GEOTECHNICAL ASSESSMENT REPORT

PROJECT:
10-LOT SUBDIVISION, MANGAKURI BEACH

CLIENT:

SR & BJ WILLIAMS CHARITABLE TRUST

19385B_03

REVISION SCHEDULE

Revision	Description	Date	Signature
193850602-01	Report Issued Draft for Comment	9/7/21	
193850602--02	Initial Report Submission for Resource Consent	6/8/21	
19385B-03	Stantec initial comment response	1/6/23	
19385B-04	Internal Review & Reissue	7/8/23	
19385B-05	Update Risk Assessment	6/10/23	

EXECUTIVE SUMMARY

SR & BJ Charitable Trust engaged Resource Development Consultants Ltd (RDCL) to undertake site investigations and reporting for a proposed ten (10) Lot Subdivision at Mangakuri Beach Subdivision.

We understand the intent is to subdivide and create ten (10) new lots from existing land (Legal Description; Lot 2 DP 481291), of which:

- Eight (8) Lots for residential development including;
  - Lots1, 3, 4, and 6 to 10.
  - Lots 2 & 5 have been excluded.
- Two (2) Lots remain balance land, include.
  - Lots 11 & 12.

BUILDING PLATFORM SUITABILITY

We consider the proposed building platforms proposed for Lots 1 to 10 (excluding Lots 2 & 5) to be suitable for residential development provided the recommendations and Consent Conditions in this report are addressed.

All sites are underlain by expansive soils requiring specific consideration for earthworks, foundations and for infrastructure (road surface stabilisation etc).

GEOLOGICAL CONDITIONS

Expansive soils are found to overlie late Cretaceous to Miocene age “Melange” Sandstone and Mudstone.

Observed land instability indicates:

- In the upper (elevated) part of the slope (Lots 3 to 10):
  - Gravitational soil creep within the near surface (< 5 m),
  - Primarily driven by loss of material strength due to shrink and swell of the expansive soils.
- In the lower part of the slope (Lot 1):
  - Historical debris lobes and soil run out from gravitational failures evacuated from slopes above.
  - There is no clear evidence of deep-seated instability encountered in borehole or CPT testing.

Slope Stability Assessment

Slope stability assessment was undertaken on six key cross-sections. The assessment confirms that the proposed building platforms are generally suitable for future development subject to recommendations in this report.

Building Platform Setout Criteria

Building platforms proposed at the top of existing slopes should be formed entirely within cut (Natural ground).

Building platforms proposed at the toe of existing slopes (Lot 1) should be setback from the toe, and excavations should be controlled or retained.

Engineered Fill may be utilised to form larger building platforms provided stability is confirmed as suitable. Fill should otherwise be used for minor structures and landscaping only.

Building Platform Design

To address the risk of expansive soils, all building platforms should be tested for expansive properties at or during the completion of the building platform.

Foundations exposed to risk of expansive type soils as defined in NZS3604 are subject to Specific Engineering Design (SED). Foundation design should also consider slope stability and setback conditions as well as usual requirements.

A building setback of at least 5 m is required for all building platforms where ground slopes away exceeding 20°, and/or where land rises above the building platform to address natural slope regression and land instability:

Building within the building setback area requires the engagement of a geotechnical engineer and may require Specific Engineering Design.

Final building setbacks will be confirmed in the Completion Report and provided within the Completion Certificate Schedule 2A (NZS4404:2010) as part of 224c.

FOUNDATION RECOMMENDATIONS

All foundations should be designed for adequate stiffness and strength to resist the expansive nature of the ground by:

- Combination of ground improvement with enhanced Foundation Design using:
  - Shallow Waffle raft slabs in accordance with Appendix F of AS2870:2011; or
  - Timber pole foundations embedded below expansive soil horizons assumed to be 1.5m to 2m bgl.

Where natural ground is exposed, inferred Ultimate Bearing Capacity (UBC) is anticipated to achieve:

- ~ 200kPa UBC from 0.2m to 0.8m bgl; or 66kPa Allowable.
- ~300kPa UBC from 0.3m to 1.5m bgl; or 100kPa Allowable.

Where engineered fill is formed bearing capacity shall be assessed following the completion of earthworks and is anticipated to achieve:

- 300 kPa UBC in accordance with NZS3604:2011, provided:
  - Expansive soils are modified by lime or cement additives;
  - Earthworks are undertaken in accordance with NZS4431:2022[1].; or
  - Piles embedded below expansive soils are used.

Due to the risk of expansive soils, buildings shall be designed with building dimension and layout restrictions in accordance with B1/AS1, 28/11/2019:

Final building setbacks will be confirmed in the Completion Report and provided at 224c.

EARTHWORKS

Expansive soils may be modified by Lime or Cement additives.

Engineered fill should comprise material as approved by a geotechnical engineer, placed in accordance with NZS4431:2022.

[1] NZS4431:2022[1]: Engineered Fill Construction for Lightweight Structures

Road Access

Road access to building platforms should be suitable provided road construction is designed to take advantage of resistant outcrops and keep away from wet, boggy terrain unless adequate drainage and ground improvement is installed and consider the following:

- Variable subgrade strength and future traffic loads including construction traffic;
- The carriageway should be designed to consider subsoil drainage and stormwater discharge.

All roads should collect stormwater by appropriate collection points using side drains, kerb and channel and discharge to appropriate discharge areas approved by the local authority.

Stormwater & Wastewater

Stormwater & wastewater design has been undertaken by StrataGroup Consulting Engineers.

Stormwater design accommodates in attenuation tanks and bubble up sumps discharged to gently sloping terrain.

Wastewater design accommodates low discharge of 1L to 1.5L /m2/day and planted to further enhance slope stability.

Storm and wastewater discharge should be located downslope and a suitable distance away from the building platform or steep slopes.

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1 INTRODUCTION

SR & BJ Charitable Trust engaged Resource Development Consultants Ltd (RDCL) to undertake geotechnical investigation and assessment report for a proposed Ten (10) Lot Subdivision at Mangakuri Beach.

This report addresses geotechnical aspects of the subdivision consent in accordance with Section 106 of the Resource Management Act (1991) (RMA).

1.1 UNDERSTANDING THE PROJECT

RDCL received the Subdivision Consent Plan completed by Surveying the Bay (Drawing no 4698-29, issued in Aug 2023).

We understand the intent is to subdivide and create ten (10) new lots from existing land (Legal Description; Lot 2 DP 481291), of which:

- Eight (8) Lots for residential development including:
  - Lots1, 3, 4, and 6 to 10.
  - Lots 2 & 5 have been excluded.
- Two (2) Lots remain balance land, include:
  - Lot 11 & 12.

A Geotechnical Report is required to address:

- Land instability risks in accordance with Section 106 of the RMA;
- Likelihood and consequence of natural hazards occurring;
- Assess suitability for new building platforms and access;
- Recommendations for avoiding, remedying or mitigation of adverse hazards; and
- Provide preliminary recommendations for earthworks, future foundations and surface water controls.

1.2 Scope of Work

The scope of work is based on the RDCL proposal (Ref 19385_03), dated 22 September 2022 and comprises:

- Review of existing reports and information;
  - RDCL report Ref 18325;
- Geotechnical site testing to confirm the subsoil conditions;
- Interpretation to develop a representative ground model and geotechnical parameters;
- Assessment of geohazards including stability analyses, expansive soil, and bearing capacity for the proposed building platform; and
- Provision of a Geotechnical Report to meet the requirements of subdivision consent.

1.3 Section 106 RMA Natural Hazards

Section 106 of the RMA states:

A consent authority may refuse a subdivision consent application, or may grant consent subject to conditions, if the land is at significant risk from natural hazards.

An assessment of the risk from natural hazards requires a combined assessment of the:

- Likelihood of the natural hazards occurring;
- Material damage that would result from natural hazards to the land where the consent is sought, other land, or structures; and

  Any likely subsequent use of the land that would accelerate or worsen the damage predicted from a natural hazard.

2 SITE DESCRIPTION

The site comprises ~112 Ha of semi-rural farmland located on the foothills above Mangakuri Beach and encompasses:

- Lot 2 PT Lots 1 & 3 DP 4588;
- Lots 2 & 3 DP 481291;
- Lots 1 & 2 DP 25804.

The general topography (Figure 1) is elevated to the west bounded by Williams Road at ~60m to ~100m elevation, and Mangakuri Beach to the east at ~20m to 30m elevation.

Three (3) separate gully catchments (North, Central and South) are defined at the western extent with a head scarp and separated by prominent ridgelines trending east.

The lower part of the slope is more gently sloping and defined by what appears to be historical landslide runout debris.

Each gully catchment appears to be spring fed the:

- Northern gully shows ongoing seepage developing into a small stream.
- Central gully shows evidence of periodic seepage, probably controlled by seasonal conditions. There is no stream in this gully.
- Southern gully also shows ongoing seepage, evident by wet and boggy ground and water tolerant vegetation.

A farm dam has been built in the southern gully with fill forming the downslope embankment.

A water tank is in the northern gully and is fed by a farm water system, location unknown.

2.1 Description by Lots

A designated 30m x 30m building platform has been assigned for each new Lot for subdivision consent (See Figure 1).

2.1.1 Lot 1 (9,307m^2)

Lot 1 is at the northeastern end of the subdivision, at the toe of the ridgeline separating the northern and central gully. The proposed building platform occupies a broad, flat and gently sloping ridge.

A shallow ephemeral stream, fed by spring water seepage crosses north of the proposed building platform and into the boundary of No 38 Okura Road. Some large Poplar trees occupy the steam channel.

2.1.2 Lot 3 (4,636m^2)

Lot 3 building platform is on the northern most ridgeline with slopes to the north and south. A spring fed stream observed as a "trickle", is in the invert of the gully to the north.

There is evidence of recent instability on Williams Road with a shoulder dropout on the opposite side of the gully to the north and on the southern side of the ridge near the building platform.

The ground is gently sloping (~10° to ~15°), with minor tension cracks observed at the edge of the northern gully, possibly caused by expansive soils and soil creep and recent instability on the south facing side of the ridgeline.

2.1.3 Lot 4 (4,844m^2)

Lot 4 building platform is to the west of Lot 3, at the head of the northern most gully with east facing slopes at ~15° to 20°.

Shallow instability in the form of rotational slips were observed coincident with seepage.

