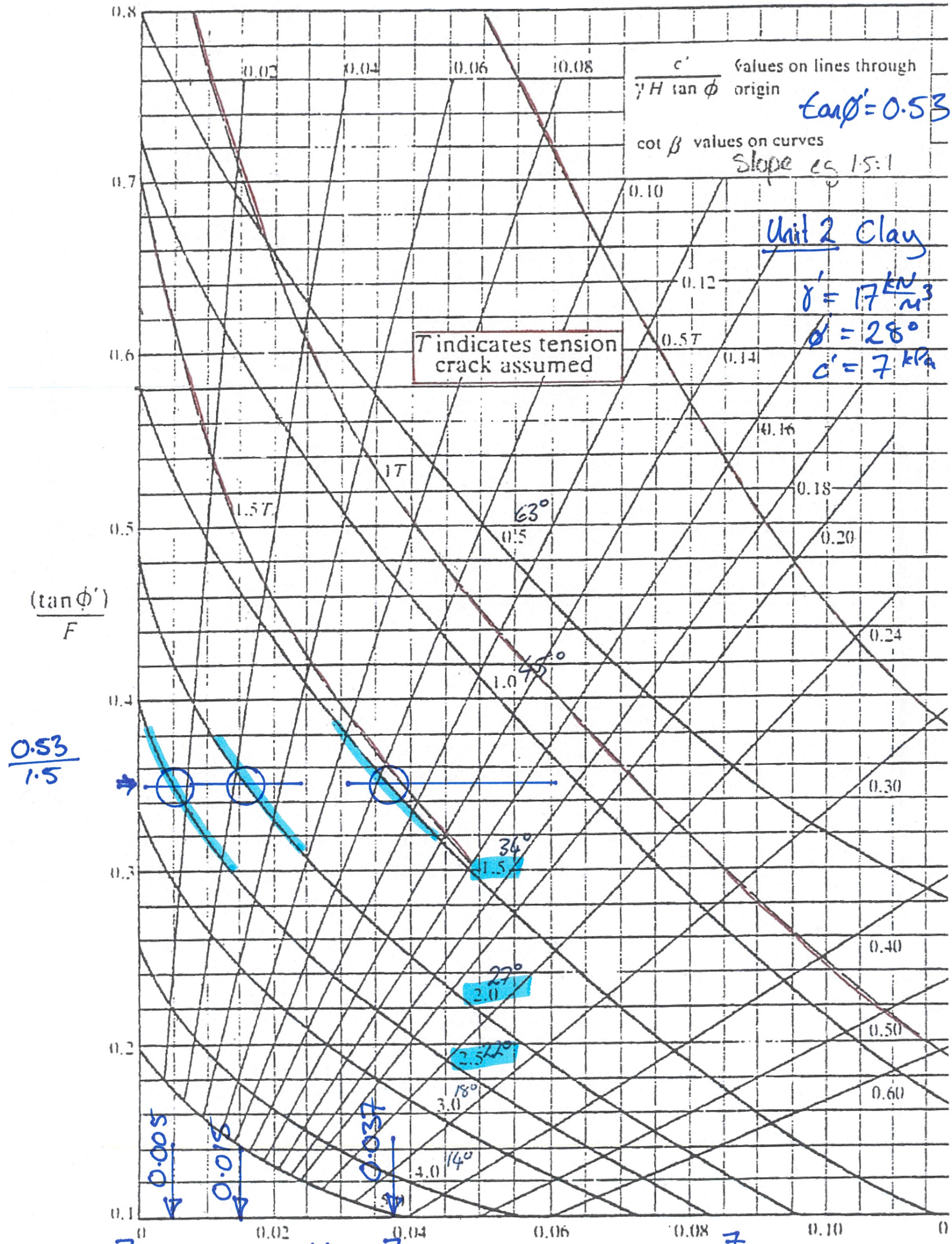
FL3

cheal

Job No.	210418 | Tony Head		
Location:	127 Onga Onga Road, Waipawa		
Test No.	FL3	Starting Depth:	
Date:	0/01/1900	Test by:	LHC