2.1.4 Lot 6 (6,757m^2)

Lot 6 building platform is at the crest of the hill to the east of Williams Road. The building platform has a western aspect with slope angles of ~15° to 20°, with no significant evidence of land instability.

Immediately to the east of the proposed platform, the slope abruptly drops at ~45° (east facing) into two separate gullies with shallow instability mapped on the steep slopes evident by hummocky ground and seepage.

2.1.5 LOT 7 (5,551M^2)

Lot 7 building platform is at the crest of a hill southeast of Williams Road. The building platform has a western aspect with slope angles of ~15° to 20°, and no significant evidence of land instability.

East of the proposed platform, the slope abruptly drops at ~45° (east facing) with evidence of slope instability on the steep face with hummocky ground and seepage.

2.1.6 LOT 8 (6,518M^2)

Lot 8 is on the southern ridgeline at the crest of the hill, surrounded by steep sloping ground.

Immediately to the east, north, and south of the proposed platform, shallow instability in form of hummocky ground is evident on steep faces.

2.1.7 LOT 9 (8,265M^2)

Lot 9 is on the southern prominent ridgeline with slopes at ~20° to 25° to the northeast. Minor gullies are located to the northwest and north-eastern side of the building platform and the land to the south continues to rise.

Shallow instability was observed and mapped outside the building platform area in the form of soil creep on 15° to 30° slopes.

2.1.8 LOT 10 (8,123M^2)

Lot 10 is to the west of Lot 9 on a prominent, gently sloping ridgeline bounded by moderate sloping gullies to the west and east, and Williams Road cut to the north.

The gullies are wet & spongy in the base with occasional rushes which indicate water seepage.

The northern gully shows evidence of shallow scarps that appear to be the result of steep slopes and a seep line.

The building platform is elevated 14m above Williams Road on a ~30° cut slope vegetated in trees and shrubs with tall pines at the crest.

2.1.9 Lot 11 (53.906ha)

Corresponds to the coastal land balance and is outside the scope of future development.

2.1.10 Lot 12 (52.535ha)

Corresponds to the coastal land balance and is outside the scope of future development.

2.2 Existing Services

2.2.1 Chorus Telecom

A Chorus Telecom service is located on the northern ridgeline, aligned along the access off Okura Road to the east.

2.2.2 Water

A buried water pipe is located along the northern ridge and connects into a small water tank assumed to be used for livestock.

2.3 Regional Geology

The 1:250,000 online GNS Science Webmap[2] indicates the site geology consists of:

- The upper (western) slope comprises Late Cretaceous to early Miocene melange of undifferentiated Whangai, Wanstead and Weber formations and Early Miocene in a sheared matrix;
- The lower (eastern) slope comprises Late Cretaceous Glenburn Formation sandstone of well-bedded, alternating sandstone, mudstone and conglomerate.

The boundary between both units is inferred to be defined by an inactive normal fault.

- The QMap published text for Hawkes Bay Area identifies the local geology (Wanstead & Weber Formations), comprise Smectite rich soils which are susceptible to expansion (and contraction), resulting in slope instability and rapid erosion; and
- Inferred colluviums from gullies infill are anticipated.

[2] http://data.gns.cri.nz/geology/

2.4 EXISTING INFORMATION

Previous geotechnical investigations and reports relevant to the site include:

- RDCL report Ref 18325 (2018) Preliminary Geotechnical Appraisal for subdivision:
  - Preliminary geotechnical appraisal for 8 houses.
  - Appraisal confirmed the proposed subdivision is generally suitable for development subject to specific requirements;
  - All building platforms and access would be subject to specific engineering design.
  - Identified complex stratigraphy resulting from successive landslide events;
  - Upper slopes prone to instability including recent displacement.
  - Poor drainage and potentially low bearing soils in low lying areas.
- Pettinga. J. (1982) Upper Cenozoic Structural history, coastal Southern Hawkes Bay, New Zealand
  - Geological paper discussing the structural high trending along the Northeast southern Hawkes Bay Coastline which comprises Upper Cretaceous to Miocene successions that have been completely folded and thrust faulted with major tectonic melange and crushed zones.

3 GEOHAZARDS

In accordance with Section 106 of the Resource Management Act, the risk of geohazards at the building platform have been summarised in Table 1 where:

- High risk is defined as likely to occur during the design life;
- Moderate risk is defined as being possible to occur during the design life;
- Low risk is defined as unlikely to occur in the design life;
- Negligible risk is defined as being very unlikely to no risk during the design life.

3.1 Landslide Risk

The Hawkes Bay Hazard Portal[3] for Land instability indicates the subdivision is within a "severe earthflow" risk zone.

Observations made by historical and recent aerial imagery and site walkover and geomorphic mapping confirms the risk of land instability indicated by recent rotational or translational landslides on slopes exceeding 30° and historical and widespread debris flow.

Active landslides are defined by:

- Translational or rotational shallow landslides on slopes exceeding 30 degrees;
  - Typically within expansive soils; with
  - Seepage.
- Undulating ground forming "hummocky mounds";
- Low to moderate angled slopes (15 to 30 degrees);
  - Typically within expansive soils; with
  - Seepage.

Active landslides impact:

- Lot 1; upslope (West);
- Lot 3 & 4; downslope (North & South);
- Lot 6 & 7; downslope (eastern side only).
- Lot 8; downslope (North, east & south);
- Lot 9 & 10; downslope (north and east).

[3] https://gis.hbrc.govt.nz/Hazards/

These landslides appear to have been caused predominantly due to:

- Heavy Rainfall induced; and
- Large (Historical) Seismic events.

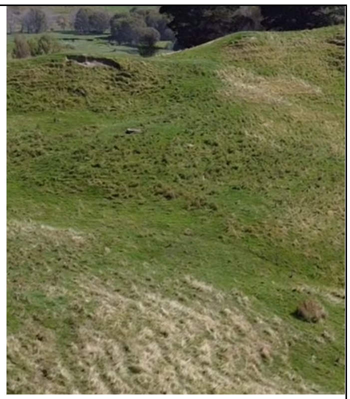
3.1.1 Rainfall Induced Landslides

Rainfall induced landslides present as shallow rotational or translational landslides at a decametre (tens of metres) scale (Figures 2a & 2b and Image 1) on steep slopes > 30°.

Evidence of recent rainfall induced landslides include:

- Below Lot 8 where a shallow translational slide has occurred on seaward facing slopes after Cyclone Gabrielle;
- Continuous minor slumping evident in the headscarp near Lots 6, 7 & 8, and above Lot 1.
- Shallow surface creep and hummocky ground in gullies near Lots 3 & 4, and Lots 9 & 10; and
- Translational movement on the steep slope above Lot 1.

Image 1: Examples of Rainfall Induced Landslides

		
Above Lot 1	Below Lot 8	Below Lot 7.

3.1.2 Seismically Induced Landslides

Seismically induced landslides are much larger and deeper and are the likely primary cause of instability that encompasses the subdivision.

Historical aerial imagery (Retrolens[4] images from 17/5/1952, 31/8/1964, 12/9/1972, 23/11/1976; and Google Earth[5] Imagery 1985 to 2022) indicates:

Seismically induced landslides are the likely origin for the southern, central and northern gully headscarps and bulbous debris runouts observed in Figure 2a, 2b and Image 2 and indicated on Figures 3a & 3b.

Review of 1952 Historical image shows a very large debris flow has occurred from the topographic high ridgeline located approx. 500m to the north of this site (Figure 3a). The debris runout can also be seen rendered on a 1m LiDAR DEM Model (Figure 3b)

The Debris flow source appears to be from the highest elevation, extends south to a saddle and then turn southeast toward the sea. Williams Road crosses the slip just below where an existing dam is and Okura Road curves around the bulge runout at the toe.

The age of the event is unknown other than to note that the trees in the toe appear to be mature which infers event seems to have occurred well before these trees established.

The future building platforms are intentionally located outside of observed active or inactive instability areas including debris runout zones.

Based on the above, we consider landslide risk to be “High” where building platforms encroach into or within 10m of steep slopes exceeding 30°, “Moderate” risk for building platforms on slopes that encroach into or within 10m of slopes 10° to 30°and “Low Risk” on slopes less than 10°.

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[4] http://retrolens.nz/

[5] https://www.google.com/earth/

IMAGE 2: SEISMICALLY INDUCED LANDSLIDES

Example of Seismically induced Landslide and debris flow below Lot 7 (left) and Lot 6 (central).

3.2 EXPANSIVE SOILS

The QMap published text for Hawkes Bay Area identifies the local geology (Wanstead & Weber Formations), comprise Smectite rich soils which are susceptible to expansion (and contraction), resulting in slope instability and rapid erosion.

Lab test results completed at the proposed building platform as part of RDCL previous version of this report indicates Liquid Limit >50% and Linear shrinkage >15% which infer expansive soils in accordance with NZS3604:2011. Based on above, the risk of expansive soil is considered “high”.

3.3 EARTHQUAKE RISK

3.3.1 SHALLOW CRUSTAL EARTHQUAKE

The 1:250,000 online GNS Active Faults Database does not indicate any known active faults on this site (GNS Science, 2022, Version 3.3.6.82).

The 1:250,000 geological map infers an inactive, normal fault may be present where the terrain changes abruptly from out of the proposed subdivision boundaries, to the east.

Based on that active Fault Risk is considered “low.

3.3.2 HIKURANGI SUBDUCTION ZONE (HSZ)

The Hikurangi Subduction zone extends between the Northeastern South Island and extends up past the East Cape and is the largest Faultline in New Zealand.

Geological evidence[6] indicates there has been Ten (10) past possible subduction earthquakes over the past 7000 years along the Hikurangi margin. The last subduction earthquake occurred at 520-470 years BP in the southern margin and strongest evidence for a full margin rupture is at 870-815 years BP.

Fault Rupture at the Hikurangi Subduction Zone is likely to generate a Megaquake capable of generating a Magnitude Mw 9 earthquake leading to large scale shaking and Tsunami risk.

The fault details are:

- Active Thrust Fault;
- Magnitude Mw 9.0;
- Recurrence Interval 500yrs.

3.4 LIQUEFACTION SUSCEPTIBILITY

The Hawkes Bay Emergency Management Group Portal[7] indicates the site is unlikely to liquefy.

This is supported by the geological age and composition of materials are unlikely to be susceptible to liquefaction. Liquefaction risk is assessed to be “Low”.

3.5 TSUNAMI RISK

The HB Hazard Portal indicates Lot 1is susceptible to Tsunami Risk from near wave source directly affecting:

- Eastern edge of the site.

with:

- Max Amplitude 13.5m;
- Return Period (yrs) 2,500.

Tsunami Risk is assessed to be “medium” based on the return period (1/2,500 yrs).

[6] Clark, K; Howarth, J; Litchfield, N; Cochran, U; Turnbull, J; Dowling, L; Howell, A; Berryman, K; Wolfe, F (1 June 2019). "Geological evidence for past large earthquakes and tsunamis along the Hikurangi subduction margin, New Zealand". *Marine Geology*. **412**: 139–72. doi:10.1016/j.margeo.2019.03.004. S2CID 135147628

3.6 FLOOD RISK

The Hawkes Bay Hazard Portal for Flood Risk indicates this site to be out of mapped area, with no indicative risk for the site.

The majority of building platform are situated at the top of hills. Topographically, we consider the risk of flooding unlikely, except for Lot 1, situated in the change of slope gradient and nearby an ephemeral stream, designed as a “Low” risk.

4 SITE INVESTIGATIONS

Site investigations were targeted at building platform locations to inform ground conditions for future land development and where land instability was observed to understand the ground model and water regime.

Engineering geological logging of materials recovered from Hand Augers, Tests pit, and Boreholes, were logged in accordance with New Zealand Geotechnical Society Guidelines (NZGS, 2005). The results are presented in the Table 2.

TABLE 1: SUMMARY OF TESTS.

Test	Completed Tests	Test ID	Termination Depth (m)	Termination Reason
Borehole (BH)	4	BH01 to BH04	Between 6m and 10.5m	Target depth or where refusal on hard ground
Test Pit (TP)	19	TP1.1 to TP9.2	Between 1.8m and 3.5m	
Hand Auger (HA)	18	HA01 to HA18	Between 0.5m and 3.0m	
Cone Penetrometer Test (CPT)	16	CPT01 to CPT14*, and CPT15 & CPT16	Between 1.5m and 9.9m	Cone Tip Resistance exceeding 20MPa or anchor failure
Soak Pit (SP)	1	SP01	1.5m	Target Depth
Dynamic Cone Penetrometer (DCP)	13	DCP01 to DCP13	Between 0.6m and 2.9m	Target depth or where double bounces

4.1 BOREHOLES, TEST PITS, HAND AUGER, CPT AND DCP TESTING

Borehole, Test Pits and Hand Auger were undertaken by RDCL Engineering Geologists and technicians.

Soil descriptions are in general accordance with the NZGS (2005) Guidelines[8].

Boreholes, Test pit, Hand Auger, CPT and DCP Logs representative of the proposed building platform are presented in Appendix A.

[8] NZGS (2005) Guidelines for Field Description of Soil and Rock

4.1.1 Lot 1 Subsoil Conditions

Table 2: Lot 1 Ground Model Summary

Top (m)	Base (m)	Description	Strength
0	0.4	Topsoil	
0.4	2.3	CLAY, light brown, very plastic	Firm to v stiff
Groundwater		Not Encountered	
Note: Referenced from CPT10, TP7.1 & 7.2			

4.1.2 Lot 3 Subsoil Conditions

Table 3: Lot 3 Ground Model Summary

Top (m)	Base (m)	Description	Strength
0	0.3	Topsoil	
0.3	3.0	CLAY, brownish grey	Soft to firm
3.0	5.4	CLAY, trace organics, grey, mod plasticity	Firm to stiff
5.4	6.5	CLAY, trace of organics, dark grey, low plasticity with HW Sandstone rock fragments	Stiff to V stiff
6.5	9	H Weathered, Carbonaceous SILTSTONE, dark grey	V Weak
Groundwater		Not Encountered	
Note: Referenced from BH01, CPT08 & 9, TP4.1 & 4.2			

4.1.3 Lot 4 Subsoil Conditions

Table 4: Lot 4 Ground Model Summary

Top (m)	Base (m)	Description	Strength
0	0.4	Topsoil	
0.4	3.0	CLAY, brownish grey	Soft to firm
3.0	5.4	CLAY, trace organics, grey, mod plasticity	Firm to stiff
5.4	6.5	CLAY, trace of organics, dark grey, low plasticity with HW Sandstone rock fragments	Stiff to V stiff
6.5	9	H Weathered, Carbonaceous SILTSTONE, dark grey moderate plasticity	V Weak
Groundwater		Not Encountered	
Note: Referenced from BH01, CPT09, HA13, HA14 & HA16			

4.1.4 Lot 6 Subsoil Conditions

Table 5: Lot 6 Ground Model Summary

Top (m)	Base (m)	Description	Strength
0	0.2	Topsoil	
0.2	1.9	CLAY,	Very stiff
1.9	2.4	SAND, light greyish brown	V Dense
Groundwater		Not Encountered	
Note: Referenced from TP3.1 & 3.2, CPT01 & 06			

4.1.5 Lot 7 Subsoil Conditions

Table 6: Lot 7 Ground Model Summary

Top (m)	Base (m)	Description	Strength
0	0.3	Topsoil	
0.3	3.8	CLAY, sandy, mod to high plasticity	Firm
3.8	6.5	SAND, clayey, low to non-plastic	Dense
6	6.5	Mudstone / Siltstone fragments	V stiff
6.5	>7.95	H Weathered, Carbonaceous Siltstone	Weak
Groundwater		2m (BH04)	

Note: Referenced from BH04, TP2.1 & CPT03

4.1.6 Lot 8 Subsoil Conditions

Table 7: Lot 8 Ground Model Summary

Top (m)	Base (m)	Description	Strength
0	0.3	Topsoil	
0.4	1.8	CLAY, grey, medium high plasticity	Medium dense
1.5	3.0	CLAY, reddish brown with Mudstone fragments	Firm to v stiff
3	>6.45	H Weathered, Carbonaceous Sandstone / Siltstone	Weak
Groundwater		1.7m bgl (BH03)	
Note: Referenced from BH03, TP1.2, CPT04, HA09 to HA11			

4.1.7 Lot 9 Subsoil Conditions

Table 8: Lot 9 Ground Model Summary

Top (m)	Base (m)	Description	Strength
0	0.4	Topsoil	
0.4	1.0	CLAY, light brown	Firm to v stiff
1.0	2.7	CLAY, reddish brown with Mudstone fragments	Very stiff
Groundwater		2.7m bgl (TP1.1)	
Note: Referenced from TP01.1, HA01 to HA3, HA07			

4.1.8 Lot 10 Subsoil Conditions

Table 9: Lot 10 Ground Model Summary

Top (m)	Base (m)	Description	Strength
0	0.2	Topsoil	
0.2	1.0	CLAY, minor fine sand, grey, medium plasticity	Stiff to v stiff.
Groundwater		No encountered	
Note: Referenced from HA04 to 06			

4.2 Groundwater & Seepage

Groundwater encountered in testing location are summarised in Table 13 below:

Table 10: Groundwater Summary

Building Platform	Groundwater depth (mbgl)
Lot 1	Not Encountered (2.3m-8m)
Lot 3	Not Encountered (9m)
Lot 4	Not Encountered (9m)
Lot 6	Not Encountered (2.4m)
Lot 7	2m
Lot 8	1.7 – 2.7m
Lot 9	Not Encountered (5m)
Lot 10	Not Encountered (5m)

Groundwater measured in the upper slopes (Lot 3 to Lot 10) was recorded at less than 2.7m bgl and is likely to represent perched groundwater and seepage from gully headwalls.

Groundwater was not encountered in the lower slopes (Lot 1) to ~8m depth.

4.3 Infiltration Testing

Infiltration testing was undertaken on Lot 1 access driveway, for disposal of storm water runoff.

Infiltration tests are required to assess soakage in the proposed areas.

Infiltration test was performed in accordance with Section 8.5.2 "Procedure for conducting an infiltration test" of the Hawke's Bay Waterway Guidance Stormwater Management Guidelines, May 2009 as follows:

- One (1) Soak pit (SP) was completed to depth of 1.5m with
  - Recovered materials descriptions are in general accordance with the NZGS (2005) Guidelines; and
  - Borehole geometry recorded prior to testing.
- The borehole was filled with water, and maintained at a static level for up to an hour (1) until static water inflow stabilised;
  - The rate of water inflow (L/s) and water level (m) above the base of the hole was recorded during filling.
- The infiltration test was completed by turning off the water inflow and recording the fall in head (m) against time (minutes).

4.3.1 Infiltration Test Result

Logging of material recovered from soak pit (Hand Auger) indicates the proposed discharge area is generally underlain by:

- Clay Topsoil to 0.15m
- Medium plasticity, moist, stiff Clay to 1.5m
- Groundwater not encountered in soak pit.

The results of infiltration testing are attached in Appendix B and are summarised in Table 12, below.

Table 11 Infiltration Test Results

Test ID	Infiltration Rate (mm/hr)
SP01	16

5 LABORATORY TESTING

Samples from Test Pit excavations were submitted for laboratory testing to classify soils and to assess suitability for reuse as engineered fill, with test including:

- Atterberg Limits (LL, PL PI)
- Linear Shrinkage; and
- Standard Compaction.

The results of Lab Testing are presented in Appendix C.

5.1 LABORATORY TEST RESULTS

5.1.1 CLASSIFICATION TESTING

A Summary of Atterberg Limit and Linear shrinkage is presented below in Table 13. Lab results are presented in Appendix C.

In accordance with Section 17 of NZS3604:2011, expansive soils are classified where Liquid Limit (LL) exceeds 50% and Linear Shrinkage exceeds 15%.

All tests in orange are considered to be expansive soils, yellow is borderline expansive and green is non-expansive.