Appendix 2

Slope Stability Charts

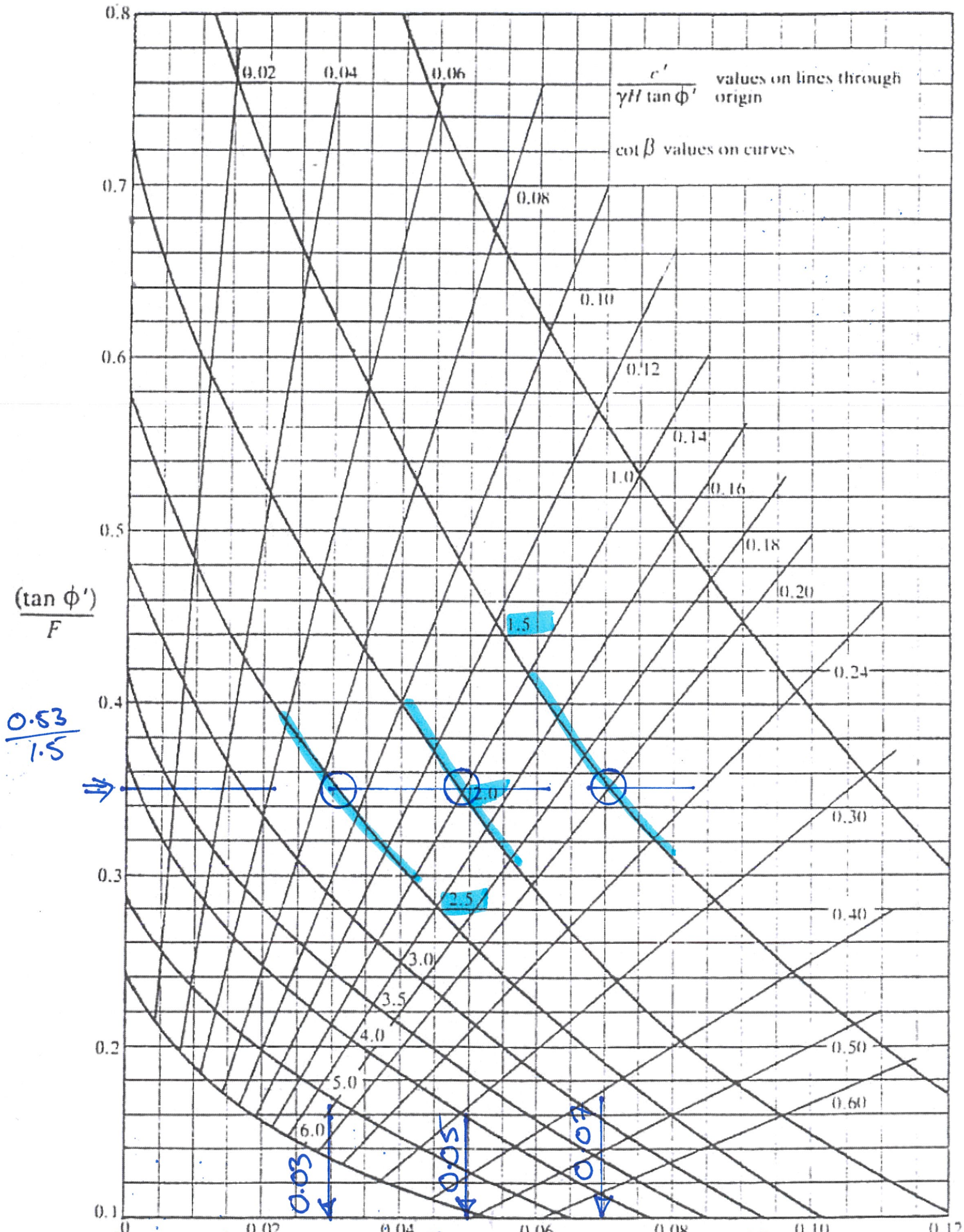


$H_{2.5} = \frac{7}{0.005 \times 17 \times 1.5}$ $\Rightarrow 55m$

$H_2 = \frac{7}{0.015 \times 17 \times 1.5}$ $\Rightarrow 18m$

$H_{1.5} = \frac{7}{0.037 \times 17 \times 1.5}$ $\Rightarrow 7.4m$

Figure 4.15 Design chart for rotational sliding. $r_u = 0$.



$H_{2.5} = \dfrac{7}{0.03 \times 17 \times 1.5}$ $\Rightarrow$ 9m

$H_{2} = \dfrac{7}{0.05 \times 25.5}$ $\Rightarrow$ 5.5m

$H_{1.5} = \dfrac{7}{0.07 \times 25.5}$ $\Rightarrow$ 4m

Figure 4.16 Design chart for rotational sliding. $r_u = 0.3$.