TABLE 12: LABORATORY TESTING SUMMARY

Lot	Sample ID	Depth	Atterberg Limits			Linear Shrinkage
		(m)	LL (%)	PL (%)	PI (%)	(%)
Lot 1	TP7.1	1.4	75	26	49	20
Lot 3	TP4.1	1.1	52	22	30	14
	TP4.2	0.6	101	29	72	21
	TP4.2	1.6	73	31	42	20
Lot 6	TP3.1	1.7	52	24	28	16
Lot 7	TP2.1	1.1	104	33	71	28
	TP2.1	2.2	46	20	26	14
Lot 9	TP1.1	0.8m	61	30	31	15
	TP1.1	1.4	80	29	51	20
	Notes: • Atterberg Limits NZS4402:1986 Test 2.2 to 2.4 • Linear Shrinkage NZS 4402: 1986 Test 2.6					

5.1.2 Standard Compaction Testing (NZ4402:1986, Test 4.1.1)

The results of standard compaction results indicate the reworked samples to be used for engineered fill is:

- Within +/- 2% of optimum water content;
- Peak shear strengths generally >100kPa;
- Residual strengths generally >25 kPa
- Peak to remoulded strength sensitivity is between 2 and 5 indicating moderately sensitive to sensitive clays (Colour coded soil from Table 13 applied)

Table 13: Standard Compaction Summary

Lot	Test ID	Depth (m)	Standard Compaction Test						Expansive Soils
			MDD t/m^3	OWC %	Nat WC %	Shear strength (kPa)			
						Peak	Resid.	Sens.	
Lot 1	TP7.1	1.4	1.47	25	24.9	UTP		--	Expansive
Lot 3	TP4.1	1.1	1.65	19	21	159	80	2	Borderline expansive
	TP4.2	0.6	1.44	29	30.3	108	24	4	Expansive
Lot 6	TP3.1	1.7	1.70	18	18.4	UTP	-	-	Expansive
Lot 7	TP2.1	1.1	1.36	31	29	183	56	3	Expansive
Lot 9	TP1.1	0.8m	1.53	24	22.9	150	30	5	Expansive
	TP1.1	1.4	1.52	26	28.5	185	60	3	Expansive

Notes:
- NZS4402:1986 Test 4.1.1

Orange indicates Expansive soils, yellow borderline and green, non-expansive.

6 GEOTECHNICAL ASSESSMENT

6.1 SITE SUBSOIL CLASS

The site subsoil class is assumed to be "Class C – Shallow Soil Site in accordance with NZS1170.5:2004, Earthquake Actions – New Zealand based on:

- Depth to soils do not exceed Class C depths in Table 3.2, 1170.5:2004;

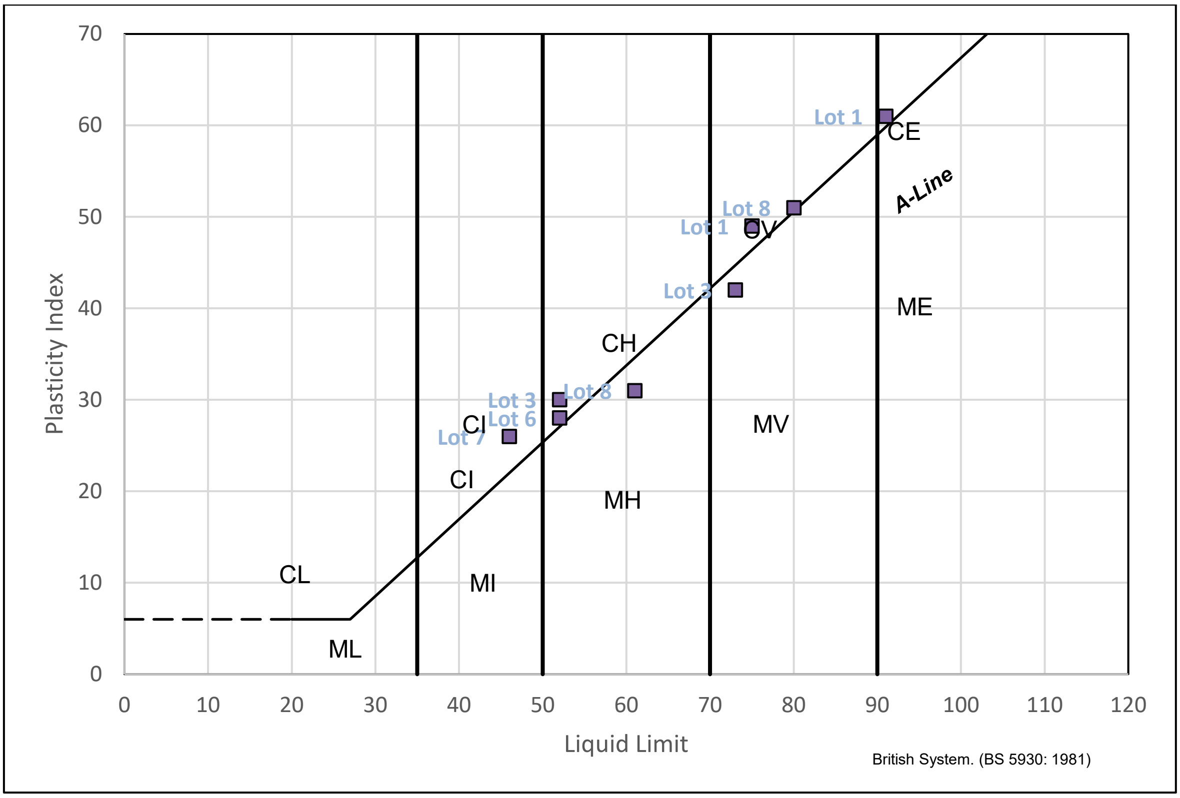
The site subsoil class was assessed based on RDCL borehole tests and nearby HBRC well database:

- RDCL Boreholes BH1 to BH4; indicating:
  - Depth to weak rock between 7m and 10m depth.
- Well 5345 & 5346; indicating:
  - Depth weak rock is ~9m bgl.

6.2 Soil Behaviour Type (A Line Plot)

The results of Liquid Limit vs Plasticity Index are Plotted against the A-Line (BS5930: 1981) (Graph 1) which indicates almost all samples site sit above the “A-Line” and exhibit low to extremely plastic “Clay like” behaviour.

Graph 1: A-Line Plot (BS5930:1981)



6.3 EXPANSIVE SOILS (SECTION 13, NZS3604:2011)

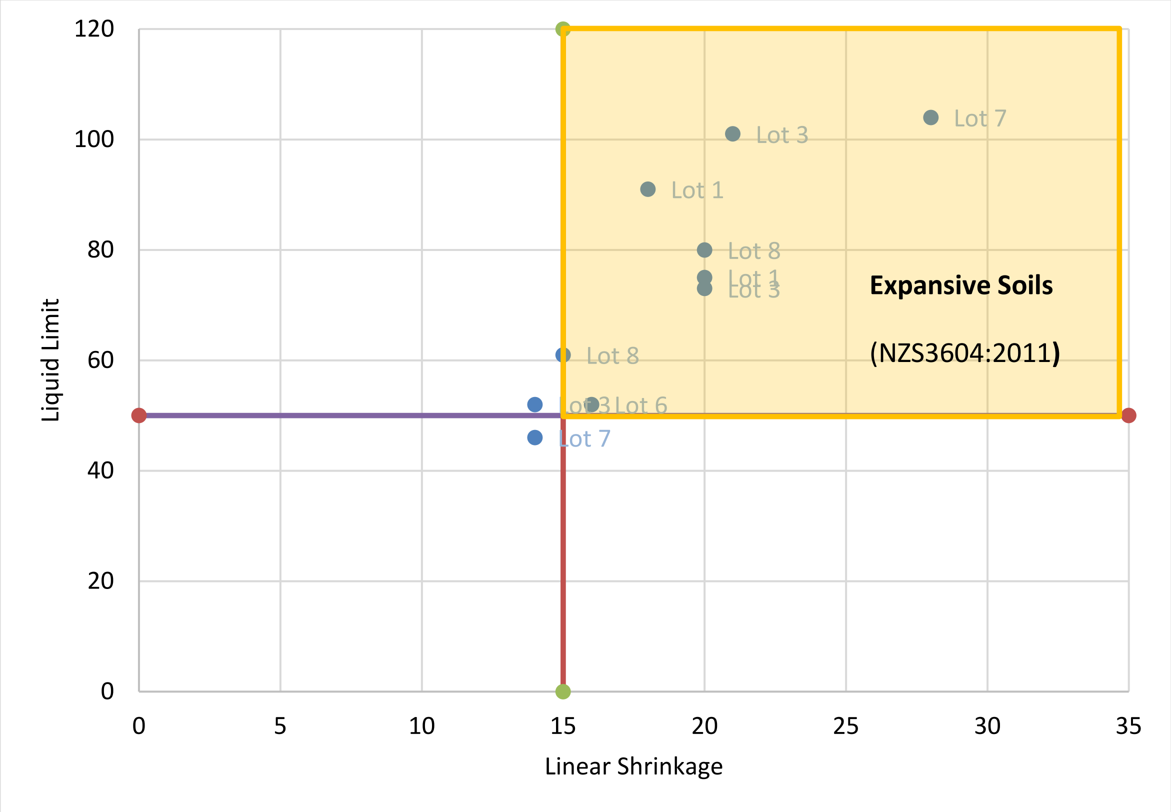
The top right quadrant with eight (8) samples with Linear Shrinkage >15% and Liquid Limit >50% plotted in Graph 2 indicates "Expansive soils" in accordance with NZS 3604:2011, Section 17.

A second group of two (2) samples are borderline expansive and may or may not display expansive behaviour.

The third group of two (2) samples indicate non-plastic (Sandy soils), Samples TP5.1, TP5.3 and are not expansive.

- Expansive and borderline expansive soils are in near surface soils at Lots 3 to 10 (upper slope) and Lot 1 & 2 (lower slope).

GRAPH 2: EXPANSIVE SOILS (NZS3604:2011).



6.4 GROUND MODEL & GEOTECHNICAL PARAMETERS

Twelve (12) critical cross-sections were developed for slope stability analyses (Figure 4). Of these, six (6) sections were assessed to be critical and have been assessed in detail. The remaining six sections have been assessed to be less critical and are no longer discussed in this report.

The cross-section locations are presented in Figure 4 and include:

- Lot 1 (Section 10);
- Lot 3 (Section 7);
- Lot 6 (Section 6);
- Lot 7 (Section 3);
- Lot 8 NS (Section 0);
- Lot 8 EW (Section 0).

Ground models were developed using site investigation information (BH, CPT, TP, HA & DCP results) interpolating between testing locations, and our knowledge of the site geology and site observations (Appendix D).

Geotechnical parameters presented in Table 15 and Appendix E and have been developed using Borehole geological descriptions and SPT N_{60} data, CPT and correlation to geotechnical parameters and references:

- CPet-IT v3.0 CPT Interpretation software;
- NZGS (2005) Field Description of Soil & Rock;
- Burt Look (2007); Handbook of Geotechnical Investigation & Design Tables;
- FHWA (April 2017) Geotechnical Site Characterisation Circular No. 5;
  - Meyerhof, 1956;
  - Hatanaka and Uchida, 1996;
  - Stroud, 1974, 1989.

TABLE 14: GEOTECHNICAL PARAMETERS

Material Type	Field Strength / Density	RAW SPT N (Average)	CPT Equiv. SPT N60 (Average)	Unit Weight (γ) kN/m3	Cohesion (c') kPa	Friction Angle (Ø') degrees	Undrained Shear strength (Su)
CLAY, mod to high plasticity, trace of organics (Expansive soil)	Stiff to Very Stiff	4 to 8 (4)	2 to 12	18	7.5	30	50
Silty SAND, yellowish brown; with Clay mixtures	Medium Dense or stiff	13 to 38 (26)	5 to 35	18	3	30	-
HW Carbonaceous SANDSTONE (Disturbed)	Very weak, Hard	-	-	21	10	40	-
HW carbonaceous SILTSTONE / MUDSTONE/ SANDSTONE	Very weak to weak	11 to 50+ (35)	(Too hard to penetrate)	21	15	40	-

6.4.1 ESTIMATED GROUND MOTION PARAMETERS

The assumptions used for residential development are:

- Importance Level 2 (IL2) in accordance with NZS1170.5:2004[9].
- 50-year design life
- Horizontal Seismic parameters Peak Ground Acceleration (pga) were derived in accordance with MBIE (Nov 2021) Module 1.
- For Serviceability Limit State (SLS):
  - Magnitude (M) = 6.4;
  - 25-year return period;
  - Peak Ground Acceleration (PGA) = 0.12g.
- For Ultimate Limit State (ULS):
  - Magnitude (M) = 7.1;
  - 500-year return period;
  - Peak Ground Acceleration (PGA) = 0.58g.

A factored seismic coefficient of 0.5 was used in ULS case which allows for ductility in soils. Soils are exposed to full peak acceleration for only a short period of time. As a ductile material, they are also able to accommodate a limited amount of displacement before catastrophic failure occurs. We have chosen:

$$PGA_{\,ULS} = a_{max} \text{ x } 0.5$$

on the basis of the following:

- NZ Geomechanics News (Dec 2018): Seismic Design of Geotechnical Structures for NCTIR. "In recognition of the fact that actual slopes and many retaining structures are not a rigid body and that the peak acceleration exists for only a short time, the pseudo-static coefficients used in practice generally correspond to acceleration values well below αmax.
- MBIE (Nov 2021) Module 6, Earthquake Resistant Retaining Wall Design uses reduced Wd factors for walls.
- ISSMGE (Feb 2015) New Zealand Simplified Seismic slope stability analysis and risk-based slope Design for earthquake resistance.

[9] NZS1170.5:2004

6.4.2 Target Factor of Safety

Based on requirements of the New Zealand Building code, MBIE Module 6, and industry standard the following minimum acceptable FoS have been adopted for each design case:

- Static condition with normal water level, FoS ≥ 1.5;
- Static condition with elevated water level, FoS ≥ 1.2; and
- ULS earthquake event with undrained shear strength, FoS ≥ 1.2.

6.5 Stability Analysis Results

The results of slope stability analyses are discussed following with outputs in Appendix F.

6.5.1 Lot 1 (Section 10) Stability Results

The stability assessment addressed the slope above (west) of Lot 1.

The stability analyses met the Target Criteria for all conditions (Table 16).

Table 15 Sections 1 & 2 (Section 10) Stability Results

Description	Groundwater condition	Target Criteria	FOS	Comment
Static (Drained)	Normal GW	1.5	1.6	
Static (Drained)	Elevated GW	1.2	1.3	Affecting building Platform – Recommend setback of 5m from toe
Pseudostatic ULS*0.5 (Undrained)	Normal GW	1.2	1.2	Affecting building Platform – Recommend setback of 5m from toe

6.5.2 Lots 3 & 4 (Section 7)

The stability analyses were undertaken assuming current landform (prior to any earthworks) on Lot 3, but applies to Lot 4 due to similar, topography, geology and groundwater regime.

The stability analyses met the Target Criteria for all conditions (Table 17).

TABLE 16 LOT 3 & 4 (SECTION 7) STABILITY RESULTS

Description	Groundwater condition	Target Criteria	FOS	Comment
Static (Drained)	Normal GW	1.5	1.5	5m setback inside building platform
Static (Drained)	Elevated GW	1.2	1.4	
Pseudostatic ULS*0.5 (Undrained)	Normal GW	1.2	1.2	Affecting building platform

6.5.3 LOT 6 (SECTION 6)

The stability analyses were undertaken assuming current landform (prior to any earthworks) and static conditions with normal and elevated groundwater conditions; and pseudostatic conditions with normal groundwater.

The stability analyses met the Target Criteria for all conditions (Table 18).

TABLE 17. LOT 6 (SECTION 6) STABILITY RESULTS

Description	Groundwater condition	Target Criteria	FOS	Comment
Static (Drained)	Normal GW	1.5	1.8	-
Static (Drained)	Elevated GW	1.2	1.4	Affecting building platform
Pseudostatic ULS*0.5 (Undrained)	Normal GW	1.2	1.5	-

6.5.4 LOT 7 (SECTION 3)

The initial stability analyses were undertaken assuming current landform (prior to any earthworks) and considered static drained with normal and elevated groundwater conditions and pseudostatic conditions with normal groundwater.

The slope stability analysis met the criteria for Static conditions assuming normal and elevated groundwater conditions (See Table 19).

The analysis did not meet the Target Criteria for pseudostatic (undrained) condition (FoS 1.1). This indicates that a rotational failure may be triggered during large seismic events.

6.5.4.1 LOT 7 STABILITY ANALYSIS (MODIFIED SLOPE PROFILE)

Lot 7 was remodelled assuming up to ~3m of cut will be undertaken at the slope crest to level the building platform. This will significantly reduce the driving force to resisting force ratio and will improve factor of safety.

The analyses achieved a factor of Safety under Pseudostatic, normal conditions of 1.2.

IMAGE 3: LOT 7 STABILITY ANALYSES RESULTS BEFORE EARTHWORKS (LEFT) AND AFTER RECOMMENDED EARTHWORKS (RIGHT)

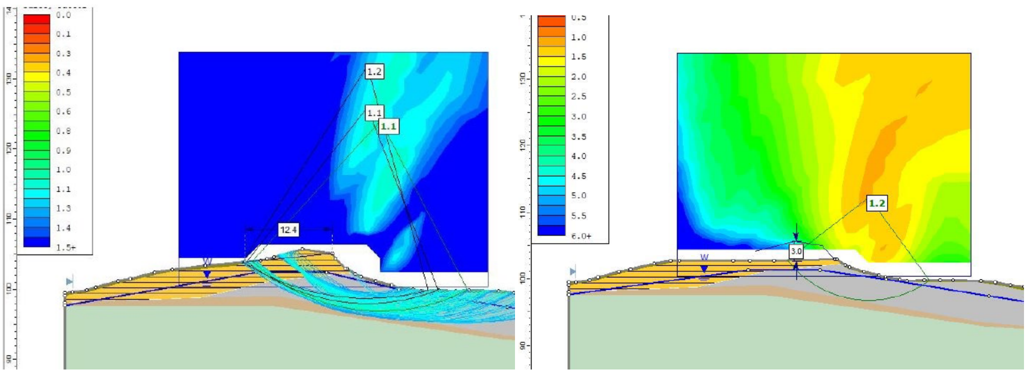


TABLE 18 LOT 7 STABILITY RESULTS SECTION 1

Description	Groundwater condition	Target Criteria	FOS	Comment
Static (Drained)	Normal GW	1.5	1.6	-
Static (Drained)	Elevated GW	1.2	1.2	-
Pseudostatic (Undrained)	Normal GW	1.2	1.1	Setback 12m from slope crest -
Pseudostatic (Undrained) With 3m excavation	Normal GW	1.2	1.2	Slope reduced by ~3m

6.5.5 LOT 8 (SECTION 0) NORTH TO SOUTH

The analysis has been undertaken on the elevated hillock assuming left to right and right to left orientations.

6.5.5.1 LEFT TO RIGHT

Stability analyses meet the Target Criteria for all conditions (Table 20).

TABLE 19:LOT 8 (SECTION 0) NS STABILITY RESULTS

Description	Groundwater condition	Target Criteria	FOS	Comment
Static (Drained)	Normal GW	1.5	2.2	-
Static (Drained)	Elevated GW	1.2	2.0	Affecting BP
Pseudostatic ULS*0.5 (Undrained)	Normal GW	1.2	1.2	-

6.5.5.2 Right to Left

Stability analyses meet the Target Criteria for all conditions (Table 21).

Table 20: Lot 8 (Section 0) NS Right to Left Stability Results

Description	Groundwater condition	Target Criteria	FOS	Comment
Static (Drained)	Normal GW	1.5	2.0	-
Static (Drained)	Elevated GW	1.2	2.0	-
Pseudostatic ULS*0.5 (Undrained)	Normal GW	1.2	1.2	Not affecting building platform

6.5.6 Lot 8 (Section 0) West to East

Stability analyses meet the Target Criteria for all conditions (Table 22).

Table 21: Lot 8 (Section 0) EW Stability Results

Description	Groundwater condition	Target Criteria	FOS	Comment
Static (Drained)	Normal GW	1.5	1.9	-
Static (Drained)	Elevated GW	1.2	1.5	-
Pseudostatic ULS*0.5 (Undrained)	Normal GW	1.2	1.2	5m setback inside building platform

6.5.7 Slope Stability Summary

Site investigations indicated the risk of land instability was confined to shallow (<5m deep) failures and with no obvious indication of “deep” (>5m deep) failure.

Slope stability is considered to be primarily due to strength loss as a result of expansive soils, and not due to weak undisturbed materials, or structurally controlled failure.

6.6 Definition of “Good Ground” and Expansive Soils

The near surface soils tested at the proposed building platform location are expansive and fall outside the definition of “Good Ground” in accordance with NZS3604:2011, Section 17, where:

- LL>50%; and
- Linear Shrinkage >15%.

Specific engineering design of foundations will be required to mitigate expansive soils.

Expansive soils have strict requirements for building structures under B1/AS1, 28/11/2019, discharging stormwater and wastewater and planting of large trees which may seasonally affect ground water.

6.7 Shallow Bearing Capacity Assumptions

Estimates of Ultimate Bearing Capacity have been inferred at building platform locations based on correlations made by M.J. Stockwell (1977).

Based on these correlations, the depth to 200kPa and 300 kPa is presented in Table 23.

Table 22: Depth to 200kPa & 300 kPa Bearing Capacity

Building Platform	Depth to 200kPa (m)	Depth to 300kPa (m)
Lot 1	0.2	1.5
Lot 3	0.8	0.8
Lot 4	0.4	0.6
Lot 6	0.2	0.3
Lot 7	0.2	1.4
Lot 8	0.8	1.0
Lot 9	0.4	0.6
Lot 10	0.4	0.5
Notes: From M.J. Stockwell (1977)		

6.8 Access to Building Platforms

According to drawing provided by the client, access to Lots 3 to Lot 10 will be from Williams Road and may cross areas where slopes are susceptible to soil creep and expansive soils that may be susceptible to long term active instability.

Access to Lot 1 will be from Okura Road via the current lane which is located over historical landslide debris runout which is currently stable when not disturbed by cut or fill.

6.8.1 California Bearing Ratio (CBR) Assumptions

Estimates of CBR subgrade strengths have been inferred at building platform locations based on correlations made by M.J. Stockwell (1977).

Based on these correlations, estimated subgrade CBR strengths are:

Table 23: Estimated CBR% Subgrade Strengths (From M.J. Stockwell, 1977)

Lot	Estimated CBR %
Lot 1	3 to 5
Lot 3	1 to 5
Lot 4	1 to 5
Lot 6	5 to 8
Lot 7	3
Lot 8	3 to 5
Lot 9	3 to 5
Lot 10	3 to 5

7 GEOTECHNICAL RECOMMENDATIONS

7.1 BUILDING PLATFORMS SUITABLE FOR DEVELOPMENT

The proposed building platforms has been chosen based on knowledge obtained from site observations and testing to be most suitable for residential development. The long-term performance of these building platforms relies on the recommendations and consent conditions made in the report.

7.2 BUILDING PLATFORM DESIGN

To address the risk of expansive soils, all suitable building platforms are subject to Specific Engineering Design (SED). Further considerations are required to address slope stability and setback conditions.

7.2.1 BUILDING PLATFORMS AT TOP OF SLOPES

Where building platforms are to be formed at the top of existing slopes, these should be formed entirely within cut (Natural ground).

Engineered Fill may be utilised to form larger building platforms provided they can be proved to be modified as suitable fill otherwise should be used for minor structures and landscaping only.

7.2.2 BUILDING PLATFORMS AT TOE OF SLOPES

Where building platforms are to be formed at the toe of existing slopes (Lot 1), these should be setback from the toe of slopes and excavation controlled or retained.

Engineered Fill may be utilised to form larger building platforms provided they can be proved to be modified as suitable fill otherwise should be used for minor structures and landscaping only.

7.3 FOUNDATION RECOMMENDATIONS

To address the risk of expansive soils, all building platforms should be tested for expansive properties at or during the completion of the building platform.

Further considerations are required to address slope stability and setback conditions.

Foundations exposed to risk of expansive type soils as defined in NZS3604 are subject to Specific Engineering Design (SED).

All foundations should be designed for adequate stiffness and strength to resist the expansive nature of the ground by:

- Combination of ground improvement with enhanced foundation design using:
    - Shallow Waffle raft slabs in accordance with Appendix F of AS2870:2011; or
    - Timber pole foundations embedded below expansive soil horizons assumed to be 1.5m to 2m bgl.

7.3.1 BEARING CAPACITY

Where natural ground is exposed, inferred Ultimate Bearing Capacity (UBC) is anticipated to achieve:

- ~ 200kPa UBC from 0.2m to 0.8m bgl; or
    - 66kPa Allowable.
- ~300kPa UBC from 0.3m to 1.5m bgl; or
    - 100kPa Allowable.
- Bearing capacity assumed to be at a depth clear of topsoil, organic or disturbed material.

Where engineered fill is formed bearing capacity shall be assessed following the completion of earthworks and is anticipated to achieve:

- 300 kPa UBC in accordance with NZS3604:2011, provided:
    - Expansive soils are modified by lime or cement additives;
    - Earthworks are undertaken in accordance with NZS4431:2022[10].; or
    - Piles embedded below expansive soils are used.

[10] NZS4431:2022[10]: Engineered Fill Construction for Lightweight Structures

7.3.2 Building Platform Design

To account for expansive soils, all foundations should be designed for adequate stiffness and strength to resist the expansive nature of the ground by:

- Combination of ground improvement; with
- Enhanced Foundation Design using:
  - Shallow Waffle raft slabs in accordance with Appendix F of AS2870:2011; or
  - Timber pole foundations embedded below expansive soil horizons assumed to be 1.5m to 2m bgl.

7.3.3 Building Setback Restriction

To address natural slope regression and land instability the following building setback restriction should be applied (Figure 5):

- Where land falls below the building platform:
  - Building setback of 5 m is recommended inside the break in slope (slope crest) for all building platforms where ground slopes away exceeding 20 degrees; and/or
- Where land rises above the building platform:
  - Building setback of 5m from the toe of slope is recommended where ground rises above the building platform (Lot 1).

Building within the building setback area requires the engagement of a geotechnical engineer and may require Specific Engineering Design including:

- Deepened piles subject to engineering design at building consent; or
- Retention systems such as walls or barriers.

Building within the building setback area requires the engagement of a geotechnical engineer and may require Specific Engineering Design including:

- Deepened piles subject to engineering design at building consent.
- Final building setback restrictions shall be reviewed at the completion of each building platform as part of the Completion Report Schedule 2a.

7.3.4 Building Design Restrictions

Due to the risk of expansive soils, buildings shall be designed with the following restrictions in accordance with B1/AS1, 28/11/2019:

- Single storey, stand-alone household unit; and
- Maximum length or width of floor of 24.0 m including any attached garage; and
- Simple plan shapes such as rectangular, L, T or boomerang; and
- Concrete slab-on-ground with a minimum thickness of 100 mm and a minimum concrete compressive strength of 20 MPa; and
- Simple roof forms, with maximum overall height of 7.0 m to roof apex; and
- Maximum span of roof truss 12.0m; and
- External walls maximum of 2.4m height studs, other than gable end walls and walls to mono-pitched roofs, which shall not exceed 4.0m.

7.4 Earthwork Recommendations

Earthworks completed by cut and fill are expected for the development of building platforms and access. All earthworks should be undertaken in accordance with NZS4431:2022.

7.4.1 Source Material Type

Site materials for construction of bulk fills for building platforms are considered to comprise:

- Material Type T (Topsoil).
- Material Type F (Fine grained soil);
- Material Type I (Intermediate-grained soil);

7.4.2 Material Condition

Material condition is anticipated to vary. Typically, site material is anticipated to comprise:

- Type U1 (Physically unsuitable); due presence of sensitive clays.

Occasional excavations will encounter:

- W (Wet) where occasional seep is encountered;
- D (Dry) which may require conditioning;
- A (acceptable).

Further classification is required at building consent stage which should incorporate the following characteristic tests:

- Particle Size Distribution (with Hydrometer method) NZS4402:1988; Test 2.7.1 & 2.8.4.
- Plasticity Index Testing (NZS4402:1988; Test 2.2 to 2.4);
- NZ Standard Compaction Tests (NZS4402:1988; Test 4.1.1).

7.4.3 Physically Unsuitable (U1) Materials

U1 material is unsuitable for immediate placement in an engineered fill and it should:

- Be classified following section C3.4.5 of NZS4431:2022;
- Determinate if U1 can be used when modified in accordance with the earthwork's specification by mechanical, chemical or other means, by geotechnical designer;
- If earthwork is successful, the material should then be reclassified.

Otherwise, these materials should be safely removed from site. Excavation depths to be verified by a geotechnical professional before reinstating with engineered fill and recorded by survey As Built.

7.4.4 Expansive Soil Modifications and Additives

Site won material will be susceptible to expansive soils unless modifications or additives are applied to the fill.

Additives may include:

- Cement stabilisation: Known to decrease liquid limit and increases plasticity index and workability of clays (Mohammed Y Fattah[11]).
    - Suitable when fine fractions (passing 200 sieve) <40% or
    - LL < 45 to 50; and
    - PI <25.
    - Typical cement content for inorganic Clay (MH or CH) is 8% to 12%.

[11] Mohammed Y. Fattah (2010) A treatment of expansive soil using different additives.

- Hydrated Lime or Quicklime stabilisation:
  - Reduce Linear shrinkage of expansive soils;
  - Plasticity reduction;
  - Reduction in moisture;
  - Increased Unconfined Compressive strength (UCS) values.

7.4.5 Cement/Lime Additive Testing

For building consent stage, relevant samples should be tested under laboratory conditions for reactivity to determine which method (Lime or cement) may be appropriate additives for construction.

7.4.6 Cut Slopes

Cut slopes shall be formed with the following criteria:

- Stiff, natural and undisturbed slopes should not exceed 1 vertical: 1.5 horizontal (1V:1.5H).
- Firm, loose or disturbed landslide materials will require specific design.
  - Some further modifications may be required to reform slopes at reduced angles where disturbed (debris flow) materials may be encountered.
- Planting of slopes with grass or small, shallow rooting plants are recommended to stabilise cut slopes.
- Planting of large tree species with large root bowls are not recommended.

7.4.6.1 Earthworks Benching

Excavations receiving fill shall be benched for earthworks in accordance with NZS4431:2022 requirements and to be confirmed onsite with the geotechnical professional.

7.4.7 Engineered Fill Construction

Engineered Fill slopes shall be formed at a maximum grade of 1 vertical to 2 horizontal (1V:2H).

All unsuitable material shall be excavated from the proposed fill area to the exposed natural ground and benched to accommodate engineered fill in accordance with TNZ F1 and shall be confirmed onsite with the geotechnical professional.

Fill shall be placed under engineering control using appropriate compaction plant and equipment and tested to meet compaction compliance.

- Engineered fill criteria should be verified by implementing a compaction trial to confirm methods for compaction and targets with the appropriate equipment onsite.
- Engineered Fill compaction to be verified by:
    - Nuclear Density Method (NDM) by an IANZ accredited laboratory;
    - Visual observation;
    - Additional verification testing by Proof Roll, Dynamic Cone Penetrometer (DCP) or shear vane testing as required.

Target Compaction Criteria shall be:

- 95% MDD at Optimum Water Content (NDM).
- Planting of slopes with grass or small, shallow rooting plants are recommended to stabilise cut slopes.
- Planting large tree species are not recommended.

7.4.8 Road Access

Road access to building platform should be suitable provided road construction is designed to take advantage of resistant outcrops and keep away from wet, boggy terrain unless adequate drainage and ground improvement is installed.

- Alignment as confirmed by the geotechnical engineer;
- Variable subgrade strength and future traffic loads including construction traffic;
- All subgrades should be stripped of disturbed material (organic loose and deleterious materials);
- The carriageway should be designed to consider subsoil drainage and stormwater discharge.

All roads should collect stormwater by appropriate collection points using side drains, kerb and channel and discharge to appropriate discharge areas approved by the local authority.

7.4.9 Preliminary Design Subgrade CBR%

For preliminary road design, Inferred CBR results from DCP testing in accordance with M.J Stockwell (1977) indicates:

- Preliminary Design Subgrade CBR varies between 1% and 5%.

Naturally, there will be some variability where localised subgrade strengths exceed or do not meet the above criteria and should be addressed during construction.

Subgrade construction should consider lime or cement modifications to address expansive soils discussed above.

7.5 Stormwater

Stormwater design has been undertaken by StrataGroup Consulting Engineers and Drawing Set J5864 checked by RDCL Geotechnical engineers for application to these sites.

The design accommodates typical stormwater detention arrangements with above ground tanks feeding a “Bubble up” Sump & subsoil drain (Drawing C500) onto gently sloping ground downslope and away from future building platforms.

7.5.1 Stormwater Pond

The existing Stormwater Pond will be utilised as a Stormwater Attenuation reservoir, discharging to the slopes below.

Site testing has not included assessment for the suitability of the stormwater pond embankment and stability and this should be completed as part of building consent.

A Soakage test in SP01 recorded 16mm/hour soakage and near surface soils comprise clays with assumed permeability k factors of between 1 x 10^{-6} & 10^{-9} considered to be “poor to impervious”.

As a minimum we recommend:

- Stormwater from the proposed buildings, access, and other impervious surfaces should be discharged to a suitable point away from the proposed building platforms.
- Any Cut slopes should incorporate cut-off drains to prevent surface water discharging over the face.
- Engineered Fill slopes should be finished with a general crossfall into the slope to prevent surface water being discharged over slopes.

7.6 WASTEWATER DISPOSAL

A wastewater disposal Plan (Sheet C300) has been prepared to demonstrate the viability of onsite wastewater disposal areas and ensure the proposed parcels are large enough to accommodate a wastewater disposal field.

This plan is not intended for construction. Future lot owners will be responsible for the construction of their own wastewater treatment and disposal, associated design, consents, and compliance with any applicable consent notices".

Design of effluent disposal fields should be undertaken by Specialist Engineers considering AS/NZS1547 -2012 On-Site Domestic Wastewater Management and Hawkes Bay Regional Resource Management Plan.

Indicative locations for effluent disposal are shown on the StrataGroup drawing Set C300.

Based on this (AS/NZS1547-2012), soil category has been classified as:

- Soil Category 5, "Medium to heavy clays", and special design techniques will be required to enable their use for land application system.
- RDCL has reviewed anticipated dosage rates and understand these to be as follows:
    - For Slopes >20%, discharge rates of ~1L/m^2/day
    - For Slopes <20%, discharge rates of ~1.5L/m^2/day

All sites have been targeted to be located on land that is not excessively steep or unstable and should be suitable provided the system occupies gentle sloping terrain preferably downslope of the proposed building platform and located a suitable distance away from the building platform or steep slopes. Specialist planting will need to be undertaken within these areas.

7.7 DRAINAGE

7.7.1 SUBSOIL DRAINAGE

Subsoil drains are anticipated to be incorporated for road construction or where collection of seepage is considered pertinent to design.

Subsoil drains may comprise geotextile wrapped aggregate subsoil drains "French Drains" in accordance with TNZ F6:2003. The subsoil drains will comprise:

- Min ~0.3 m wide x 0.5 m to 1.0 m deep trench;
- Wrapped with a geotextile filter fabric Geotextile Strength Class B (TNZF7: 2003); with
- 100 mm diameter punched HDPE pipe (Nexus©, or equivalent); and
- Filled with <40 mm drainage aggregate.

The French drains shall connect to sumps or manholes using a sold walled unpunched PE Pipe (Nexuscoil©, or equivalent).

7.7.2 CUT-OFF DRAINS

Cut-off drains should be installed at the crest of all cut slopes where surface sheet flow can be collected.

A cut-off drain should incorporate a Min 0.5 m deep "v-drain" formed with minimum 1:100 ratio slope to be discharged into existing gullies away from proposed building platforms.

Cut-off drains may need to be required above building platforms.

7.7.3 RETAINING STRUCTURES

Any retaining walls greater than 1.5 m high or supporting building platforms and/or access require:

- Specific design by a suitably qualified structural engineer with input by a geotechnical engineer as required:
- Appropriate parameters for the design of retaining walls should be confirmed at building consent stage based on soils encountered at the specific site of construction.
- Stormwater management to ensure retained soils remain drained.

All retaining walls should be backfilled with free-draining materials with sub-soil drains to capture and direct water away for adequate disposal.

7.8 PLANTING

Due to the expansive nature of soils, strict control on planting is required. We recommend all cut and fill slopes to be planted with small shrubs and shallow rooting plants.

Large tree species may not be planted within a horizontal distance equivalent to the mature tree height of any pertinent structure (house, road, stormwater, drainage).

8 AS-BUILT RECORDS

As-Built records should be recorded for the following;

- Documenting excavation works including topsoil strip by survey;
- Documenting fill volume progress by survey records;
- Compiling earthworks records including;
  - Material source testing;
  - Modification by additives;
  - Compaction verification certification;
  - Recording and certifying failed compaction areas;
- Installation of buried services including subsoil drains and ground improvement by survey;
- Installation of buried structures including geofabric or geogrid reinforcement;
- Any site Instructions.

9 RESIDUAL RISK ASSESSMENT

The risk assessment was undertaken in accordance with AGS Guideline for Landslide Susceptibility, Hazard & Risk Zoning for Land Use Planning to meet compliance in accordance with Section 106 of the Resource Management Act.

The risk assessment considers likelihood of threats occurring and resulting consequence to Property and Health & Safety (Table 25).

Mitigation measures outlined below have been carried through to recommendations and consent conditions in this report.

Based on these mitigation measures, and where these are applied correctly under engineering guidance and control, we consider the residual risk of land instability affecting each building platform and access way to be no more than "Low".

Where the residual risk cannot be practicably mitigated to "Low", we consider the mitigation measures taken to reduce the natural hazard is as low as reasonably practicable. The proposed mitigation measures will not exacerbate the natural hazard risks in other areas.

9.1 EXPANSIVE SOILS

For expansive soils, cycles of wetting and drying may lead to differential movement causing cracking or damage to rigid surfaces such as concrete foundations, wall cladding and linings or pavement surfaces.

The Likelihood of expansive soils causing property damage to buildings and road access before engineering control are considered to be "Likely" (B).

The consequence of expansive soils before engineering control is likely to be:

- "Minor" (4) due to indicative cost of damage to be 5% of property values.
- Risk level Moderate Risk requiring treatment options to reduce the risk to "Low".

Expansive Soils can be mitigated with engineering control by (See Section 7):

- Testing for expansive soil properties at building platforms;
- Incorporate enhanced foundation design for IL2 building structures & building design restrictions; and
- Mitigate risk to road access with expansive soil modifications (Lime or cement).

The effect of engineering control mitigation will reduce Likelihood of expansive soils from "Likely" (B) to "Unlikely" (D).

The Residual Risk Level after mitigation is considered to be "Low Risk", usually acceptable to regulators.

9.2 LAND STABILITY – SHALLOW CRUSTAL EARTHQUAKE

Based on the review of historical images and site records, there is little evidence of shallow crustal earthquakes generating significant shaking intensity to cause large scale landslides or activate existing debris flows.

The Likelihood of shallow crustal earthquakes occurring during the design life (50 years) is considered "Likely" (B), considering a 1/100-year event.

The consequence of shallow crustal earthquake before engineering controls leading to risk of property damage is Minor (4) but is unlikely to impact loss of life.

The initial Risk level is Moderate Risk.

Seismically induced landslides can be mitigated by:

- Strategic location of building platforms outside of known earth & debris flow;
- Stability assessment to address theoretical risk.
- Reduction of building platform and slope height significantly reduces the risk; and
- Building setback criteria moves the risk outside of the hazard;
- Planting and erosion protection.

With engineering control mitigation, the Likelihood of property damage to the building platforms and road is considered to be "Unlikely (D)".

The Residual Risk Level after mitigation is considered to be "Low Risk", usually acceptable to regulators.

9.3 LAND STABILITY – HSZ INDUCED LANDSLIDE

Land damage associated with the Hikurangi Subduction Zone (HSZ) Fault Rupture is likely to lead to widespread across the North Island region.

The Likelihood of this event occurring is proportional to the recurrence interval (1 in 500yr) event and is considered "Possible" (3).

The consequence of a HSZ event without engineering control is considered to be "Medium (3) due to large scale land instability leading to significant property damage and risk to life. Property damage includes damage to the building platform, structural damage to buildings, road access and potential loss of life in a large-scale landslide event.

The likelihood and consequence of damage can be partially mitigated by:

- TA accepting 1 in 500-year Recurrence Intervals for large earthquake events.
- Strategic location of building platforms outside of known earth & debris flow;
- Building in accordance with current Building Act regulations and guidelines.
- 1170.0 Importance Level 2 (IL2) structures designed to meet ULS (Life Safety) objectives.

With the above engineering controls implemented, the likelihood of a large earthquake occurring remains “Possible”. The engineering control implemented could arguably reduce the consequence of damage from “Medium” to “Minor” on the basis of approximate cost of damage.

The Risk level remains “Moderate” and may be tolerated in certain circumstances (Subject to regulatory approval).

9.4 LAND STABILITY – RAINFALL INDUCED LANDSLIDE

Rainfall induced landslide could lead to instability impacting future building platforms, house foundations and road access.

The Likelihood of rainfall induced landslides are associated with larger 1 in 100-year events (1×10^{-2}) considered to be “Likely” (B).

The consequence of rainfall induced Landslides before engineering control could lead to “Minor” (4) property damage but is unlikely to lead to loss of life.

The likelihood of Rainfall Induced Landslides after engineering control can be reduce by implementing:

- Strategic location of building platforms outside of known earth & debris flow;
- Stability assessment to address theoretical risk.
- Reduction of building platform and slope height significantly reduces the risk; and
- Building setback criteria moves the risk outside of the hazard;
- Planting and erosion protection.

The effect of engineering control mitigation will reduce Likelihood of rainfall induced landslides from “Likely” (B) to “Unlikely” (D).

The Residual Risk Level after mitigation is considered to be “Low Risk”, usually acceptable to regulators.

TABLE 24: LOT 1, 3, 4, 6, 7, 8, 9 & 10 RESIDUAL RISK ASSESSMENT

Hazard	Initial risk			Mitigation	Residual Risk		
	Likelihood	Consequence	Initial risk		Likelihood	Consequence	Residual Risk
Expansive Soils risk to building foundations & damage to access	Likely (B)	Minor (4)	Moderate	Building Platforms & site won fill to be tested for expansive properties. Road access to consider expansive soil modifications. Enhanced Foundation Design for IL2 building structures & building design restrictions (Section 7).	Unlikely (D)	Minor (4)	Low
Land Instability (Shallow Crust EQ)	Likely (B)	Minor (4)	Moderate	Strategic location of building platforms outside of known earth & debris flow. Stability assessment to address theoretical risk. Reduction of building platform and slope height significantly reduces the risk; and	Unlikely (D)	Minor (4)	Low

Hazard	Initial risk			Mitigation	Residual Risk		
	Likelihood	Consequence	Initial risk		Likelihood	Consequence	Residual Risk
				Building setback criteria moves the risk outside of the hazard. Planting and erosion protection.			
Land Instability (HSZ Induced)	Possible (C)	Medium (3)	Medium	Strategic location of building platforms Building in Accordance with current Building Act regulations and guidelines. IL2 structures designed to meet ULS (Life Safety) objectives.	Possible (C)	Minor (4)	Moderate
Land Instability (Rainfall Induced)	Likely (B)	Minor (4)	Medium	Strategic location of building platforms Stability assessment to address theoretical risk. Reduction of slope height at building platform significantly reduces the risk. Apply building setback restrictions. Planting	Unlikely (D)	Minor (4)	Low

Hazard	Initial risk			Mitigation	Residual Risk		
	Likelihood	Consequence	Initial risk		Likelihood	Consequence	Residual Risk
Tsunami Risk (Lot 1 Only)	Unlikely (D)	Medium (3)	Low	No practicable engineering mitigation for Tsunami Risk due to likelihood of occurrence.	Unlikely (D)	Medium (3)	Low
Flood Risk	Possible (C)	Minor (4)	Medium	NZBC E1 minimum requirements for flood hazard	Unlikely (D)	Minor (4)	Low
Liquefaction Susceptibility	Unlikely (D)	Minor (4)	Low	Material composition and qualitative liquefaction assessment	Unlikely (D)	Minor (4)	Low

TABLE 25: AGS QUALITATIVE RISK ANALYSIS

QUALITATIVE RISK ANALYSIS MATRIX – LEVEL OF RISK TO PROPERTY

LIKELIHOOD		CONSEQUENCES TO PROPERTY (With Indicative Approximate Cost of Damage)				
	Indicative Value of Approximate Annual Probability	1: CATASTROPHIC 200%	2: MAJOR 60%	3: MEDIUM 20%	4: MINOR 5%	5: INSIGNIFICANT 0.5%
A – ALMOST CERTAIN	10^{-1}	VH	VH	VH	H	M or L (5)
B - LIKELY	10^{-2}	VH	VH	H	M	L
C - POSSIBLE	10^{-3}	VH	H	M	M	VL
D - UNLIKELY	10^{-4}	H	M	L	L	VL
E - RARE	10^{-5}	M	L	L	VL	VL
F - BARELY CREDIBLE	10^{-6}	L	VL	VL	VL	VL

Notes: (5) For Cell A5, may be subdivided such that a consequence of less than 0.1% is Low Risk.
(6) When considering a risk assessment it must be clearly stated whether it is for existing conditions or with risk control measures which may not be implemented at the current time.

RISK LEVEL IMPLICATIONS

Risk Level		Example Implications (7)
VH	VERY HIGH RISK	Unacceptable without treatment. Extensive detailed investigation and research, planning and implementation of treatment options essential to reduce risk to Low; may be too expensive and not practical. Work likely to cost more than value of the property.
H	HIGH RISK	Unacceptable without treatment. Detailed investigation, planning and implementation of treatment options required to reduce risk to Low. Work would cost a substantial sum in relation to the value of the property.
M	MODERATE RISK	May be tolerated in certain circumstances (subject to regulator's approval) but requires investigation, planning and implementation of treatment options to reduce the risk to Low. Treatment options to reduce to Low risk should be implemented as soon as practicable.
L	LOW RISK	Usually acceptable to regulators. Where treatment has been required to reduce the risk to this level, ongoing maintenance is required.
VL	VERY LOW RISK	Acceptable. Manage by normal slope maintenance procedures.

Note: (7) The implications for a particular situation are to be determined by all parties to the risk assessment and may depend on the nature of the property at risk; these are only

10 CONSENT CONDITIONS

The following consent conditions apply for Mangakuri Subdivision:

- Lots 3 to 11 building platforms should be lowered (excavated) to form a level building platform and to reduce the risk of further land instability.
- Lot 1 should not be subjected to excavation at the toe of the slope due to risk of land stability.
- Lot 1 may be subjected to fill with geotechnical consideration.
- Where land falls below the building platform:
    - Building setback of 5 m is recommended inside the break in slope (slope crest) for all building platforms formed on cut where ground slopes away exceeding 20 degrees; and/or
- Where land rises above the building platform:
    - Building setback of 5m from the toe of slope is recommended where ground rises above the building platform (Lots 1).
- Building Platforms should be formed entirely within Natural ground (Cut). Engineered Fill should be designated for minor structures and landscaping only unless modified and certified acceptable.
    - All materials excavated from this site in preparation for being used as engineered fill should be tested to confirm the presence of expansive clay soils in accordance with NZS3604:2011.
    - Expansive clay soils can only be reused if modified.
- All cut slopes should be formed at 1V:1.5H and fills at 1V:2H.
- Subsoil drains should be installed where seepage occurs relative to the building footprint or fill placement and in particular on the eastern side of the building platform and where appropriate for road access where seepage is observed.
- Cut-off drains to be installed above building platforms and road cuts.
- Due to the expansive nature of soils, strict control on planting is required. We recommend all cut and fill slopes and stormwater and effluent discharge areas to be planted with small shrubs and shallow rooting plants.
- Large tree species may not be planted within a horizontal distance equivalent to the mature tree height of any pertinent structure (house, road, stormwater, drainage).

- Stormwater Pond to be assessed and designed by competent engineers considering embankment suitability and slope stability.

11 FURTHER GEOTECHNICAL INPUT

A suitably qualified geotechnical professional should be engaged:

- To provide further detailed design assessment for Building Consent;
- To provide geotechnical supervision and recording during construction in the form of a completion certificate, Form 6 for 224c.
- To provide geotechnical supervision and documentation to confirm earthworks are in accordance with NZS 4431:2022 "Engineered Fill Construction for Lightweight Homes"; including.
  - Documenting strip of organic and soft or loose soils; and
  - Documenting benching of surfaces for fill placement;
  - Documenting cut and fill progress.
- For inspection of excavations for retaining walls before piles are installed;
- To confirm suitable drainage for seepage in exposed cut slopes;
- To confirm the geotechnical suitability of finished building platforms, including:
  - confirmation of appropriate angles for finished batters;
  - review of compaction test results on engineered fill;
  - review as-built information for delineation of cut and fill.

12 LIMITATIONS

- This report has been prepared for the particular purpose outlined in the project scope and no responsibility is accepted for the use of any part in other contexts or any other purpose.
- Ground conditions assessed in this report are inferred from published sources, site inspection and the investigation described. Variations from the interpreted conditions may occur, and special conditions relating to the site may not have been revealed by this investigation, and which are therefore not considered. No warranty is included either expressed or implied that the actual conditions will conform to the interpretation contained in this report.
- No responsibility is accepted by Resource Development Consultants Ltd for inaccuracies in data supplied by others. Where data has been supplied by others, it has been assumed that this information is correct.
- Groundwater conditions can vary with season or due to other events. Any comments on groundwater conditions are based on observation at the time.
- This report is provided for sole use by the client, section owners, and CHBDC. No responsibility whatsoever for the contents of this report shall be accepted for any person other than the client.

We trust this meets your current needs. Should you wish to discuss any aspect of the contents of this document please contact the undersigned on 06 877-1652.

Sincerely,

Prepared by:	Approved by:
T Bunny	CA Wylie
PGDip EngGeol | CPEng | CMEngNZ	MSc, CMEngNZ; CPEng
Principal Engineering Geologist	Principal

FIGURE 1: SITE INVESTIGATION PLAN

FIGURE 2: GEOMORPHIC MAP

FIGURE 2A FIELD MAP

FIGURE 2B GIS RENDERED GEOMORPHIC MAP

FIGURE 3: HISTORICAL IMAGERY & DEM MAPS

FIGURE 4: GEOTECHNICAL CROSS SECTION LOCATIONS

FIGURE 5: BUILDING SETBACK PLAN

APPENDIX A: SITE INVESTIGATION LOGS

APPENDIX B: INFILTRATION TEST RESULTS

APPENDIX C: LAB TEST RESULTS

APPENDIX D: GEO PARAMETERS

APPENDIX E: CROSS SECTIONS

APPENDIX F: SLOPE STABILITY OUTPUTS